Permeability of Fly Ash and Fly Ash Stabilized Soils

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16. Abstract

Fly ash, to be produced in Arkansas, is a good potential stabilizing agent for many Arkansas Soils. The fly ash, produced from Wyoming low sulfur coal, reacts chemically like quick lime, generating heat and possessing self hardening characteristics.

Two soils, an organic clay and a sand (classified OH and SP-SM by the Unified system) were tested for permeability with the fly ash. As the percent of fly ash increases, the permeability of soil-fly ash mixtures decreases. For the sand, permeability decreased from 3.3×10^{-3} cm/sec for the sand alone to 1.5×10^{-6} cm/sec for a 50% sand, 50% fly ash mixture. For the clay, permeability decreased from 8.0×10^{-6} cm/sec for the clay alone to 2.0×10^{-6} cm/sec for a 50% clay, 50% fly ash mixture. For 100% fly ash samples, cracks developed in the samples creating secondary permeability of 1.2×10^{-3} cm/sec. Cracks in 100% fly ash samples were due to a reaction in the self hardening fly ash. Fly ash deposited in a slurry at a water content of 25 to 40% has a permeability of 3×10^{-6} cm/sec.

Permeability decreased with time within the first seven days. For 16.7 percent fly ash and 83.3 percent sand, permeability decreased from 6×10^{-5} cm/sec to 2×10^{-5} cm/sec.

Effluent from the tests had a high pH, alkalinity, and dissolved solids concentrations.

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PERMEABILITY OF FLY ASH AND FLY ASH STABILIZED SOILS

by
David G. Parker
Sam I. Thornton

FINAL REPORT
HIGHWAY RESEARCH PROJECT 47

conducted for
The Arkansas State Highway Department
in cooperation with
The U.S. Department of Transportation
Federal Highway Administration

December, 1976

SUMMARY

Fly ash, a pozzolanic by-product of coal burning power plants, is an abundant potential source of highway and embankment construction material. Some fly ashes are suitable for use as a supplement or replacement for lime and portland cement in soil stabilization applications. Production of lime and portland cement requires heat and will become more costly as energy costs rise. Fly ash, however, is a by-product of power production.

Production of fly ash in the United States was 3.67×10^{10} kilograms (40.4 million tons) in 1974 and is projected to be 4.53×10^{10} kilograms (50 million tons) by 1980. Less than 15% of the fly ash produced is used in commercial applications. The remainder of the fly ash is wasted, either by sluicing to ponds or hauling to solid waste disposal areas. Disposal operations are quite expensive and require the use of land which could be used for other purposes.

FLY ASH PROPERTIES

Fly ash generally exhibits a wide range in chemical and physical properties. These properties determine the effectiveness of the ash for use in soil stabilization. The characteristics of a particular fly ash is dependent on the coal source, coal preparation procedures, boiler type and the ash collection device.

The fly ash used was collected by an electrostatic precipitator from a 350 megawatt tangential burner boiler. The coal was a low sulfur coal obtained from Campbell County, Wyoming and was pulverized before injection into the burner. The fly ash has a light cream color and particles are spherical in shape. Chemical and physical properties of the fly ash are shown in Summary Table 1.

The fly ash has self-hardening characteristics when mixed with water. Twenty-eight day unconfined compression strengths in excess of 70 kgs/sq cm (1000 psi) were obtained from samples compacted immediately after mixing with water. Furthermore, temperatures up to 66^{o} C (150^{o} F) were observed within 30 minutes after compaction of fly ash soil mixtures.

One possible explanation for the apparent reactivity of the fly ash is the relatively

high calcium oxide (CaO) content of the ash. Most investigators report CaO contents between 1 and 11% while the ash under study has a concentration of 20%. The CaO in the ash may be acting like quick lime, causing the observed temperature increases and enhancing the pozzolanic activity of the other constitutents in the ash.

SUMMARY TABLE 1

PROPERTIES OF FLY ASH

Property	Value
Loss on Ignition	0.0%
pH	11.2
CaO	20.0%
Water Soluble Fraction	1.0%
Specific Gravity	2.75
Maximum Density at 9% moisture (Modified Proctor)	1.89 g/cc

PROPERTIES OF SOILS

Property	Clay	Sand
Percent Sand	0.0%	92.9%
Percent Silt	41.0%	1.8%
Percent Clay	59.0%	5.3%
Liquid Limit	54.0%	Non Plastic
Plastic Index	19.0%	Non Plastic
pH	3.9	4.3

