EVALUATION OF THRESHOLD GRADIENTS IN CLAY-WATER SYSTEMS AT SMALL HYDRAULIC POTENTIALS

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CHAPTER I

INTRODUCTION

The flow of water in soils has always been of prime importance in the evaluation and design of engineering projects constructed of earth material. Analysis of the flow of water in soils is based on Darcy's (1856) law, which states that the rate of flow is a function of the hydraulic gradient, cross-sectional area of flow, and a constant called the coefficient of permeability. In recent years doubt has been cast upon the validity of Darcy's law when the gradient of hydraulic potential is small. It has been supposed that in clay minerals the anomalous reduction of hydraulic conductivity, where it has been observed, is due to interaction between the water and the surface-active clay minerals. Although the volume of research work on non-Darcy flow is meager compared with the work which has been accomplished on Darcy flow, the interest in investigating non-Darcy flow has started growing recently. Prediction of soil-water movement is important for a number of environmental assessments, including water resources management and pollutant transport problems.

Recently, there has been an increasing interest in measuring and understanding the permeability of both saturated and partially saturated soils. The need for permeability measurements in fine-grained soils seems to be increasing as a result of several recent developments. One such development is increased concern over the long-term environmental effects associated with burying toxic wastes in the ground and the movement of pollutants in the subsurface. Another important concern with regard to fine-grained soils is its use as liners in waste-disposal facilities. Lining waste-disposal impoundments with compacted clay materials of low permeability has been demonstrated to be an effective and economical means of preventing leachate and waste liquid components from leaking from an impoundment and subsequently entering and polluting the groundwater.

The use of impoundments and landfills to store, treat, and/or dispose of unwanted materials has been common practice for industry and municipalities, and since these types of facilities often prove to be cost-effective solutions to solid and hazardous waste handling requirements, their use is expected to continue until other economical waste-treatment technologies are developed. Permeability, or saturated conductivity, remains the primary criterion for evaluating the suitability of clay soils for the lining of solid and hazardous waste landfills and impoundments. The use of fine-grained soils as liners will become more important as the problem of water supply and shortage gains more attention.

The concept of initial gradient, while both interesting and significant, has not been incorporated into the overall risk assessment of contaminant transport and environmental safety at waste storage/disposal sites. For such facilities, the groundwater system becomes the significant medium through which any waste constituents accidentally released from storage structures can be transported to the biosphere. Hydrogeology and geochemistry of storage sites become critical factors in evaluating waste containment, contaminant transport, and overall environmental safety. An element of redundancy generally prevails in the management of such wastes, calling for the provision of multiple barriers and conservatism in facility design. Many believe that the exclusion of the concept of initial gradient from the safety analysis of waste management sites permits a more objective, technical investigation of these facilities.

Most of the theoretical expressions given for permeability of soils have applicability only for coarse-grained soils and generally are of limited use for fine-grained materials, especially clays. The reasons for this are that several factors which affect the permeability of fine-grained soils are not contained in the theoretical equations (i.e., soil composition and structure, permeant characteristics, etc., are not represented); the fact that the various terms are not independent, but are interrelated in a very complex manner; and the difficulty of selecting the effecting "constants" and soil characteristics. Various factors which might affect the permeability of soils in general and fine-grained soils in particular include factors related to the porous media (particle size, void ratio, composition, fabric, and degree of saturation), and factors related to the fluid (viscosity, unit weight, and polarity).

Soil structure (i.e., orientation of the particles in a soil mass) has great influence on the permeability of fine-grained soils. Two extremes of soil structure, namely "dispersed" and "flocculated," exhibit a great difference in permeability of a given soil at the same void ratio. Generally, the more dispersed the structure, the lower will be the permeability, and a more flocculated structure provides higher permeability. This is one of the important factors which is not considered in the ordinary expressions for permeability. The structure of the soil in most laboratory experiments (disturbed samples) is controlled by the compaction process and depends on whether the soil is compacted on the "wet" or "dry" side of optimum. Compaction on the wet side provides a more dispersed structure (parallel orientation of particles) and lower permeability, while compaction on the dry side of optimum gives a more flocculated structure and consequently a higher permeability. Minimum permeability of a compacted soil occurs at a water content greater than optimum.

Some evidence indicates that Darcy's law is not linear for all values of hydraulic gradient, especially the larger values. On the other hand, there is evidence that for fine-grained soils (clays) there may be some "threshold gradient" below which no flow will take place. Laboratory permeability tests have been conducted in which low gradients were applied to fine-grained saturated soils. As a result, some experimenters have reported finding a threshold or initial gradient which had to be exceeded before flow would take place. Others have suggested that an initial gradient does not exist, but under very low gradients deviations from Darcy's law may be observed. More recent work suggests that the deviations from the predicted flow are not due to non-Darcian flow, but rather to a combination of experimental artifacts and changes in the soil fabric as flow is taking place. In either case, these deviations from predicted flow using Darcy's law are likely to be most important in the prediction of transport phenomena and the time rate of consolidation of cohesive soils in the field, where low hydraulic gradients occur. It is not clear, at this time, what magnitude of error results from using Darcy's law and a constant coefficient of permeability for such small gradients.

Darcy's law states that the flow velocity is directly proportional to the applied hydraulic gradient. The presence of non-Darcian behavior is marked by the existence of an initial or threshold gradient which must be exceeded before flow will take place. This initial gradient, however, does not necessarily imply that there is absolutely no flow at smaller gradients. Flow may occur under such circumstances, but the rate may be too low to be detected with any accuracy in the laboratory. The permeability coefficient at gradients smaller than the critical (if any) may be substantially smaller than the corresponding value at higher gradients. Hysteretic flow is possible, resulting in a threshold gradient with increasing head but with little or no threshold gradient with decreasing head.

The proposed research principally involves evaluation of the existence of general non-Darcy behavior, and specifically threshold gradients, for water flow in saturated, natural, compacted, cohesive soils typically used in Oklahoma and elsewhere for construction of earth structures. The existence of an initial or threshold gradient will be tested for through conduct of laboratory flow studies in clay samples compacted to various known placement conditions, using small hydraulic gradients within the practical range of engineering interest. The apparatus generally used for laboratory permeability testing for the existence of threshold gradients in clay soils are described in the literature. Basically, laboratory testing apparatus fall into two categories; oscillating permeameters and modified conventional triaxial cell apparatus (Chapter III). The modified triaxial permeability apparatus, which consists of an ordinary triaxial cell connected to a system for applying hydraulic gradients and for measuring the resulting flow rates through the sample, will be used in laboratory flow studies. Further, a two-phase testing procedure (described in Chapter III) will be utilized in conduct of laboratory flow studies, consisting of sample saturation under back pressure followed by soil permeability testing.

Some clay soils (as montmorillonites) are unstable even in water and decompose, releasing salts into solution. Research has shown that such decomposition, or contamination of apparatus, can result in unexpected osmotic effects. If flow studies are initiated and any osmotic effects present are not accounted for, apparent threshold gradients can be found. To minimize the potential for unexpected osmotic effects as a result of soil decomposition and the subsequent releasing of salts (electrolytes) into solution, leading to apparent threshold gradients, an Oklahoma soil with stable clay mineralogy properties (also described in Chapter III) will be used in laboratory flow studies. Further, replicate samples will be compacted and tested for each placement condition (density and moisture content), to insure reliability of test results.

It is hoped that as a result of experimentation a better understanding of the various factors which affect the permeability of fine-grained soils will be achieved, and the question of the existence of general non-Darcy behavior (and specifically threshold gradients) in saturated clay systems will be answered. As previously mentioned, soil characteristics affecting permeability are interrelated in a very complex manner. The various properties which affect the permeability of fine-grained soils include particle size, void ratio, composition, fabric, and degree of saturation, as well as permeant characteristics.

The proposed research will attempt to identify and/or quantify the effecting "constants" and soil characteristics in terms of commonly used engineering properties. The current state of the art in permeability testing is such that most of the theoretical expressions given for permeability of fine-grained soils do not directly consider the various factors which affect soil permeability, including soil composition and structure. The need for reliable and representative soil hydraulic characteristics has become increasingly important with the application of simulation models to phenomena involving soil-water flow.

The research will provide a better idea of the magnitude of error resulting from using Darcy's law and a constant coefficient of permeability in prediction of soil-water movement in environmental assessments, including water resources management and pollutant transport problems. These deviations from predicted flow using Darcy's law are likely to be most important in the prediction of transport phenomena and time rate of consolidation of cohesive soils in the field, where low hydraulic gradients occur.

In addition to documenting the accomplishment of previously described research objectives, this paper will also serve to document the use of a constant-head triaxial cell test apparatus for determining the hydraulic conductivity characteristics of a compacted clay soil. In recent years, a controversy has developed over the use of "fixed-wall" versus "flexible-wall" permeameters for measuring the hydraulic conductivity of clay soils. Conventional fixed-wall permeameters include compaction-mold and consolidation-cell permeameters, while flexible-wall permeability tests are normally conducted using modified triaxial cell apparatus.

Proponents of fixed-wall devices point to low cost, ease of operation, and applicability to compacted soils when defending the use of fixed-wall permeameters. Critics of fixed-wall permeameters argue that there may be imperfect contact between the soil and the inside of the fixed-wall cell, which can lead to so-called "sidewall" leakage and erroneously large measurements of permeability. Flexible-wall devices not only tend to minimize sidewall leakage, but are also convenient for testing with back pressure, for measuring volume change within the soil specimen, and for controlling both the horizontal and vertical effective stress on the specimen. However, if the lateral effective stress applied to the soil in a flexible-wall cell exceeds the lateral effective stress in the field, the measured permeability may be much too low.

Few details are given in the literature regarding specific equipment and testing procedures used in permeability testing of fine-grained soils with flexible-wall permeameters. This paper will discuss factors related to the equipment and testing procedures used in this study which may have influenced the value of coefficient of permeability as determined in the laboratory using the triaxial, constant-head, permeability testing technique. Further, recommendations will be made regarding future application of triaxial apparatus in determining the permeability of fine-grained soils.

CHAPTER II

LITERATURE REVIEW

Movement of Water in Soils and Concept of Permeability

Since the discovery by Darcy in 1856 of the proportionality of the discharge of a fluid through a porous medium to the product of the cross-sectional area of the medium and the first power of the hydraulic gradient, many engineers and scientists have investigated the variables which influence the coefficient of permeability (proportionality coefficient in Darcy's equation) and the methods of determining it. Because of the numerous factors involved in the problem (described in the following sections), no universally valid relationship has been obtained which adequately describes the permeation process for all soils and fluids.

Most of the theoretical expressions given for permeability of soils have applicability only for coarse-grained soils, and generally are of limited use for fine-grained materials, especially clays. In the case of fine-grained soils, factors such as mineralogy of the particles, particle size and shape, adsorbed ions, and physical characteristics of the permeating fluid affect permeability significantly. Many aspects of this problem have been recognized and solved, while many others are still under investigation.

Soil-Water System

As soon as water, a liquid composed of dipolar molecules, comes into contact with a wettable solid surface, its physical properties are greatly altered from those of the bulk liquid, depending upon the nature of the surface and the solutes in the liquid (44). In the case of soil particles, the nature of the solid surface is influenced by a variety of factors including the physical, chemical, and mechanical properties of the particle. In coarse-grained soils (sands and silts) the predominant phenomenon is the physical adhesion of the water molecule to the soil particle, while for fine-grained materials (clays) chemical behavior of the particle has a very important role.

Grim (21) has studied adsorption and orientation of water molecules on clay mineral surfaces, and discussed its implications with respect to the properties of a clay-water system. According to his findings, the structure and organization of adsorbed water molecules depends on the clay mineral composition and its adsorbed ions. He showed that water molecules tend to group into a network around the soil particles. This initial water is adsorbed in a rigidly oriented state, and as the adsorbed water layer increases in thickness, there is a point at which this orientation is lost or greatly reduced in rigidity of organization.

The forces that cause adsorption of water molecules on soil particles are chemical or electrical in nature. They may originate from broken bond forces caused by interruption, at the particle surface, of the normal sequence and balance of the molecular or atomic force fields within the crystal lattice. These surface atoms tend to establish bonds with adsorbed atoms by sharing electrons or orbitals. The attraction may also arise from Van der Waals' forces, which cause bonding of adsorbed molecules by lowering the total energy of the system through mutual harmonic motion of the electrons in the electron clouds (65).

The manner of adsorption of water molecules around clay particles has important consequences in its effects on the physical and mechanical behavior of a clay-water system. Grim (21) has indicated that the mineral composition of the clay particles has a great influence on the thickness and rigidity of the adsorbed water layer. The thickness of the water layer around different clay mineral particles generally decreases in the following sequence: montmorillonite, vermiculite, illite, chlorite, kaolinite, halloysite, and allophane. However, the type of adsorbed ions on the surface of a given clay also has a great effect on the thickness of the water layer. Among different ions, Mg^{++} and Ca^{++} tend to develop a very well oriented system of water molecules to a thickness of about two-to-four molecular layers, while Na^{+} provides a very thick layer of water (tens of molecules), but with a very loose orientation. Other ions such as K^{+}, H^{+}, Al^{+++}, and Fe^{+++}, form a light bond between particles with very low potential for the growth of thick oriented water layers (18, 21).

Water Movement in Soil

Like other bodies in nature, a soil-water system can contain energy in different quantities and forms (26). Between the two principal forms of energy, namely kinetic and potential energy, the former is often negligible because it is proportional to the square of the velocity which is commonly quite low, especially in fine-grained soils. The latter form of energy (i.e., potential energy), which is due to position or internal conditions, is of primary importance in determining the state and movement of water in soils. The potential energy due to water may be different in different parts of the soil mass, causing water movement from a point of higher potential energy to a point of lower potential energy (i.e., in the direction of decreasing potential energy). The rate of decrease of potential energy with distance is in fact the motivating force causing flow. If the change in potential energy, p, in a distance, dx, is presented by dp, the force acting on water directed from a zone of higher potential to a zone of lower potential will be equal to the negative of the potential gradient, $-\frac{dp}{dx}$.

Hydrostatic potential gradient is the main form of energy that causes water movement in soils, although the nature of water movement under thermal and electrical potential gradients has also been investigated (81). Research has shown that water will move from a location of higher temperature to that of a lower temperature in a soil mass subjected to a temperature gradient (32, 81). Further, the highly electrical character of the soil-water interaction phase renders pore water in soil susceptible to movement if an electrical potential is applied. In a moist clay soil system in which the electrical charges are assymetrically distributed between the predominantly negatively charged clay particle surfaces and predominantly positively charged water phases, any electrical interference in the stable system may result in the movement of water molecules. This process is commonly referred to as electro-osmosis, and many references concerning it are found in the literature (12, 79, 80, 81).

The water phase in soils can be found in two different forms. First, there is a rigidly adsorbed form called fixed or adsorbed water, the thickness of which depends on the nature of the soil particles. In coarse-grained soils, this layer is very thin and negligible in comparison to the particle size. However, in fine-grained soils such as clays, this layer is relatively thick and has a very important effect on the movement of water in the soil. Second, there is free water which can flow through soil pore spaces under an applied potential gradient (58). Actually, there is no distinct boundary between these two phases -- the rigidity of the adsorbed water layer generally decreases with distance from the particle surface, so that at some point water is not under any adsorptive force from the particle surface. Some investigators believe that the thickness of the adsorbed water (fixed layer) is a function of the applied hydraulic gradient (39, 58, 62, 70, 81).

The early theories of fluid mechanics were based on the properties of a perfect fluid (i.e., one that is both frictionless and incompressible). In the flow of real fluids, however, adjacent layers do impose tangential stresses (friction or drag forces). In the case of water movement in soils, boundary friction must also be taken into consideration (26, 65). The degree of interaction (fixation) between fluid and soil particles will determine the thickness of the stationary boundary layer (adsorbed water) and, as previously discussed, it is a function of mineral composition of the soil, ions adsorbed by particles, and physical properties of the fluid (65).

Concept of Permeability

The ease with which a fluid can move through a porous medium is called permeability, and accordingly, the moving fluid is called a permeant. The medium being permeated is called a permeate, and the system is called a permeation system. From the above definitions, it is seen that the permeability of a medium indicates its ability to pass a given fluid through it, therefore, it is related to the properties of both the medium and fluid. In this regard, the problem is one of mutual composite interaction of both phases, and the permeability of the system cannot be investigated by analysis of the properties of each phase separately. For this reason, permeability is neither a dimensionless coefficient nor a material constant, but rather a property of the permeation system. The many factors affecting permeability will be discussed in a separate section.

Darcy's Law and Concept of Threshold Gradient

During his experimental studies on sand filters, Darcy (1856) found that the velocity, v_x, of purely viscous flow through an element under a hydrostatic pressure difference, dp, between two points separated by a distance, dx, is given by (69):

$$v_x = -k\left(\frac{dp}{dx}\right)$$

In the above equation, the factor k is called the coefficient of permeability (or simply permeability). The ratio $\frac{dp}{dx}$ (or i) represents the applied pressure (hydraulic gradient) causing flow, and the negative sign in the equation indicates that flow is opposite to the direction of pressure increase. Because the hydraulic gradient is a dimensionless quantity, the dimensions of the coefficient of permeability are the same as those of velocity. In soil mechanics, the dimensions of k are usually expressed in cm/sec. From the above equation, the rate of flow, q, through an area, A, is given by the following expression:

$$q = -k \cdot A \cdot \frac{dp}{dx}, \text{ from which}$$

$$k = -\frac{q}{A \cdot \frac{dp}{dx}}$$

Thus, the coefficient of permeability is simply the quantity of fluid forced through a unit area in unit time under an applied pressure gradient of unity.

Validity of Darcy's Law

Permeability computed on the basis of Darcy's law is limited to conditions of laminar flow and complete saturation of soil voids (10). Since Darcy's law is valid only for laminar flow, it cannot be used for extremely coarse sediments, in which water moves at a high velocity and a turbulent condition exists (69). In turbulent flow, the flow is no longer proportional to the first power of the hydraulic gradient. Further, under incomplete saturation, the flow is in a transient state and is time-dependent. Unsaturated voids, containing some entrapped air, cannot transmit water as well as can saturated voids. Recent investigations (16, 23) have shown, however, that complete saturation of soil voids is not necessary, and that Darcy's law is valid for nonsaturated flow with a modified definition of gradient. As a general rule, the higher the degree of saturation, the higher the permeability (38).

The main factors affecting the permeability of partially saturated cohesive soils are pore water pressure and degree of saturation. Pore water pressures in partially saturated soils are negative compared with the pore air, and this negative pore water pressure is termed "suction." With regard to the effect of degree of saturation on partially saturated soils, measurements have shown that the degree of saturation decreases as the suction increases, and the permeability decreases rapidly as the degree of saturation decreases (27, 58). Although considerable research is currently being conducted in this area of soil mechanics, many questions still remain unanswered regarding measurement of permeability in partially saturated soils.

Stability of the permeation system as a requirement for the validity of Darcy's law has been discussed by Schmid (65), who states that several conditions should be either absent or negligible during the permeation process. These include volume change of permeate and permeant, ion exchanges or dissolution and leaching of the permeate, deposition of solid, liquid, or gaseous matter by the permeant within the permeate, structural changes of the permeate due to dispersion, aggregation, or change in the crystal lattice, and temperature changes. When these changes take place, they generally occur at the beginning of the permeation process, but sometimes may continue after an otherwise stable condition is reached.

Magnitude of Hydraulic Gradient

The applicability of Darcy's law for different hydraulic gradients has also been investigated. There appear to be both upper and lower limits to the magnitude of hydraulic gradient for which Darcy's law has been verified for a variety of soils and fluids. In investigating the upper limit, it was found that Darcy's law accurately represents the flow through a porous medium if the flow is approximately laminar. For the normal maximum gradients or velocities encountered in soils, Muskat (55) suggests that laminar flow can be expected in soils up to a range in size of medium-to-coarse sands. However, Anandakrishnan and

Varadarajulu (1) found nonlaminar flow in fine and medium sands under moderate hydraulic gradients, and suggested a nonlinear form of Darcy's law, $v = (k'i)^{1/n}$, in which k' is a modified coefficient of permeability and n an exponent between 1 and 2.

Laboratory permeability tests have also been conducted in which low gradients were applied to fine-grained saturated soils. As a result, some experimenters have reported finding a threshold or initial gradient which had to be exceeded before flow would take place (49, 56). Others (24, 70, 71) have suggested that an initial gradient does not exist, but under very low gradients deviations from Darcy's law may be observed. More recent work (53, 57) suggests that the deviations from the predicted flow are not due to non-Darcian flow, but rather to a combination of experimental artifacts and changes in the soil fabric as flow is taking place.

Slepicka (67) has expressed the general form of Darcy's law as:

$$v = k(i)^n$$

where

$n > 1$ for very small velocities
$n = 1$ for intermediate velocities
$n < 1$ for very high velocities

This has been confirmed by Hansbo (24), Swartzendruber (70), and Abelev cited in Olson and Daniel (58) for very low velocities, and by Muskat (55) and Muskat and Botest (54) for high velocities. Darcy himself realized that his equation was not valid for high fluid velocities, and during the past forty years considerable effort has been directed toward a fuller understanding of the problem (17, 44, 55). It seems well established that when the hydraulic gradient exceeds a critical value, the flow velocity is no longer proportional to it, but increases less rapidly than the gradient. Burmister (10) reported a decrease in permeability with an increase in hydraulic gradient. It is believed that this might be true only for very coarse-grained soils, where flow under high pressure is no longer laminar and the energy losses of turbulent flow cause a decrease of the velocity gain resulting from increased gradients.

There are many different opinions with regard to very low hydraulic gradients. Fishel (17) has indicated that for his experiments, Darcy's law was valid for a hydraulic gradient as low as two or three inches per mile. King (31) has reported that for flow through porous media under very low hydraulic gradients, the velocity was not proportional to the gradient, but increased more rapidly than the gradient. Terzaghi (74) also observed a distinct departure from Darcy's law for very low hydraulic gradients. He reasoned that when water percolates through a clay mass under a considerable head it produces elastic and nonelastic deformations and grain displacement similar to the deformations produced by a stream of water forced through a system of very expansible rubber tubes, but at lower hydraulic gradients, the elastic deformations disappear and the coefficient of permeability changes accordingly.

A more extensive study of this subject was conducted by Swartzendruber (70), who explains the nonproportionality of velocity and hydraulic gradient using the theory of Newtonian and non-Newtonian flows. According to this, there are three types of flow with respect to the relationahip between the rate of shear and shear stress in a liquid (Figure 1). These are Newtonian flow, which is shown as a straight line passing through the origin; non-Newtonian flow, which is represented as a curved line passing through the origin and concaving upward; and

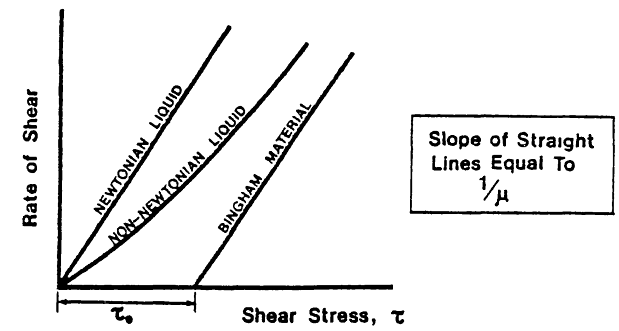


Figure 1. Shear Rate versus Shear Stress for Three Flowing Materials (70)

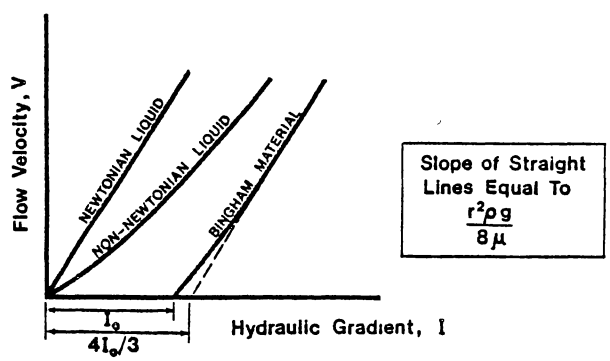


Figure 2. Flow Velocity versus Gradient for Three Fluids in Figure 1 (70)

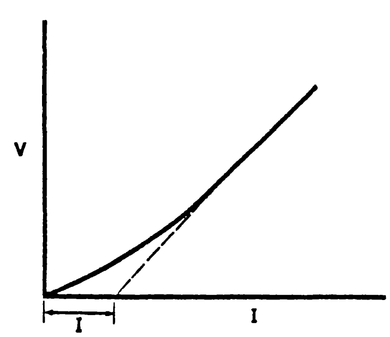


Figure 3. General Aspects of Suggested Flow Relationship in Figure 2 (70)

Bingham flow, which is shown as a straight line but does not pass through the origin.

Swartzendruber compared these three relationships with the velocity-hydraulic gradient relationship of fluid movement in a porous medium. He categorizes as a Newtonian liquid those fluids for which the v-i relationship is a straight line passing through the origin; as a non-Newtonian liquid, those fluids for which the v-i relationship is a curve passing through the origin and concaving upward; and as a Bingham material, those fluids for which the v-i relationship is linear above a given velocity, and nonlinear for velocities less than that (Figure 2). The slopes of straight lines are given as:

$$\frac{r^2 \cdot \rho \cdot g}{8\mu}$$

where r is the radius of capillary channels, ρ is the density of the liquid, μ is the coefficient of viscosity of the fluid, and g is the acceleration of gravity. It appears that this concept can be useful for understanding the v-i relationship for low hydraulic gradients in porous media.

Hansbo (24) and Slepicka (67) have suggested that the curved part of the Bingham line be represented mathematically by a power function such as $v = k(i)^n$. This suggestion shows that they did not believe in the existence of a threshold gradient in very fine-grained soils, a topic which has been the subject of many papers. The general shape of the v-i relationship according to Hanbo's suggestion would be as shown in Figure 3. However, many workers in this area believe that there is a threshold gradient (lower limit for i) below which there is no flow and therefore $v = 0$ (15, 39, 44, 49, 56, 65). This problem, which is the topic of this paper, will be discussed in more detail in the following section.

Concept of Threshold Gradient

As is evident from the material presented in the previous section, the effect of magnitude of hydraulic gradient on permeability has been the main topic of a number of investigations. Some studies support the theory of a dependency of permeability on hydraulic gradient, while many others do not. Threshold gradient (i.e., the minimum hydraulic gradient required to start the permeation process in a given soil) is one aspect of this problem, and the topic of this paper. The problem basically relates to the nonproportionality of flow velocity and hydraulic gradient for very low and very high gradients, and to the nonlinear v-i relationship for these two extremes.

Considering the theory of stationary boundary layer (a viscous or rigid layer of water adsorbed by soil particles), it is reasonable to assume that only a part of the channel voids are available for flow (65). It has also been shown that the thickness of this fixed layer is related to the applied pressure. Some of those who have worked on the threshold gradient problem believe that there is a hydraulic gradient below which no flow occurs, and that a linear v-i relationship having an intercept with the i-axis at a value i_o greater than zero, exists for values of i greater than i_o (see Figure 2). Thus, it is assumed that Darcy's law may be applied over the entire range of flow velocities (39, 49, 81). Other investigators have found that v-i realtionship to be nonlinear for very low values of i, represented by a curve passing through the origin and exhibiting an upward concavity (see Figure 3).

A similar nonlinear v-i relationship has sometimes been reported also for very large values of hydraulic gradient. For very high values of i, the velocity increases more rapidly than i. In cases where the flow remains laminar, the nonlinearity appears to be associated with a decrease in the thickness of adsorbed water film and the consequent enlargement of flow channels and increase in effective porosity resulting from the increased stress level (24, 44, 60, 70). The enlargement of pore openings may be related to a reversible orientation of the particles along the stream line or a surpassing of the yield value of the smaller pores as i increases (new pores become available to flow). These phenomena are also consistent with the threshold gradient theory. For some soil types, none of the behavior described above has been observed. The aberrant behavior is observed especially in the more clayey soils (49). As a general rule, there is a straight-line relationship between v and i for intermediate values of i.

Nonlinear saturated flow has been found in soils by King (31), von Engelhardt and Tunn (76), and Hansbo (24); and in clays by Micheals and Lin (47), Lutz and Kemper (44), and Miller and Low (49). Threshold gradients have been reported for ceramic filters by Derjaguin and Krylov (15) and for clay systems by Oakes (56), Li (41), and Miller and Low (49). To account for these observations, one or a combination of several explanations is usually invoked. These are: (i) quasi-crystalline water, (ii) particle reorientation, (iii) electrokinetic effects, and (iv) a range in pore sizes. The prevalent hypotheses advanced are those relating to matrix effects and a quasi-crystalline water structure resulting in shear-dependent viscosities.

Von Engelhardt and Tunn, Hansbo, Lutz and Kemper, and Miller and Low accounted for the nonlinearity they observed by invoking the idea that clay surfaces alter water structure. This altered water is thought to be more ordered because of surface-induced hydrogen bonding and is most pronounced at the clay-water interface. The amount of hydrogen-bonded water decreases with distance from the clay surface until normal water is reached. The distance to normal water depends on the surface, its charge, and the exchangeable ions present. When low hydrostatic heads are applied, water flows in the center of the pores only, where the least altered water exists. With each higher pressure increment, additional layers of water are "sheared," giving increased permeabilities or flow rates. This continues with further increases in head until the whole pore is conducting water.

Low (43) has suggested that the presence of a threshold gradient would be a critical test for the presence of quasi-crystalline water on clay surfaces. Below the threshold gradient, no water would flow and thus electroviscous or plugging effects could be discounted. Miller and Low (49) argued that if clay surfaces do order water, then at some clay concentration this quasi-crystalline water would extend across the entire pore. A finite energy, or head, would be needed to break down this structured barrier before flow would begin. On the basis of the threshold gradients they reported and from experiments which indicated that the activation energy for water flow in clay pastes increased as the applied gradient decreased, Miller and Low concluded that a quasi-crystalline water structure does exist on clay surfaces.

Micheals and Lin (47), Martin (46), and Mitchell and Younger (53) suggest that particle reorientation is the most important effect in non-Darcy flow. These matrix effects need not be just a simple movement of particles, but may include bending and flexing of particles or the breaking of edge-to-surface bonds to permit particle reorientation with the flow path. Micheals and Lin (47) also postulated an electroviscous resistance to flow which decreases with increasing head, as flow restricting cations were swept into the larger pores. Kemper (30) has suggested a similar mechanism. However, Micheals and Lin (48) concluded that these effects were probably small. Since electrokinetic theory and experimental results suggest that there is a linear relationship between the electroviscous effect and applied pressure, this factor is probably not of major importance.

Miller and Low (49) suggested that nonlinear flow could be caused by a range in pore size. As the hydraulic gradient was increased, the threshold gradient was exceeded in smaller pores increasing the flow rate. The existence of a threshold gradient is necessary for this explanation. Miller and Low (49) and Kolaian and Low (33) suggest that the development of water structure in a clay-water system is a time-dependent process. If this is true, transient measurements may not be a meaningful test of threshold gradients. Hysteretic flow is possible, resulting in a threshold gradient with increasing head but with little or no threshold gradient with decreasing head. That this occurs is unlikely since no evidence of a time-dependent water structure in bulk water has been found. In addition, Miller and Low (49) found that once the applied gradient was decreased below the threshold, the threshold gradient reestablished itself in approximately three hours.

Flow studies were conducted by Miller, Overman, and Peverly (50) in concentrated Wyoming bentonite and kaolinite clay samples in an attempt to verify the existence of threshold gradients in these clay-water systems. Kaolinite is a 1:1 layer silicate with the individual platelets held together by hydrogen bonding, forming packets of various sizes. Because of the orderly stacking of plates, kaolinite has a relatively small surface area. Montmorillonite, on the other hand, is a 2:1 layer silicate with expanding characteristics. Because it is an expanding mineral, montmorillonite has a large surface area. Due to its characteristics, montmorillonite would appear to be more subject to matrix rearrangements than kaolinite. It has a larger component of particles that can bend, flex, and move, whereas, kaolinites do not have many small particles to move or shift.

Similarly, because of their low surface area, kaolinites are usually thought of as minimizing water structure, if they do in fact alter water structure. The effect kaolinite has on water will be as large per unit of area, but since there is so little area the water effects would be hard to measure. Montmorillonite, having both a large surface area and associated exchange capacity, would be expected to maximize any water effect that is a result of surface, exchangeable cations, or charge-induced phenomena. In the system tested no threshold gradients were found, and it was concluded that water in these systems was not stabilized by the clay surfaces to the extent a finite pressure is needed to cause the water to flow. The water apparently retained fluid properties at all clay concentrations.

Portions of the above non-Darcy data have been criticized for various reasons. Li's (41) data was questioned by Olsen (57) because of uncorrected experimental errors which could account for the reported threshold gradients. Oakes (56) cited only one of several experiments in which threshold gradients were found. Since no experimental details were given, it is possible that evaporation and other errors were not compensated for. Jackson (28) contends that the threshold gradient that Derjaguin and Krylov (15) observed in ceramics could have been a result of osmotic effects. Olsen (57) has shown that air-contaminated capillaries could account for some of the non-Darcy and threshold gradient characteristics reported. Hansbo's reported deviations can be entirely accounted for this way, and the data of von Engelhardt and Tunn, Miller and Low, and Lutz and Kemper can be partially discredited on this basis.

Of the threshold gradients reported in the literature, Miller and Low (49) were the only ones to present experimental detail and data. Of the four systems they tested, three had negative or reverse flow at zero gradient. Jackson (28) contends this was probably a result of osmotic forces. Even though Miller and Low corrected for the reverse flow, two of these three samples showed a threshold gradient. The flow at zero head makes the threshold gradient observed in these two cases questionable. The fourth sample had the highest threshold gradient and no flow at zero head. Since the clays tested by Miller and Low (49) were prepared with a hydrous aluminum oxide film, it can be argued that the threshold gradient observed by them was a result of such a film, although the evidence is incomplete on this point. In flow studies conducted by Miller, Overman, and Peverly (50) using clay-water pastes prepared without hydrous aluminum oxide surface films, no threshold gradient was observed.

Bondarenko (5, 6) and Bondarenko and Nerpin (7) have presented evidence of the failure of Darcy's law when water flows through a quartz capillary tube under small potential gradients. Since surface activity of the quartz can hardly account for such a phenomenon, these authors rely upon the hydrogen bonding between water molecules to provide shear strengths, and attribute to water a Bingham type of plastic flow at sufficiently low hydraulic gradients. If water does in fact sustain a shear stress up to a limit, τ_o, before yielding, then as shown by Buckingham (9) and Reiner (61), flow in a capillary tube of radius r will not take place with a hydraulic gradient less than a threshold value, i_o, where τ_o and i_o are related by the expression:

$$\tau_o = \frac{g\rho r i_o}{2}$$

In Bondarenko's experiment r had a value of about 10^{-2} cm, departure from Darcy's law became noticeable when i_o was reduced to about 1, and by extrapolation it was estimated that i_o was about 10^{-2}. Thus from the above equation, τ_o was of the order of 5×10^{-2} dyne/cm^2. Childs and Tzimas (13) argued that certainly an ephemeral quartz-like type of water structure, as proposed by Bernal and Fowler (2), accounts for many of the striking properties of water, including its maximum density at 4°C and the anomalous mobility of the hydrogen ion, but the ephemeral nature of the structure hardly seems consistent with appreciable rigidity, and certainly plasticity of water has not heretofore been recorded in simple flow systems. Most of the experiments conducted by Bondarenko and Nerpin were carried out at temperatures near 0°C, and these authors report that the anomaly which they observed disappeared at higher temperatures, although it was still marked at 20°C.

Miller, Overman, and King (51) report experiments which purport to show that Darcy's law is obeyed in contradiction of Bondarenko, when water flows through sintered glass porous membranes of various grades, at low potential gradients, if precautions are taken to keep the pores clear. Experiments conducted by Childs and Tzimas (13), using a granular porous material of glass spheres with pore sizes on the same order as the capillary radius reported by Bondarenko and Nerpin, indicated conformity to Darcy's law (i.e., Newtonian flow) down to one-tenth of the threshold gradients reported by the Russian workers.

Non-Darcian flow behavior in a dense glacial till was investigated by Law and Lee (40) using an ordinary triaxial cell connected to a system for applying hydraulic gradients, which permitted accurate measurement of minute flow under low hydraulic gradients. Law and Lee (40) found that above an initial gradient the flow velocity is linear with the hydraulic gradient, and the permeability coefficient is of the same order of magnitude as that obtained by other conventional tests. Below the initial value the flow behavior is characterized by a permeability coefficient either equal to zero or significantly smaller than that at higher gradients. Initial gradients reported by Law and Lee ranged from 0.10 to 2.9, with coefficients of permeability from 4.2×10^{-9} cm/sec to 1.3×10^{-7} cm/sec.

Darcy's law states that the flow velocity is directly proportional to the applied hydraulic gradient. The non-Darcian flow behavior observed by Law and Lee was marked by the existence of an initial or threshold gradient. This initial gradient, however, does not necessarily imply that there is absolutely no flow at smaller hydraulic gradients. Flow may occur under such circumstances, but the rate may be too low to be detected with any degree of accuracy in the laboratory. The permeability coefficient at gradients smaller than the critical (if any) may be substantially smaller than the corresponding value at high gradients. A modified triaxial apparatus similar to that used by Law and Lee (described in Chapter III) will be used in this study to test for threshold gradients in clay-water systems.

Factors Affecting Permeability

As mentioned previously, most of the theoretical expressions given for permeability of soils have applicability only for coarse-grained soils and generally are of limited use for fine-grained soils, especially clays. The reasons for this are that several factors which affect permeability of fine-grained soils are not contained in the theoretical equations; the fact that the various terms are not independent, but are interrelated in a very complex manner; and the difficulty in selecting the effecting "constants" and soil characteristics.

In the following sections, the influence of various factors which might affect the permeability of soils in general and fine-grained soils in particular will be discussed. These factors are classified in four major groups: (i) factors related to the porous media (permeate), (ii) factors related to the fluid (permeant), (iii) factors related to the permeameter device, and (iv) other factors.

Factors Related to the Porous Media (Permeate)

Mineral Composition of the Soil. This factor has a great influence on the permeability of clay soils. Such soils are composed of particles of varying grain size; however, the distinctive behavior of these soils is generally determined by the presence of certain very small crystalline particles called clay minerals. These particles are hydrous aluminum silicates often containing iron, magnesium, and other metals. It is the electrochemical properties of these minerals that produce many of the properties we associate with clay soils. The results of hundreds of experiments made by different investigators have shown that the permeability of clay soils changes with changes in mineralogical composition of the soil, keeping all other factors the same. A major reason for this is the difference in thickness and rigidity of the water film around the clay particles, which was discussed previously (37, 44). Considering the mineral composition (clay mineralogy) of the soil, the following trends are usually accepted:

"k" montmorillonite < "k" attapulgite < "k" kaolinite (37)

"k" Wyoming bentonite < "k" Bladen clay < "k" Utah bentonite < "k" halloysite (44)

Adsorbed Cations. Because clays are negatively charged and of the appropriate size, the particles act as colloids when placed in aqueous suspensions. When a clay particle is in water, two primary effects occur; first, exchangeable cations are attracted to the vicinity of the particle surface (some anions may be attracted to the positively charged edges of the particle as well), and secondly, within the zone near the particle surface that is strongly influenced by particle charges, the water molecules are also affected. The kind of adsorbed ions on fine-grained soils has a great influence on the thickness of water film around the particles, and consequently, on the permeability of the soil. The ability of a particle to attract (adsorb) cations to its surface is determined by the net negative charge on the particle. Typical ranges for cation exchange capacity (CEC) of the predominant clay minerals are given below:

CEC of Clay Mineral (Millequivalent/100 grams)	
Kaolinite	3- 15
Halloysite	5- 40
Illite	10- 40
Montmorillonite	80-150

Experiments have indicated that the lower the ion exchange capacity of a soil, the lower the effect of the exchangeable ion on permeability.

The cations from the pore water that are attracted to the surface of the clay particles are often termed "exchangeable" ions because other ions can be exchanged for them. The relative ease with which one ion replaces another of the same concentration depends primarily upon the valence and size of the hydrated ion as shown below:

$$Li^{+} < Na^{+} < H^{+} < K^{+} < NH_4^{+} < Mg^{++} < Ca^{++} < Al^{+++}$$

That is, the replacing power of a given ion is less than that of the higher valence or smaller hydrated ion to the right of it. The exception to this is potassium which, once it has been adsorbed onto the clay, is exceedingly difficult to replace because of the way it fits cavities in the surface of the particle where it is very tightly held. The exchanging power is also a function of ion concentration. If a calcium clay is leached with a sodium solution of high concentration, the sodium will replace some or all of the calcium. However, it requires a significantly higher concentration for a monovalent ion to replace a divalent one.

Sodium clays are generally much less permeable to both water and electrolytes than are Ca^{++} and H^{+} clays (3, 25, 44). Still, the effect of Ca^{++} and H^{+} depends on the type of clay. For example, the permeability of various ionic forms of montmorillonite at the same void ratio varies as $K^+ < Na^+ < H^+ < Ca^{++}$, and for kaolinite varies as $Na^+ < K^+ < Ca^{++} < H^+$ (37). Sodium clays, particularly sodium montmorillonite, are generally the least permeable soil mineral and are therefore widely used by engineers as an impermeabilizing additive for other soils.

Void Ratio. Considering the different expressions relating permeability to the void ratio (e) of soils, one would expect a straight-line relationship to exist between k and $\frac{e^3}{1+e}$; but the experimental results given by several authors (47) do not show such a linear relationship. Instead, a number of investigators have obtained a straight-line relationship between log k and e, assuming other factors are held constant (35, 43, 73). Schmid (65) has shown mathematically that if k is directly proportional to the first power of porosity (n), there will be a logarithmic relationship between k and e. Some investigators have attempted to find a linearity between k and n (or n^2), but only Winterkorn and Moorman (78) have succeeded. They reported a straight-line relationship to exist between k and n^2.

Soil Structure. As previously mentioned, soil structure or the orientation of the particles in a soil mass influences the permeability of fine-grained soils. Two extremes of soil structure, namely "dispersed" and "flocculated," exhibit a great difference in permeability of a given soil at the same void ratio. Generally, the more dispersed the structure, the lower will be the permeability, and a more flocculated structure provides a higher permeability (38).

The structure of the soil in most laboratory experiments (disturbed samples) is controlled by the compaction process and depends on whether the soil is compacted on the "wet" or "dry" side of optimum. Compaction on the wet side provides a more dispersed structure (parallel orientation of particles) and lower permeability. Compaction on the dry side of optimum gives a more flocculated structure and consequently higher permeability (38). Minimum permeability of a compacted soil occurs at a water content greater than optimum (36, 37, 68, 77).

Soil Texture/Particle Size and Shape. In general, the permeability of a soil decreases with an increase in clay or silt content (16). Finer particles in a soil have a high impermeabilizing effect and normally control the permeability of a mixture (10, 37). Normally, the effect of particle size is expressed by specific surface area in permeability equations, but there is no way of considering the effect of the shape of particles, especially for fine-grained soils (3, 37). For coarse-grained soils such as sand and gravel, the effect of particle size has been expressed as an effective grain size, such as in Hazen's equation (29).

Pore Size Distribution. The effect of pore size distribution on soil permeability was investigated by Marshall (45) and Smith, Browning, and Pohlman (68). The effective pore size distribution in soil was measured by water removal at different levels of tension forces. Smith, Browning, and Pohlman found that the saturated pores contribute to flow approximately in proportion to their diameters, while Marshall derived an equation relating permeability to the pore size distribution.

Degree of Saturation. As the degree of saturation is increased, more void spaces in the soil are filled with water and air is forced out or entrapped and compressed. So long as the soil is not saturated, air voids in the form of continuous and discontinuous channels will be present in the system. Unsaturated voids, containing some entrapped air, cannot transmit water as well as can saturated voids. There have been several attempts to take the degree of saturation into account in some theoretical relationships (27, 65). As a general rule, the higher the degree of saturation, the higher the permeability of the soil (38), although Lambe (37) has indicated that the influence of the degree of saturation on permeability is relatively minor in comparison with the composition, structure, and void ratio.

Factors Related to the Fluid (Permeant)

Density and Viscosity. These are the two principal fluid characteristics affecting the permeability of soils (37). These major effects of the permeant on permeability can be eliminated by using absolute or specific permeability, $K = k\frac{\mu}{\gamma}$, where μ and γ are the coefficient of viscosity and unit weight of the fluid, respectively. In general, the lower the viscosity and the higher the density of the fluid, the higher will be the soil permeability.

Type of Fluid. Results of experiments conducted by Micheals and Lin (47) indicated that for two different fluids having the same viscosity and density and under the same conditions, the soil permeabilities were different. This discrepancy was ascribed (37) to electro-osmotic backflow and thickness of the immobilized fluid layer, both of which increase with polarity of the fluid. The experimental results show that at any given void ratio, soil permeability decreases regularly with increasing polarity of the permeant, due to more orderly packing of the solid (47).

<u>Chemical Composition of the Fluid</u>. Chemical composition of the permeant and especially the concentration of ions in a solution has great effect on the permeability of soils. For example, soils remain moderately permeable when leached with a high-sodium solution so long as the salt concentration remains fairly high. The reason for this is that salt tends to maintain the flocculated structure of the soil. If the sodium concentration is lowered, however, the flocculated structure of the soil is destroyed and a dispersed structure with much lower permeability is produced (16).

Factors Related to the Permeameter Device

The equipment available to measure the hydraulic conductivity of saturated, compacted clay generally falls into two categories: (1) "fixed-wall" permeameters, and (2) "flexible-wall" permeameters (82). Two types of fixed-wall permeameters commonly used in the laboratory are the compaction-mold permeameter and the consolidation-cell permeameter. Flexible-wall permeability tests are normally performed in conventional or modified triaxial cells. Each of these devices is described, in more detail, in subsequent portions of this section.

The major differences between fixed-wall and flexible-wall permeameters are the methods of confining and saturating the sample. Most fixed-wall permeameters do not have the capability of saturating the soil under back pressure, and saturation is achieved by "soaking" the sample. Samples tested in fixed-wall permeameters are not usually fully saturated, so the conductivities measured using these devices will normally be lower than the corresponding value for fully saturated soil (82). Perhaps more important than the method of saturation, however, is the method of sample confinement.

Flexible-wall permeameters confine the sample in a flexible membrane. The cell is similar to those used for triaxial shear strength testing, but provisions must be made for top and bottom drainage of the sample. The membrane is held tightly in place against the soil by pressure applied to the cell fluid. Because the membrane is flexible, it can easily conform to surface irregularities along the side of the sample. In this way, flow along the sample-membrane interface is minimized or eliminated.

Fixed-wall permeameters confine the sample in a rigid ring. To prevent side flow, compaction permeameters rely on compaction stresses to provide a good contact between the soil and the permeameter device. The consolidation permeameter uses vertical stresses to force the soil laterally against the ring, thus promoting a good seal. However, as Zimmie (82) has shown, one can never be certain that side flow is not occurring in a fixed-wall device.

Mitchell, Hooper, and Campanella (52) used compaction permeameters and checked their results against tests performed in a flexible-wall permeameter. They found little difference between the measured hydraulic conductivities. Acar cited in Boynton (8) performed tests in a flexible-wall permeameter using acetone as the permeant. By measuring the concentration of acetone in the effluent, he concluded that side flow did not occur in his experiments. If it had, acetone would have appeared in trace quantities well before the acetone front broke through samples. Although it would seem that flexible-wall permeameters would be more effective in preventing side flow than fixed-wall permeameters, the available data is insufficient at this time to conclude with certainty whether or not side flow occurs in either type of permeameter.

Research conducted by Boynton (8) on compacted clays has also shown that only subtle differences exist in the hydraulic conductivities of samples when measured with the previously described permeameter devices. These differences may be attributed to a variety of differences in the equipment, testing procedures, and applied stresses in the various tests. Some of these differences are listed in Table I. Based on the results obtained by Boynton, it would appear that either the parameters listed in Table I were not very significant for those soils tested, or the various differences tended to offset one another.

Carpenter and Stephenson (11) investigated the influence of various test parameters on the measured coefficient of permeability of clays determined using triaxial apparatus. They found that the coefficient of permeability generally decreased with an increase in the length of the sample. They concluded that most of this decrease can be attributed to an increase in the applied effective stress at the outflow end of the sample. This increase in effective stress is necessary to achieve the same gradient as the length of the sample is increased.

The influence of sample diameter on the measured coefficient of permeability was investigated by Carpenter and Stephenson (11) using 4.0-inch-diameter and 2.8-inch-diameter samples. They found that the measured coefficient of permeability was much less sensitive to changes in the applied effective stress (outflow end of sample) for the 4.0-inch-diameter samples as the applied effective stress was increased. Further, for the clay used in their study, the test data for the

TABLE I

DIFFERENCES IN SELECTED TEST PARAMETERS FOR VARIOUS TYPES OF PERMEAMETERS

Test Parameter	Type of Permeameter		
	Compaction-Mold	Consolidation-Cell	Flexible-Wall
Sidewall Leakage	Leakage is Possible	Applied Vertical Stress Makes Leakage Unlikely	Leakage is Unlikely
Void Ratio (e)	Relatively High Since Applied Vertical Stress is Zero	Relatively Low Because a Vertical Stress is Applied	Relatively Low Because an All-Around Confining Pressure is Applied
Degree of Saturation	Specimen May Not Be Fully-Saturated	Specimen May Not Be Fully-Saturated	Application of Back Pressure Will Likely Result in Complete Saturation
Voids Formed During Trimming	None - Soil is Tested in the Compaction Mold and is Not Trimmed	Voids May Have Formed, but Application of a Vertical Stress Should Help in Closing Any Voids	Not Relevant - the Flexible Membrane Conforms to the Irregular Surface of the Soil Specimen
Portion of Sample Tested	All of the Compacted Specimen is Tested, Including the Relatively Loose Upper Portion; the Dense Lower Portion May Lead to Measurement of a Relatively Low Permeability Value	Only the Central Portion of the Specimen is Tested; the Upper and Lower Third of the Specimen are Trimmed Away	Approx One Centimeter of Soil is Normally Trimmed Off Both Ends of the Compacted Sample

2.8-inch-diameter samples resulted in a measured coefficient of permeability that was about twice that of the 4.0-inch-diameter samples, when the applied effective stress at the outflow end was equal to the preconsolidation pressure (i.e., applied during the back-pressure saturation phase). In this regard, these researchers believe that tests should be performed on 4.0-inch-diameter samples (not less than 2.8-inch-diameter) with a length-to-diameter ratio of between 0.5 and 1.0.

Carpenter and Stephenson (11) investigated the effects of gradient magnitude on the coefficient of permeability when determined in the triaxial permeability apparatus, and found it to decrease as the gradient was increased. This decrease in permeability was also attributed to the necessary high effective stress applied to the outflow end of samples during permeability testing. These researchers concluded that tests using triaxial permeability apparatus should be conducted at a gradient that results in an applied effective stress at the outflow end of the sample less than the preconsolidation stress of the material.

Fireman (16) studied the effects on permeability of the size and shape of the permeameter device for fixed-wall permeameters. In his experiments, he found no difference in the measured permeability of samples having different lengths varying from one inch to 34 inches and different diameters of one inch to six inches. He found, however, that with increasing diameter and length, the nonuniformity in packing and particle size distribution is diminished. Fireman's experiments also indicated that a cylindrical form is the most suitable shape for a permeameter and soil sample to avoid any kind of flow restriction.

Franzini (19) and Rose and Rizk (63) investigated the effects of the permeameter wall on permeability for fixed-wall permeameters. The effect of the permeameter wall depends on Reynold's number and the ratio of the permeameter diameter (D) to the mean particle diameter (d). If the $\frac{D}{d}$ ratio is greater than 40, the effect of the permeameter wall is of negligible magnitude (19, 63). The main effects of the permeameter wall arise from an increased resistance to flow along the surface of the wall and a greater soil porosity in the immediate vicinity of the wall than in the body of the medium.

Fireman (16) also conducted extensive research on the suitability of various porous materials used for retaining the sample in permeameter devices. The permeability of these materials should be much greater than that of the soil, and should be constructed so that a relatively uniform water movement is maintained through the sample and soil sloughing is prevented. He found perforated brass disks covered by a thick layer of coarse asbestos, sand supported by a thin fiberglass screen, and lathing screen covered by one thickness of "fast" filter paper were particularly well suited for this purpose.

Details regarding the three types of permeameter devices previously mentioned is presented in the material which follows. In each case, only the essential elements of the device will be described.

Compaction-Mold Permeameter. The most common compaction-mold permeameter consists of a standard 4.0-inch-diameter mold with a modified base plate, and a special collar and top plate (Figure 4). The sample is compacted in the mold and the top surface of the sample is trimmed flat. The permeameter is then assembled, and the permeant is ponded on top of the specimen. Air pressure may be applied to the permeant to promote flow through the sample. The measured outflow is used to calculate hydraulic conductivity.

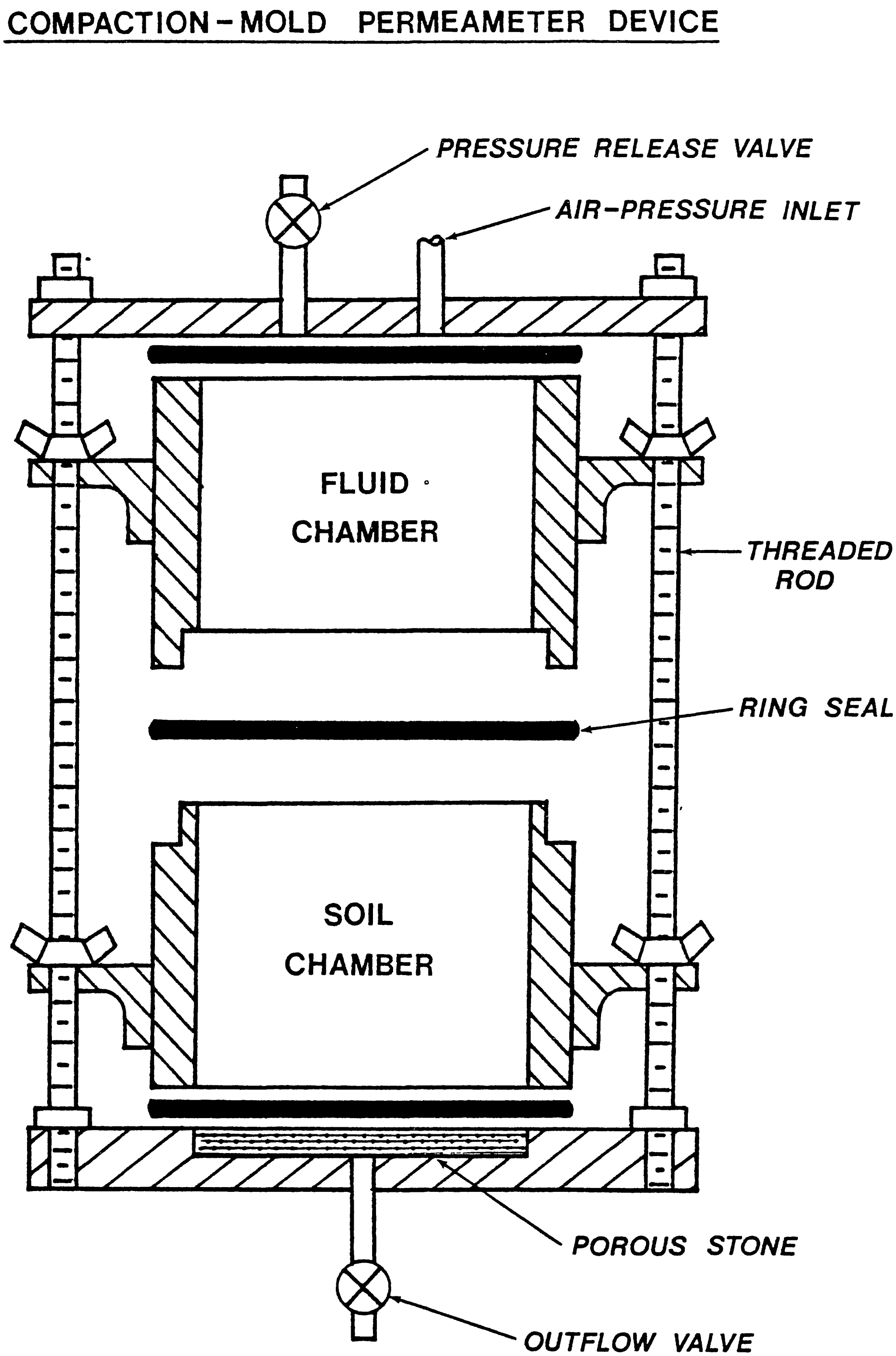


Figure 4. Schematic Diagram of a Typical Compaction-Mold Permeameter (8)

Two main advantages of this type of permeameter are its simplicity and economy. The only preparation required after compaction is trimming the top of the sample and assembling the apparatus. The peripheral equipment required to perform the test is less elaborate than for tests with flexible-wall permeameters, and the cell itself is less expensive. One disadvantage associated with the compaction permeameter is the way the sample is saturated. Most fixed-wall permeameters have no provisions for back-pressure saturation. Although a vacuum may be applied to the outflow end of the cell, it is still unlikely that the sample will be saturated completely.

One can never be sure if side flow is occurring when fixed-wall permeameters are being used. This problem is compounded by the lack of control over the state of stress in a compaction-mold permeameter. Vertical pressure can be helpful in sealing any gaps between the soil and the permeameter wall. If organic solvents are used as permeants, this can be a particular problem. Organic solvents cause some clays to shrink and pull away from the permeameter wall, which leads to excess side leakage. Compaction-mold permeameters are best suited for testing laboratory-compacted specimens of clay that will be subjected to low overburden stresses in the field. Any permeant fluid may be used.

Consolidation-Cell Permeameter. A modified consolidation cell, such as the one shown in Figure 5, can be used to perform permeability tests. One advantage the consolidation permeameter has over the compaction permeameter is that vertical loads may be applied to the specimen. This helps prevent side leakage, by forcing the soil laterally against the ring during consolidation. Some consolidation cells are available

CONSOLIDATION - CELL PERMEAMETER DEVICE

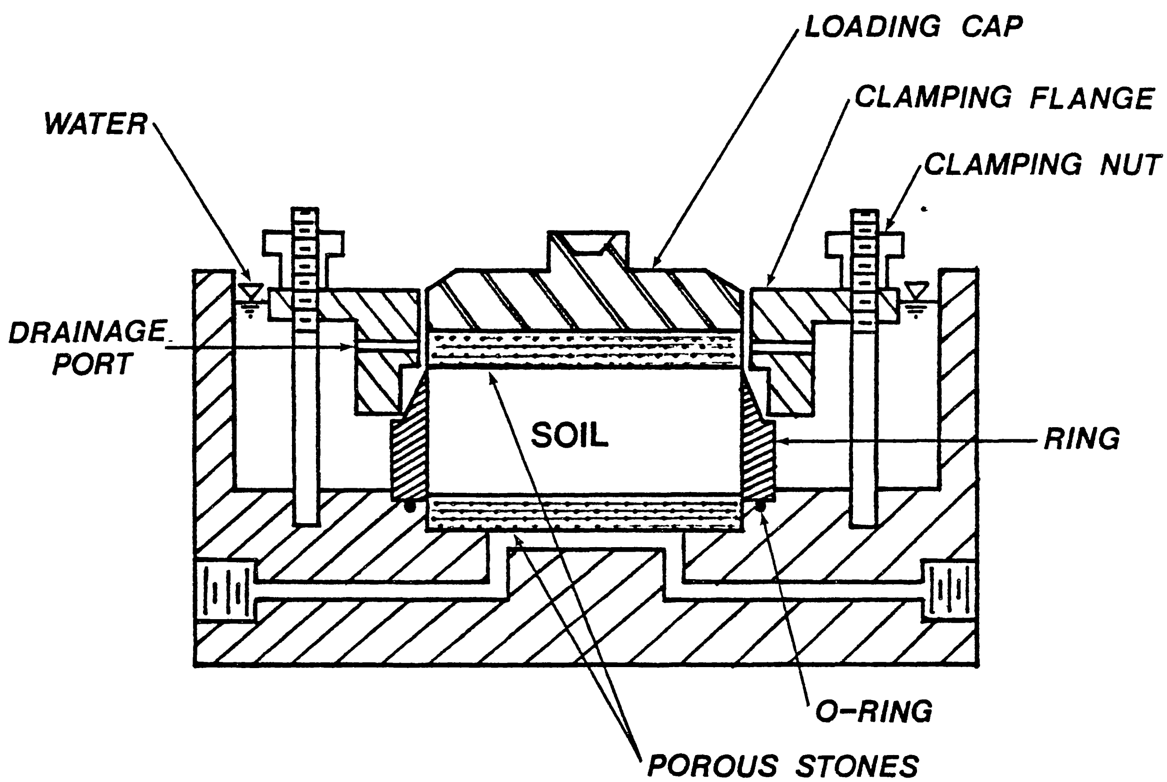


Figure 5. Schematic Diagram of a Typical Consolidation-Cell Permeameter (8)

with back-pressure capability, but this equipment is not used very extensively and is significantly more expensive than consolidation apparatus without back-pressure capability. One disadvantage of this apparatus is that thin samples of relatively small diameter are normally required. The consolidation-cell permeameter is best suited to testing undisturbed samples of relatively compressible soil, such as soft-to-moderately stiff clays, that will be subjected to significant overburden pressure.

Flexible-Wall Permeameter. Flexible-wall permeameters have several advantages over fixed-wall permeameters. The ability to minimize side flow has already been mentioned as one of the main differences between the two types of permeameters. The ability to saturate under back pressure and to control the state of stress are two other important advantages of flexible-wall permeameters. A flexible-wall permeameter may also be desirable when testing permeants other than water. When a permeant which causes soils to shrink is used, a flexible membrane can adjust itself to the new sample dimensions, thus preventing side flow. The main disadvantage of flexible-wall permeameters may well be their cost.

In general, flexible-wall permeameters are best suited to soil samples with irregular sidewalls (e.g., undisturbed samples and compacted samples that contain sand or gravel). The device, however, may be difficult to use with certain caustic chemicals due to incompatibility between the membrane and chemical. The flexible-wall permeameter is also ideal for soils that will be subjected to substantial overburden pressure, such as natural samples obtained at depth or compacted clay that will be overlain by significant thicknesses of soil and/or solid waste.

Other Factors Affecting Permeability

Effect of Compaction. Since cohesive soils are often used in compacted form, the structure and thus the engineering properties of compacted soils will depend greatly on the method or type of compaction, the compactive effort, and the compaction water content. Usually the water content of compacted soil is referenced to the Optimum Moisture Content (OMC) for the given type of compaction. Depending on procedures adopted, the compaction water content may be "dry of optimum" or "wet of optimum." Research on compacted clays has shown that when they are compacted dry of optimum, the structure of the soil is essentially independent of the type of compaction. The type of compaction has a significant effect on the soil structure and thus on the strength, compressibility, and permeability of soil when it is compacted wet of optimum.

Permeability is related more to the structure of the soil than to any other single variable. The structure itself is influenced greatly by shear strains associated with the method of compaction wet of optimum moisture content (52). Different compaction methods induce different amounts of shear strain. The shear strain and, therefore, the degree of dispersion of the soil particles, is related to the method of compaction. Thus, permeability of the soil will be influenced by the method of compaction. Seed and Chan (66) have shown that shearing strain and degree of dispersion for different compaction methods increases in the following order: static, vibration, and kneading. Therefore, samples prepared by kneading compaction have lower permeability than do samples prepared by static compaction. This is true for compaction wet of optimum, but for dry of optimum, the effect will be negligible because no method of compaction induces appreciable shear strain under such conditions.

A pronounced decrease in permeability will be produced by increasing the compactive effort at any given water content. Mitchell, Hooper, and Campanella (52) indicated that permeability decreases more than 100-fold (i.e., two orders of magnitude) without a change in density or moisture content, as a result of increasing the kneading compactive effort. The effect of the higher compactive effort is to produce a more dispersed structure and consequently a lower permeability.

Effect of Flow Direction. Many papers have indicated that an upward direction of flow for a permeability test is more effective in removing entrapped air (to be discussed) from the sample. On the contrary, the results of experiments conducted by Fireman (16) indicated no difference with respect to removal of entrapped air for downward and upward flows. The measured permeabilities were different only during the early stages of flow, and during later stages were in excellent agreement. The same result was obtained by Smith, Browning, and Pohlman (68) and Christiansen (14). Bodman (3) states that downward flow velocities are affected more than upward flow velocities by the nonlinear influences existing for lower values of i.

Effect of Time. It has often been observed that permeability changes with time (4, 10, 32, 59). Generally, it is found that a decrease in permeability will occur during the early stages on an experiment. In some instances, the initial decrease is followed by a gradual increase before a constant permeability is reached. The initial decrease in permeability is believed to be due to several factors, one of which is the increased dispersion and migration of particles. The wetting process weakens the bonds between colloidal particles and, with movement of water through the soil, such loosened particles are moved to more stable positions, decreasing the effective pore size and producing a more dispersed structure with a consequent decrease in permeability (16, 59). Further, removal of electrolytes can also cause this initial decrease in permeability, as a more dispersed structure and reduced permeability is produced (4, 16). Gupta and Swartzendruber (22) have suggested bacterial activity as one of the reasons for reduction in permeability.

During the second stage of the permeability-time relationship there is generally an increase in permeability. The reasons for this phenomenon appear related to the dissolving of entrapped air in the voids of an unsaturated soil, which produces a larger effective pore size and higher permeability (37), and thixotrophy, or a tendency toward a flocculated structure. If a transition to a more flocculant structure takes place, accompanied by thixotrophic hardening, the permeability should be expected to increase (52). An increase in permeability may well depend on whether one of the above processes takes place in a given soil (4).