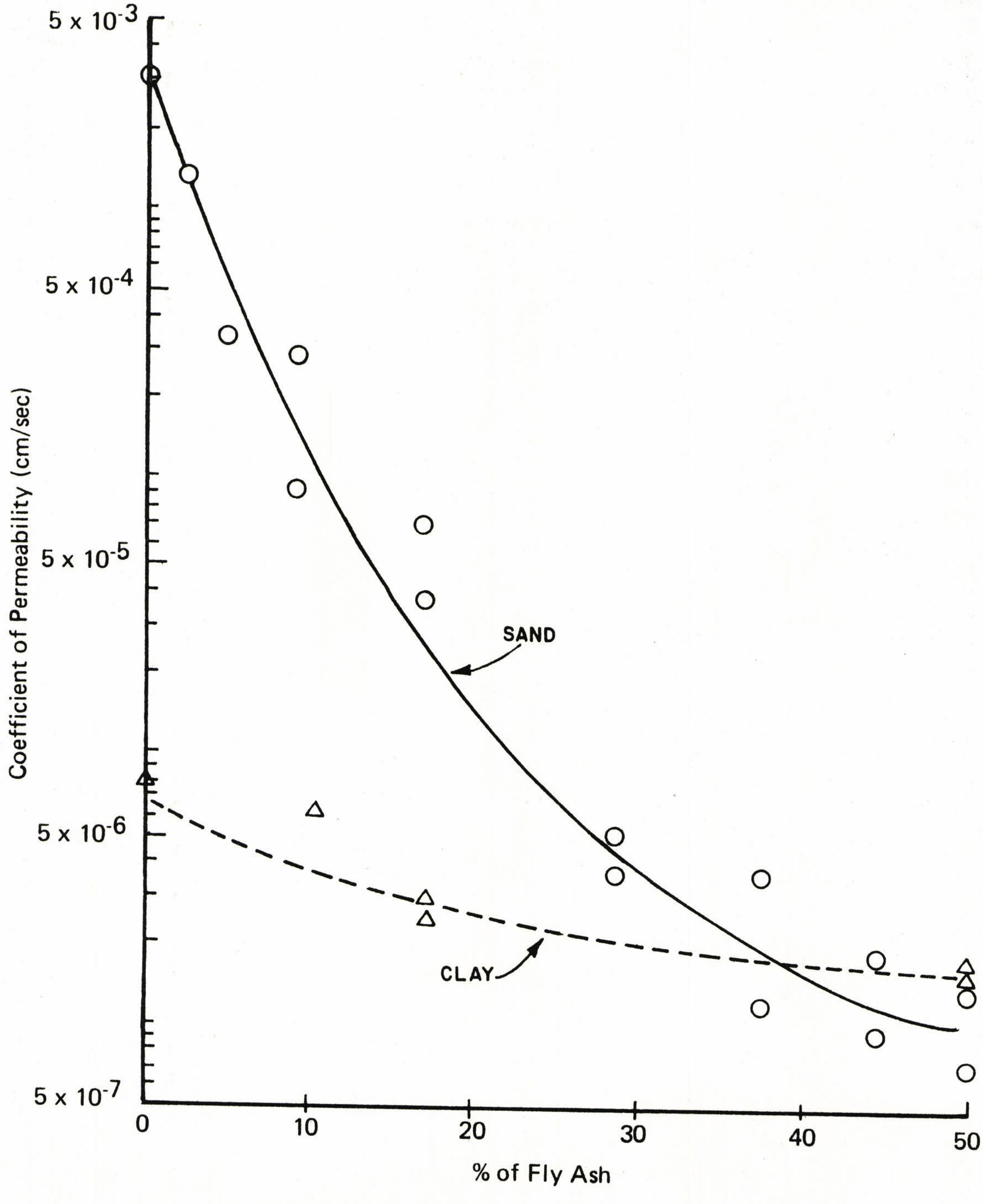
SOIL PROPERTIES

Two Arkansas soils, a clay (OH) and a sand (SP-SM) were tested. The clay, 59% by weight less than 2 micron, contained 11.4% organic material. Liquid limit of the clay was 59 and the plastic index was 19 (Summary Table 1).

Effect of Fly Ash on Permeability of Soil

As the percentage of fly ash increased, the coefficient of permeability of both sand and clay-fly ash mixtures decreased (Summary Figure 1).

The coefficient of permeability of sand-fly ash mixtures decreased from 3.3×10^{-3} cm/sec to 1.5×10^{-6} cm/sec, as the percent of fly ash increased up to 50%. The



Summary Figure 1. Effect of Fly Ash on Permeability of Sand and Clay.

self-hardening process and chemical reaction of fly ash appeared to cement the sand particles and made sand-fly ash mixtures less permeable. The soil-fly ash mixtures were compacted dry according to ASTM-D 2434-68 procedures. The samples were then saturated to between 90% and 100% saturation before testing.

The coefficient of permeability of clay-fly ash mixtures decreased from 8.0×10^{-6} cm/sec (0% fly ash) to 2.0×10^{-6} cm/sec (50% fly ash). The fly ash was less effective in reducing the permeability of clay than sand.

For 100% fly ash sample compacted dry, vertical cracks formed in the sample when saturated creating secondary permeability. The vertical cracks were due to the chemical reaction and shrinkage of the fly ash. The measured coefficient of permeability of fly ash including secondary permeability was found to be about 1.2×10^{-3} cm/sec, only a little lower than value of sand (3.3×10^{-3} cm/sec).

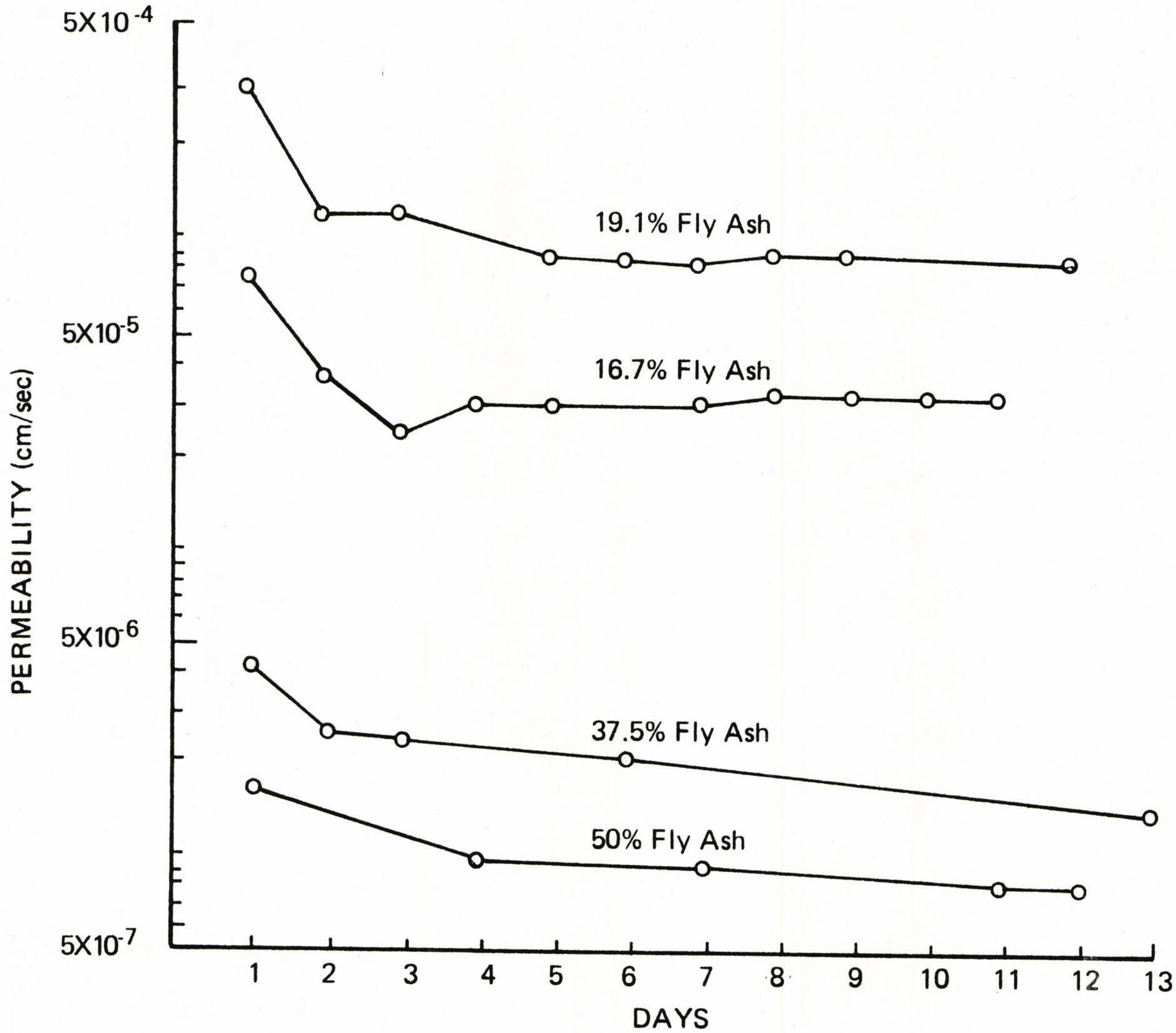
Effect of Time on Permeability

The coefficient of permeability was found to change with time. Sand-fly ash samples were kept in permeameters in a saturated condition without water flowing and tested periodically for up to 13 days (Summary Figure 2). In general, the coefficient of permeability decreased to a constant value for each of the fly ash mixtures used. The decrease in permeability with time was probably due to chemical reactions between the fly ash and the soil. Samples tested continuously were slightly more permeable, possibly due to dissolution of salts and chemicals.

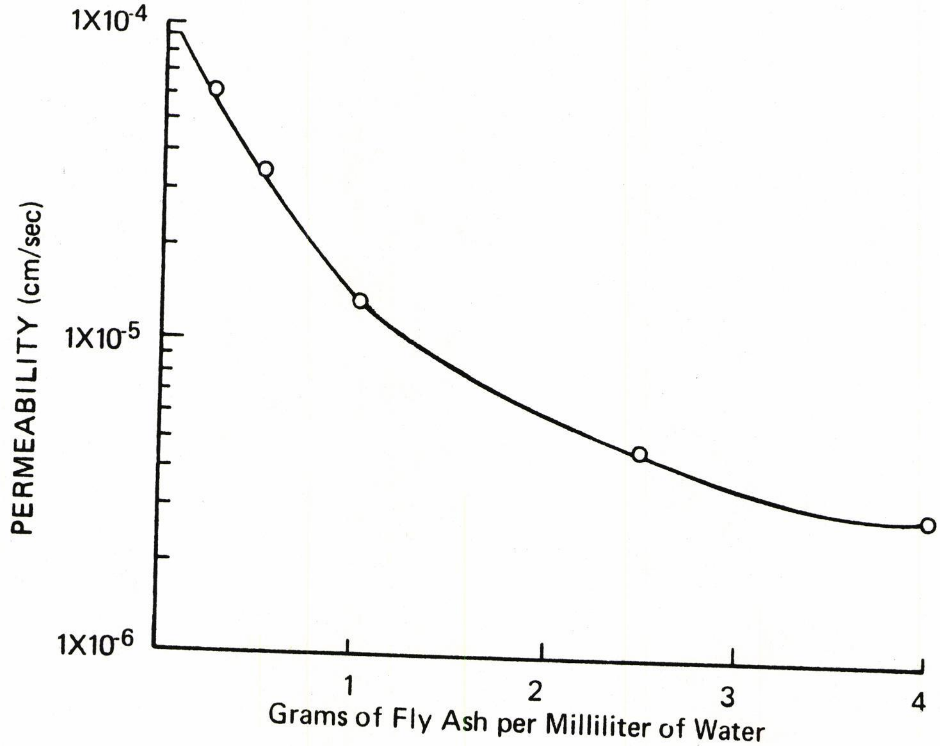
Permeability of Fly Ash-Water Slurry Mixtures

The coefficient of permeability of fine fly ash-water slurry mixtures was determined. The fly ash solution was mixed for one minute and poured into an ASTM-D 2434-68 (plastic) permeameter uncompacted. The coefficient of permeability was measured after a 24 hour set time.

As the concentration of fly ash solution increased, the coefficient of permeability decreased (Summary Figure 3). The coefficient of permeability of 0.25 grams of fly ash per milliliter of water slurry was 5.8×10^{-5} cm/sec. The coefficient of permeability of 4 grams of fly ash per milliliter of water slurry was 3.2×10^{-6} cm/sec. Generally, there was a slight (less than 10%) decrease in permeability between 24 hours and 48 hours set time for the slurries tested.