Based on an experimental study, Bodman (4) concluded that the decrease in permeability with time is related primarily to the silt content of the soil. He found no relation to clay content, moisture, or other factors which have frequently been observed to affect permeability, although clay content is believed the most influential factor governing the permeability of soil.

Effect of Temperature. An increase in temperature generally causes an increase in soil permeability because of the decrease in viscosity of the permeant. Further, if a temperature differential exists in the sample, the movement of permeant caused by the thermal gradient will affect the measured permeability (81).

Effect of Entrapped Air. This problem has been studied by Christiansen (14) and others. The general effect of entrapped air is to decrease the permeability by plugging smaller voids and decreasing the effective soil porosity (i.e., area available for water movement). However, most of the entrapped air will eventually dissolve in the percolating water, causing an increase in permeability (second stage of k-t curve). Air dissolved in the water used for the actual permeability test causes no trouble in normal testing as long as it does not come out of solution. One procedure used to prevent any air from coming out of solution is to use water which has less than its capacity of air dissolved in it (i.e., "deaired" water).

Effect of Disturbance. Laliberte and Corey (34) studied the effect of sample disturbance on the permeability of soils. Because sampling produces a change in the macrostructure of the soil, the permeability is always lower for disturbed samples than for corresponding undisturbed samples. Disturbance by pulverization during compaction may produce very great changes in the properties of soil material from a particular site.

Laboratory Measurement of Soil Permeability

The permeability of soils may be measured in the laboratory using either a direct method or by the use of the results of other tests on the soil sample. The various procedures are outlined and described in the following sections.

Direct Methods

Direct methods are based on measurement of the rate of flow of water through the soil, generally assuming the validity of Darcy's equation. Direct methods in common use include the constant-head method and falling-head method.

Constant-Head Method. In this method, an undisturbed or disturbed sample of soil of given dimensions is subjected to a constant hydraulic potential and the average rate of flow during a given time interval is measured. This method is normally used for both plastic and nonplastic soils (38). Advantages of this test include the fact that the soil is not disturbed during the entire test, the effluent can be quickly and accurately measured, and the reservoir requires a minimum of attention (16). Disadvantages of this method are the necessity for maintaining a constant head, which is often troublesome, and the fact that a constant head is seldom encountered under natural conditions.

Falling-Head Method. In this method, a soil sample is placed under a hydraulic head which decreases in magnitude with time as water passes through the soil and the level of water in the reservoir goes down. Lambe and Whitman (38) believe this method should be limited to saturated soils of rather high permeability. Fireman (16) expresses an opposite opinion, and considers the method best suited for finer material of very low permeability. The advantages of this method are the ease of control of head and the insignificant effect of evaporation on the results. Disadvantages are the greater complexity required, the possible effect of refilling the reservoir periodically, and the effect of entrapped air on the calculated permeability. This method is generally less effective in dissolving entrapped air in the percolating water (16).

Possible Sources of Error. In an effort to reduce testing time, large hydraulic gradients may be imposed on samples, causing a washout of fine material to the boundary, with a possible reduction in permeability. The field hydraulic gradient may be on the order of 0.5 to 1.5, whereas in the laboratory it may be 5 or more. If Darcy's law is valid, such gradients will not alter the measured permeability. Swartzendruber (72) surveyed the then existing literature and found many experiments in which k (as defined by Darcy's law) increased as the gradient increased, with ratios of the maximum to minimum permeabilities typically between 1 and 5. Other studies, conducted by Mitchell and Younger (53) and Gairon and Swartzendruber (20), found decreasing values of permeability as the gradient was increased, apparently as a result of particle migration causing plugging.

There are several problems associated with superimposing excessive heads, including the fact that water tends to become saturated with gas at elevated pressures. If water is forced through the soil using compressed air, the water entering the sample may contain a higher gas concentration than that corresponding to gas saturation at a lower pressure, and thus gas bubbles may form as the pressure in the flowing water decreases. As water flows through the soil sample and the water pressure drops, there may be a tendency for air bubbles to evolve in the sample.

Indirect Methods

The permeability of soils may be determined indirectly using theoretically derived equations and average properties of the soil mass and soil particles. In this method, soil permeability may be determined by analysis of the size, shape, and arrangement of the soil particles, or by some related properties such as void ratio, effective porosity, pore size distribution, etc. (16). These methods do not provide an accurate quantitative value for soil permeability, but rather yield quantitative values which are normally used for preliminary evaluation and comparison of permeabilities of different soils. The most common relationships of this nature are discussed in the literature and include Hazen's equation (42), Slichter's equation (10, 42, 74), Terzaghi's equation (42, 74), Kozeny's equation (42, 43), the Kozeny-Carman equation (37, 38), Taylor's equation (37, 38), Rose's equation (42), London's equation (42), and the Childs-Marshall equation (43, 45). The results of a consolidation test can also be used for an indirect determination of soil permeability. This method is based on Terzaghi's theory of consolidation (75).

Laboratory Measurement of Threshold Gradient

The apparatus generally used for laboratory permeability testing for the existence of threshold gradients in clay soils are described in the literature (13, 40, 50). Basically, laboratory testing apparatus fall into two categories; oscillating permeameters and modified conventional triaxial cell apparatus. Oscillating permeameters may be either of the flow-through type or simple harmonic motion (SHM) type. Details regarding the equipment used to test for threshold gradients, including the modified triaxial apparatus, is presented in Chapter III which describes materials, sample preparation, equipment, and testing procedures used in this study.

Many other aspects of soil permeability addressed in numerous papers have no applicability to this study, and are not discussed herein. Tens of papers have been written pertaining to the characteristics of special devices used for permeability measurement, and a discussion of these and many other subjects can be found in the literature given in the references (83). The best overall laboratory device to determine the permeability of fine-grained soils appears to be the triaxial device, utilizing the proper application of back pressure. The use of rigid-wall permeameters and/or consolidation theory to determine the hydraulic conductivity of fine-grained soils is not recommended, since unconservative results are generally obtained.

Despite the uncertainties and complexities involved in permeability investigations, continued study appears justified because of the increasing importance of permeability in solving engineering-related problems involving earth materials of construction. A national desire to prevent the pollution of surface water and groundwater supplies has recently lead to the practice of retaining many kinds of waste behind dikes (dams) or in lined reservoirs, which can threaten the safety of people and property if these waste-disposal structures are not properly designed and constructed. It is clear that the question of the existence of general non-Darcy behavior and specifically threshold gradients for water flow in saturated clay systems is unanswered.

CHAPTER III

MATERIALS, SAMPLE PREPARATION, EQUIPMENT, AND TESTING PROCEDURE

This chapter provides information regarding the physical and engineering properties of the material used throughout the study as well as the method of preparation of soil samples. The apparatus generally used for laboratory permeability testing for the existence of threshold gradients in clay soils are described, and details of the specific equipment and testing procedure used in this study are also presented. Information concerning the properties of materials is limited to that which might have some effect on the results of the experiments. In describing testing procedures, details are given only for the triaxial, constant-head, permeability test, which was the technique used to evaluate threshold gradients in this study.

Materials

Some clay soils, such as montmorillonites, are unstable in water and decompose releasing salts into solution. Research has shown that such decomposition can result in unexpected osmotic effects. If flow studies are initiated and any osmotic effects present are not accounted for, apparent threshold gradients can be found. To minimize the potential for unexpected osmotic effects as a result of soil decomposition and the subsequent releasing of salts (electrolytes) into solution, leading to apparent threshold gradients, an Oklahoma soil with stable clay mineralogy properties was used in the laboratory flow studies. As clay soils used in construction of earth structures are normally used in their compacted state, disturbed (remolded) soil samples were used in this study. Disturbed soils are those which have been dug or excavated from a site, dried, pulverized, and passed through some given sieve size (usually No. 40). The materials are then stored for later use in preparing remolded samples.

The Union City Red Clay used in this study was selected from among available clay soils stored in the Soil Mechanics Laboratory at Oklahoma State University (OSU). This soil was used solely in its natural state and was originally obtained from the inclement weather materials stockpile at the Oklahoma Brick Corporation manufacturing plant located in southern Canadian County near Union City, Oklahoma. The material has a distinctive red color due to its high iron content, and is commonly called "Permian Red Clay." Based on the results of electron microscope and X-ray diffraction analysis techniques, the clay fraction of this soil consists almost entirely of the mineral illite (64), which is a 2:1 layer hydrous aluminum silicate with a fixed (nonexpanding) lattice.

A summary of the physical and engineering properties of the Union City Red Clay (disturbed material) is presented in Table II. The listed properties include those which may have an influence on soil permeability, and more specifically threshold gradients, or which may be useful or necessary for analysis of the results. Listed properties include natural moisture content, specific gravity, Atterberg limits, Unified Soil Classification, grain-size distribution, activity number, and compaction data (used in molding samples).

TABLE II

PHYSICAL AND ENGINEERING PROPERTIES
(Disturbed Material)

UNION CITY RED CLAY (ILLITE)	
Natural Moisture Content, w_{nat} (%)	9.7
Specific Gravity (G_s)	2.74
Atterberg Limit Data:	
Liquid Limit, LL (%)	32.8
Plastic Limit, PL (%)	17.6
Plasticity Index, PI (%)	15.2
Unified Soil Classification	"CL"
Grain Size Analyses:	
% Pass. No. 200 Sieve	86.4
% CLAY (<0.002 mm)	28.5
% SILT (0.002-0.074 mm)	57.9
% SAND (>0.074 mm)	13.6
Dia. at 60% (D_{60})	0.011 mm
Effect. Dia. ($D_e = D_{10}$)	0.0003 mm
Uniformity Coeff. (C_u)	32 (Well-Graded)
Activity Number	0.53 (Inactive)
Compaction Criteria:	
Standard Proctor Test[1] (C.E. = 12,400 ft-lb/ft^3)	w_{opt} = 16.4%/γ_d(max) = 113.3 pcf
Modified Proctor Test[2] (C.E. = 56,300 ft-lb/ft^3)	w_{opt} = 13.8%/γ_d(max) = 119.7 pcf
Harvard Miniature Test[3] (C.E. = 13,800 ft-lb/ft^3)	w_{opt} = 17.9%/γ_d(max) = 106.0 pcf

[1] 3 Layers/25 Blows per Layer; 5.5-lb Hammer; 12-in Drop Ht (-4 Mesh).
[2] 5 Layers/25 Blows per Layer; 10.0-lb Hammer; 18-in Drop Ht (-4 Mesh).
[3] 3 Layers/25 Blows per Layer; 0.81-lb Hammer; 6-in Drop Ht (-40 Mesh).

Based on the results of laboratory soils testing (Table II), the soil consists of a well-graded silty clay of low plasticity, classified "CL" by the Unified Soil Classification System. Grain-size analysis indicates that approximately 86 percent of the material passes the No. 200 sieve. The material has an activity number of 0.53, indicative of a relatively inactive (stable) clay.

Sample Preparation

In an investigation such as this, in which low hydraulic gradients are applied to a soil compacted to various known placement conditions, the greatest care must be taken in preparing test specimens to allow a relative comparison of results. To improve reliability of test results, replicate samples were prepared and tested for each placement condition (density and moisture content). It is essential that replicate samples be closely similar, if not identical, if comparisons are to be meaningful and have the desired degree of validity. In this regard, several specimens were discarded during and after preparation because of discernible differences in macroscopic structural features which might affect permeability. In several instances, tests already in progress were abandoned because of gnawing doubts concerning some feature of the specimen (or test conditions).

The structure of disturbed (remolded) soils in most laboratory experiments is controlled by the compaction process and depends on whether the soil is compacted on the "wet" or "dry" side of optimum. Compaction on the wet side provides a more dispersed structure (parallel orientation of particles) and lower permeability, while compaction on the dry side of optimum gives a more flocculated structure (random orientation of particles) and consequently a higher permeability. Minimum permeability of a compacted soil occurs at a water content greater than optimum.

Clay materials used in the construction of earth structures are usually compacted in relatively thin lifts in the field according to prescribed conditions of moisture content and density. Moisture-density curves are developed for each soil by compacting samples in the laboratory at various water contents and determining the unit weight of samples. The optimum moisture content and maximum density attainable in the field, for a given soil and compactive effort, is estimated from the curve. A typical specification might require compaction to at least 95 percent standard Proctor density at optimum water content. Alternatively, a range in compaction water content may be specified, such as 3 percent dry to 3 percent wet of optimum water content, assuming minimum density requirements can be satisfied.

Remolded specimens were prepared in the laboratory by compacting the soil under desired conditions of moisture content and compactive effort. To obtain a range in placement conditions, samples were compacted at optimum water content (w_{opt}), 3 percent dry of optimum (-3% w_{opt}), and 3 percent wet of optimum (+3% w_{opt}). Because the triaxial shear strength testing equipment in the OSU Soil Mechanics Laboratory easily accommodates Harvard miniature samples, the Harvard-size mold (1-5/16 in. inside diameter and 2.816 in. long) was used to compact all soil samples. Compaction criteria obtained from moisture-density relationships developed for the soil using the Harvard miniature, standard Proctor (ASTM D-698), and modified Proctor (ASTM D-1557) methods is given in Table II. An optimum water content (w_{opt}) of 17.9% was determined for the soil using the Harvard miniature method, which approximates the compactive effort used in the standard Proctor test (13,800 ft-lbs/ft^3 versus 12,400 ft-lbs/ft^3).

In preparing samples, care was taken to make lifts equal in thickness and lifts were scarified with a spatula to improve bonding. Two samples were required for testing at each water content (i.e., placement condition) and two additional samples were prepared as spares. After extruding each sample from the mold, the soil was immediately wrapped in Saran Wrap, waxed, labeled, and placed in the moist room until required for testing. Prior to testing, a flexible membrane was placed around each specimen, and samples were then mounted in separate triaxial cells and tested, using procedures described in a subsequent section of this chapter. When the tests were completed, the triaxial cells were disassembled and each specimen was cut into thirds and oven-dried, to determine the final water contents.

Equipment

The apparatus generally used for laboratory permeability testing for the existence of threshold gradients in clay soils are described below. Basically, laboratory testing apparatus fall into two categories; oscillating permeameters and modified conventional triaxial cell apparatus (13, 40, 50). As described in the literature, oscillating permeameters may be either of the flow-through type or simple harmonic motion (SHM) type. A modified triaxial permeability apparatus (to be described) was specifically used to test for threshold gradients in this study.

<u>Oscillating Permeameter Apparatus</u>

<u>Flow-Through Type</u>. The flow-through oscillating permeameter apparatus typically used for flow studies in saturated clays is shown schematically in Figure 6. In the apparatus shown in Figure 6, fritted glass disks F_1 and F_2 are used to confine the sample (horizontally-oriented) in a glass flow cell approximately 2.5 cm long. Stopcocks S_1 and S_2 and a constant-head reservoir, A, are used to regulate the applied head and flow direction (left or right) of the system. The head drop across the flow cell is monitored continuously using a Pace model KP15 differential pressure transducer, with a ±1 psi linear-response diaphragm, D. As the diaphragm flexes in response to a pressure difference across the cell, a linear millivolt-pressure output signal is generated which can be continuously monitored. By using the Pace carrier demodulator model CD10 and a 10-inch stripchart recorder calibrated in centimeters of water, a precision of ±0.5 mm of water can be obtained.

Measurements are made in a controlled temperature environment by applying a positive initial head, h_o, at P (Figure 6), which deflects the diaphragm to the right from the vertical (equilibrium) position. Stopcock S_1 is then closed and the recorder is switched "on" to monitor the pressure dissipation with time. After stopcock S_1 is closed, the mechanical force exerted by the flexed diaphragm (equal to the applied head) causes flow to occur from left to right until the pressure has dissipated and the diaphragm is again in the equilibrium position. An identical procedure can be carried out on the right side using stopcock S_2, to produce flow in the opposite direction (right to left). If there is no threshold gradient, pressure-time plots should fall to the zero pressure line, the diaphragm's equilibrium position. If there is a

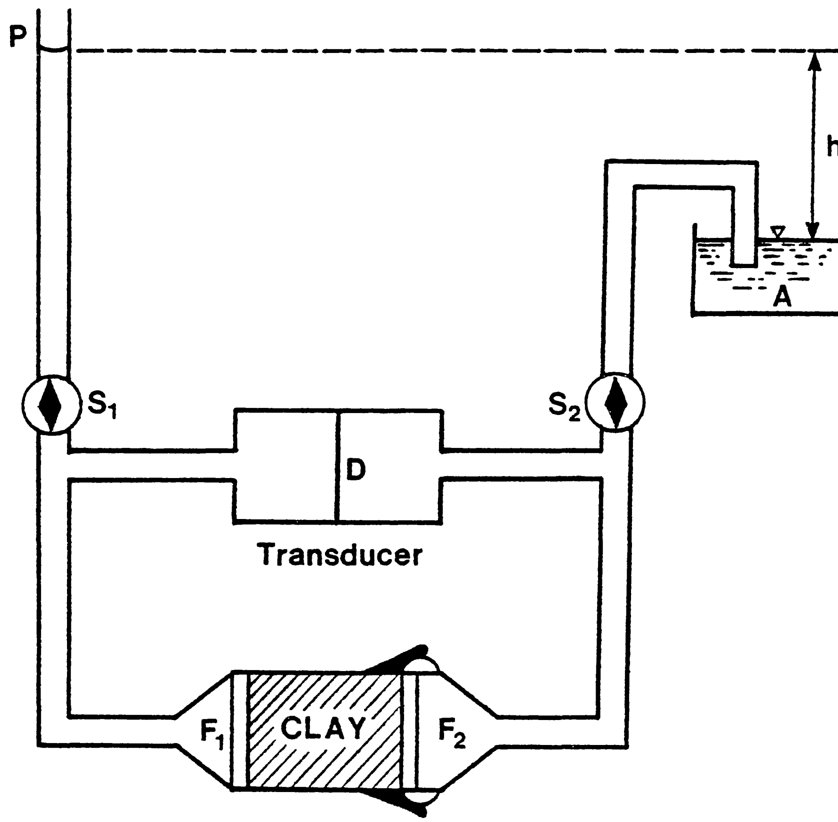


Figure 6. Schematic Diagram of Oscillating Permeameter Apparatus (flow-through type) used for flow studies in saturated clays (50)

threshold gradient, flow will stop when the threshold value is reached. At this point, diaphragm movement towards the equilibrium position will stop and a horizontal plateau will be observed in the time-pressure plot above the zero pressure line (i.e., all pressure dissipated). The magnitude of the plateaus should be equal from both flow directions.

The flow-through oscillating permeameter offers the advantage of measuring osmotic effects (pressures), if present. If flow studies are initiated and any osmotic effects present are not accounted for, apparent threshold gradients can be found. Osmotic effects can be tested for by placing equal heads on both sides of the diaphragm, closing both stopcocks, and recording the output. If there are any osmotic effects present, water will move in the direction of greatest electrolyte concentration, generating a pressure, and hence changing the recorder signal. The stable osmotic pressure generated is then taken as the new zero pressure line, with no effects on threshold gradient determinations. The percent head dissipation, h/h_o, with time can be calculated by adding or subtracting the previously determined osmotic pressure, depending on the horizontal flow direction. The new osmotic zero pressure line will probably change over extended times, because the clay sample is not a perfect semipermeable membrane. However, for the times typically involved in experiments of this nature, the osmotic zero pressure line should remain essentially constant. Semilog plots of relative head $(h - h_f/h_o - h_f)$ versus time (t) should be linear if k (the hydraulic conductivity) of the material is constant.

Simple Harmonic Motion (SHM) Type. A typical simple harmonic motion (SHM) oscillating permeameter apparatus is shown schematically in Figure 7. This apparatus appears particularly well suited to the

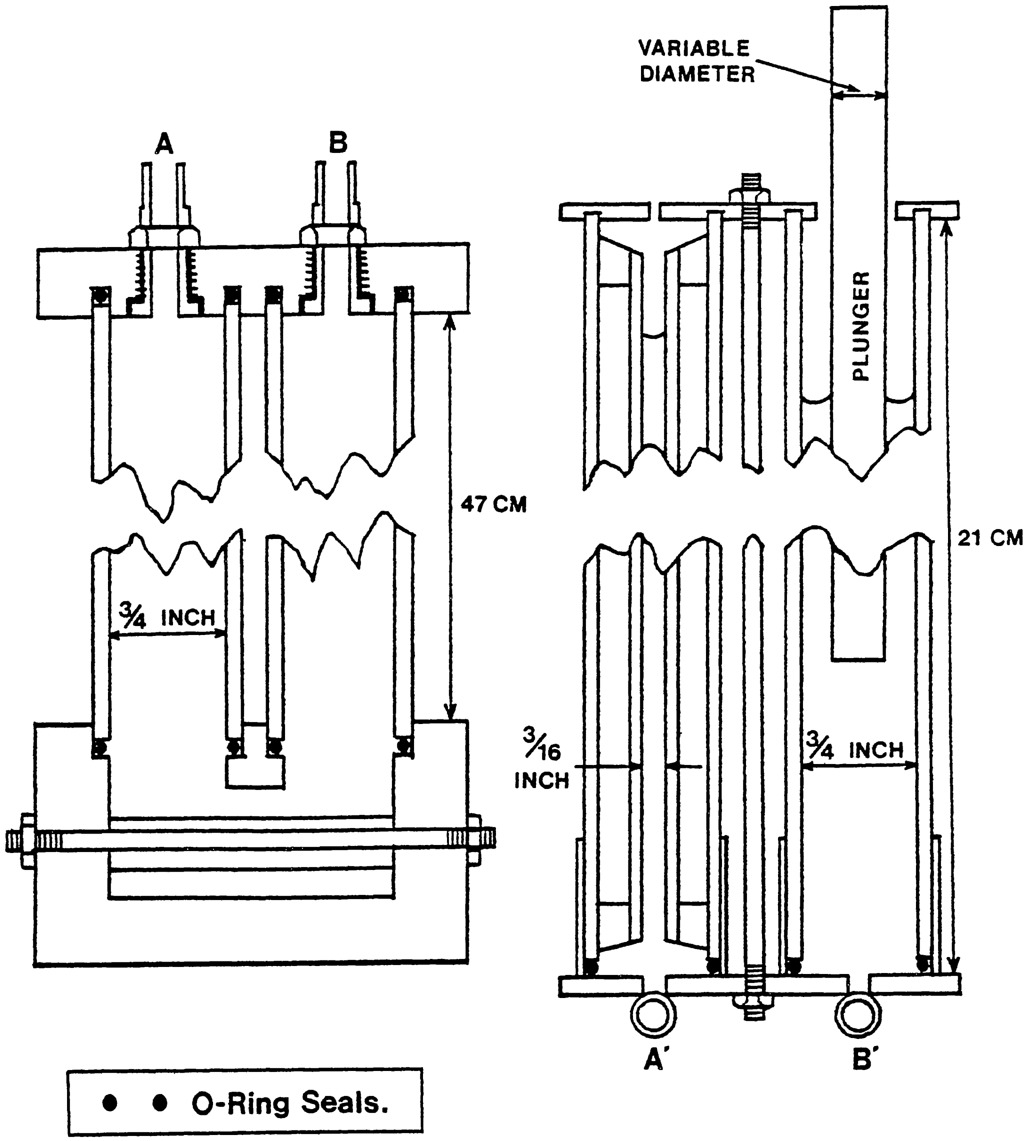


Figure 7. Schematic Diagram of Oscillating Permeameter Apparatus (SHM type) showing composite U-tube containing sample (left) and plunger which provides alternating potential difference (right) (13)

present purpose, since not only is the applied potential gradient under close control, but it passes through zero potential twice in each cycle of operation. The essential elements of the apparatus are shown in Figure 7. A U-tube is constructed in two parts (shown left and right) connected A to A' and B to B' by flexible plastic tubing. The longer left-hand section contains the soil sample to be tested while the other provides the open terminal tubes in which the liquid meniscuses are located. In the right-hand terminal tube, a vertical cylindrical plunger (fixed amplitude of approx 5 cm) is moved up and down with simple harmonic motion by a motor-driven mechanism through a variable speed gear assembly. The displacement of water so caused in the right-hand arm produces an alternating potential difference in the system, and the consequent alternating flow of water is indicated by the movement of the meniscus in the left-hand (unobstructed) arm. A series of plungers of decreasing diameters can be used to obtain the required range of decreasing amplitudes of variation of potential gradient in the sample. That part of the U-tube which contains the sample may be accommodated in a large vacuum flask for better maintenance of a constant temperature.

The displacement z and Z of the plunger and the meniscus in the unobstructed arm, respectively, may be measured by a cathetometer at regular and frequent intervals. In general, the smaller the plunger diameter, the smaller the resulting meniscus displacement. For the largest to smallest amplitude of potential variation, the meniscus should describe simple harmonic motion. This would not be true if there was an appreciable decrease in hydraulic conductivity with potential gradient. At the peak of the meniscus displacement the meniscus is stationary, so that the rate of flow is zero here and the potential gradient is also zero. Hence on each side of this stationary state the potential gradient is small, and if there is a range in which the conductivity (k) is zero, the stationary state must have an appreciable width (i.e., the wave form of Z would be flat-topped). Disadvantages of this system, as compared to the previously described system, include the fact that only a limited volume of water is forced through the sample in each cycle of operation, and the fact that insufficient time is provided for any time-dependent (transient) effects to establish themselves.

Modified Triaxial Permeability Apparatus

Geotechnical engineers most commonly use triaxial devices in shear strength testing to determine the Coulomb strength parameters (c and ϕ) of soils. However, due to increased activity in the waste-disposal area, triaxial devices are being more commonly used for permeability measurement of fine-grained soils. There are many variations for triaxial setups, and no great significance should be attached to the specific locations of valves and fittings. Most triaxial devices utilize back pressure to saturate soil samples in shear strength tests, but are not equipped for permeability tests under back pressure. The modifications necessary to do permeability tests with back-pressure saturation are relatively minor and will be described. In addition to the ability to control the vertical and horizontal stresses, many triaxial devices are equipped to measure the vertical and volumetric deformations of the soil. The essential elements of the triaxial cell are a soil specimen surrounded by a thin, flexible membrane enclosed in a fluid-pressurized chamber.

A typical modified triaxial permeability apparatus used to test for threshold gradients in clay-water systems is shown schematically in Figure 8. The apparatus basically consists of an ordinary triaxial cell connected to a system for applying hydraulic gradients and for measuring the resulting flow rates through the sample. The use of the triaxial cell permits the application of any principal stress system to the specimen. A back pressure is used to promote complete saturation of samples prior to and during permeability tests. By application of the proper back pressure and vertical load, the soil specimen can be stressed to "in-situ" or any other desired field condition. Drain holes at both ends of the specimen allow the performance of permeability tests. The flexible membrane surrounding the specimen is pressed tightly against the soil by the pressure in the chamber fluid, thus preventing flow along the sides of the specimen. Separate pressure controls are used to regulate the cell pressure and back pressure acting on the two ends of the soil specimen.

For the apparatus shown in Figure 8, the hydraulic gradient (i) is applied using reservoirs (pots) connected to the specimen ends. The reservoir at the bottom end is stationary while the other reservoir can be adjusted up or down. The difference between water levels in the reservoirs, after subtracting the hydraulic resistance in the system, is equal to the total head difference across the specimen length (50± mm). For the system, the applicable range of head difference is from 0 to 1.0 m, corresponding to an i of 0 to 20. For an accuracy of measurement of head difference of about ±1.0 mm, the precision for i is about ±0.02. For i less than 20, both pots are pressurized by the same back pressure; for i greater than 20, a higher back pressure is applied to the

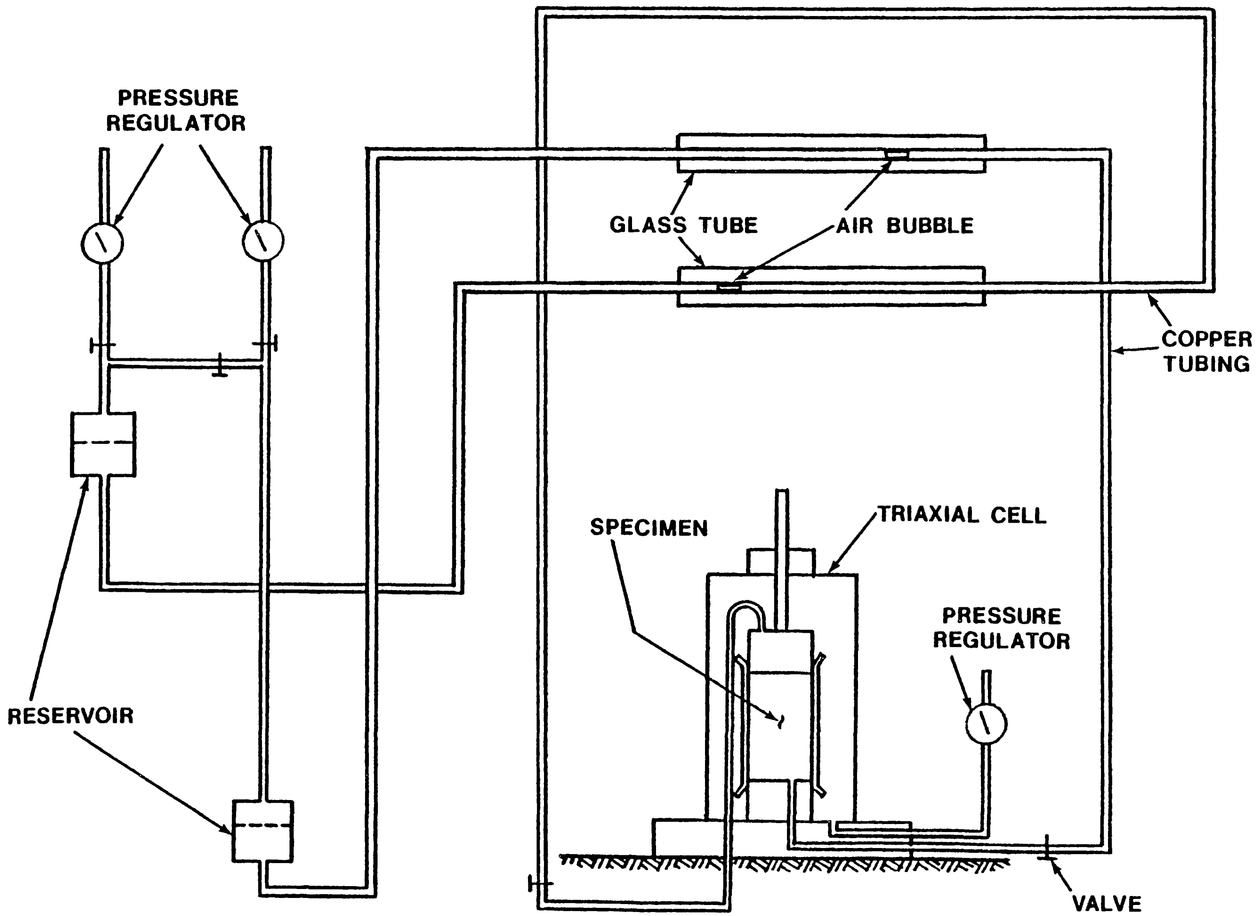


Figure 8. Schematic Diagram of Modified Triaxial Permeability Apparatus used to test for threshold gradient in clay-water systems (40)

reservoir connected to the top end of the specimen. If large reservoirs are used, the test is essentially conducted under a constant head because the small quantity of water transported between the reservoirs does not significantly alter the water levels during the test.

Two horizontal, calibrated, glass tubes are installed between the reservoirs and the specimen. An elongated air bubble is trapped in each tube and the flow rate (volume) "into" and "out" of the sample can be determined by reading the movement of the bubbles regularly. Any discrepancy between the two flow rates is either due to system leakage or to the process of swelling of the specimen. Should swelling occur during testing, the volume flow "into" the sample will be slightly larger than the volume flow "out." For these cases, the actual flow may be taken as the average of the two flows. As compared to the flow-through oscillating permeameter, disadvantages of this system include the inability to easily quantify osmotic effects (pressures), should such exist, and the fact that flow direction in the system is fixed.

Apparatus Used In Research

A modified triaxial permeability apparatus, consisting of two ordinary triaxial cells connected to a system for applying hydraulic gradients and for measuring the resulting flow rates through the samples, was used in laboratory flow studies. The triaxial setup used in this study is located in the OSU Soil Mechanics Laboratory and was originally designed and built by Dr. T. A. Haliburton, former professor of soil mechanics. The triaxial setup allows the concurrent testing of a maximum of three samples. Air pressure (150 psi) to each setup passes through separate pressure regulators mounted on control panels connected to each cell. Separate pressure controls on each panel are used to maintain the cell pressure and back pressure acting on the two ends of the soil specimen.