Summary Figure 2. Permeability versus Time.



Summary Figure 3. Permeability of Fly Ash - Water Slurry Mixtures.

Environmental Effects

The potential for water quality problems caused by the fly ash studies was found to be limited to a high pH, alkalinity and hardness. The maximum values of these parameters was found to be pH 11.2, alkalinity 580 mg/l, and hardness 640 mg/l. All values decreased with increasing volume of water passed through the permeameter.

Alkalinity, pH, and hardness are the same parameters that are affected by lime. Therefore, it is felt that the use of fly ash to stabilize soils presents no more hazard to water quality than the use of lime.

GAINS, FINDINGS, AND CONCLUSIONS

The following conclusions are based on the results of a study using a fly ash produced from Wyoming low sulfur coal and two Arkansas soils.

1. Addition of fly ash to clay or sand reduces the permeability. The fly ash was more effective in reducing the permeability of sand (permeability reduced three orders of magnitude at 50% fly ash) than in clay (reduced by a factor of 4 at 50% fly ash).
2. Permeability does not vary greatly with time. Variation of permeability with time was less than an order of magnitude, usually less than a factor of two. Change in permeability was most rapid the first three days with little change after three days.
3. Increased compactive effort increases density and reduces permeability in soils. However, reduction in permeability due to increased compactive effort are usually small.
4. The permeability of fly ash, placed in a slurry, varies between 10^{-4} cm/sec and 10^{-6} cm/sec depending on the amount of water in the slurry.
5. Fly ash placed dry, then saturated, developed shrinkage cracks which created secondary permeability.

IMPLEMENTATION

Fly Ash from coal fired power plants now under construction is a good potential building resource for construction of highways in Arkansas. This study, (HRP 47) and an earlier study (HRP 43) show that fly ash can, if compacted immediately, develop high strength and presents little danger of pollution. Fly Ash, however, has not been proven in a field test.

Before implementation of this research, three problems must be overcome:

1. No fly ash is now being produced in Arkansas, however, completion of the coal fired electric generating facilities is scheduled for the summer of 1978, at which time fly ash production will begin.
2. Construction procedures must be developed and/or retarding additives found, which will allow the properties of the fly ash to be fully utilized.
3. A test section should be constructed and monitored to prove the fly ash and procedures in a field application.

The first problem will be solved by the passage of time. The second problem is the subject of a research project (HRP 52) given conditional approval and scheduled to begin in January, 1977. The third problem may be taken care of by an expansion of HRP 52, submission in one year of another research project, or construction of a test section by the Highway Department.

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INTRODUCTION

The purpose of this report is to review the laboratory and in-situ tests of permeability for soil, to review literature on permeability of fly ash and fly ash mixtures, and to review the relationship between permeability, frost heave, and compressive strength. Also, the permeability of soil-fly ash mixtures are to be evaluated.

Two Arkansas soils, a kaolinite clay (OH) and a fine sand (SP-SM) were used in the study. Both soils can be stabilized effectively with fly ash (Thornton, Parker & White, pp. 8). However, the permeability of fly ash and soil-fly ash mixtures is not well known. Permeability affects the potential for frost heave, durability and potential for pollution through percolation of ground water. Therefore, the permeabilities of these two Arkansas soil-fly ash mixtures were evaluated.

LITERATURE REVIEW

In general, solids have continuous voids,especially coarse soil-gravels, sands, and even silts. In clays, because of their plate-shaped particles, a small percentage of isolated voids may be possible. But electron photomicrographs of the finest clays show that the voids are interconnected. So water can flow through all soils.

Since soil particles are randomly distributed, water does not travel in a straight line at constant velocity. Water travels in a winding path from pore to pore. The velocity of a drop of water at any point along the winding path depends on the size of the pore and its position in the pore.

In soil engineering problems, the flow path in the soil grain is considered as macroscopic. The water is assumed to flow from point A to point B along a straight line at an effective velocity (Figure 1).

DARCY'S LAW

In the 1850's, H. Darcy performed a classical experiment. He used a set up similar to that which is shown in Figure 2 to study the flow properties of water through a sand filter bed. Darcy varied the length of sample L and the water pressure at the top and bottom of the sample. He measured the rate of flow Q that passed through the sand.

Darcy experimentally found that the volume rate of flow, Q, through the filter bed in a given time was directly proportional to levels Δh and inversely proportional to the length, L, between the piezometers.

$$\frac{Q}{t} = q \;\alpha\; \frac{A \Delta h}{L}$$

$$q = \text{constant} \times \frac{A \Delta h}{L} \qquad \text{(Eq. 2-1)}$$

The classic experiment provided the physical basis for the analysis of flow through porous media, and (Eq. 2-1) is known as Darcy's Law. In Equation 2-1, q is the total volume flow rate and L is the length of filter bed between the piezometers or the macroscopic flow path. The difference in piezometric level, Δh is very nearly equal to the total head lost over the length L. The difference in piezometric level Δh is an accurate measure of the total head lost due to the flow of water in soil mass, that is, pressure head + elevation head + velocity head. Since the maximum velocity in practical problem is small,