The modified triaxial apparatus used in this study generally consists of three parts: (1) a pressure chamber (triaxial cell) in which the specimen is enclosed with provisions for applying confining pressure, back pressure, and axial loads to the specimen, (2) a system for saturating the specimen, applying hydraulic gradients, and measuring flow rates through the sample (pressure saturation device), and (3) a pressure source for supplying fluid under pressure to the sample and pressure chamber. Figure 9 shows photographs of the modified triaxial apparatus used in this study. The triaxial cell (including peripheral equipment) and pressure saturation device used in experiments are shown in the photographs of Figure 10. Each of these devices is described in more detail.

A two-phase testing procedure was utilized in conduct of laboratory flow studies, consisting of sample saturation under back pressure followed by soil permeability testing. To simulate the predominant condition existing in the field, all laboratory permeability tests were performed with samples oriented vertically and flow direction downward. Hydraulic gradients were imposed on samples using air pressure applied through a system of burettes, which also allowed the measurement of flow "into" and "out" of specimens. To avoid the saturation of samples with gas at elevated pressures, distilled, deaired water was used in laboratory permeability testing. As the applied pressures substantially exceeded the hydrostatic head in burettes, the test was essentially conducted under constant-head conditions.

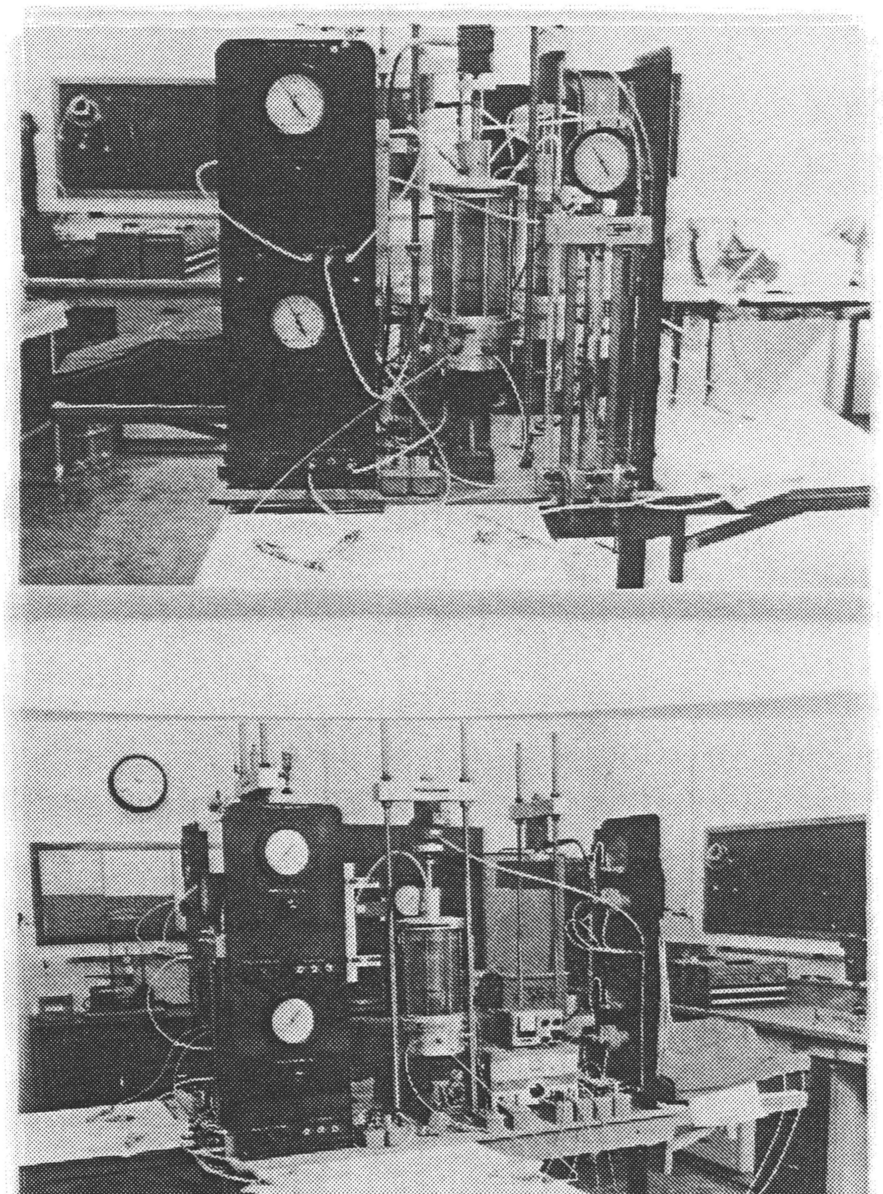


Figure 9. Photographs of Front (Top View) and Side (Bottom View) of Modified Triaxial Apparatus Used to Test for Threshold Gradient

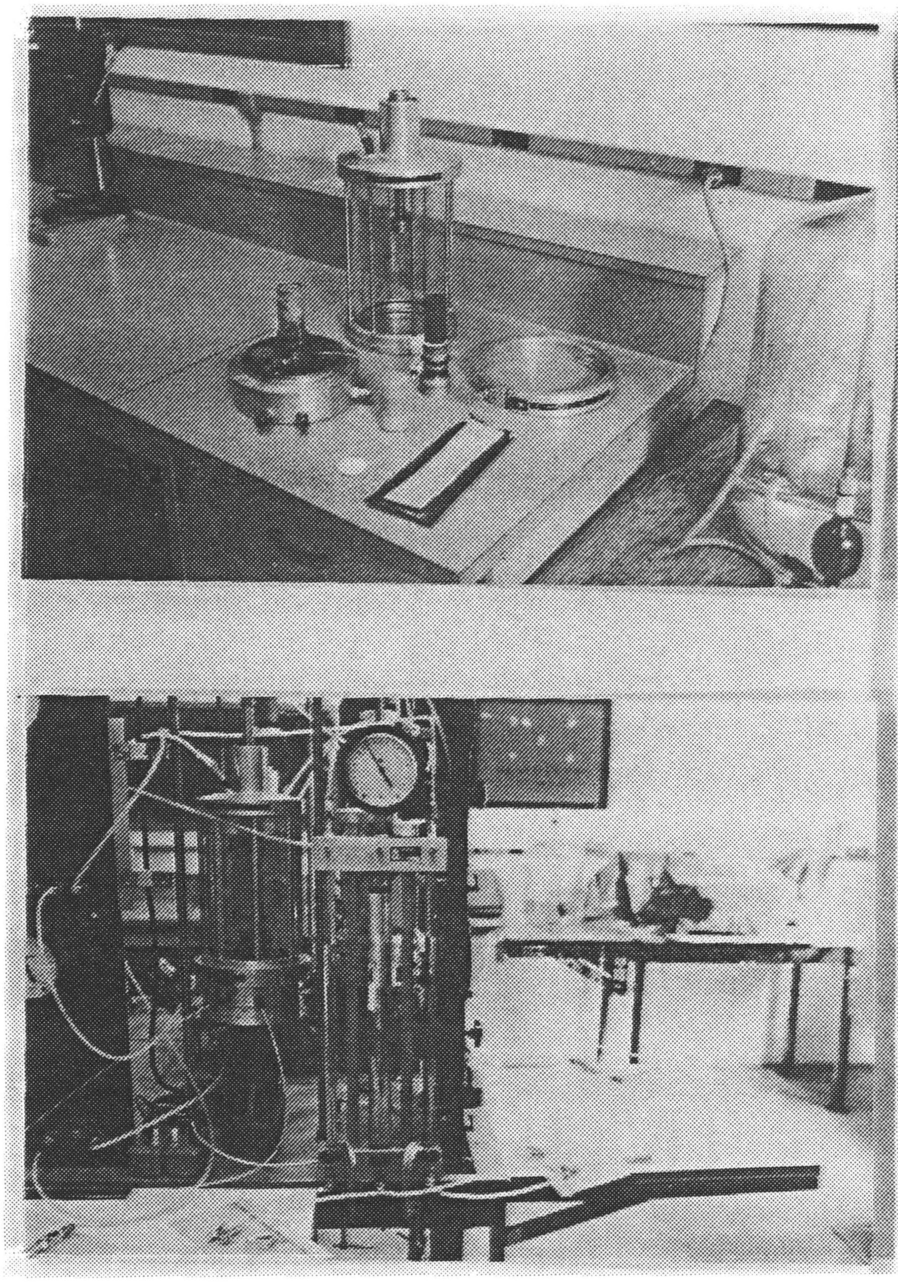


Figure 10. Photographs of Triaxial Cell Apparatus (Top View) and Karol-Warner Model PS-1 Pressure Saturation Device (Bottom View) Used for Soil Permeability Testing

<u>Permeameter Cell</u>. The essential elements of the triaxial cell are a soil specimen surrounded by a thin, flexible membrane enclosed in fluid-pressurized chamber. A typical triaxial cell setup for soil permeability testing is shown schematically in Figure 11. The sample to be tested is placed between two pedestals containing porous stones which have the same diameter as the sample. The sample is covered by an impervious (rubber) membrane and sealed to the pedestals by means of O-rings, such that the sample is isolated from exterior influence except through porous stones in the pedestals. The base of the unit contains inlets which allow control of sample drainage, through the stones at the top and bottom of the sample. The sample is further isolated in a cylindrical lucite pressure chamber that seals to the base of the unit, beneath the lower pedestal. A loading piston is inserted through the top of the cell (through pressure seals, to maintain an airtight cell) onto the top pedestal, so that a uniaxial load (principal stress) can be applied to the sample. The cell chamber is filled with a mixture of water and antifreeze (to prevent corrosion), and a confining pressure is applied to all sides of the sample. The sample can be back-pressure saturated by forcing water into the sample under air pressure. Although shown schematically in Figure 11, the loading piston was not used in testing other than for vertical confinement of the sample, as all samples were loaded isotropically in the laboratory.

<u>Pressure Saturation Device</u>. To determine the maximum or fully saturated permeability of fine-grained soils it is necessary to apply a back pressure to the specimen while maintaining the same effective stress on the soil specimen. This technique is widely used in geotechnical engineering practice and several devices are commercially

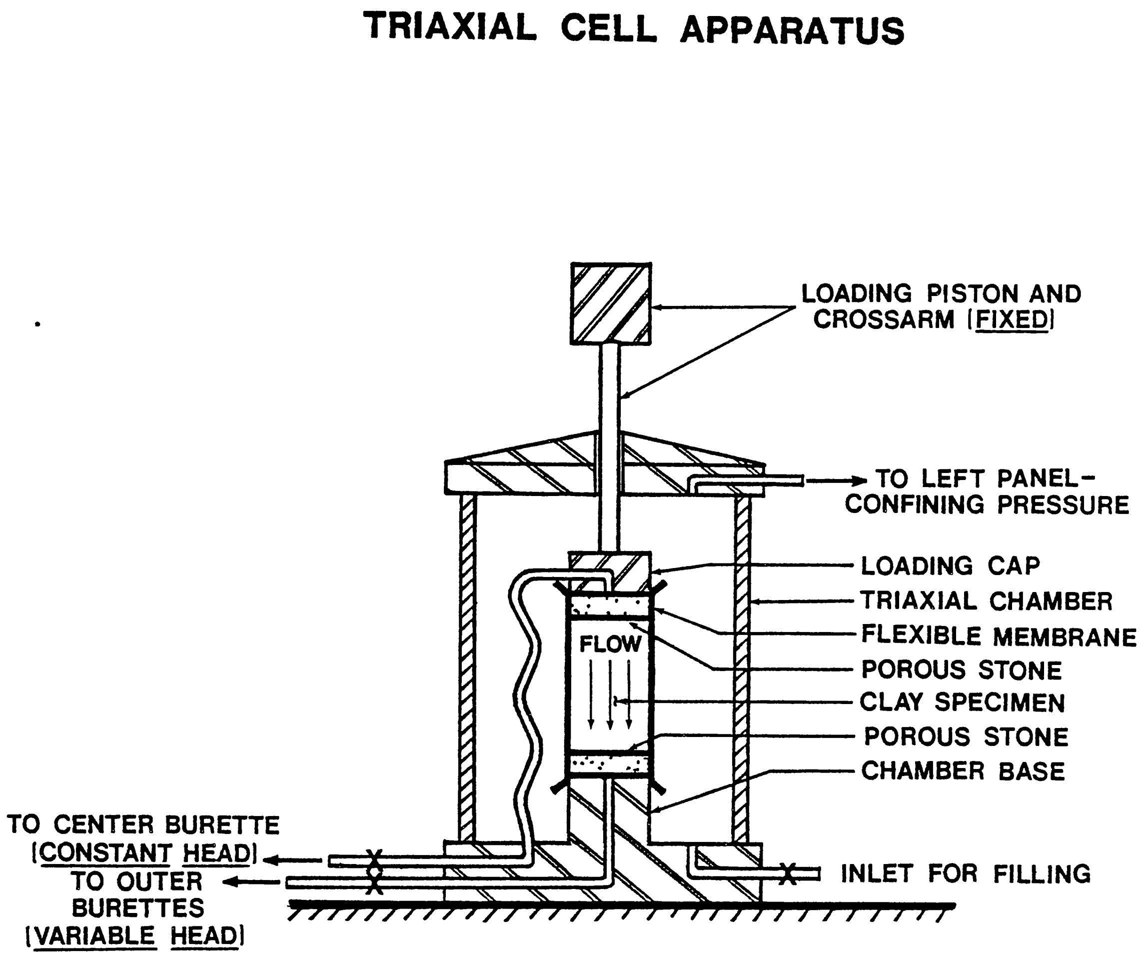


Figure 11. Schematic Diagram of Typical Trıaxial Cell Apparatus Used for Soil Permeability Testing

available which serve this purpose well. A Karol-Warner model PS-1 pressure saturation device, shown schematically in Figure 12, was used throughout this study. Pressure capacity of this system is 200 psi. The back pressure applied to the sample dissolves any entrapped air and promotes complete saturation of samples. As previously mentioned, hydraulic gradients were imposed on samples using air pressure applied through burettes, which also allowed for the measurement of flow "into" and "out" of samples. To avoid the saturation of samples with gas at elevated pressures, distilled, deaired water was used in laboratory permeability testing.

The pressure saturation device consists of three polished lucite burettes mounted top and bottom in aluminum block manifolds, individually piped and valved so that air pressure may be applied to the upper ends and liquid may move "into" and "out" of the lower ends. Scales are provided for accurate measurement of liquid levels in burettes. The two smaller outer burettes (1/2 in. inside diameter) were used to apply variable heads to soil samples during permeability testing and were connected to the outflow (bottom) end of samples. The larger center burette (1-1/4 in. inside diameter) was used to maintain a constant head on soil samples during permeability testing and was connected to the inflow (top) end of samples. With this configuration, replicate (two) samples could be tested concurrently using the three burettes.

Prior to use, it was necessary to calibrate the burettes by reading levels as known amounts of liquid were added. Scales were calibrated to 1/64 inch increments, which for the outer burettes corresponded to a volume of 0.05 ml (cm^3). Inlets for filling are provided on top of each of the outer burettes, and outflow from each burette is regulated

KAROL–WARNER MODEL PS–1
PRESSURE SATURATION DEVICE

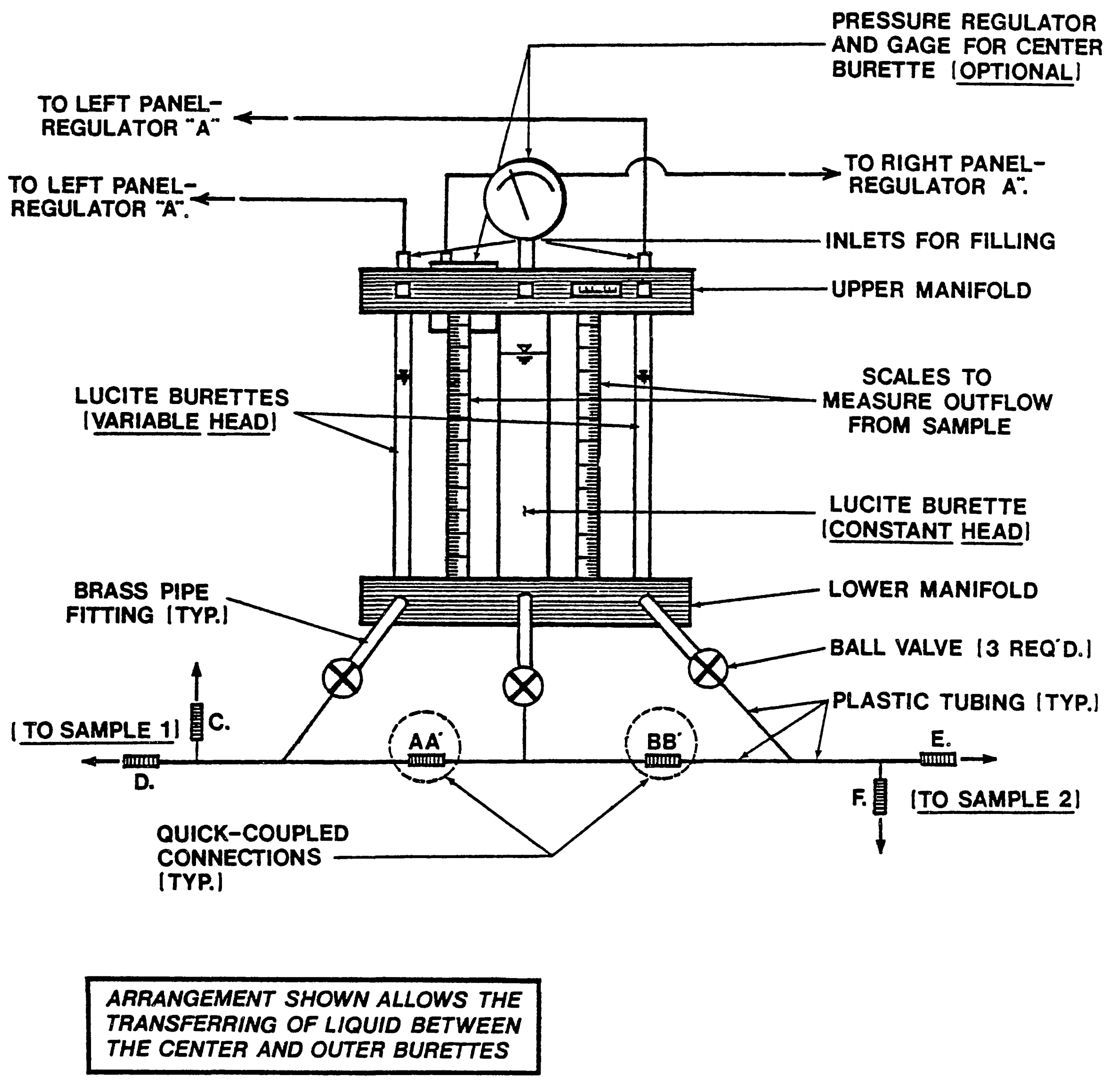


Figure 12. Detail of Karol-Warner Model PS-1 Pressure Saturation Device Used for Soil Permeability Testing (Pressure Capacity of System - 200 psi)

through high-pressure valves (ball-type) connected to the lower manifold through brass pipe fittings. Plastic tubing (1/8 in. inside diameter) terminated with quick-connect couplings completes the system, and with the arrangement shown in Figure 12, allows the transferring of liquid between the center and outer burettes with valves positioned appropriately. Transparent tubing was used for all connections, to allow the monitoring of entrapped air bubbles in the lines.

As previously mentioned, a two-phase testing procedure was followed in conduct of laboratory flow studies, consisting of sample saturation under back pressure followed by soil permeability testing. A constant pressure was applied to all burettes during back-pressure saturation, although the center burette was not used during this phase of testing. During back-pressure saturation, all valves were kept in the open position and couplings AA' and BB' (Figure 12) were disconnected. The back pressure applied through the left burette enters the top and bottom of Sample No. 1 through quick-connect couplings at C and D. In like manner, the back pressure applied to the right burette enters the top and bottom of Sample No. 2 through quick-connect couplings E and F. In permeability testing, a constant head was maintained on the top of samples (connections at A' and B) using the center burette, while a variable (decreasing) head was applied to the bottom end of samples (connections at D and E) using the outer burettes. Readings of outflow from each sample were made in the outer burettes only.

Arrangement for Testing. A schematic diagram of the typical arrangement of equipment for back-pressure saturation of samples is shown in Figure 13. A constant back pressure was applied to each sample

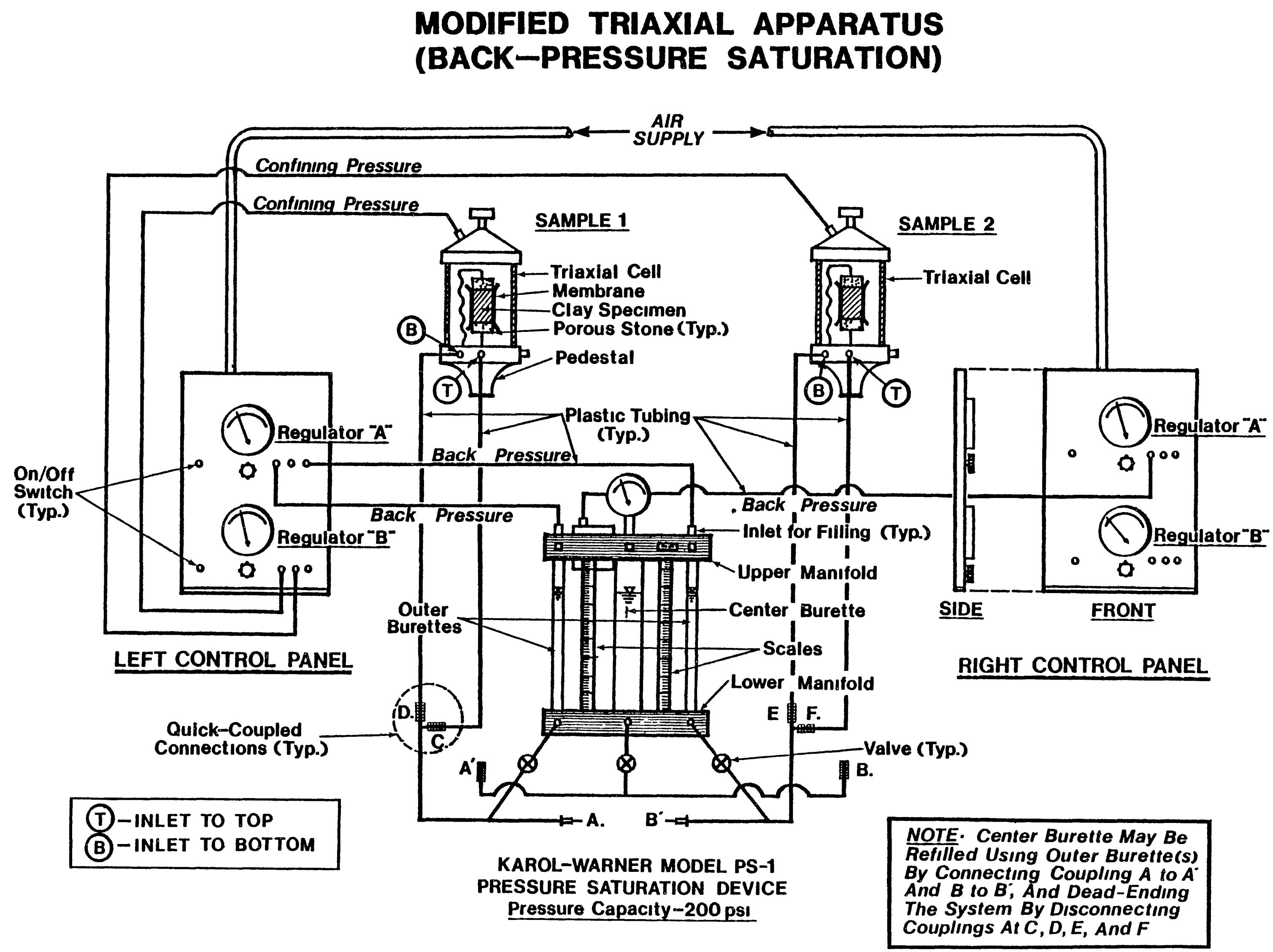


Figure 13. Schematic Diagram of Modified Triaxial Apparatus Used to Test for Threshold Gradient (Typical Setup for Back-Pressure Saturation)

through the outer burettes using Regulator A on the Left Control Panel. The center burette was also pressurized using Regulator A on the Right Control Panel. A confining pressure in excess of the back pressure was applied to each triaxial cell using Regulator B on the Left Control Panel. During the back-pressure saturation phase, measurement of flow into Sample No. 1 and Sample No. 2 was made by measuring the decline in water levels in the left and right burettes, respectively.

A schematic diagram of the typical arrangement of equipment for permeability testing is shown in Figure 14. The modifications necessary to the system to allow the application of hydraulic gradients across samples include disconnecting coupling C to form coupling C'A' (Sample No. 1), and disconnecting coupling F to form coupling F'B (Sample No. 2). In this manner, a constant head was maintained on the inflow (top) end of samples through the center burette, while a variable (decreasing) head was maintained on the outflow (bottom) end of samples through the outer burettes. Sample flow from top to bottom required, by necessity, that the pressure in the outer burettes be less than that in the center burette.

In constructing the previously described apparatus, considerable effort was made to use like fittings and equal lengths of plastic tubing on either side of the system (left or right), in order that the head losses would be similar. All valves should be maintained in the open position throughout the back-pressure saturation and permeability testing phases of experiments.

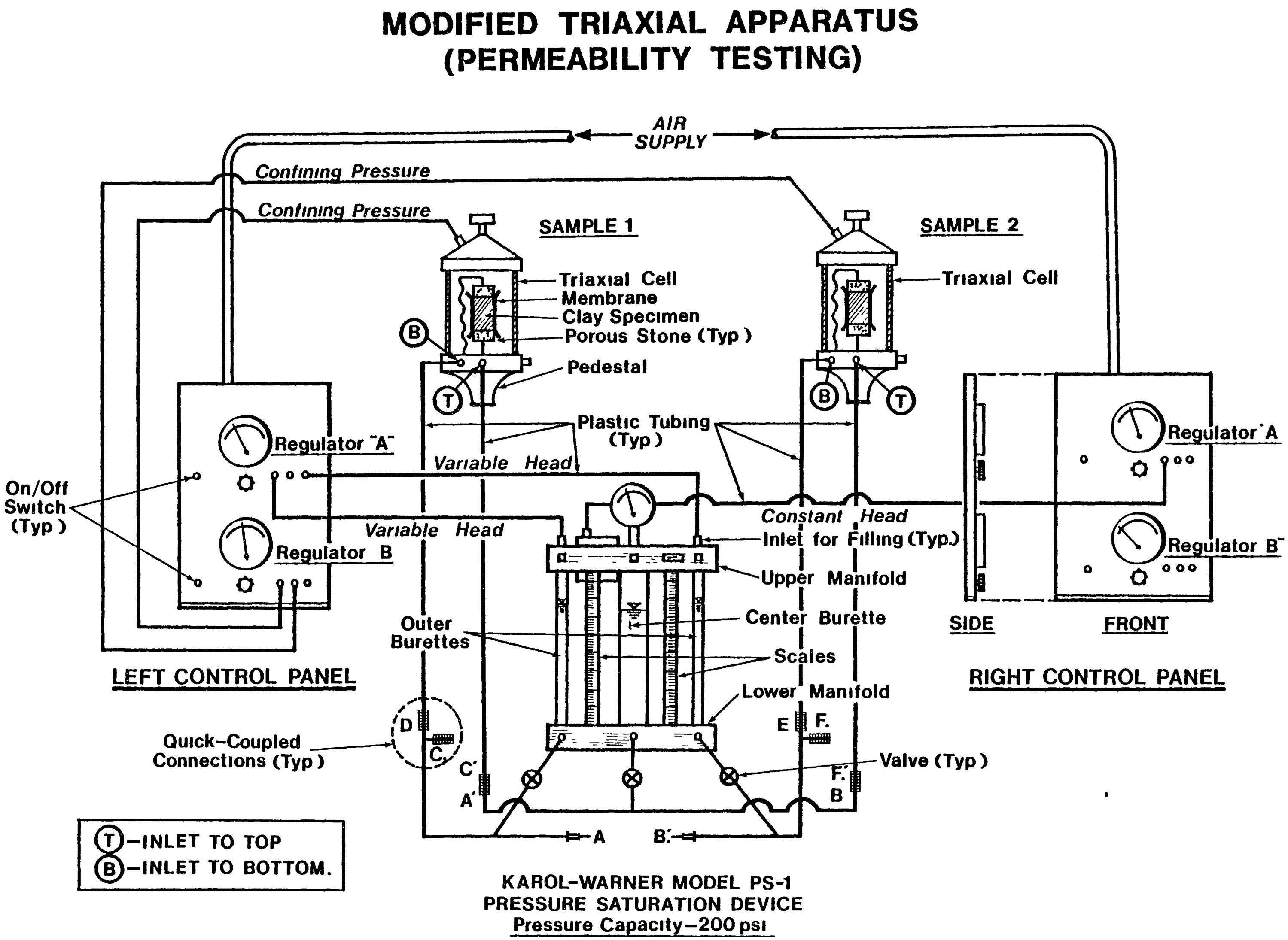


Figure 14. Schematic Diagram of Modified Triaxial Apparatus Used to Test for Threshold Gradient (Typical Setup for Permeability Testing)

Testing Procedure

The proposed research primarily involves evaluation of the existence of general non-Darcy behavior, and specifically threshold gradients, for water flow in saturated, natural, compacted, cohesive soils typically used for construction of earth structures. The existence of an initial or threshold gradient was tested for through conduct of flow studies in clay samples compacted to various known placement conditions prior to testing, using small hydraulic gradients within the practical range of engineering interest.

In addition to accomplishing the previously described research objectives, this paper also serves to document the use of a constant-head triaxial cell test apparatus for determining the hydraulic conductivity characteristics of a compacted clay soil. Factors related to the equipment and testing procedures used in this study which may have influenced the value of coefficient of permeability as determined in the laboratory using the triaxial, constant-head, permeability test will be discussed. Hydraulic gradients generally outside the practical range of engineering interest were applied to prepared sample, to investigate the influence of magnitude of hydraulic gradient on the value of permeability measured using the triaxial device.

A two-phase testing procedure was followed in laboratory flow studies, consisting of saturation of samples under equal back pressure followed by soil permeability testing under applied hydraulic gradients (pressure heads). In the following sections, the applicable range in hydraulic gradients, assembly of apparatus, test procedures followed in back-pressure saturation and permeability testing of soil samples, distribution of stresses on triaxial samples, and other testing procedures are discussed.

Applicable Range in Hydraulic Gradients

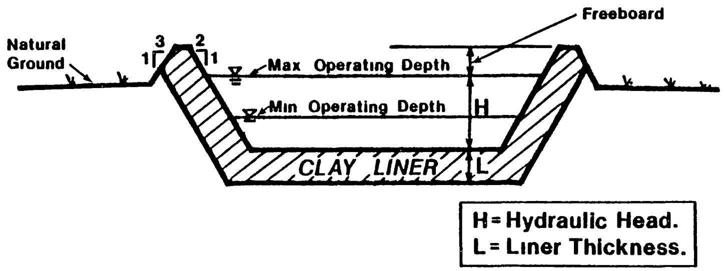
In this paper, a laboratory test procedure is described for evaluating the saturated hydraulic conductivity characteristics of a fine-grained soil using a constant-head triaxial cell test apparatus. Specific emphasis has been placed on evaluation of soil permeability to evaluate the performance of waste-disposal facilities. The constant-head triaxial cell test appears particularly well suited for the hydrologic assessment of the long-term performance of a new (or existing) waste-disposal facility, in that the test provides a proper means for simulating anticipated field conditions. Further, seepage quantity can be estimated under various placement conditions. Permeability testing is directly applicable to several areas of waste-disposal facility design, including selection of suitable soil liner material, determination of required liner thickness, and determination of acceptable impoundment fluid levels.

Regulations governing the disposal of waste materials often require 1 ft - 3 ft compacted clay liners with a saturated hydraulic conductivity on the order of 10^{-7} cm/sec or less for lagoons and disposal pits associated with such facilities. Other design considerations include, for waste-disposal lagoons, establishing a minimum and maximum operating depth for the lagoon. A minimum depth is desirable to avoid the drying of clay liner material and the establishment of vegetative growth, while the maximum depth should consider a minimum 3 ft - 5 ft dike freeboard. The depth of lagoons is generally limited by construction considerations, including size requirement (areal extent) with required dike side slopes. In this regard, multi-lagoon systems are often utilized for handling large volumes of waste.

The range in field hydraulic gradients for liquid depths up to 45 ft and liner thicknesses of 1 ft - 3 ft is shown, for a typical waste-disposal lagoon, in Figure 15. The maximum lagoon depths for each liner thickness shown in Figure 15 are recommendations of the author, and are based solely on his experience in working in the areas of solid and hazardous waste disposal. Considering each liner thickness and maximum recommended lagoon depth, the practical range in field hydraulic gradients lies to the right of the bold diagonal line in the table, and includes gradients of 15 and less. A hydraulic gradient of 1.0 or less would generally exist on lagoon liners if a minimum 1 ft depth were maintained in lagoons, for reasons previously stated. The applicable range in hydraulic gradients for a lagoon liner, based on conditions existing in the field, therefore appears generally restricted to gradients of 15 or less.

One disadvantage of testing at extremely low gradients with triaxial (air-pressured) apparatus includes the fact that most are not equipped to accurately regulate the small pressures required to test samples at such gradients. For example, in order to apply a gradient of 1.0 on a Harvard-size sample a pressure difference of 0.1 psi would be required across the sample. An accuracy in the reading of air pressure to 0.5 psi is obtainable with the current triaxial setup in the Soil Mechanics Laboratory, which allows the testing of samples at a minimum hydraulic gradient of 5. The main disadvantage in applying hydraulic gradients using strictly hydrostatic pressures (nonpressurized system)

HYDRAULIC GRADIENTS IN THE FIELD



CROSS-SECTION — TYPICAL LAGOON

- **REGULATIONS TYPICALLY REQUIRE:**
  - 1 ft – 3 ft Compacted Clay Liner.
  - $K = 1 \times 10^{-7}$ cm/sec or less
- **DESIGN CONSIDERATIONS:**
  1. Minimum Operating Depth – Avoid Drying of Liner and Establishment of Vegetation
  2. Maximum Operating Depth – Should Consider a Min 3 ft – 5 ft Dike Freeboard
  3. Maximum Depth – Limited by Construction Considerations, Including Size Requirement/Areal Extent with Max Dike Side Slopes Multi-Lagoon Systems are Often Used
  4. Field Gradients – Generally Less Than 15

HYDRAULIC GRADIENTS FOR VARIOUS LAGOON DEPTHS AND LINER THICKNESSES
(I = H/L)

L \ H	45 FT.	40 FT	35 FT.	30 FT.	25 FT	20 FT	15 FT	10 FT	5 FT	1 FT.	RECOMMENDED MAX DEPTH[1]
1 FT	45	40	35	30	25	20	15	10	5 0	1 0	≤ 15 FT.
2 FT	27 5	20	17.5	15	12 5	10	7 5	5 0	2 5	0 5	≤ 30 FT.
3 FT.	15	13 3	11 7	10	8 3	6 7	5 0	3 3	1 7	0 3	≤ 45 FT.

[1] Including Freeboard

Figure 15. Hydraulic Gradients in the Field for Various Lagoon Depths and Liner Thicknesses

is associated with the length of standpipes required to apply larger hydraulic gradients, unless relatively thin samples are used. For example, in order to apply a gradient of 15 on a Harvard-size sample, a hydrostatic head of approximately 42 inches would be required. Further, even with this apparatus and relatively thin samples, testing times required to obtain measurable flow under low gradients may still be impractical for all but research applications.

Air pressure was used in the experiments described herein, with the intent that if indications of non-Darcy flow were present during early stages of the test (hydraulic gradients greater than 5), further testing would be conducted at lower gradients using more sensitive pressure gages or nonpressurized flow in a standpipe. The applicable range in hydraulic gradients, based on considerations of conditions existing in the field and instrumentation of available laboratory equipment, was therefore initially restricted in experiments to gradients between 5 and 15. Permeability testing at these gradients would require a pressure difference across each sample of 0.5 psi and 1.5 psi, respectively.

Assembly of Apparatus

Prior to testing, the hydraulic system of the triaxial apparatus was pressure tested to insure no leaks were present in the system which might affect results. This was accomplished by partially filling (with water) the three burettes mounted in the pressure saturation device and applying a back pressure to the top of each burette. The pressure applied to burettes (and triaxial cells) was controlled in experiments through regulators mounted on separate control panels connected to each triaxial cell. A pressure of 100 psi was maintained on the system for a minimum period of 24 hours prior to testing. Water-sensitive blotters were placed under fittings, couplings, and valves to insure that any minute leakage was detected. After pressure testing the apparatus for a period of approximately 24 hours, the pressure was removed from the system and the burettes were refilled with distilled, deaired water through inlets provided in the top manifold of the pressure saturation device. The valves regulating flow out of each burette were left open while filling burettes, to facilitate the removal of any entrapped air from drainage lines.

Two identically prepared samples were obtained from the moist room, unwrapped, and placed in each triaxial cell. Each cell was disassembled and double O-rings placed on the top and bottom pedestals. The top pedestal was then inverted and one end of the flexible membrane used to confine the sample pulled down onto the pedestal and secured with a double O-ring seal. The membrane was then rolled down onto the pedestal, and a porous stone inserted into the recessed end of the pedestal. Wetted filter paper was placed on either side of the porous stone, as well as the porous stone in the bottom pedestal. Prior to placing the test specimen on the triaxial cell pedestals, the quick-connect fittings on the base of the triaxial cell leading to the top and bottom porous stones should be connected to a distilled water supply and distilled water flushed through the lines and out the top and bottom porous stones, under gravity head, to remove air and facilitate future saturation.

With the top pedestal in the inverted position, the membrane was rolled upward onto the sample a distance of approximately 2 inches. The top pedestal with sample attached was then inverted again and placed on the bottom pedestal, making sure to center the sample on the porous stone and filter paper. The membrane was then rolled onto the bottom pedestal and secured with a double O-ring seal. The top portion of the triaxial cell was placed on its base, the loading piston fixed into the top pedestal, and the triaxial cell sealed. Prior to cell assembly, it is suggested that (using a compressed air source) various triaxial cell fittings be checked to insure that they are tight and leakproof.

After sealing, the top chamber quick-connect fitting was vented to the atmosphere and the lower chamber quick-connect fitting in the base of the triaxial cell used to fill each cell with a mixture of antifreeze and water. First, an approximate 1 inch depth of antifreeze was allowed to flow, by gravity head, from the antifreeze container into the triaxial cell. After the antifreeze had been placed in each cell, the connection to the antifreeze container was removed and, using the same outlet, the cell filled with distilled water until the antifreeze-water mixture reached a level above the top triaxial cell pedestal. This mixture will provide corrosion resistance to the aluminum triaxial cell components and also allows observation of any leakage through the membrane during the test. After filling, each triaxial cell was placed on the round loading platen of the triaxial unit load frame. For each of the triaxial units, the load frame crosshead was lowered until the load cell platen was in contact with the ball bearing on top of the triaxial cell piston.

Back-Pressure Saturation

A schematic diagram of the typical equipment arrangement for back-pressure saturation of samples is shown in Figure 13. For each control panel in turn, the appropriate plastic tubing was connected from the regulator outlets to the top of burettes and the top inlet of the triaxial cell. In like manner, the appropriate connections were made to each triaxial cell, as shown in Figure 13, from the pressure saturation device. The air supply to the triaxial apparatus was turned "on," making sure that the regulator controls were turned counterclockwise such that a pressure of zero was indicated on the regulator pressure gages. A constant back pressure was applied to each sample through the outer burettes connected to Regulator A on the Left Control Panel, while the center burette was pressurized using Regulator A on the Right Control Panel. A confining pressure in excess of the back pressure was applied to each triaxial cell chamber using Regulator B on the Left Control Panel. Regulators were turned clockwise to apply pressure to each sample and triaxial chamber.

A back pressure of 75 psi (5.4 tsf) was applied to both ends of samples during the back-pressure saturation phase. A confining pressure of 80 psi (5.8 tsf) was used during both the back-pressure saturation and permeability test phases. Pressures applied to each sample were increased concurrently in 5 psi increments to their maximum values, always maintaining a minimum 5 psi difference in confining pressure over back pressure. The rate and volume of water entering each sample were measured by reading declines in water levels in the outer burettes until the system had stabilized. A period of 3.5 days to 6.5 days was generally required for water levels to stabilize in burettes, indicating essential complete saturation of samples. Since burettes were filled prior to application of back pressure, there was no need to refill burettes during (or after) this phase of the test. After burette water level readings had stabilized, pressures were maintained on samples for an additional period of approximately 36 hours. If no additional inflow was indicated, samples were assumed fully saturated and ready for permeability testing.

Permeability Testing

A schematic diagram of the typical equipment arrangement for permeability testing of samples is shown in Figure 14. While maintaining a back pressure of 75 psi on each burette and a confining pressure of 80 psi on each triaxial cell chamber, coupling C was disconnected to form coupling C'A' (Sample No. 1) and coupling F was disconnected to form coupling F'B (Sample No. 2). In this manner, a constant head of 75 psi was maintained on the inflow (top) end of samples during testing using the center burette connected to Regulator A on the Right Control Panel, while a variable head (less than 75 psi) was maintained on the outflow (bottom) end of samples through the outer burettes connected to Regulator A on the Left Control Panel. All valves were kept in the open position. With a lower pressure in the outer burettes than in the center burette, flow occurred in samples from top to bottom, and was measured by monitoring increasing water levels in each of the outer burettes. To apply increasing hydraulic gradients to samples, the back pressure applied to each of the outer burettes was decreased incrementally while maintaining a constant head on the center burette.

As previously mentioned, the applicable range in hydraulic gradients initially selected for soil permeability testing was for gradients between 5 and 15. For a gradient of 5, this would require a pressure difference of 0.5 psi across each sample, or a pressure equal to 74.5 psi in the outer burettes. A pressure difference across each sample of 1.5 psi (pressure equal to 73.5 psi in outer burettes) was required for a gradient of 15. At each gradient, concurrent measurements of outflow from each sample were made on frequent and regular intervals (normally 6 hrs maximum). A period of nonsteady-state flow usually immediately followed the application of a hydraulic gradient, in which the rate of flow (flow velocity) tended to decrease with time to a steady-state (constant) flow value. Measurement of a minimum of two consecutive readings of equal outflow was required for each sample prior to application of a new hydraulic gradient. During the early stages of permeability testing (gradients between 5 and 15), a nonlinear increase in flow velocity and decreasing soil permeability was observed with increasing hydraulic gradient. This aberrant behavior was reflected by a nonlinear v-i relationship, such that the flow velocity (v) was not proportional to the hydraulic gradient (i), but increased less rapidly than the gradient.

To further investigate the apparent effects of magnitude of hydraulic gradient on permeability, gradients were incrementally increased in testing to a maximum value of 300. A gradient of 300 required a pressure difference across the sample of 30 psi, or a pressure equal to 45 psi in the outer burettes. This type phenomenon appears consistent with particle migration and the clogging of soil voids, or with consolidation of samples under increasing hydraulic gradients. The potential for particle migration and the clogging of voids seems remote, as effluent from samples remained clear throughout tests. In an attempt to determine whether or not consolidation was the predominant phenomenon occurring, it was decided to continue tests, only this time under the condition of decreasing hydraulic gradients. If consolidation was in fact occurring, a relatively linear v-i relationship (and constant permeability) should be obtained for the condition of decreasing hydraulic gradients. During the conduct of all tests, frequent inspections were made of the equipment to insure it was in proper working order and no leaks had developed in the apparatus.

Distribution of Stresses

The assumed distribution of stresses and deformed shapes of triaxial samples, both during back-pressure saturation and soil permeability testing, is shown conceptually in Figure 16. It must be remembered that when a load is applied to a saturated soil mass and drainage is prevented or impeded, compressive stresses are developed in the pore water (assumed incompressible) over and above compressive stresses existing in the water from hydrostatic effects. These compressive stresses are called "excess pore pressures." These stresses are initially absorbed by the pore water only and are not felt by the soil, thus they can produce no relative displacement of soil particles within the soil mass. Instead, soil particles are deformed only in response to the stresses they feel, which have been denoted as "effective stresses." The effective stress felt by the soil particle equals the total stress applied to the soil, less the excess pressure developed in the water. Consolidation of a saturated soil mass under a constant total stress generally occurs with an increase in effective stress after pore pressures have dissipated with time and drainage.

In addition to accurately simulating the sequence of loading anticipated in the field, a laboratory test must also be capable of

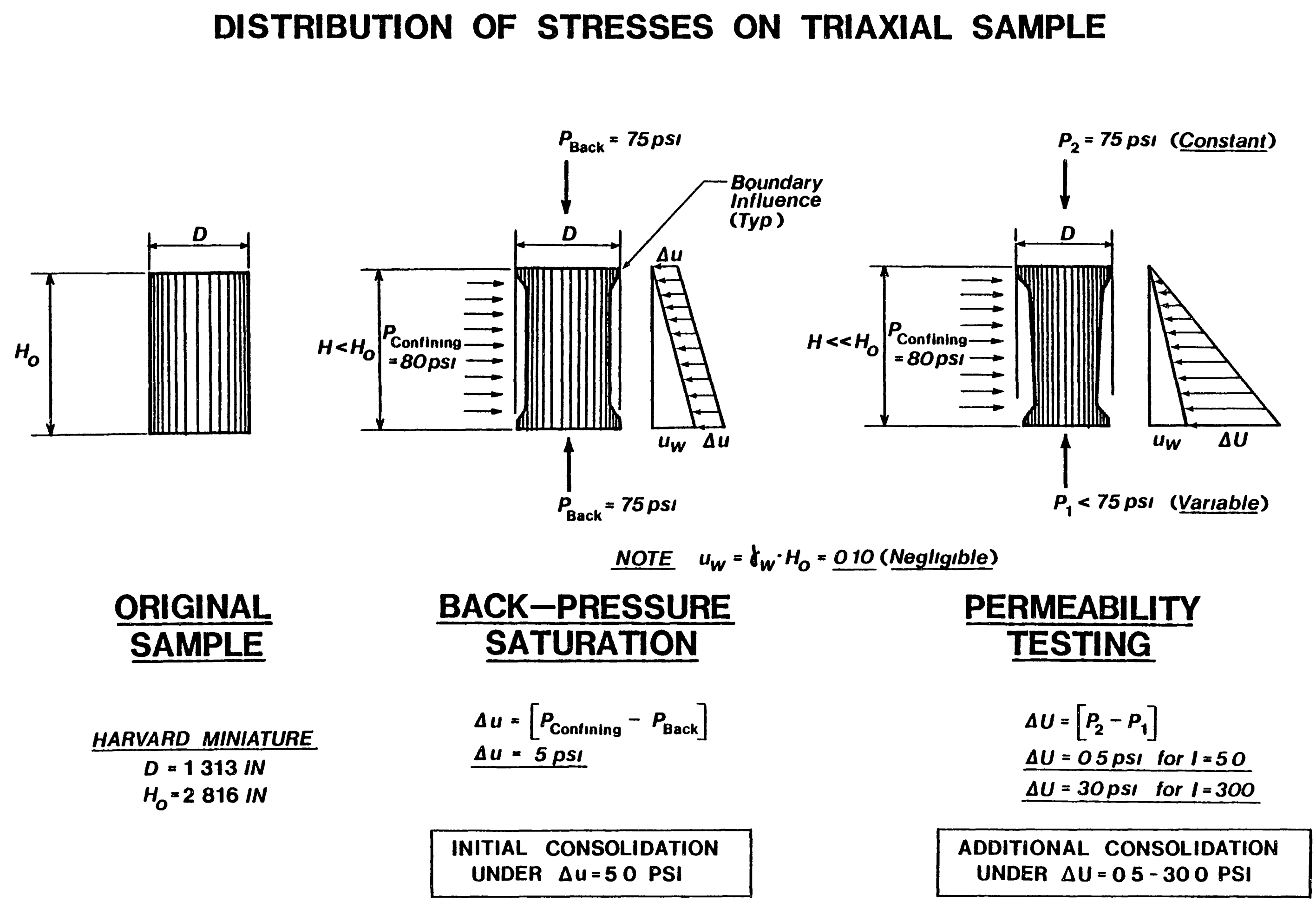


Figure 16. Assumed Distribution of Stresses on Triaxial Sample During Back-Pressure Saturation and Permeability Testing

simulating the conditions of drainage expected in the field during and after construction. All clay soils are only partially saturated when initially placed in the field. In the case of a compacted clay liner, a fluid head applied on the liner for an extended period of time would tend to saturate the soil and promote the flow of fluid through the liner. Increased fluid (hydrostatic) heads would result in increased hydraulic gradients on the liner, and thus, increased flow rates. Further, assuming unimpeded drainage and significant hydrostatic heads in the field, consolidation of clay liner material would begin, and would tend to increase with increasing hydraulic gradients. For the range in hydraulic gradients normally encountered in the field (generally less than 15), the total amount of consolidation would be relatively small. The normal sequence of loading in the field would be for strictly increasing, or increasing then decreasing, hydraulic gradients.

For the most part, consolidation of samples was prevented during the back-pressure saturation phase in experiments in order to allow samples to consolidate under increasing effective stresses resulting from applied hydraulic gradients. In this manner, the effects of magnitude of hydraulic gradient on permeability can be investigated. At the beginning of the back-pressure saturation phase, the soil is only partially saturated and voids contain some entrapped air. When the back pressure and confining pressure are applied, excess pore pressures develop in the soil voids. Initially, the excess pore pressure (5 psi) is equal to the difference in the confining pressure (80 psi) and back pressure (75 psi), as without drainage all the applied stress is taken by the pore water. However, as air entrapped in soil voids is relatively compressible as compared to water (or even soil particles), some of the initial stress will likely be taken by the soil grains as effective stress as the air compresses. As the soil is progressively saturated from either end under back pressure, more and more entrapped air is forced into solution. Under high pressures, essentially all the entrapped air is forced into solution, such that a pore pressure of 75 psi (in excess of hydrostatic pressure) exists in samples at the end of the back-pressure saturation phase.

The consolidation of samples primarily occurs during the permeability testing phase of experiments, as the conditions for dissipation of pore pressures with drainage and time are satisfied. When the back pressure at the outflow end of samples is reduced and a hydraulic gradient is applied to samples, the excess pore pressure at the top of each sample (constant head end) would be less than 5 psi, while the excess pore pressure at the bottom (variable head end) would equal that at the top plus the pressure difference across the sample due to the applied hydraulic gradient. For a given hydraulic gradient (total stress), excess pore pressures would dissipate with time and drainage, resulting in increased effective stresses and consolidation of samples. This occurrence would be most pronounced at the outflow end of samples, where the lateral effective stresses are highest. Consolidation would continue at a given gradient until all excess pore pressures were dissipated, or until a new hydraulic gradient (total stress) was applied to the soil. In this manner, consolidation as a result of high lateral effective stresses at the outflow end of samples would continue to occur as long as increasing hydraulic gradients were applied. As the soil was consolidated at a given gradient in experiments, it is continually undergoing minor volume changes. During this period, nonsteady-state flow was generally measured through the sample, as the volume of voids was in a transient state. Once pore pressures were dissipated, the volume of voids was essentially fixed and a steady-state (constant) flow was measured through the sample.

Other Testing Procedures

After conclusion of testing, the pressure was removed from triaxial cells, triaxial cells disassembled, samples removed, and final water content determinations made. In determining final water contents, samples were cut into thirds and the water content and degree of saturation determined for each portion. If consolidation occurred during permeability testing as a result of an increase in the applied effective stress at the outflow end of samples, this would be reflected by a decreasing moisture profile across samples from top to bottom. Since portions nearest the outflow end of samples are stressed (consolidated) to the greatest extent, the final water content at the outflow end of samples should be less than that measured for the middle (and top) portion of samples.

CHAPTER IV

RESULTS AND DISCUSSION

In this study, the permeability of six remolded samples was determined directly in the laboratory using the triaxial, constant-head test and procedures described previously (Chapter III). To investigate the effects of placement condition on permeability, the samples were compacted to various known placement conditions (density and moisture content) prior to testing. Permeability tests were conducted on samples compacted at optimum water content (w_{opt}), 3 percent dry of optimum (-3% w_{opt}), and 3 percent wet of optimum (+3% w_{opt}).

Because the triaxial shear strength testing equipment in the OSU Soil Mechanics Laboratory easily accommodates Harvard miniature samples, the Harvard-size mold was used to compact all soil samples. An optimum water content (w_{opt}) of 17.9% was determined for the soil using the Harvard compaction method. Replicate (two) samples were prepared and tested for each placement condition, to improve reliability of test data. To allow a relative comparison of test results, all samples were compacted using the same compactive effort (13,800 ft-lbs/ft^3).

This chapter presents data regarding initial soil parameters as well as the results of permeability tests conducted on remolded soil samples. In the following sections, back-pressure saturation and water content data are presented for each sample and placement condition, and the results of laboratory permeability tests are presented for both increasing and decreasing hydraulic gradients. For each sample, permeability test data are shown graphically in the form of separate plots of soil permeability versus hydraulic gradient and flow velocity versus hydraulic gradient. An analysis and discussion of test results is presented in a separate section.

Initial Soil Parameters

Soil parameters of particular interest in this study and used for evaluation of results included water content (degree of saturation) and void ratio. The analysis of test results generally required some simplifying assumptions regarding "initial" and "final" parameters of compacted samples, particularly with regard to the density (i.e., void ratio) of samples. Necessary assumptions regarding the initial void ratio of compacted samples were based on average water content and density values calculated for samples at the end of tests (to be described). A specific gravity (G_s) of 2.74 was determined for the soil and used in calculations involving weight-volume relationships.

A dry density of 105.1 pcf was calculated for samples compacted at optimum water content (17.9%), which corresponded to an initial void ratio and degree of saturation of 0.627 and 78.1%, respectively. A dry density of 102.0 pcf was calculated for samples compacted dry of optimum (14.9%) and wet of optimum (20.9%), which corresponded to an initial void ratio for both of 0.677. These values are only slightly lower than the values predicted from the moisture-density curve developed for the soil using remolded samples (described previously). The initial degree of saturation of samples compacted dry of optimum and wet of optimum was calculated to be 60.3% and 84.6%, respectively.

Results of Back-Pressure Saturation

The equipment used and procedures followed in the back-pressure saturation of compacted soil samples were described in Chapter III. During the back-pressure saturation phase of experiments, a constant back pressure of 75 psi was maintained on either end of samples. A confining pressure of 80 psi was used throughout the back-pressure saturation (and permeability test) phase. The pressures applied to samples were increased concurrently in 5 psi increments to their maximum values, always maintaining a minimum 5 psi difference in confining (chamber) pressure over back pressure. The rate and volume of water entering each sample was measured by reading declines in water levels in the outer burettes until the system had stabilized. Concurrent readings of flow into each sample were made on frequent and regular intervals (normally 6 hrs maximum). After conclusion of testing, the triaxial cells were disassembled and each sample was cut into thirds and oven-dried, to determine the final water content (and degree of saturation) of each portion.

The average rate and volume of flow into samples during the back-pressure saturation phase of experiments is shown in Figure 17. A brief period of rapid inflow normally accompanied the application of a back pressure to samples, followed by a period in which the rate of flow into samples steadily decreased until full saturation was achieved. A period of 3.5 days to 6.5 days was generally required for water levels to stabilize in burettes, indicating essential complete saturation of samples. Samples compacted dry of optimum showed the highest initial rates of inflow, but generally required a longer saturation time than samples

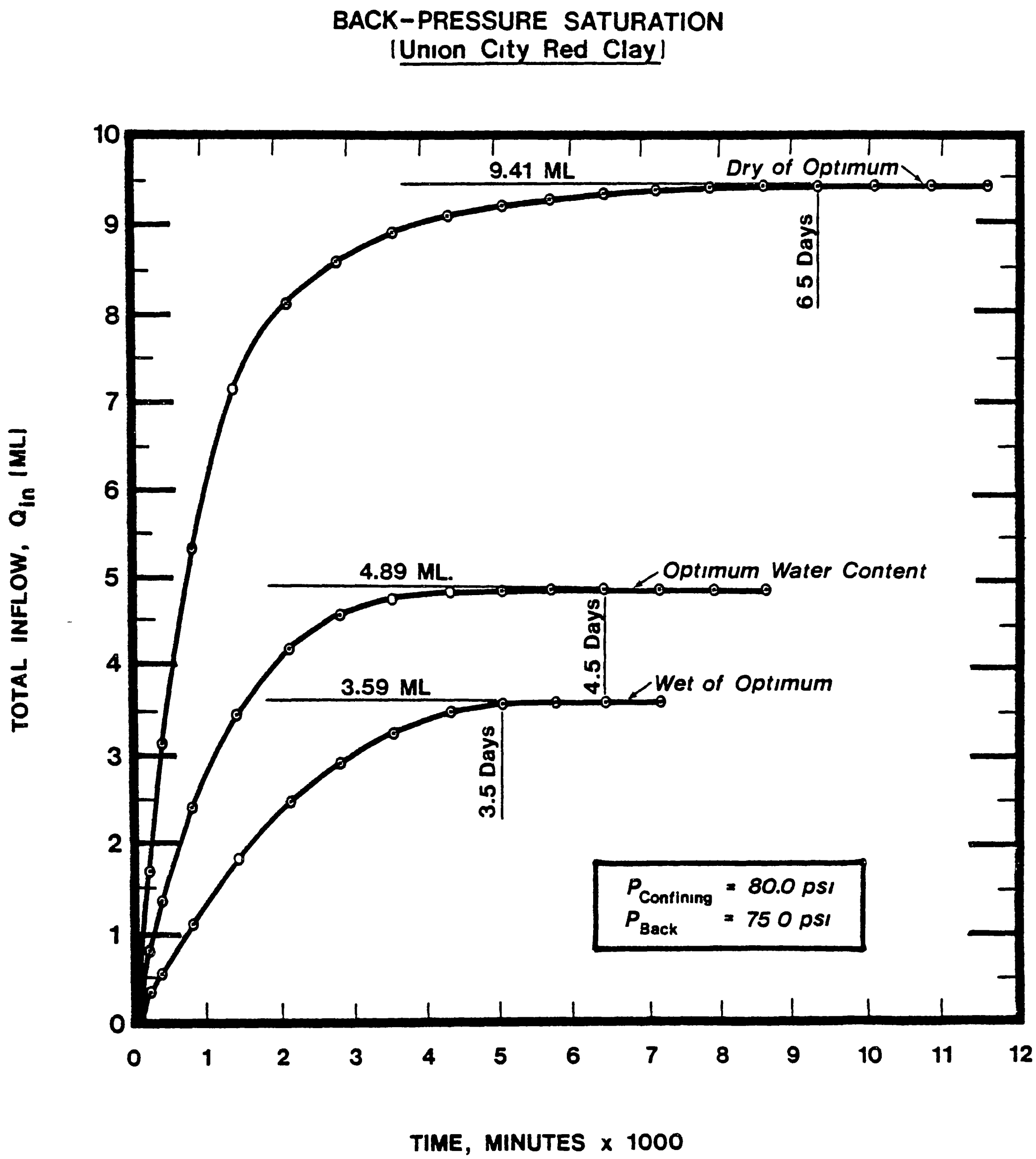


Figure 17. Back-Pressure Saturation Data for Various Placement Conditions

compacted at (and wet of) optimum water content. After burette water level readings stabilized, pressures were maintained on samples for an additional period of approximately 36 hours. If no additional inflow was indicated, samples were assumed fully saturated and ready for permeability testing.

The initial and final water contents and degree of saturation calculated for the top, middle, and bottom portions of each sample are summarized in Table III. This data is shown graphically for each placement condition in Figures 18 - 20. The saturation values given in Table III (shown in Figures 18 - 20) were calculated assuming no change in the initial void ratio of samples during testing (i.e., no sample consolidation). However, the actual degree of saturation of samples would be slightly higher than the previously described values since consolidation occurred in samples during testing. This would be especially true for the bottom portion of samples, where the greatest consolidation (change in void ratio) occurred.

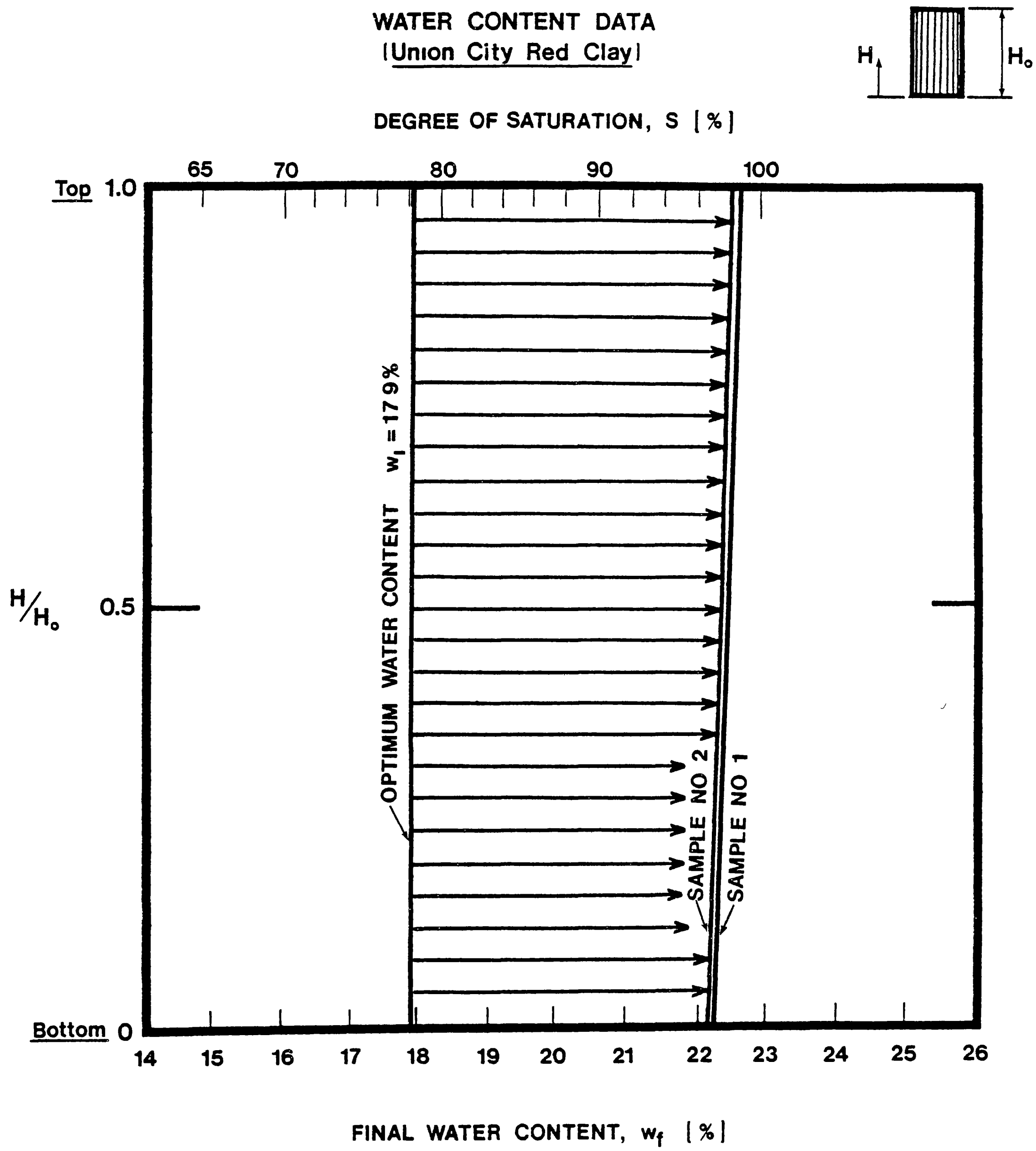
A linearly decreasing profile from top to bottom was generally indicated in the final water content (and degree of saturation) of samples. The moisture profiles shown in Figures 18 - 20 indicate the total variation in the final water content (degree of saturation) was relatively consistent for each sample and placement condition. The final degree of saturation calculated for the top portion of samples (Table III) generally ranged from 97.5% to 99.0%. Similar values calculated for the bottom portion of samples generally ranged from 95.6% to 97.0%. In general, a final degree of saturation of 98.0% or more would be necessary to infer complete saturation of samples (or portions thereof) during testing.