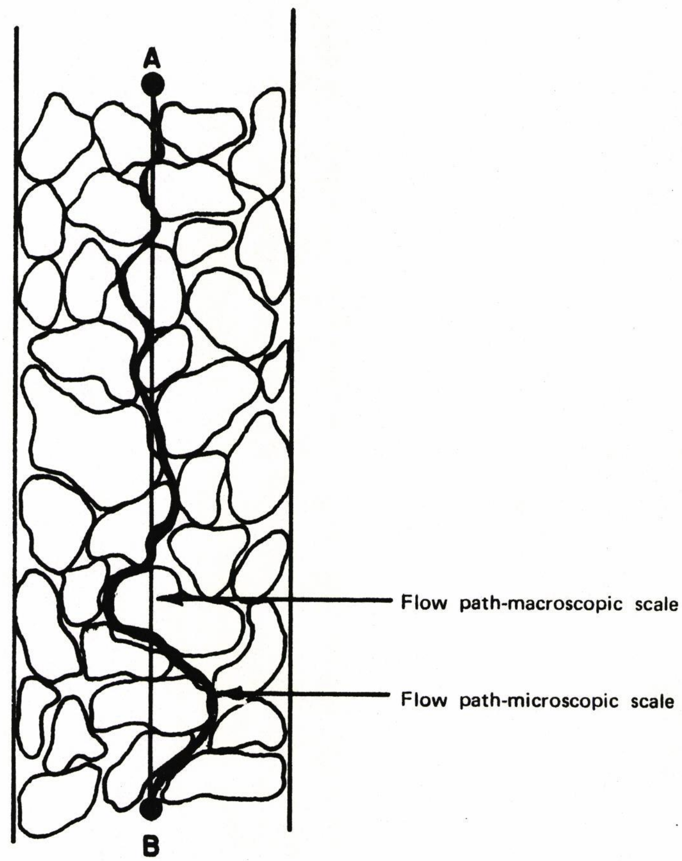


Figure 1. Flow Path of Water (from Lambe & Whitman, 1969, P. 251).

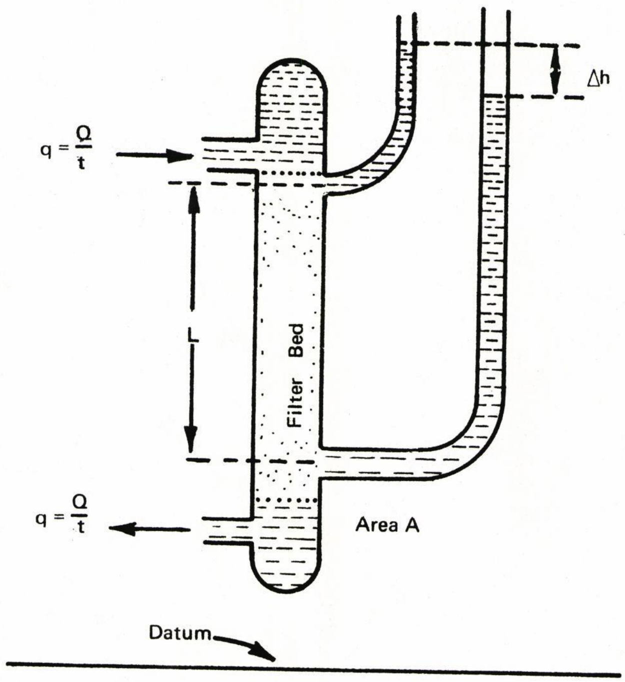


Figure 2 . Schematic diagram of Darcy's sand filtration experiment (from Leonards, 1962, P. 109).

the velocity head is negligible. A piezometer only measures the sum of the pressure and elevation head.

Rearranging Eq. 2-1

$$\frac{q}{A} = v \; \alpha \; \frac{\Delta h}{L} \quad = \text{constant } i = ki$$

where v = total volume flow rate per unit of total cross-section area perpendicular to direction of macroscopic flow commonly called discharge (or approach) velocity.

i = total head lost per unit length of macroscopic flow path, called (macroscopic) hydraulic gradient.

k = constant of proportionality, variously designated, but referred to as Darcy's coefficient of permeability, or simply the coefficient of permeability.

Thus

$$v = ki$$

is also referred to as Darcy's Law.

Darcy's Law is valid for most types of fluid flow in soils. For liquid flow at very high velocity and for gas flow at very low or high velocity, Darcy's Law becomes invalid. Darcy's Law is based on the following boundary assumptions (Leonards, 1962, pp. 128):

1. Homogeneous porous medium.
2. Continuous (saturated), two dimensional flow.
3. Homogeneous fluid.
4. Steady-state flow condition.
5. No change in voids of porous medium.
6. Imcompressible fluid.
7. Laminar flow.

The coefficient of permeability is actually a parameter which indicates the soil's ability to transmit water. The coefficient of permeability is defined as the rate of discharge of water at a temperature of 20° C under conditions of laminar flow through a unit cross sectional area of a soil medium under a unit hydraulic gradient. The coefficient of permeability has the dimensions of a velocity and is usually expressed in centimeters per second or meter per second per square meter in S.I. units or feet per second or gallons per day per square foot in British units. Figure 3 shows some average permeability coefficients

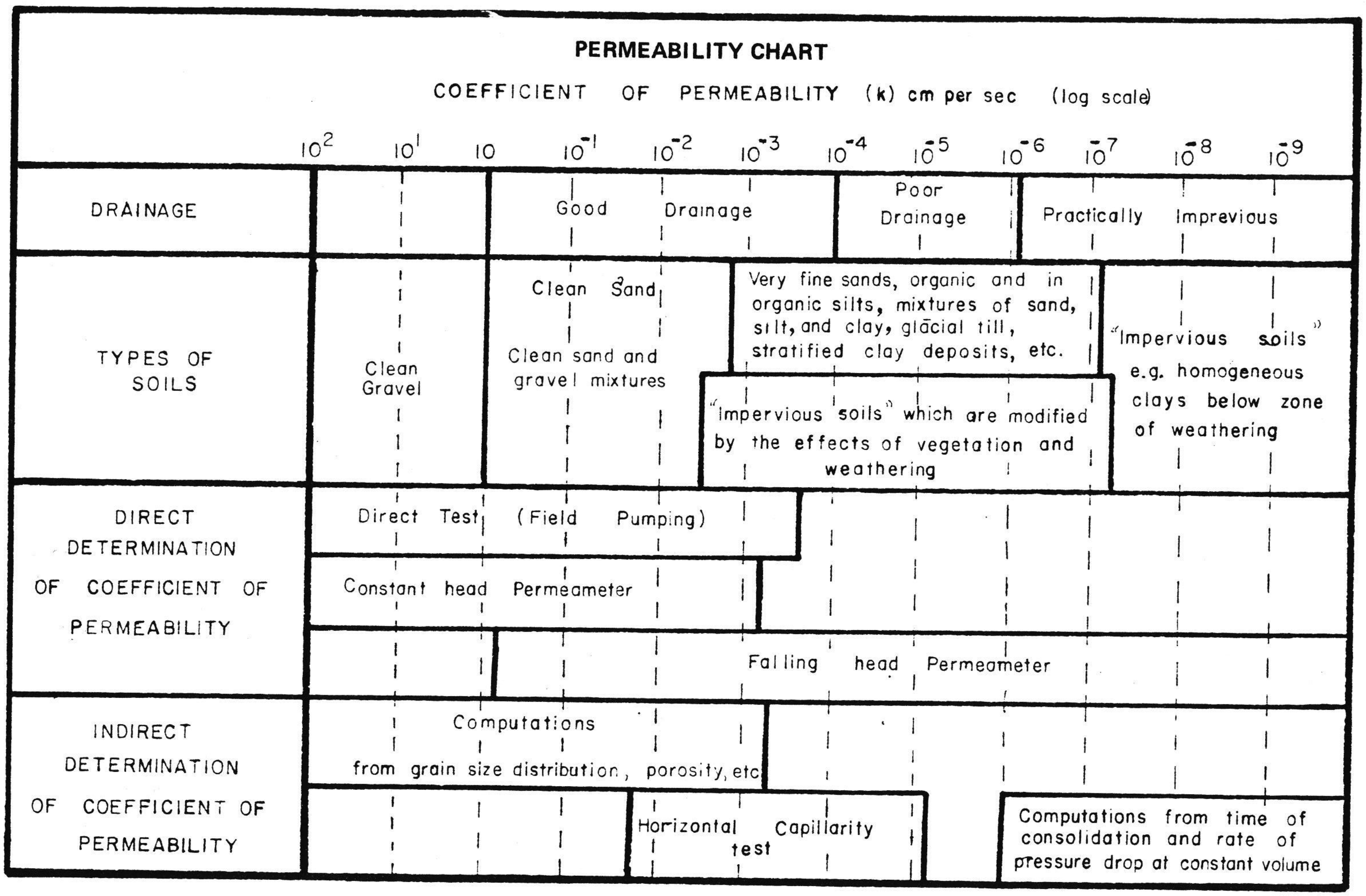


Figure 3 . Permeability Chart (From Leonards, 1962, P. 134).

and appropriate permeability test methods.

EFFECT OF HYDRAULIC GRADIENT

The simple relationship expressed by Eq. 2-1 may not be valid in soils containing clay, particularly under conditions of high hydraulic gradient (Mitchell & Younger, 1967, pp. 109). But no evidence for a threshold gradient was found for saturated, compacted silty clay (Mitchell & Younger, 1967, pp. 137). This is contrary to other evidence in the literature. Since deviations from Darcy's law are most severe at high gradients and gradients in the field are seldom much greater than unity. Therefore the coefficient of permeability obtained in the laboratory is greater than actually developed in the field if non-Darcy flow exists.

FACTORS AFFECTING PERMEABILITY

The coefficient of permeability is an important property of soil. The magnitude depends upon the size, shape and state of packing of soil particles. As an example, a clay soil which is composed of mainly small particles, will have a much smaller permeability coefficient than a sand with relatively coarse particles, even if the void ratios are approximately the same. The individual soil characteristics including particle size, void ratio, composition, and degree of saturation influence the coefficient of permeability.

Size of the Soil Grain

The coefficient of permeability increased with increasing grain size. But, no simple relationship exists between permeability and grain size except for fairly coarse soils with rounded grains.

Taylor suggested that the permeability of sands varies approximately as the square of the grain size (Taylor, 1960, pp. 112).

Hazen found experimentally that the permeability of filter sand may be correlated with the effective diameter of D_{10} (Hazen in Taylor, 1960, pp. 112):

$$k \text{ (cm/sec)} = C\, D_{10}^{2} \qquad \text{(Eq. 2-3)}$$

where D_{10} = effective size (cm)

C = Constant which varies from 100 to 150 (cm/sec)

But Eq. 2-3 makes no allowance for variations in porosity or in the shape of particles.

Viscosity of Soil Water

The permeability is directly proportional to the unit weight of water γw and inversely proportional to the viscosity of soil water, μ from the Kozeny-Carmen Equation:

$$k = D_s^2 \frac{\gamma w}{\mu} \frac{e^3}{1+e} C \qquad \text{(Eq. 2-4)}$$

where k = Coefficient of permeability

D_s = Diameter of spherical grain

γw = Unit weight of water

μ = Viscosity of water

e = Void ratio of soil

C = A composite shape factor

The unit weight of water is essentially constant, but the value of viscosity varies with temperature. Therefore the effect of fluid properties on the value of the permeability when other factors are constant is give by:

$$\frac{k_1}{k_2} = \frac{\mu_2}{\mu_1} \qquad \text{(Eq. 2-5)}$$

The viscosity of water decreases as temperature increases. Therefore the permeability is higher for higher temperature.