TABLE III

WATER CONTENT AND DEGREE OF SATURATION DATA

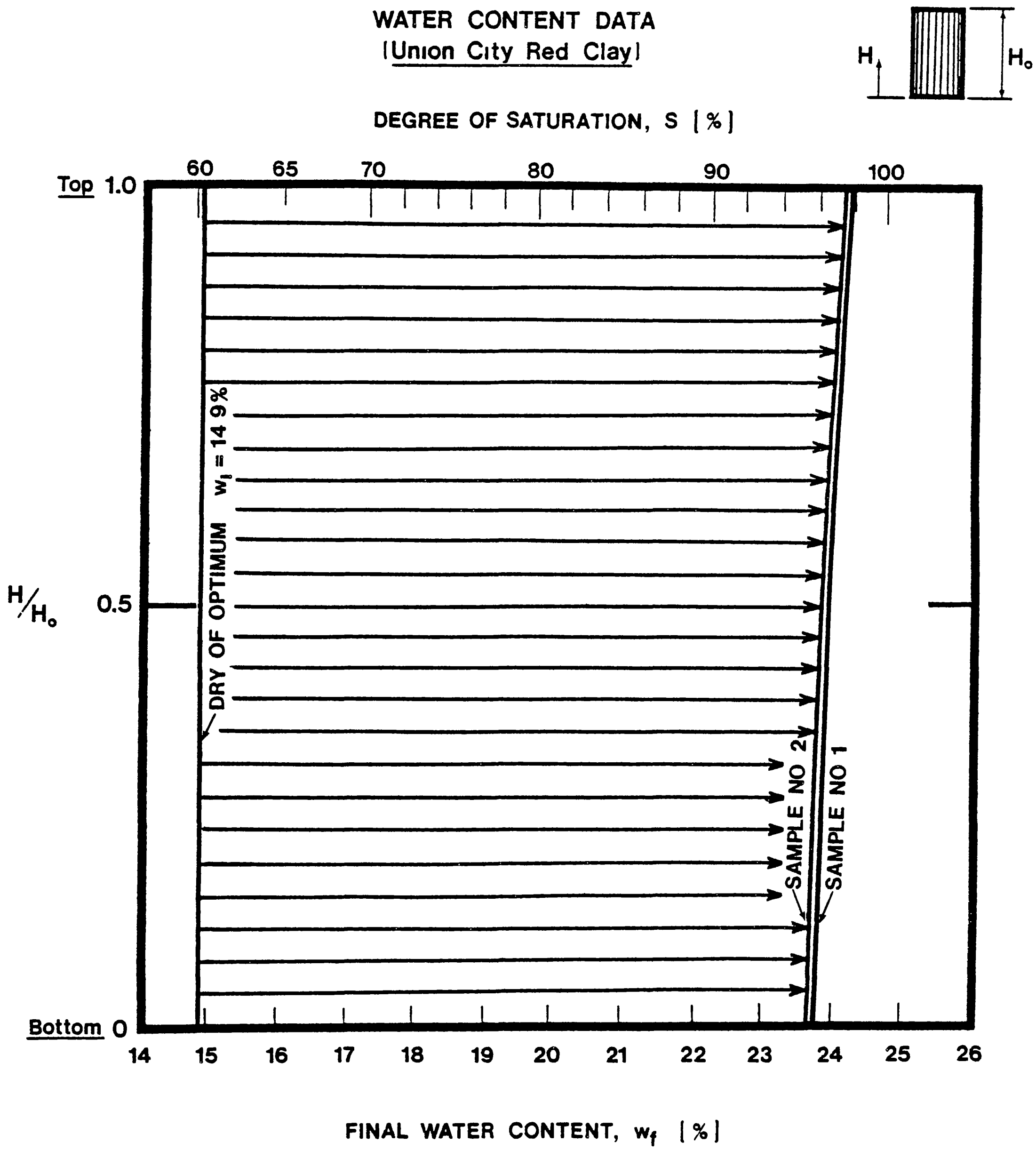
UNION CITY RED CLAY						
Placement Condition	Initial Moisture Content/Degree of Saturation	Portion of Sample	SAMPLE NO. 1		SAMPLE NO. 2	
			Final Water Content, w_f	Final Degree of Saturation, S_f[1]	Final Water Content, w_f	Final Degree of Saturation, S_f[1]
OPTIMUM WATER CONTENT	$w_i = 17.9\%$	TOP	22.60%	98.6%	22.48%	98.1%
		MIDDLE	22.40%	97.8%	22.29%	97.3%
	$S_i = 78.1\%$	BOTTOM	22.23%	97.0%	22.11%	96.5%
		(Avg.)	22.41%	97.8%	22.29%	97.3%
DRY OF OPTIMUM	$w_i = 14.9\%$	TOP	24.20%	97.9%	24.11%	97.5%
		MIDDLE	23.94%	96.9%	23.86%	96.5%
	$S_i = 60.3\%$	BOTTOM	23.75%	96.1%	23.63%	95.6%
		(Avg.)	23.96%	97.0%	23.87%	96.5%
WET OF OPTIMUM	$w_i = 20.9\%$	TOP	24.39%	98.7%	24.46%	99.0%
		MIDDLE	24.25%	98.1%	24.31%	98.4%
	$S_i = 84.6\%$	BOTTOM	24.06%	97.3%	24.13%	97.6%
		(Avg.)	24.23%	98.0%	24.30%	98.3%

[1] Values calculated assuming no change in void ratio ($e_o = e_f$). Actual degree of saturation would be slightly higher for all portions of sample since consolidation occurred during testing.



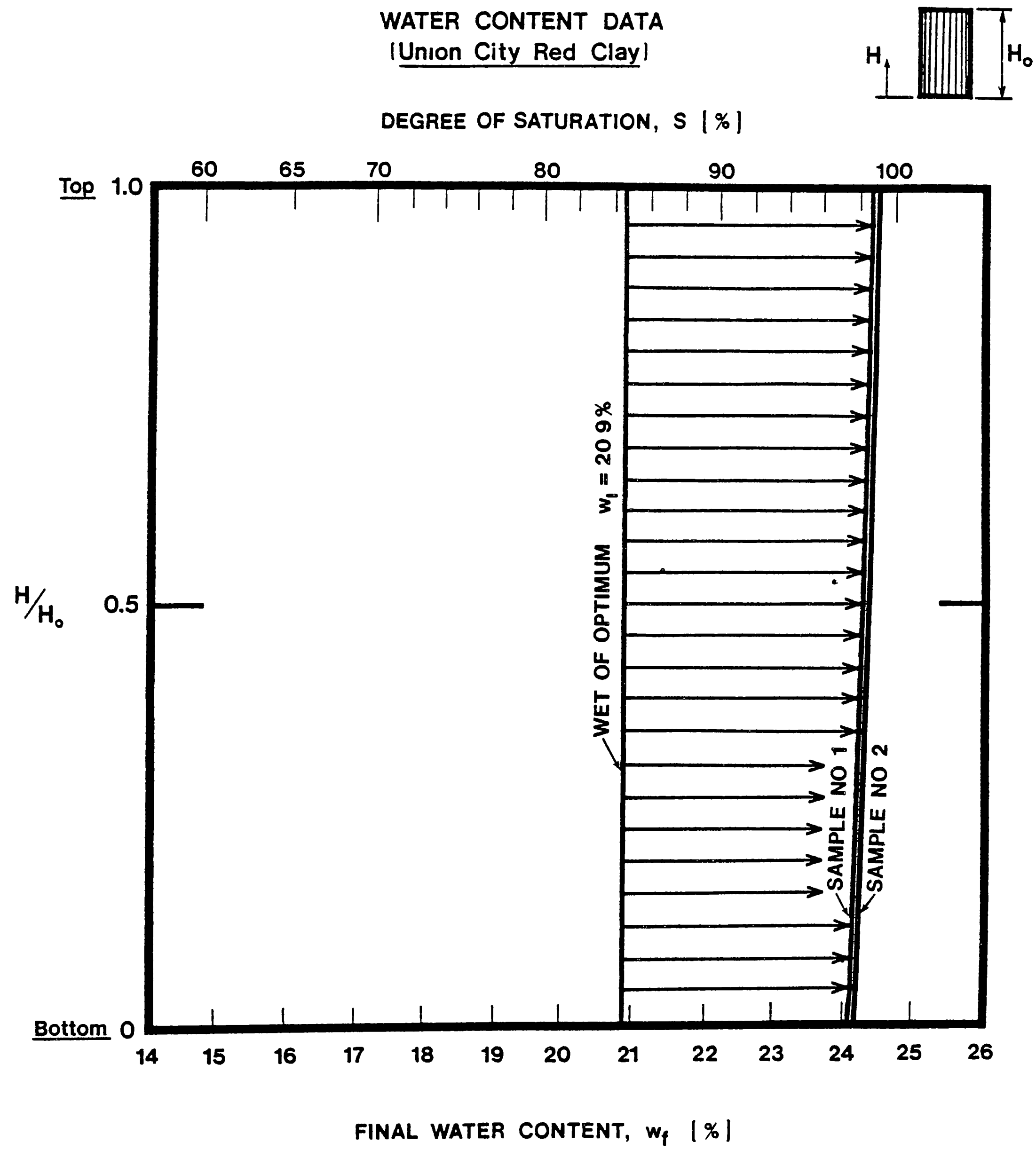
	SAMPLE 1	SAMPLE 2
TOP	22.60%	22 48%
MIDDLE	22 40	22 29
BOTTOM	22 23	22 11

Figure 18. Final Water Content and Degree of Saturation Data for Samples Compacted at Optimum Water Content



	SAMPLE 1	SAMPLE 2
TOP	24 20%	24 11%
MIDDLE	23 94	23 86
BOTTOM	23 75	23 63

Figure 19. Final Water Content and Degree of Saturation Data for Samples Compacted Dry of Optimum



	SAMPLE 1	SAMPLE 2
TOP	24 39%	24 46%
MIDDLE	24 25	24 31
BOTTOM	24 06	24 13

Figure 20. Final Water Content and Degree of Saturation Data for Samples Compacted Wet of Optimum

The significance of a decreasing moisture profile from top to bottom across samples at the end of tests was discussed in Chapter III, and appears related to consolidation of samples under increasing hydraulic gradients as a result of an unavoidable increase in the applied effective stress at the outflow end of samples. In order to apply increasing hydraulic gradients, the back pressure at the outflow end of samples must be reduced while maintaining a constant head on samples. Since portions nearest the outflow end of samples are stressed (consolidated) to the greatest extent during testing, the final water content measured at the outflow end of samples should be less than that measured for the middle and top portions of samples. This general trend was, in fact, reflected in the water content of samples at the end of tests (Table III). The consolidation of samples will be discussed, in more detail, in a subsequent section.

Results of Permeability Testing

The equipment used and procedures followed in permeability testing of compacted soil samples were described in Chapter III. During permeability tests, a constant back pressure of 75 psi was maintained on the inflow (top) end of samples using the center burette, while a variable head (less than 75 psi) was maintained on the outflow (bottom) end of samples using the outer burettes. With a lower pressure in the outer burettes than in the center burette, flow occurred in samples from top to bottom, and was measured by monitoring increasing water levels in each of the outer burettes. To apply increasing hydraulic gradients to samples, the back pressure applied to each of the outer burettes was decreased incrementally while maintaining a constant head on the center burette. At each gradient, concurrent measurements of outflow from each sample were made on frequent and regular intervals (6 hrs maximum).

As previously discussed, the applicable range in hydraulic gradients initially selected for soil permeability testing was for gradients between 5 and 15. A period of nonsteady-state flow usually immediately followed the application of a hydraulic gradient (pressure difference), in which the rate of flow (flow velocity) tended to decrease with time to a steady-state (constant) flow value. Measurement of a minimum of two consecutive readings of equal outflow was required for each sample prior to application of a new hydraulic gradient. With an accuracy in measurement of water levels in burettes to 1/64 inch, the minimum outflow which could be measured from samples corresponded to 0.05 ml (cm^3).

During the early stages of permeability testing (gradients less than 15), a nonlinear increase in flow velocity and decreasing soil permeability was observed with increasing hydraulic gradients. This behavior was reflected by a nonlinear v-i relationship in which the flow velocity (v) was not proportional to the hydraulic gradient (i), but increased less rapidly than the gradient. To further investigate the effects of magnitude of hydraulic gradient on permeability, gradients were increased incrementally in testing to a maximum value of 300. Further, in an attempt to determine whether this type behavior was due to consolidation of samples under increasing hydraulic gradients, permeability tests were continued, only under the condition of decreasing (versus increasing) hydraulic gradients. If consolidation was occurring under increasing hydraulic gradients, a relatively linear v-i relationship (and constant permeability) should be obtained for the condition of decreasing hydraulic gradients.

In permeability testing of samples under increasing hydraulic gradients, gradients of 5, 10, 15, 20, 40, 80, 150, and 300 were applied to samples using the pressure differences summarized in Table IV. These same gradient values were also applied to samples in permeability testing under decreasing hydraulic gradients, with the exception that testing was omitted at gradients of 10 and 20. Separate plots of soil permeability versus hydraulic gradient and flow velocity versus hydraulic gradient were developed for each sample using the increasing gradient and decreasing gradient steady-state flow values measured at each hydraulic gradient. Plots of soil permeability versus hydraulic gradient are shown for each placement condition in Figures 21 - 26. Similar plots of flow velocity versus hydraulic gradient are shown for each placement condition in Figures 27 - 32. The laboratory times required for measurement of an outflow of 0.10 ml (1/32 inch movement in burettes) under steady-state (constant) flow conditions are summarized in each plot.

In developing the previously described plots, the flow velocity of samples at each hydraulic gradient was calculated by dividing the measured steady-state flow value (volume/time) by the cross-sectional area of the sample (8.73 cm^2). The coefficient of permeability of samples at each gradient was calculated by dividing the previously determined flow velocity (ordinate in v-i plot) by the magnitude of hydraulic gradient. As shown in Figures 21 - 26, permeability generally decreased with increasing hydraulic gradients for all samples and placement conditions. The coefficient of permeability of samples compacted dry of optimum and

TABLE IV

HYDROSTATIC HEADS AND PRESSURE DIFFERENCES FOR VARIOUS APPLIED HYDRAULIC GRADIENTS[1]

Hydraulic Gradient (I)[2]	Hydrostatic Head (H)	Pressure Difference (Δp)[3]
0.5	1.41 in.	0.05 psi.
1.0	2.82 in.	0.10 psi.
1.5	4.22 in.	0.15 psi.
2.0	5.63 in.	0.20 psi.
2.5	7.04 in.	0.25 psi.
3.0	8.45 in.	0.31 psi.
3.5	9.86 in.	0.36 psi.
4.0	11.26 in.	0.41 psi.
4.5	12.67 in.	0.46 psi.
5.0*	14.08 in.	0.51 psi.
10.0**	28.16 in.	1.02 psi.
15.0*	42.24 in.	1.53 psi.
20.0**	56.32 in.	2.03 psi.
25.0	70.40 in.	2.54 psi.
30.0	84.48 in.	3.05 psi.
35.0	98.56 in.	3.56 psi.
40.0*	112.64 in.	4.07 psi.
45.0	126.72 in.	4.58 psi.
50.0	140.80 in.	5.08 psi.
60.0	168.96 in.	6.10 psi.
70.0	197.12 in.	7.12 psi.
80.0*	225.28 in.	8.14 psi.
90.0	253.44 in.	9.15 psi.
100.0	281.60 in.	10.17 psi.
150.0*	422.40 in.	15.25 psi.
200.0	563.20 in.	20.34 psi.
250.0	704.00 in.	25.42 psi.
300.0*	844.80 in.	30.51 psi.
350.0	985.60 in.	35.59 psi.
400.0	1126.40 in.	40.68 psi.

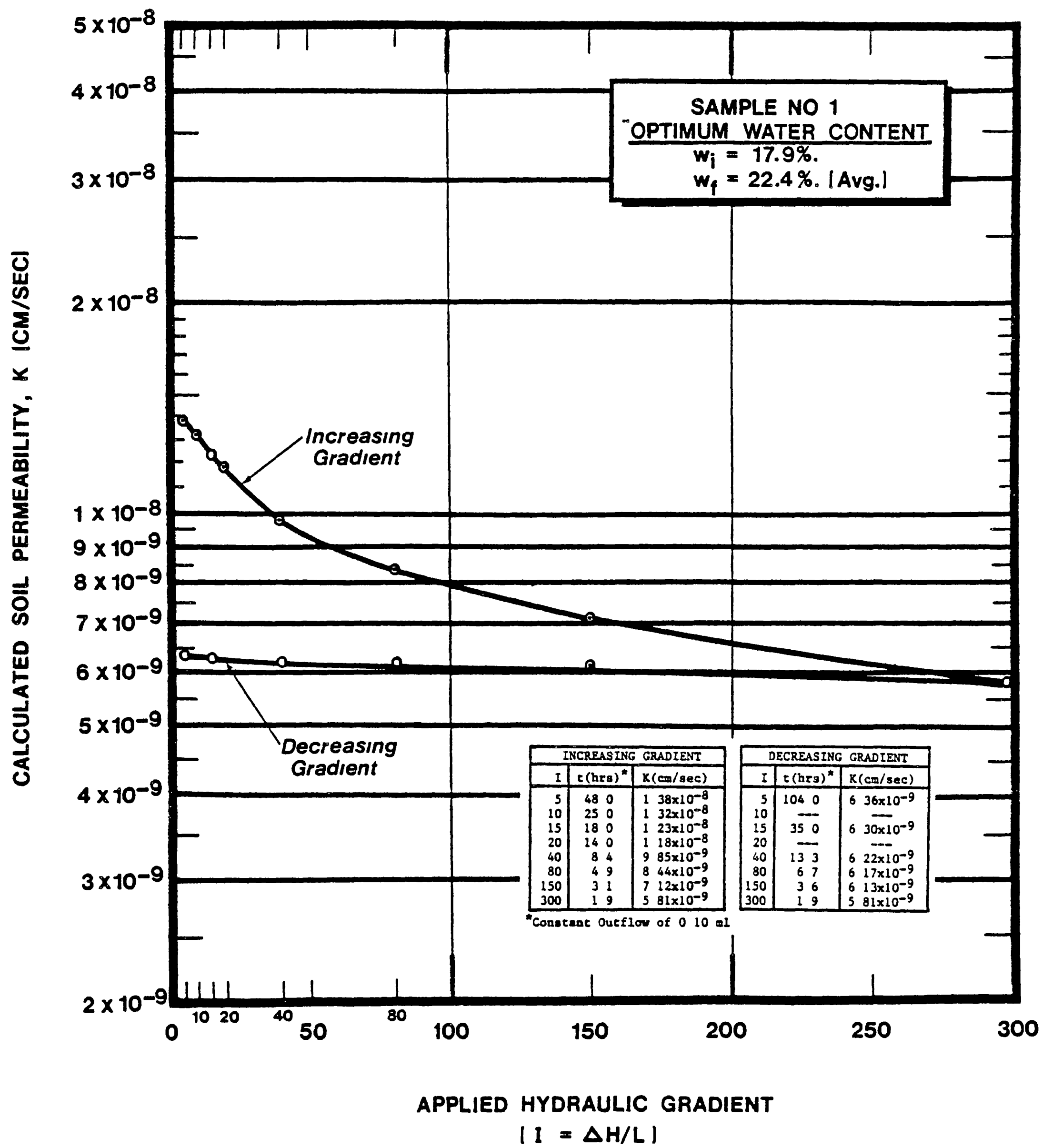
* Increasing/Decreasing Gradient value.
** Increasing Gradient value only.

[1]Assumes samples are fully-saturated.

[2]$I = H/L$, where $L = 2.816$ in. (for Harvard miniature samples).

[3]$\Delta p = \gamma \cdot H$, where $\gamma = 62.4$ pcf. (for water).

PERMEABILITY TESTING
(Union City Red Clay)

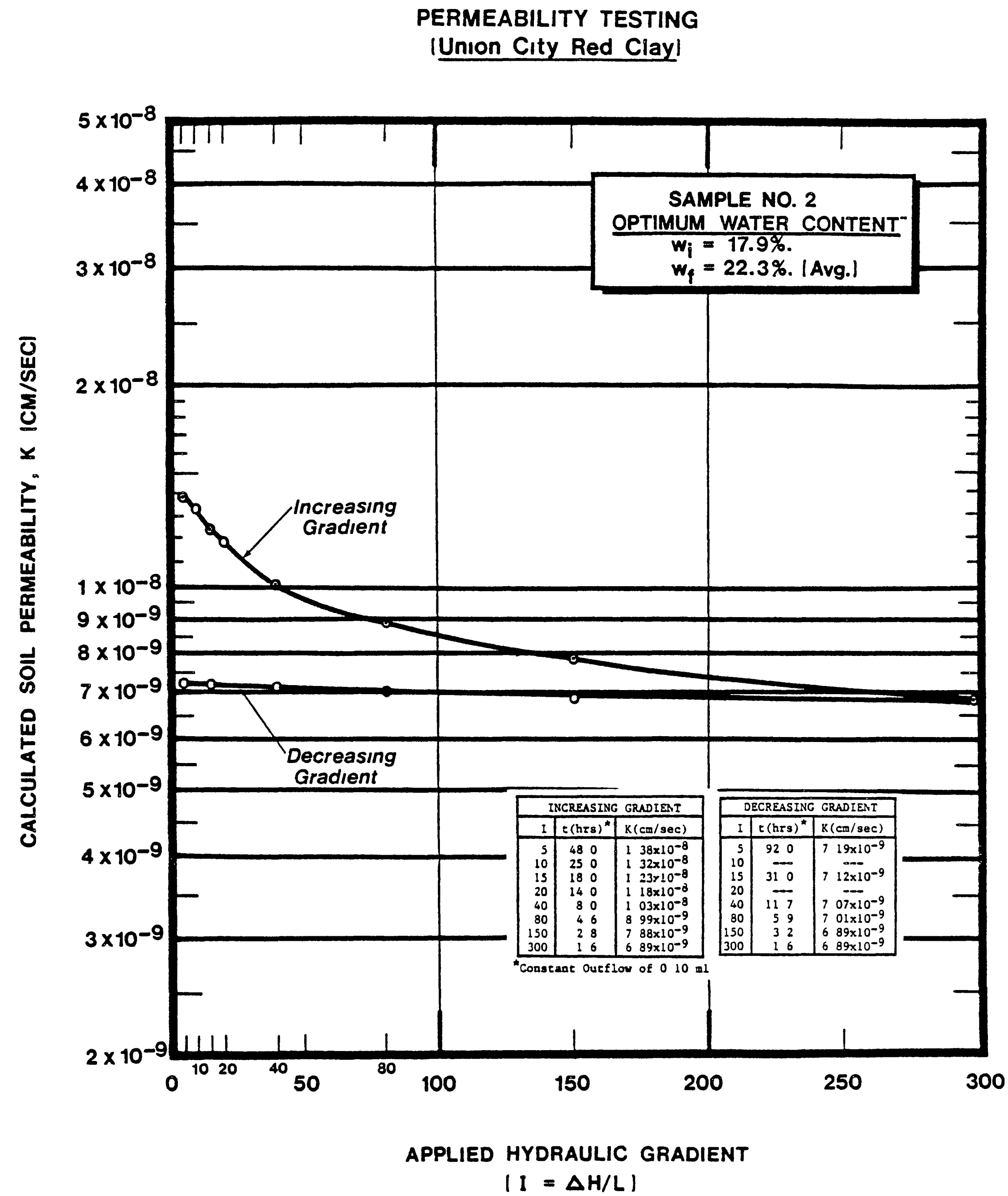


INCREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	48 0	1 38x10^{-8}
10	25 0	1 32x10^{-8}
15	18 0	1 23x10^{-8}
20	14 0	1 18x10^{-8}
40	8 4	9 85x10^{-9}
80	4 9	8 44x10^{-9}
150	3 1	7 12x10^{-9}
300	1 9	5 81x10^{-9}

DECREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	104 0	6 36x10^{-9}
10	---	---
15	35 0	6 30x10^{-9}
20	---	---
40	13 3	6 22x10^{-9}
80	6 7	6 17x10^{-9}
150	3 6	6 13x10^{-9}
300	1 9	5 81x10^{-9}

*Constant Outflow of 0 10 ml

Figure 21. Permeability-Gradient Relationship for Soil Compacted at Optimum Water Content (Sample No. 1)

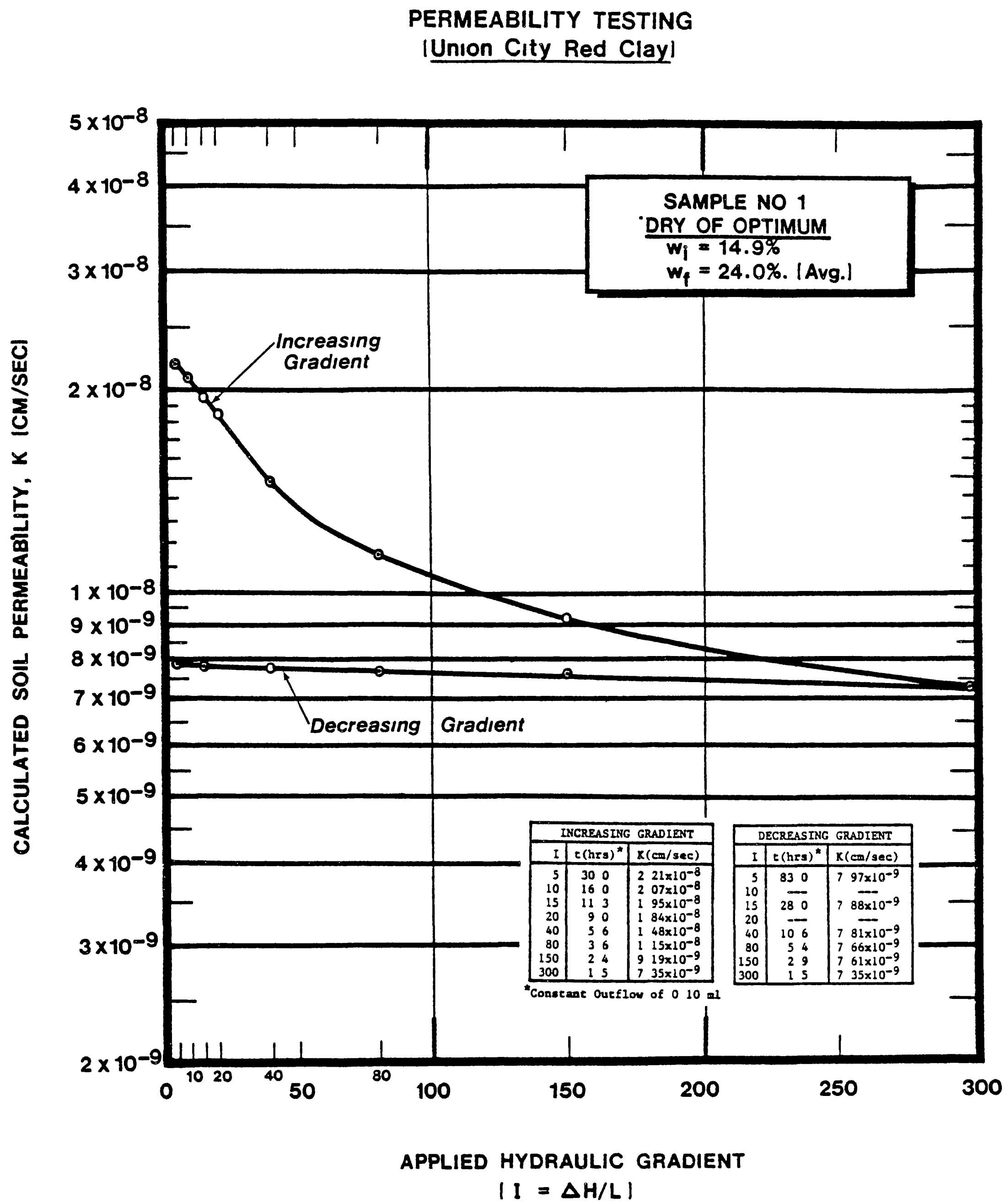


INCREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	48 0	1 38x10^{-8}
10	25 0	1 32x10^{-8}
15	18 0	1 23x10^{-8}
20	14 0	1 18x10^{-8}
40	8 0	1 03x10^{-8}
80	4 6	8 99x10^{-9}
150	2 8	7 88x10^{-9}
300	1 6	6 89x10^{-9}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	92 0	7 19x10^{-9}
10	---	---
15	31 0	7 12x10^{-9}
20	---	---
40	11 7	7 07x10^{-9}
80	5 9	7 01x10^{-9}
150	3 2	6 89x10^{-9}
300	1 6	6 89x10^{-9}

Figure 22. Permeability-Gradient Relationship for Soil Compacted at Optimum Water Content (Sample No. 2)

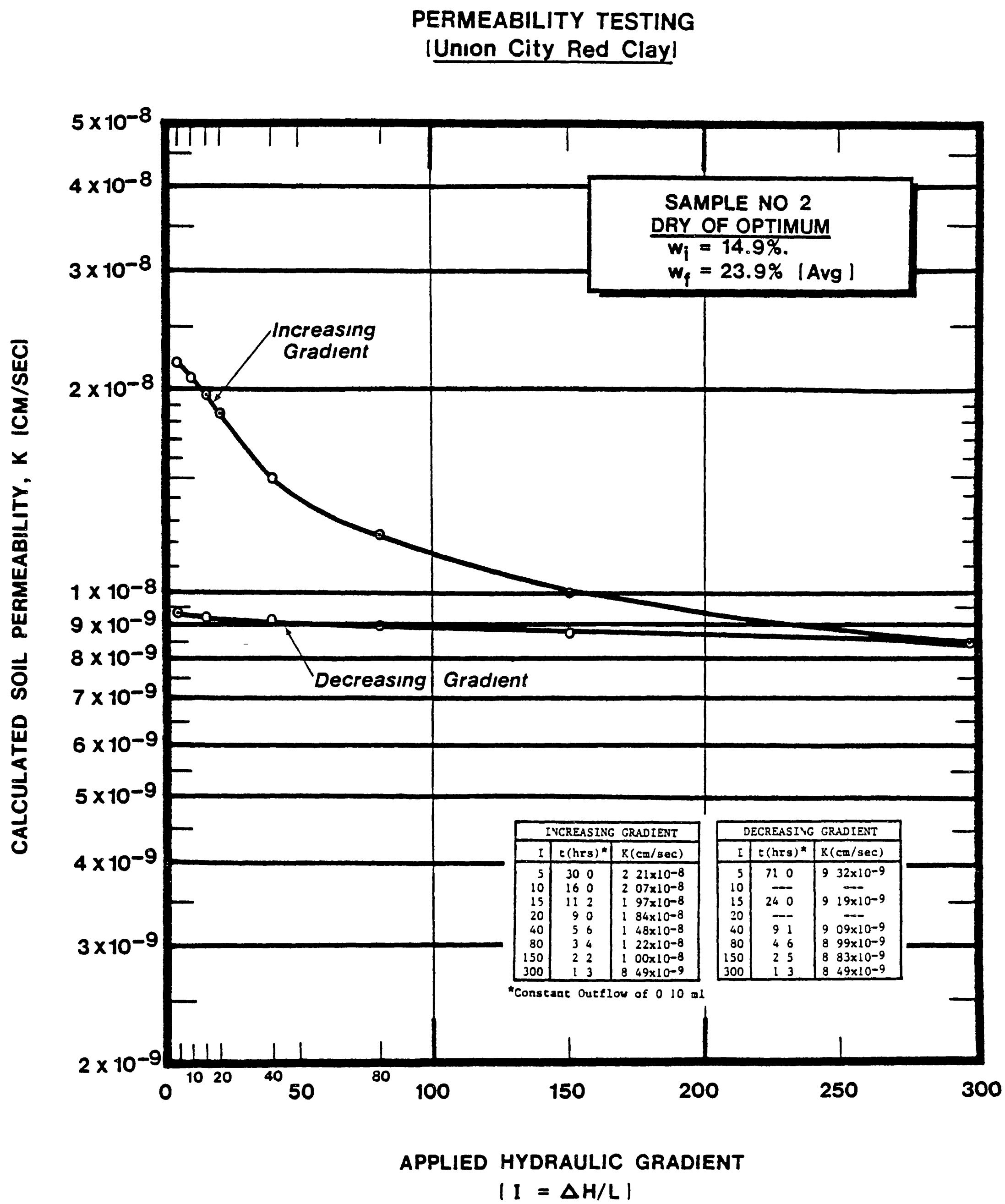


INCREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	30 0	2 21x10^{-8}
10	16 0	2 07x10^{-8}
15	11 3	1 95x10^{-8}
20	9 0	1 84x10^{-8}
40	5 6	1 48x10^{-8}
80	3 6	1 15x10^{-8}
150	2 4	9 19x10^{-9}
300	1 5	7 35x10^{-9}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	83 0	7 97x10^{-9}
10	---	---
15	28 0	7 88x10^{-9}
20	---	---
40	10 6	7 81x10^{-9}
80	5 4	7 66x10^{-9}
150	2 9	7 61x10^{-9}
300	1 5	7 35x10^{-9}

Figure 23. Permeability-Gradient Relationship for Soil Compacted Dry of Optimum (Sample No. 1)