In practical laboratory testing, the test of permeability if run at the most convenient temperature and reported at 20 C. So Equ. 2-5 becomes

$$k_{20^o C} = \frac{\mu_T}{\mu_{20^o C}} \cdot K_T \qquad \text{(Eq. 2-6)}$$

where k_{20^oC} = permeabilityat temperature 20°C

k_T = permeability at temperature T°C in the lab

μ_T = viscosity of water at temperature T°C in the lab

μ_{20^oC} = viscosity of water at temperature 20°C

By the Equation 2-6 and the relative viscosity values from Figure 4, we can get the permeability at 20°C.

Size and Shape of Voids and Flow Path

Void ratios are related to permeability because a decrease in void ratio also decreases

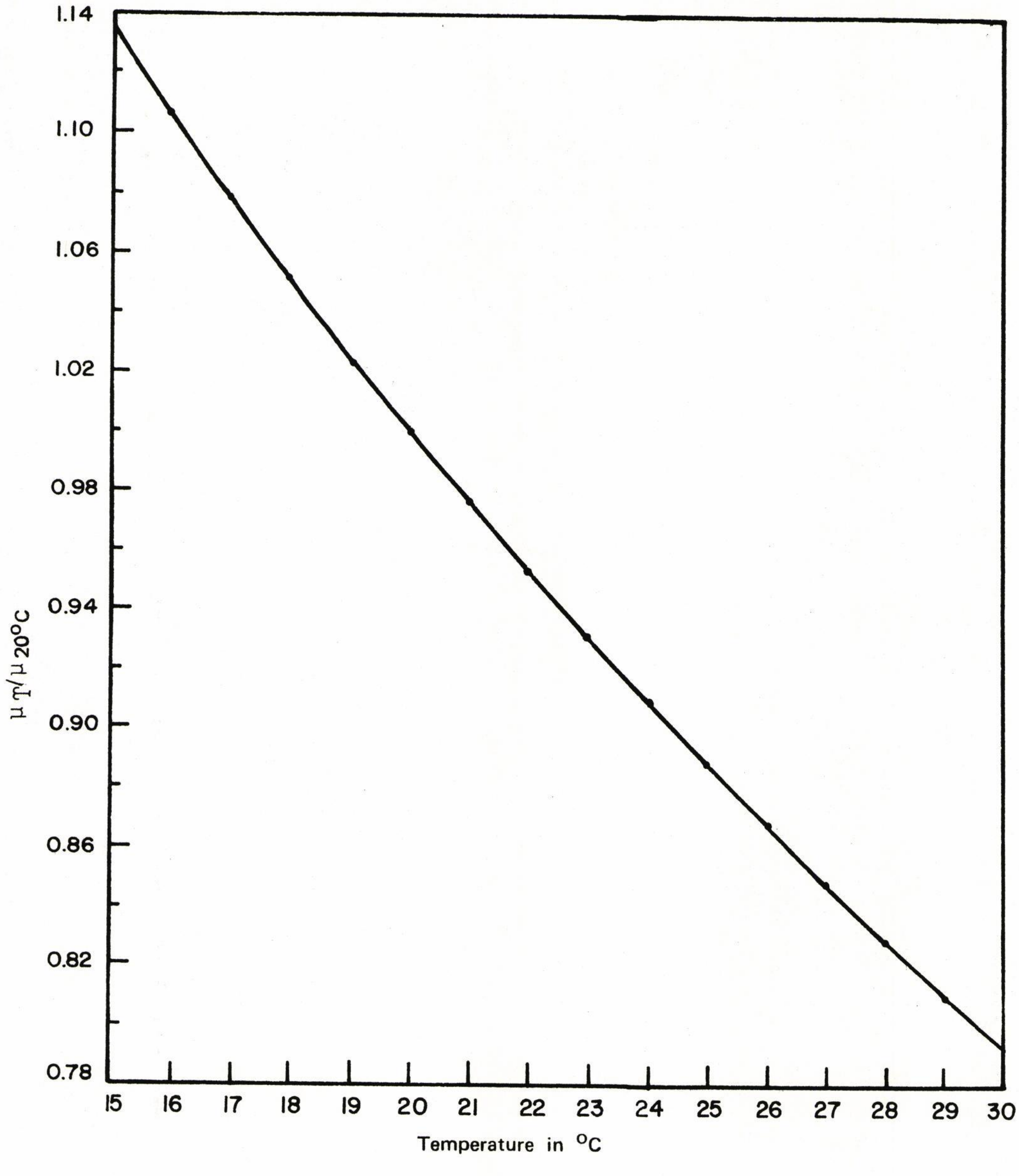


Figure 4. Temperature versus Relative Viscosity (from Lambe, 1960, P. 61).

the permeability of a soil. Casagrade (Terzaghi and Peck, 1968, pp. 51) presented an equation which relates the void ratio to coefficient of permeability as follows:

$$k = 1.4\, e^2\, k_{0.85} \qquad \text{(Eq. 2-7)}$$

where k = coefficient of permeability

e = void ratio

$k_{0.85}$ = coefficient of permeability at void ratio of 0.85

Graphs which illustrate the influence of void ratio on permeability are presented in Figures 5, 6, and 7. The general rule of these graphs is that e verse log k is approximately a straight line for nearly all soils.

Degree of Saturation

An increase in the degree of saturation (Sr) of a soil causes an increase in permeability. For values of Sr greater than about 85% much of the air in soil is held in the form of small occluded bubbles. Darcy's Law is still approximately valid when Sr is as low as 85%. But, the bubbles block some of the pores and reduce the permeability considerably. If the degree of saturation is less than 85% much of the air is continuous through the voids. Darcy's Law no longer holds.