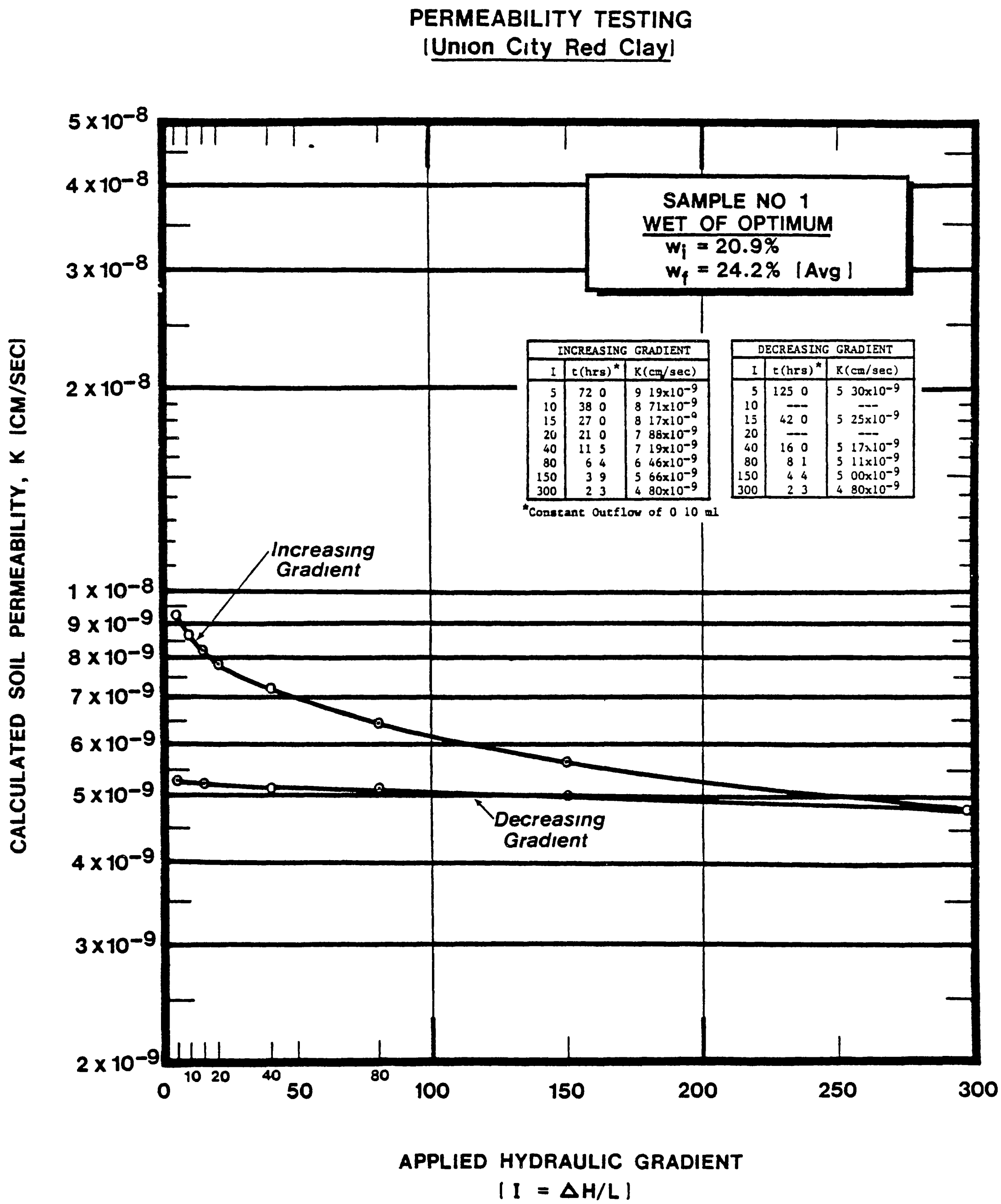


INCREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	30 0	2 21x10^{-8}
10	16 0	2 07x10^{-8}
15	11 2	1 97x10^{-8}
20	9 0	1 84x10^{-8}
40	5 6	1 48x10^{-8}
80	3 4	1 22x10^{-8}
150	2 2	1 00x10^{-8}
300	1 3	8 49x10^{-9}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	71 0	9 32x10^{-9}
10	---	---
15	24 0	9 19x10^{-9}
20	---	---
40	9 1	9 09x10^{-9}
80	4 6	8 99x10^{-9}
150	2 5	8 83x10^{-9}
300	1 3	8 49x10^{-9}

Figure 24. Permeability-Gradient Relationship for Soil Compacted Dry of Optimum (Sample No. 2)

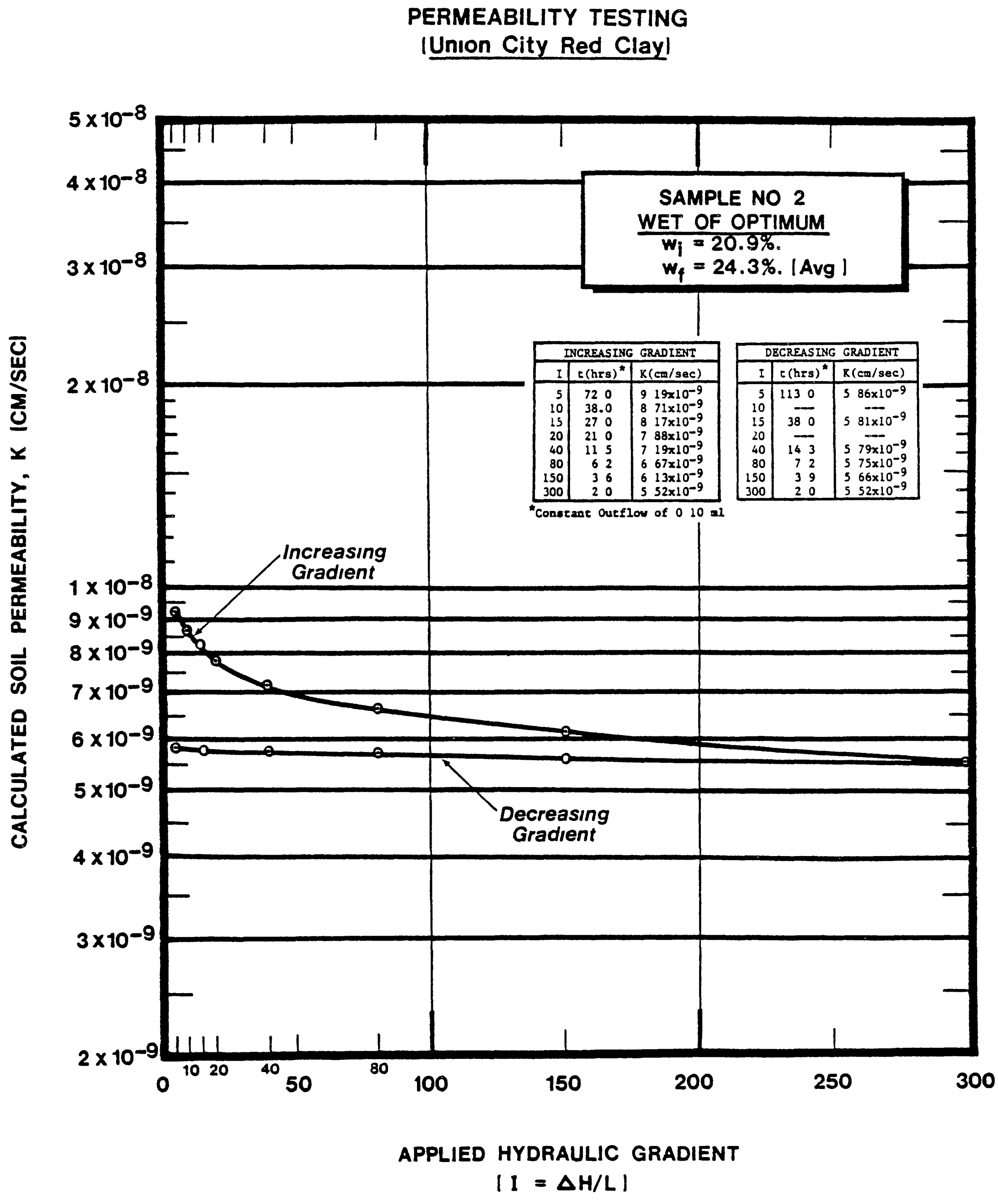


INCREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	72 0	9 19x10^{-9}
10	38 0	8 71x10^{-9}
15	27 0	8 17x10^{-9}
20	21 0	7 88x10^{-9}
40	11 5	7 19x10^{-9}
80	6 4	6 46x10^{-9}
150	3 9	5 66x10^{-9}
300	2 3	4 80x10^{-9}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	125 0	5 30x10^{-9}
10	---	---
15	42 0	5 25x10^{-9}
20	---	---
40	16 0	5 17x10^{-9}
80	8 1	5 11x10^{-9}
150	4 4	5 00x10^{-9}
300	2 3	4 80x10^{-9}

Figure 25. Permeability-Gradient Relationship for Soil Compacted Wet of Optimum (Sample No. 1)

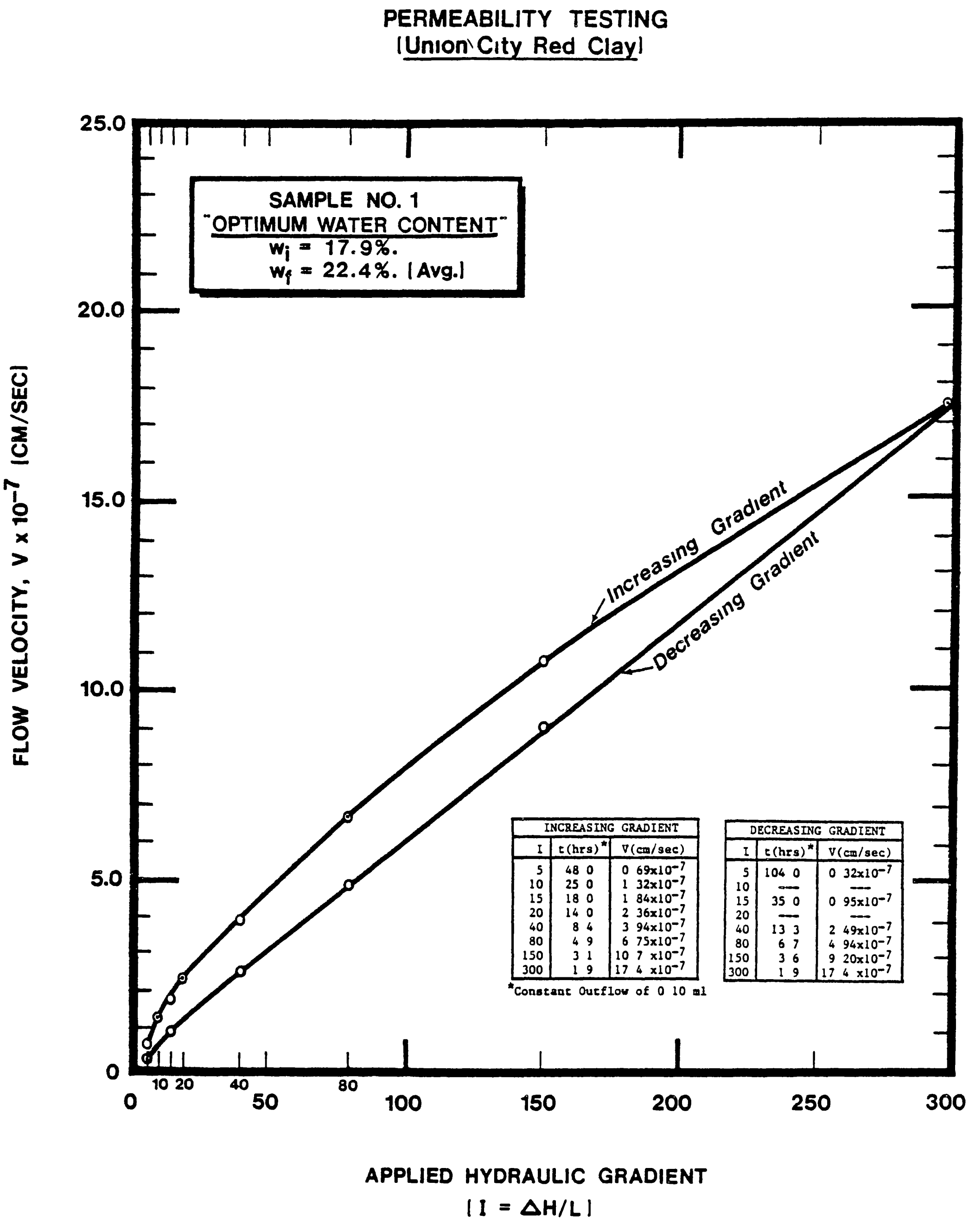


INCREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	72 0	9 19x10^{-9}
10	38.0	8 71x10^{-9}
15	27 0	8 17x10^{-9}
20	21 0	7 88x10^{-9}
40	11 5	7 19x10^{-9}
80	6 2	6 67x10^{-9}
150	3 6	6 13x10^{-9}
300	2 0	5 52x10^{-9}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	K(cm/sec)
5	113 0	5 86x10^{-9}
10	---	---
15	38 0	5 81x10^{-9}
20	---	---
40	14 3	5 79x10^{-9}
80	7 2	5 75x10^{-9}
150	3 9	5 66x10^{-9}
300	2 0	5 52x10^{-9}

Figure 26. Permeability-Gradient Relationship for Soil Compacted Wet of Optimum (Sample No. 2)

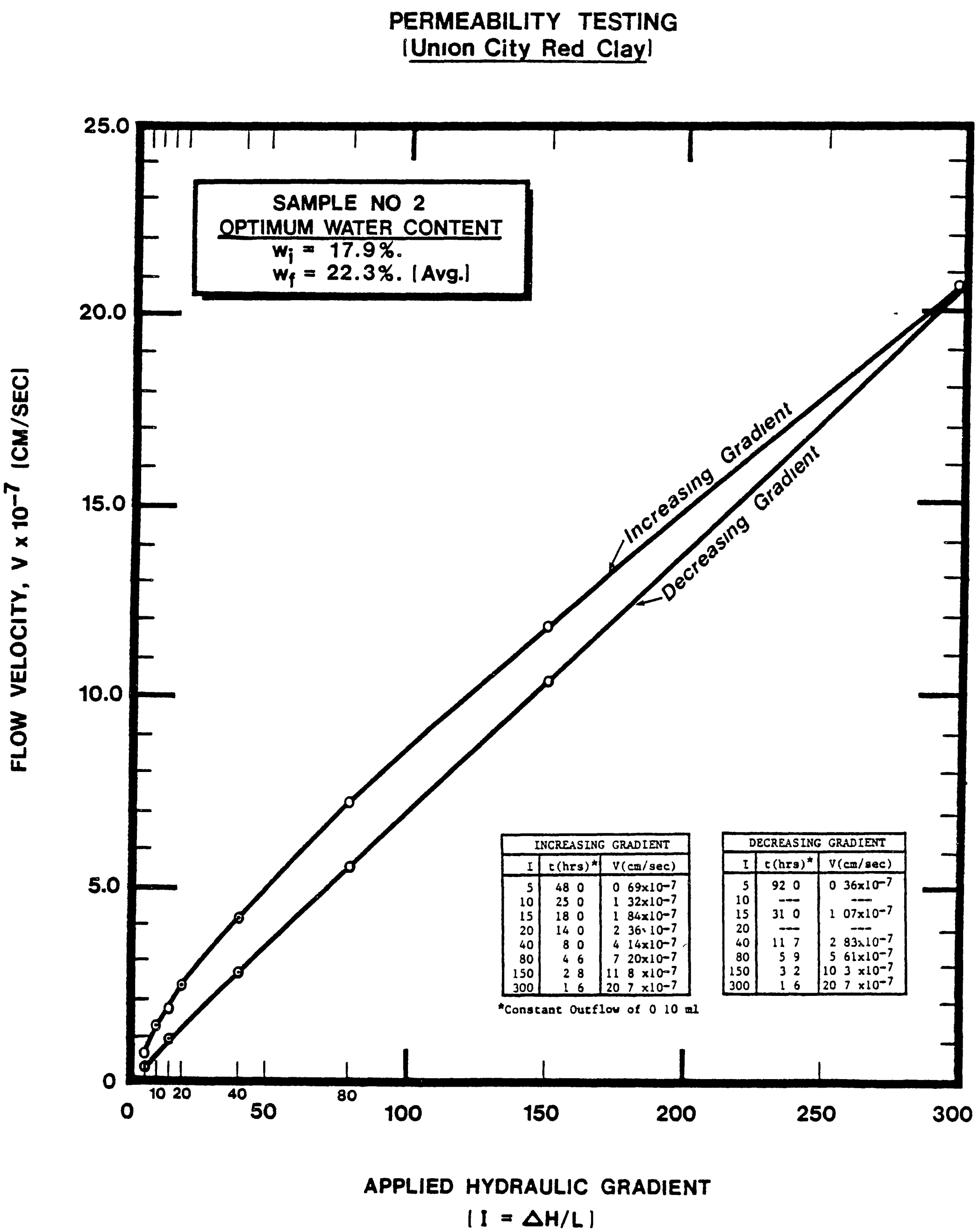


INCREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	48 0	0 69x10^{-7}
10	25 0	1 32x10^{-7}
15	18 0	1 84x10^{-7}
20	14 0	2 36x10^{-7}
40	8 4	3 94x10^{-7}
80	4 9	6 75x10^{-7}
150	3 1	10 7 x10^{-7}
300	1 9	17 4 x10^{-7}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	104 0	0 32x10^{-7}
10	----	----
15	35 0	0 95x10^{-7}
20	----	----
40	13 3	2 49x10^{-7}
80	6 7	4 94x10^{-7}
150	3 6	9 20x10^{-7}
300	1 9	17 4 x10^{-7}

Figure 27. Velocity-Gradient Relationship for Soil Compacted at Optimum Water Content (Sample No. 1)



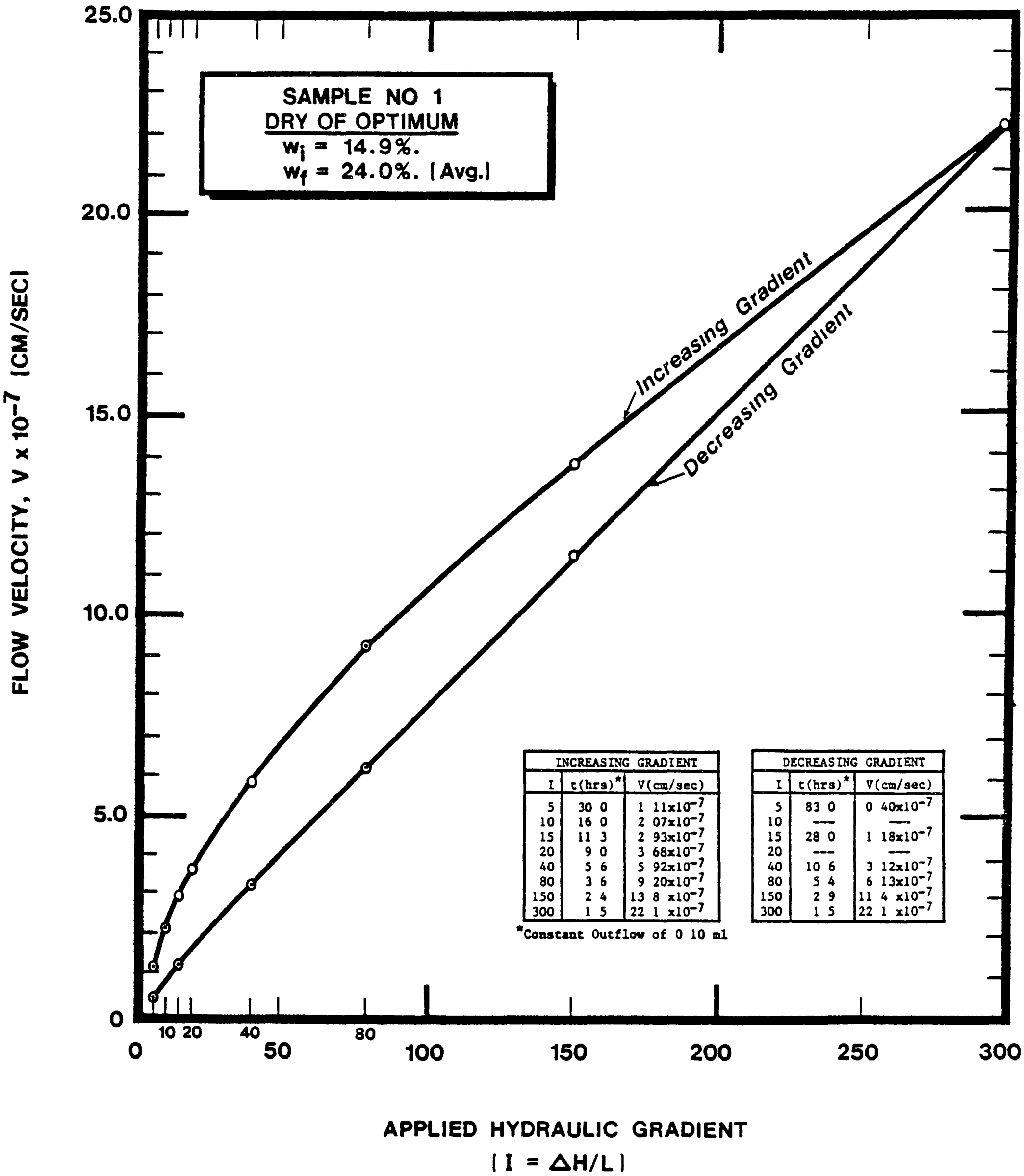
INCREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	48 0	0 69x10^{-7}
10	25 0	1 32x10^{-7}
15	18 0	1 84x10^{-7}
20	14 0	2 36x10^{-7}
40	8 0	4 14x10^{-7}
80	4 6	7 20x10^{-7}
150	2 8	11 8 x10^{-7}
300	1 6	20 7 x10^{-7}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	92 0	0 36x10^{-7}
10	---	---
15	31 0	1 07x10^{-7}
20	---	---
40	11 7	2 83x10^{-7}
80	5 9	5 61x10^{-7}
150	3 2	10 3 x10^{-7}
300	1 6	20 7 x10^{-7}

Figure 28. Velocity-Gradient Relationship for Soil Compacted at Optimum Water Content (Sample No. 2)

PERMEABILITY TESTING
(Union City Red Clay)

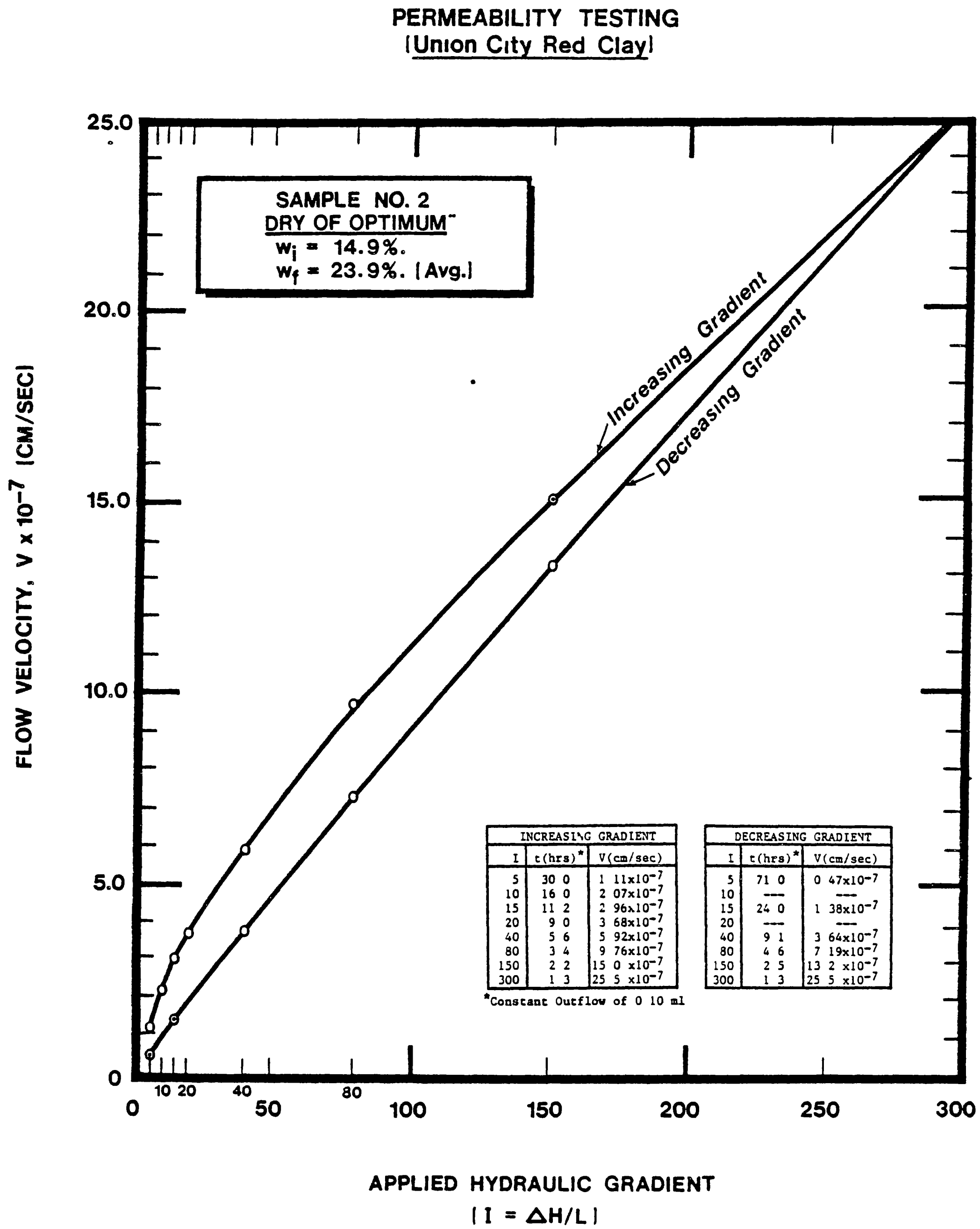


INCREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	30 0	1 11x10^{-7}
10	16 0	2 07x10^{-7}
15	11 3	2 93x10^{-7}
20	9 0	3 68x10^{-7}
40	5 6	5 92x10^{-7}
80	3 6	9 20x10^{-7}
150	2 4	13 8 x10^{-7}
300	1 5	22 1 x10^{-7}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	83 0	0 40x10^{-7}
10	---	---
15	28 0	1 18x10^{-7}
20	---	---
40	10 6	3 12x10^{-7}
80	5 4	6 13x10^{-7}
150	2 9	11 4 x10^{-7}
300	1 5	22 1 x10^{-7}

Figure 29. Velocity-Gradient Relationship for Soil Compacted Dry of Optimum (Sample No. 1)

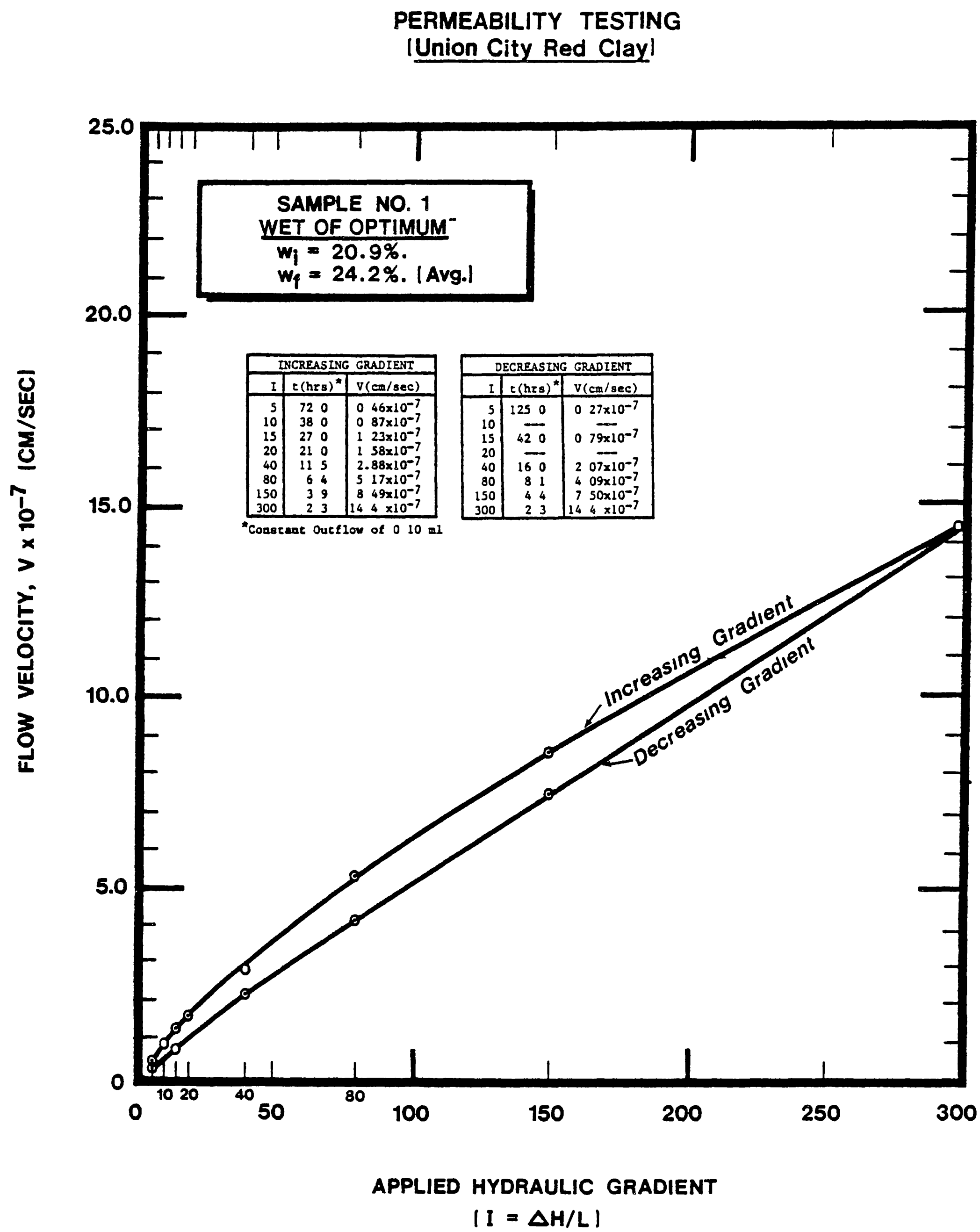


INCREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	30 0	1 11x10^{-7}
10	16 0	2 07x10^{-7}
15	11 2	2 96x10^{-7}
20	9 0	3 68x10^{-7}
40	5 6	5 92x10^{-7}
80	3 4	9 76x10^{-7}
150	2 2	15 0 x10^{-7}
300	1 3	25 5 x10^{-7}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	71 0	0 47x10^{-7}
10	---	---
15	24 0	1 38x10^{-7}
20	---	---
40	9 1	3 64x10^{-7}
80	4 6	7 19x10^{-7}
150	2 5	13 2 x10^{-7}
300	1 3	25 5 x10^{-7}

Figure 30. Velocity-Gradient Relationship for Soil Compacted Dry of Optimum (Sample No. 2)

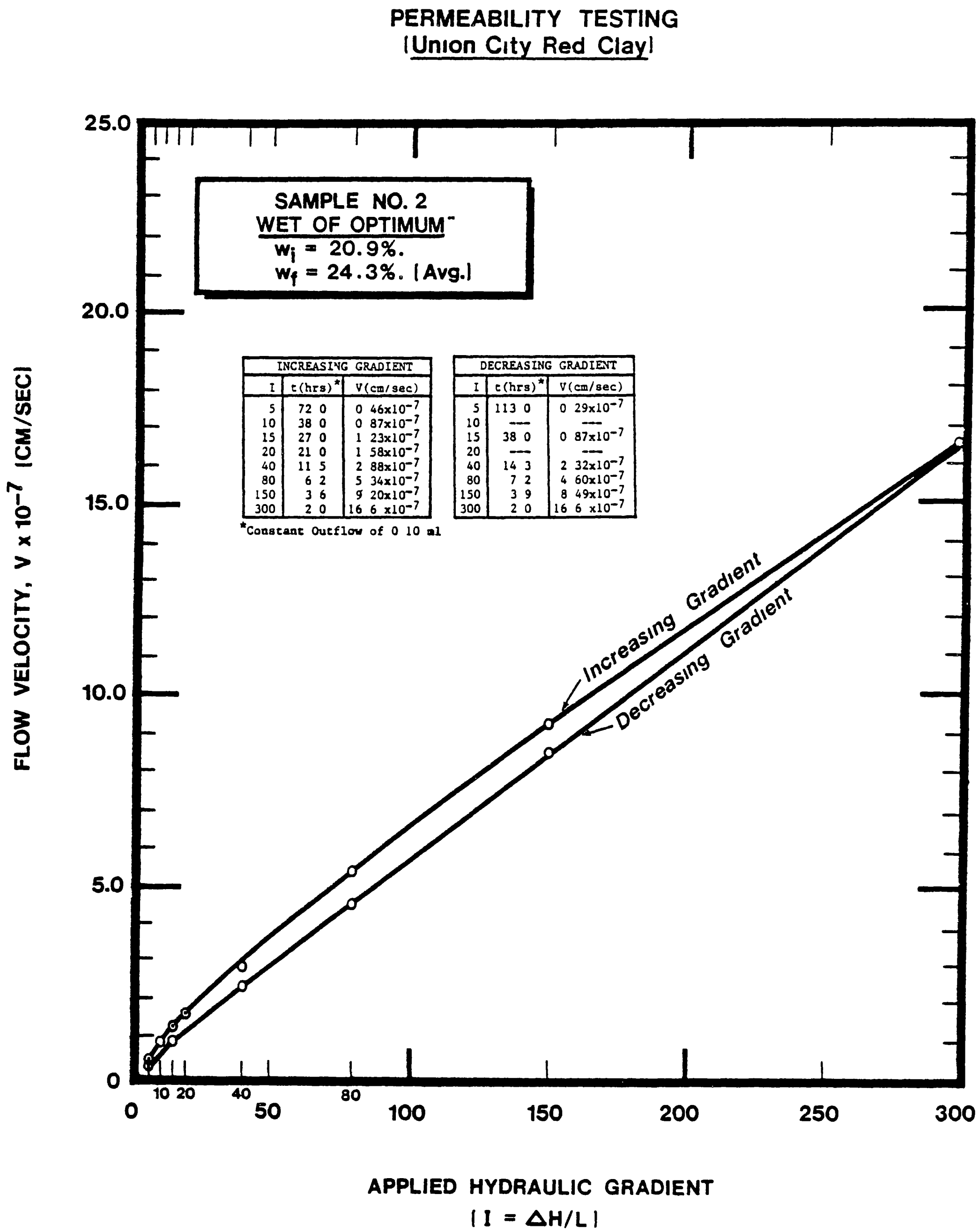


INCREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	72 0	0 46x10^{-7}
10	38 0	0 87x10^{-7}
15	27 0	1 23x10^{-7}
20	21 0	1 58x10^{-7}
40	11 5	2.88x10^{-7}
80	6 4	5 17x10^{-7}
150	3 9	8 49x10^{-7}
300	2 3	14 4 x10^{-7}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	125 0	0 27x10^{-7}
10	—	—
15	42 0	0 79x10^{-7}
20	—	—
40	16 0	2 07x10^{-7}
80	8 1	4 09x10^{-7}
150	4 4	7 50x10^{-7}
300	2 3	14 4 x10^{-7}

Figure 31. Velocity-Gradient Relationship for Soil Compacted Wet of Optimum (Sample No. 1)



INCREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	72 0	0 46x10^{-7}
10	38 0	0 87x10^{-7}
15	27 0	1 23x10^{-7}
20	21 0	1 58x10^{-7}
40	11 5	2 88x10^{-7}
80	6 2	5 34x10^{-7}
150	3 6	9 20x10^{-7}
300	2 0	16 6 x10^{-7}

*Constant Outflow of 0 10 ml

DECREASING GRADIENT		
I	t(hrs)*	V(cm/sec)
5	113 0	0 29x10^{-7}
10	---	---
15	38 0	0 87x10^{-7}
20	---	---
40	14 3	2 32x10^{-7}
80	7 2	4 60x10^{-7}
150	3 9	8 49x10^{-7}
300	2 0	16 6 x10^{-7}

Figure 32. Velocity-Gradient Relationship for Soil Compacted Wet of Optimum (Sample No. 2)

at optimum water content was on the order of 10^{-8} cm/sec at the start of tests ($i = 5$), and decreased to a value on the order of 10^{-9} cm/sec ($i = 300$). The coefficient of permeability of samples compacted wet of optimum was on the order of 10^{-9} cm/sec throughout tests.

Analysis of Test Results

In the preceding sections, data were presented regarding initial soil parameters as well as the results of permeability tests conducted on remolded soil samples. Data regarding the final water content of compacted samples were presented in Table III (shown in Figures 18 - 20). Back-pressure saturation data, showing the average rate and volume of flow into samples, were presented in Figure 17. Separate plots of soil permeability versus hydraulic gradient are shown for each placement condition in Figures 21 - 26. Similar plots of flow velocity versus hydraulic gradient are shown for each placement condition in Figures 27 - 32. Steady-state flow times (0.10 ml outflow) measured in experiments under the conditions of increasing and decreasing hydraulic gradients are summarized on each plot.

As may be seen from the preceding data, a nonlinear increase in flow velocity and decreasing soil permeability was observed in all experiments with strictly increasing hydraulic gradients. This behavior was reflected by a nonlinear v-i relationship (concaving downward) in which the flow velocity was not proportional to the hydraulic gradient, but increased less rapidly than the gradient. Further, this behavior was consistently reflected throughout the range in hydraulic gradients ($i = 5$-300), but was most pronounced for a low-to-intermediate range in gradients ($i = 5$-150). A relatively linear v-i relationship and essentially constant permeability was obtained for all samples under decreasing hydraulic gradients. This data appears consistent with consolidation of samples under strictly increasing hydraulic gradients, which was also reflected in the linearly decreasing moisture profile from top to bottom across samples at the end of tests.

For a range in hydraulic gradients of 5 to 300, and for the conditions of increasing then decreasing hydraulic gradients, the presence of an "initial" or "threshold" gradient in samples was not indicated for any placement condition. The existence of a threshold gradient (i.e., minimum gradient to initiate the permeation process) would generally be reflected by a nonlinear v-i relationship (concaving upward) in which the flow velocity was not proportional to the gradient, but increased <u>more rapidly</u> than the gradient. This is in contrast to the case observed in this study, where the flow velocity increased <u>less rapidly</u> than the gradient. The v-i relationship described above typically has an intercept with the i-axis at a value i_o (i.e., threshold gradient) greater than zero for which a flow velocity of zero is indicated.

As previously mentioned, a decreasing permeability was measured with increasing hydraulic gradients for all samples and placement conditions. In all cases, the lowest permeability value calculated for samples was for a gradient of 300. A slightly higher coefficient of permeability was generally calculated for samples at the end of testing under decreasing hydraulic gradients. This type behavior could be attributed to elastic deformations occurring in the soil under increasing hydraulic gradients, resulting in a reduced void ratio as soil particles are compressed more and more tightly together. During the period of decreasing hydraulic gradients, the soil is effectively "unloaded" and compressive stresses acting on the soil particles would be reduced such that more area (void space) becomes available for flow. Some swelling might also occur in soil samples.

The coefficient of permeability calculated for samples at hydraulic gradients of 0, 15, and 300 are summarized in Table V. In each case, permeability is given for the condition of both increasing and decreasing hydraulic gradients, to allow a relative comparison of test results. The permeability values given in Table V for a gradient of zero were estimated by extrapolation from plots of soil permeability versus hydraulic gradient, and are indicative of the "intrinsic" permeability of the soil. The values given in Table V for a gradient of 15 correspond to the permeability of the soil at the maximum practical field gradient. The final permeability value given in Table V is for a gradient of 300, which was the maximum hydraulic gradient used in testing.

Analysis of the data shown in Table V indicates that the average percent difference in the estimated value of intrinsic permeability from start to finish of tests was greatest for samples compacted dry of optimum (62.3%). Samples compacted wet of optimum showed the least average percent difference (40.8%) in the estimated value of intrinsic permeability. An average percent difference of 53.0% was calculated for samples compacted at optimum water content. The average percent difference in the value of permeability calculated for samples at a gradient of 15 (maximum practical field gradient) corresponded to 56.5% (dry of optimum), 45.5% (optimum water content), and 32.3% (wet of optimum).

The same trends in permeability data with regard to placement condition were generally observed throughout the range in test hydraulic

TABLE V

CALCULATED SOIL PERMEABILITY VALUES FOR
VARIOUS PLACEMENT CONDITIONS[1]

UNION CITY RED CLAY					
Placement Condition	Permeability Value (K_I)	SAMPLE NO. 1		SAMPLE NO. 2	
		Increasing I	Decreasing I	Increasing I	Decreasing I
OPTIMUM WATER CONTENT	K_0*	1.45×10^{-8}	6.40×10^{-9}	1.45×10^{-8}	7.25×10^{-9}
	K_{15}**	1.23×10^{-8}	6.30×10^{-9}	1.23×10^{-8}	7.12×10^{-9}
	K_{300}	5.81×10^{-9}	5.81×10^{-9}	6.89×10^{-9}	6.89×10^{-9}
DRY OF OPTIMUM	K_0*	2.30×10^{-8}	8.00×10^{-9}	2.30×10^{-8}	9.35×10^{-9}
	K_{15}**	1.95×10^{-8}	7.88×10^{-9}	1.97×10^{-8}	9.19×10^{-9}
	K_{300}	7.35×10^{-9}	7.35×10^{-9}	8.49×10^{-9}	8.49×10^{-9}
WET OF OPTIMUM	K_0*	9.50×10^{-9}	5.35×10^{-9}	9.50×10^{-9}	5.90×10^{-9}
	K_{15}**	8.17×10^{-9}	5.25×10^{-9}	8.17×10^{-9}	5.81×10^{-9}
	K_{300}	4.80×10^{-9}	4.80×10^{-9}	5.52×10^{-9}	5.52×10^{-9}

[1]Values obtained from K-I plots (refer Figures 21-26) and expressed in cm/sec.

*By Extrapolation.

**Maximum Practical Field Gradient.

gradients. The rather large variation in the permeability of samples compacted dry of optimum was somewhat expected, and appears consistent with collapse of the flocculated (random) structure of soil particles to a more dispersed structure (parallel orientation of soil particles) under increasing hydraulic gradients. The average percent difference calculated in the permeability of samples within the practical range in hydraulic gradients ($i = 5\text{-}15$) was relatively small, and corresponded to values of 14.8% (dry of optimum), 15.2% (optimum water content), and 14.0% (wet of optimum).

Data regarding the total time required for back-pressure saturation and laboratory permeability testing of samples is presented in Table VI. The total times required for permeability testing of compacted soil samples have been summarized in Table VI for the conditions of nonsteady-state and steady-state flow, and both increasing and decreasing hydraulic gradients. The average laboratory testing times at each gradient for the conditions of nonsteady-state flow and increasing hydraulic gradients are given in Table VII. Similar data is presented for the condition of steady-state flow and both increasing and decreasing hydraulic gradients in the individual plots of soil permeability (flow velocity) versus hydraulic gradient.

In the experiments described herein, time periods of approximately 23 days, 27 days, and 34 days were required to test samples compacted dry of optimum, at optimum water content, and wet of optimum. For the condition of increasing hydraulic gradients, approximately 60% of the total time required for tests consisted of nonsteady-state flow measurements (consolidation occurring), while this trend was essentially reversed for the condition of decreasing hydraulic gradients. Samples

TABLE VI

TOTAL LABORATORY PERMEABILITY TESTING TIMES FOR
VARIOUS PLACEMENT CONDITIONS

UNION CITY RED CLAY							
Placement Condition	Back-Pressure Saturation[1]	Permeability Testing[2]					Total Testing Time
		Flow Condition	Increasing Gradient		Decreasing Gradient		
			Time (t)	% of Total (t/T)	Time (t)	% of Total (t/T)	
OPTIMUM WATER CONTENT	108.0 hrs	Nonsteady	187.2 hrs	60.4	78.3 hrs	33.8	649.4 hrs (27.1 days)
		Steady	122.7 hrs	39.6	153.2 hrs	66.2	
		Total (T)	309.9 hrs	100.0	231.5 hrs	100.0	
DRY OF OPTIMUM	156.0 hrs	Nonsteady	124.8 hrs	61.2	65.5 hrs	35.2	545.9 hrs (22.7 days)
		Steady	79.0 hrs	38.8	120.6 hrs	64.8	
		Total (T)	203.8 hrs	100.0	186.1 hrs	100.0	
WET OF OPTIMUM	84.0 hrs	Nonsteady	275.6 hrs	61.6	95.1 hrs	33.8	812.3 hrs (33.8 days)
		Steady	171.8 hrs	38.4	186.0 hrs	66.2	
		Total (T)	447.4 hrs	100.0	281.1 hrs	100.0	

[1]Refer to Figure 17.

[2]Average values given. For Nonsteady-State Increasing Gradient values, refer to Table VII.

TABLE VII

PERMEABILITY TESTING TIMES AND PRESSURE DIFFERENCES FOR VARIOUS PLACEMENT CONDITIONS AND HYDRAULIC GRADIENTS (Increasing Gradient)

UNION CITY RED CLAY						
Placement Condition	Hydraulic Gradient	Pressure Difference (Δp)	% of Maximum Difference (Δp/ΔP)	Avg Testing Time (t)[1]	% of Total Time (t/T)	
					Incremental	Cumulative
OPTIMUM WATER CONTENT	5	0.5 psi	1 7	72 0 hrs	38 5	38 5
	10	1 0 psi	3 3	37 5 hrs	20.0	58 5
	15	1 5 psi	5 0	27 0 hrs	14.4	72 9
	20	2.0 psi	6 7	21.0 hrs	11.2	84 1
	40	4 0 psi	13 3	12.3 hrs	6 6	90 7
	80	8 0 psi	26.7	7.1 hrs	3 8	94 5
	150	15 0 psi	50 0	5 9 hrs	3.2	97 7
	300	30.0 psi	100.0	4.4 hrs	2.3	100 0
	Maximum Difference (ΔP) = 30.0 psi (2 2 tsf)			Total Time (T) = 187.2 hrs (7.8 days)		
DRY OF OPTIMUM	5	0 5 psi	1 7	45 0 hrs	36 1	36 1
	10	1 0 psi	3.3	24 0 hrs	19.2	55 3
	15	1.5 psi	5.0	16 9 hrs	13 5	68 8
	20	2 0 psi	6.7	13.5 hrs	10.8	79 6
	40	4.0 psi	13 3	8 4 hrs	6.7	86 3
	80	8.0 psi	26.7	7 0 hrs	5 6	91 9
	150	15 0 psi	50 0	5 8 hrs	4.7	96 6
	300	30 0 psi	100 0	4 2 hrs	3 4	100 0
	Maximum Difference (ΔP) = 30.0 psi (2 2 tsf)			Total Time (T) = 124 8 hrs (5 2 days)		
WET OF OPTIMUM	5	0 5 psi	1 7	108.0 hrs	39 2	39 2
	10	1 0 psi	3 3	57 0 hrs	20.7	59 9
	15	1 5 psi	5 0	40.5 hrs	14.7	74 6
	20	2.0 psi	6 7	31 5 hrs	11 4	86 0
	40	4 0 psi	13 3	17 3 hrs	6 3	92 3
	80	8.0 psi	26 7	9 5 hrs	3 4	95 7
	150	15 0 psi	50 0	7.5 hrs	2 7	98 4
	300	30.0 psi	100 0	4 3 hrs	1 6	100 0
	Maximum Difference (ΔP) = 30.0 psi (2 2 tsf)			Total Time (T) = 275 6 hrs (11.5 days)		

[1]Values given are for nonsteady-state flow conditions, assuming consolidation occurs only during this time period Refer to Figures 21-32 for constant outflow values

compacted dry of optimum required the longest saturation times but had the shortest overall testing times, while samples compacted wet of optimum required the shortest saturation times but had the longest overall testing times.

For all samples and placement conditions investigated in this study, a nonlinear v-i relationship was observed between flow velocity and hydraulic gradient for the condition of strictly increasing hydraulic gradients. This behavior appears directly related to consolidation due to an unavoidable increase in the effective stress at the outflow end of samples as increased hydraulic gradients are applied. In order to apply increasing gradients, the back pressure at the outflow end of samples must be reduced while maintaining a constant head on samples. For a given hydraulic gradient, the effective stress at any point along the sample is equal to the difference in the chamber (confining) pressure and back pressure within the sample. The effective stress at the outflow end of samples would have a minimum value of 5.5 psi at the start of permeability tests ($i = 5$) and would increase to a maximum value of 35 psi ($i = 300$).

It has been shown that at a given hydraulic gradient (stress condition) the effective stress increases towards the outflow end of samples. In this regard, it would appear logical to assume that consolidation also increases towards the outflow end of samples. This was substantiated by measurement of a linearly decreasing moisture profile from top to bottom across samples at the end of tests (Table III). As drainage was effectively prevented during the back-pressure saturation phase of experiments, minimal (if any) consolidation probably occurred during this phase of tests. Further, it would appear reasonable to assume that during permeability testing, consolidation of samples occurred only during the initial period of nonsteady-state flow (Table VII). During this period, the volume of voids in samples would be in a continually changing (transient) state.

The effects of consolidation were also quantitatively evaluated in terms of changes in the initial void ratio of samples during testing. In this analysis, simplifying assumptions regarding sample saturation and the final water contents shown in Table III were used to calculate the change in the initial void ratio of each portion (top, middle, and bottom) of samples. An initial void ratio of 0.627 was calculated for samples compacted at optimum water content, while a value of 0.677 was calculated for samples compacted dry of optimum and wet of optimum. Assuming incomplete saturation (less than 100%) prior to tests and a final degree of saturation for the entire sample equal to that calculated for the top portion, the change in the initial void ratio of various portions of samples for this case are given in Table VIII. In the analysis, the assumption that no change in void ratio occurred in the top portion of samples is somewhat unrealistic, in that the top portion of samples would be consolidated under a minimum 5 psi pressure difference in tests.

Similar void ratio data is presented in Table IX assuming complete saturation (100%) prior to tests and a change in the initial void ratio of all portions of samples. This analysis is probably not totally correct either, as the change in the initial void ratio at the top of samples appears disproportionate to that at the bottom (i.e., one-half), considering the maximum effective stress at the bottom of samples was seven times that applied at the top (5 psi versus 35 psi). In both

TABLE VIII

VOID RATIO DATA
(S_f < 100%)

UNION CITY RED CLAY				
Placement Condition	Initial Parameters	Portion of Sample	Final Void Ratio, e_f[2]	Change in Void Ratio, Δe
OPTIMUM WATER CONTENT	γ_d = 105.1 pcf[1] w_i = 17.9% S_i = 78.1% e_o = 0.627	TOP	0.627	None
		MIDDLE	0.623	0.004
		BOTTOM	0.617	0.010
DRY OF OPTIMUM	γ_d = 102.0 pcf[1] w_i = 14.9% S_i = 60.3% e_o = 0.677	TOP	0.677	None
		MIDDLE	0.670	0.007
		BOTTOM	0.664	0.013
WET OF OPTIMUM	γ_d = 102.0 pcf[1] w_i = 20.9% S_i = 84.6% e_o = 0.677	TOP	0.677	None
		MIDDLE	0.674	0.003
		BOTTOM	0.668	0.009

[1]Average values given.

[2]Values calculated assuming a final degree of saturation (S_f) for entire sample equal to S_f of Top Portion. Refer to Table III for Final Water Content and Degree of Saturation Data.

TABLE IX

VOID RATIO DATA
(S_f = 100%)

UNION CITY RED CLAY						
Placement Condition	Initial Parameters	Portion of Sample	SAMPLE NO. 1		SAMPLE NO. 2	
			Final Void Ratio, e_f[2]	Change in Void Ratio, Δe	Final Void Ratio, e_f[2]	Change in Void Ratio, Δe
OPTIMUM WATER CONTENT	γ_d = 105.1 pcf[1] w_i = 17.9% S_i = 78.1% e_o = 0.627	TOP	0.619	0.008	0.616	0.011
		MIDDLE	0.614	0.013	0.611	0.016
		BOTTOM	0.609	0.018	0.606	0.021
DRY OF OPTIMUM	γ_d = 102.0 pcf[1] w_i = 14.9% S_i = 60.3% e_o = 0.677	TOP	0.663	0.014	0.661	0.016
		MIDDLE	0.656	0.021	0.654	0.023
		BOTTOM	0.651	0.026	0.648	0.029
WET OF OPTIMUM	γ_d = 102.0 pcf[1] w_i = 20.9% S_i = 84.6% e_o = 0.677	TOP	0.668	0.009	0.670	0.007
		MIDDLE	0.664	0.013	0.666	0.011
		BOTTOM	0.659	0.018	0.661	0.016

[1]Average values given.

[2]Values calculated assuming a final degree of saturation for entire sample equal to 100 percent.

analysis, the change in the initial void ratio from top to bottom across samples was essentially linear. Samples compacted dry of optimum showed the greatest change in void ratio, while samples compacted wet of optimum showed the least change in void ratio.

The practical significance of the decreasing gradient curve shown in each plot of soil permeability (flow velocity) versus hydraulic gradient (Figures 21 - 32) is that it provides a qualitative means for estimating the magnitude of error introduced in permeability testing under excessive hydraulic gradients. In the case of relatively impervious clays, substantial time may be needed to obtain measurable flow under low hydraulic gradients. In an effort to reduce testing time, excessive or large hydraulic gradients are often imposed on samples using air pressure. If Darcy's law is valid, such gradients will not alter the measured permeability. However, research has shown that excessive hydraulic gradients can cause consolidation or particle migration and the clogging of soil voids, resulting in a reduced soil permeability. As effluent from samples remained clear throughout tests, the possibility that particle migration occurred in the experiments described herein seems remote.

Using the high-gradient test method, a constant coefficient of permeability is normally calculated for the soil based on specific test conditions, and a linear flow relationship is "assumed" over a wide range in gradients. Although the sequence of load application is important when discussing consolidation, differences in the "true" flow characteristics of the soil used in this study versus that which might be "predicted" using the high-gradient test method, could be approximated over a range in gradients by the relative differences in the increasing and decreasing gradient curves of Figures 21 - 26. As shown in the permeability plots of Figures 21 - 26, relatively large differences appear to exist in soil permeability at low hydraulic gradients, the magnitude of which decreases with increasing hydraulic gradients. Within the practical range in hydraulic gradients ($i = 5$-15), data previously presented in this chapter indicated percent differences in permeability in excess of 50%.

Similar inferences regarding flow characteristics of the soil could be made from the increasing gradient and decreasing gradient curves shown in each plot of flow velocity versus hydraulic gradient. It must be cautioned, however, that a direct relationship does not exist between the magnitude of "differences" in the flow velocity curves and permeability curves at a particular hydraulic gradient. That is, large differences in the flow velocity curves at high gradients produce relatively small differences in the soil permeability curves, while large differences in the flow velocity curves at low hydraulic gradients produce relatively large differences in the soil permeability curves. Further, as the decreasing gradient curve reflects flow characteristics under the condition of full consolidation while the increasing gradient curve reflects flow characteristics under the condition of incremental (partial) consolidation, the total area between the two curves would be indicative of the relative amount of consolidation which occurred in each sample. When these areas are compared, the total amount of consolidation was greatest for samples compacted dry of optimum (related to collapse of flocculated structure), while samples compacted wet of optimum experienced the least consolidation.

CHAPTER V

CONCLUSIONS

The primary objective of this research was to evaluate the existence of general non-Darcy behavior, and specifically threshold gradients, for water flow in compacted clay soils typically used in Oklahoma (and elsewhere) for construction of earth structures. The existence of an initial or threshold gradient was tested for through conduct of laboratory permeability tests on clay samples compacted to various known placement conditions prior to testing, using small hydraulic gradients within the practical range of engineering interest. The soil used in this study had its origin in Oklahoma, and consisted of a well-graded silty clay of low plasticity with stable clay mineralogy (illite). No threshold gradients were found in the soil tested in this study, and it was concluded that water was not more tightly held by the clay surfaces (or other phenomenon) to the extent a finite pressure is needed to initiate the flow of water.

In accomplishing this research, a laboratory test procedure has been described for evaluating the hydraulic conductivity of a fine-grained soil using a constant-head triaxial cell test apparatus. Specific emphasis has been placed on use of this equipment to evaluate the long-term performance of new and existing waste-disposal facilities. Hydraulic gradients generally outside the practical range of engineering interest were also applied to compacted samples, to investigate the influence of magnitude of hydraulic gradient on the measured permeability using the triaxial device. In this study, it was concluded that the effects of magnitude of hydraulic gradient and placement condition on permeability were relatively small for the soil tested, and insignificant from a design point of view. The triaxial device could cause significant errors in the measurement of threshold gradients, due to unavoidable consolidation of samples during testing.

The following specific conclusions and recommendations pertaining to soil permeability and its determination using the constant-head triaxial cell test apparatus follow directly from the results of this investigation.

1. For a range in hydraulic gradients of 5 to 300, and for the test conditions of increasing then decreasing hydraulic gradients, the presence of an "initial" or "threshold" gradient in samples was not indicated for any placement condition. Darcy's law states that the flow velocity (v) is directly proportional to the applied hydraulic gradient (i). The existence of a threshold gradient (i.e., minimum gradient to initiate flow) would generally be reflected by a nonlinear v-i relationship (concaving upward) in which the flow velocity was not proportional to the gradient, but increased more rapidly than the gradient. The v-i relationship described above typically has an intercept with the i-axis at a value i_o (i.e., threshold gradient) greater than zero for which a flow velocity of zero is indicated.

For the soil tested in this study, it was concluded that water was not more tightly held by the clay surfaces or other phenomenon to the extent that a finite pressure is needed to initiate the flow of water. Sufficient time was provided in tests at low gradients for any time-dependent phenomenon to manifest itself, and the possibility of hysteretic flow was investigated under the test conditions of increasing and decreasing hydraulic gradients. Although the existence of threshold gradients was not investigated for hydraulic gradients less than 5, the practical significance of a threshold gradient in soil much less than this value, for anything but research applications, is questionable. Of the threshold gradients reported in the literature, most values are for gradients less than 3, with a majority for gradients less than 1.

2. A nonlinear increase in flow velocity and decreasing soil permeability was observed in all experiments with strictly increasing hydraulic gradients. This behavior was reflected by a nonlinear v-i relationship (concaving downward) in which the flow velocity was not proportional to the hydraulic gradient, but increased <u>less rapidly</u> than the gradient. This is in contrast to the case observed for threshold gradients, in which the flow velocity increases <u>more rapidly</u> than the gradient. This behavior was consistently reflected throughout the range in hydraulic gradients ($i = 5$–300), but was most pronounced for a low-to-intermediate range in hydraulic gradients ($i = 5$–150). A relatively linear v-i relationship and essentially constant permeability was obtained for all samples under decreasing hydraulic gradients. This data is consistent with consolidation of samples under strictly increasing hydraulic gradients.

3. Consolidation is directly related to an unavoidable increase in the effective stress at the outflow end of samples as increased hydraulic gradients are applied. In order to apply increasing gradients, the back pressure at the outflow end of samples must be reduced while maintaining a constant head on samples. For a given hydraulic gradient, the

effective stress at any point along the sample is equal to the difference in chamber (confining) pressure and back pressure within the sample. Since portions nearest the outflow end of samples are stressed (consolidated) to the greatest extent during testing, the final water content measured at the outflow end of samples should be less than that measured for the middle and top portions.

A linearly decreasing moisture profile, in fact, was observed from top to bottom across samples at the end of tests. The effects of consolidation were also quantitatively evaluated in terms of changes in the initial void ratio of samples during testing. The change in the initial void ratio from top to bottom across samples was relatively uniform. In these experiments, the effective stress at the outflow end of samples had a minimum value of 5.5 psi at the start of tests ($i = 5$), and increased to a value of 35 psi ($i = 300$). The top portion of samples was consolidated under a 5 psi pressure difference in tests.

4. Although the best overall laboratory device to determine permeability appears to be the triaxial device with back-pressure capability, this device is not particularly well suited for measurement of threshold gradients. In order to maintain contact between the membrane and soil sample, the pressure in the cell fluid must be higher than the pore pressure in the test sample. At any gradient, the effective stress at the outflow end of the sample will always be larger than that at the top. In this regard, the effective confining pressure at the bottom of samples cannot be less than the pressure drop across the sample. It would appear that even minor consolidation in samples under low hydraulic gradients (effective confining pressures) could cause significant errors in the measurement of threshold gradients. Further, the

consolidation of samples would tend to produce a concaving downward v-i relationship, while the presence of a threshold gradient would be indicated by a concaving upward v-i relationship.

5. In this study, it was concluded that the effects of magnitude of hydraulic gradient and placement condition on permeability were relatively small for the soil tested, and insignificant from a design point of view. The value of "intrinsic" permeability ($i = 0$) estimated for samples compacted dry of optimum, at optimum water content, and wet of optimum were 2.30×10^{-8} cm/sec, 1.45×10^{-8} cm/sec, and 9.50×10^{-9} cm/sec, respectively. These values were only two-to-three times higher than the value calculated at a gradient of 300, with the greatest change in permeability for samples compacted dry of optimum. Samples compacted wet of optimum showed the least change in permeability throughout the range in test gradients ($i = 5$-300). The current state of the art in permeability testing of fine-grained soils is such that reliability is normally given only to the "order-of-magnitude" determined for the soil in the laboratory. As used herein, an order-of-magnitude change would require a 10-fold variation in the value of soil permeability.

Differences in the "true" flow characteristics of the soil used in this study versus that which might be "predicted" using a high-gradient test method were inferred over a range in gradients by the relative differences in the increasing and decreasing gradient curves. Relatively large differences appear to exist in the soil permeability curves at low hydraulic gradients, the magnitude of which decreases with increasing hydraulic gradients. However, increasing gradient permeability values were only two-to-three times higher than decreasing gradient values. Although the values "predicted" by the decreasing gradient curve are unconservative, these differences do not represent order-of-magnitude changes in permeability. The largest differences occurred in samples compacted dry of optimum, while relatively little difference was noted for samples compacted wet of optimum. For the total range in test hydraulic gradients ($i = 5$-300) the data indicated no order-of-magnitude changes in soil permeability, and only insignificant changes within the practical range in hydraulic gradients ($i = 5$-15).

6. In this paper, a laboratory test procedure has been described to evaluate the performance of waste-disposal facilities. It has been demonstrated that reproducible hydraulic conductivity measurements can be obtained using a constant-head triaxial cell apparatus. Advantages of this type of testing include better sample saturation and elimination of short-circuited permeant flow. In addition, this test provides a proper means for simulation of actual field loading and drainage conditions. Seepage quantity can be estimated under various field placement conditions. Permeant testing using the triaxial device is directly applicable to several areas of waste-disposal facility design, including selection of suitable soil liner material, determination of required liner thickness, and determination of acceptable impoundment fluid levels.

However, there are certain limitations that should be considered in implementing the test. Using the triaxial cell apparatus, the only way to test samples from shallow depths without exceeding the confining pressure in the field is to use low gradients. For some clays of extremely low permeability, a permeability test at low hydraulic gradients may be somewhat impractical, due to excessive testing time required to establish steady-state flow. Field-time conditions are accelerated in the triaxial test, often to the extent that several decades of fluid flow are predicted in only a few days (or weeks) of laboratory testing. This normally is accomplished by permeating fluids through the soil under higher gradients than would typically occur in the field.

The use of a large pressure difference (hydraulic gradient) across the sample leads to a high effective pressure at the outflow end of the sample. These high effective stresses can close any cracks or fissures that might be present or reduce the void ratio in a homogeneous sample, either of which would lead to a measured hydraulic conductivity that is too low. If high gradients are used, it is recommended to start the test at a low gradient and to increase it progressively to the value needed to complete the test. By doing this, one at least has some idea of the effect of hydraulic gradient on the permeability of the soil. The applicable range in field hydraulic gradients for clay-lined lagoons and waste-disposal pits is generally restricted to a gradient of 15 or less, based on design and operational considerations.

7. Since the increasing gradient curve reflects flow characteristics for the condition of incremental (partial) sample consolidation while the decreasing gradient curve reflects flow characteristics for the condition of full sample consolidation, the total area between the curves would indicate the relative amount of consolidation which occurred in each sample. When these areas were compared, the total amount of consolidation was greatest for samples compacted dry of optimum (related to collapse of flocculated structure), while samples compacted wet of optimum experienced the least consolidation. The greatest change in void ratio occurred in samples compacted dry of optimum, while samples compacted wet of optimum showed the least change in void ratio.

CHAPTER VI

RECOMMENDATIONS FOR FUTURE RESEARCH

1. Although no threshold gradients were indicated for the particular soil tested in this study (an illite), the evidence is inconclusive and further research is necessary to more fully evaluate the existence of threshold gradients. If the surfaces of clay particles do, in fact, adsorb water to the extent that a finite pressure (i.e., threshold gradient) must be exceeded before flow will occur, other clay soils with different clay mineralogy should be tested.

2. If future tests are conducted at low gradients using the constant-head triaxial cell test apparatus described herein, more sophisticated instrumentation of equipment is recommended. In this regard, more sensitive pressure gages should be used, perhaps incremented to 0.1 psi (25 psi full-scale reading). Further, if the diameter of the outer burettes was reduced, more frequent water level readings could be made for the same volume of outflow. With the present setup, a horizontal sliding bar or similar device is needed to facilitate the accurate reading of water levels in burettes using the ruled scales provided on the pressure saturation device. Alternatively, ruled scales could be inscribed on each burette.

3. The consolidation of samples observed in this study is directly related to a necessary increase in the lateral effective stress at the outflow end of samples as hydraulic gradients are applied. As

previously discussed, the lateral effective stress at the outflow end of the sample cannot be less than the pressure drop across the sample. If future tests are conducted using the modified triaxial cell apparatus, consideration should be given to the use of thinner samples. For a given hydraulic gradient, this would allow a smaller pressure difference across the sample, and thus a reduced lateral effective stress at the outflow end of the sample.

4. The "flow-through" oscillating permeameter device described in Chapter III (shown in Figure 6) might be more suited to laboratory testing for threshold gradients. Because this is a "fixed-wall" permeameter, any inaccuracies due to lateral consolidation of samples should be avoided.

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