The ratio of the permeability of the unsaturated soil to that of the saturated material at the same void ratio varies approximately as the degree of saturation (Sr/100) to the power 3.5 over the range of saturation from zero to 100% (Polubarinova - Kochina in Scott, 1963. pp. 75). But in the degree of saturation from 80% to 100%, the ratio of the permeability to (Sr/100) is nearly a linear function of the degree of saturation (Figure 8). The ratio also varies as $\{1\text{-}m(1\text{-}Sr/100)\}$, where m is a constant with values between 2 and 4. The linear approximation to the power curve in the 80% to 100% range of saturation has an m-value of 3.5 (Scott, 1963 pp. 75). The lower values of m hold for soils of uniform grain size and m increases in well-graded material (Orlob and Radhakrishna in Scott, 1963. pp. 75).

Absorbed Water in Clays

Because the ions in the crystalline layers of finegrained soil such as clay have net electrical charge (Wu. 1966. pp. 392), water is held against these ionic surfaced by hydrogen bonds. The water molecules that are held to the clay are called absorbed water. Absorbed

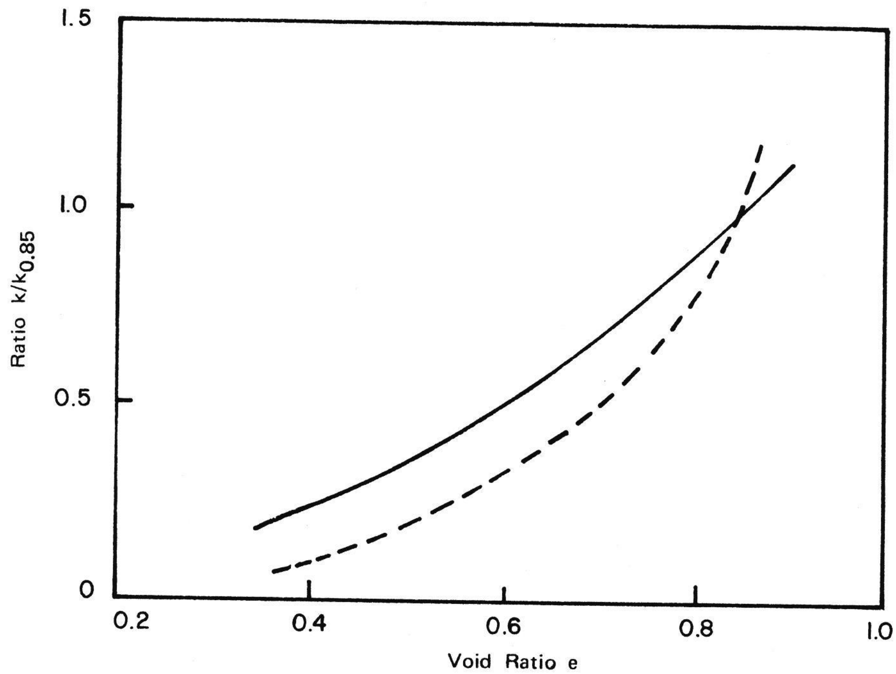


Figure 5. Relation between void ratio and permeability of mixed-grained sand (solid line) and soil with flaky constituents (dash line) (from Terzaghi and Peck, 1968, P. 51).

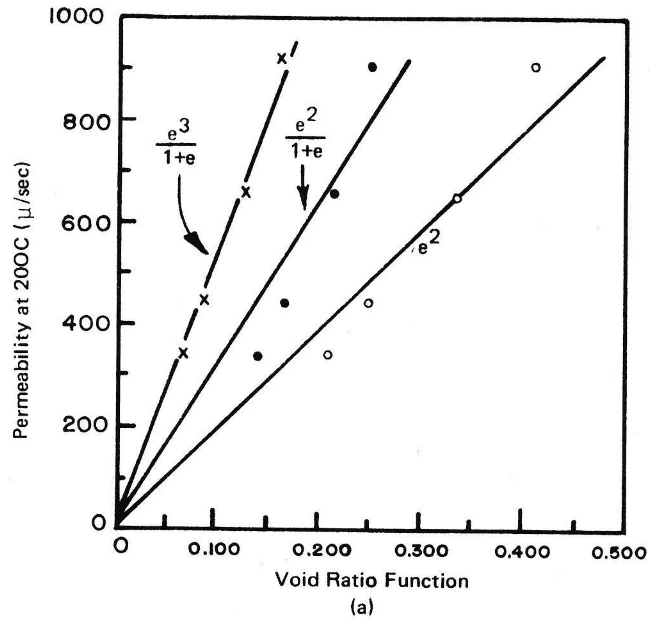


Figure 6. Void Ratio versus Permeability from different equations (falling head test) (from Lambe, 1951).

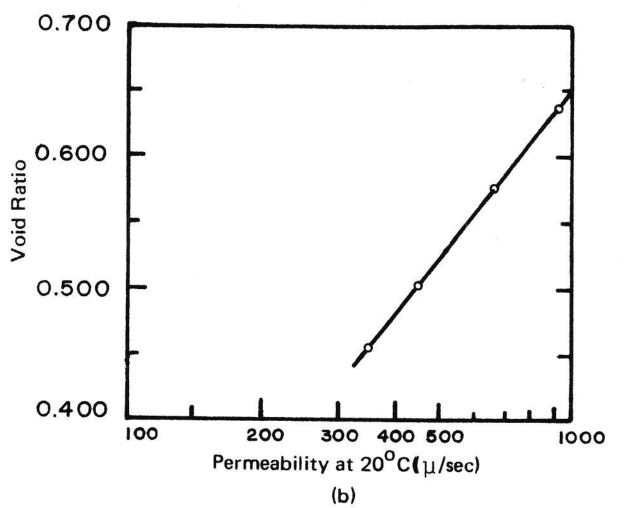


Figure 7. Void ratio versus log k (falling head test)(from Lambe, 1951).

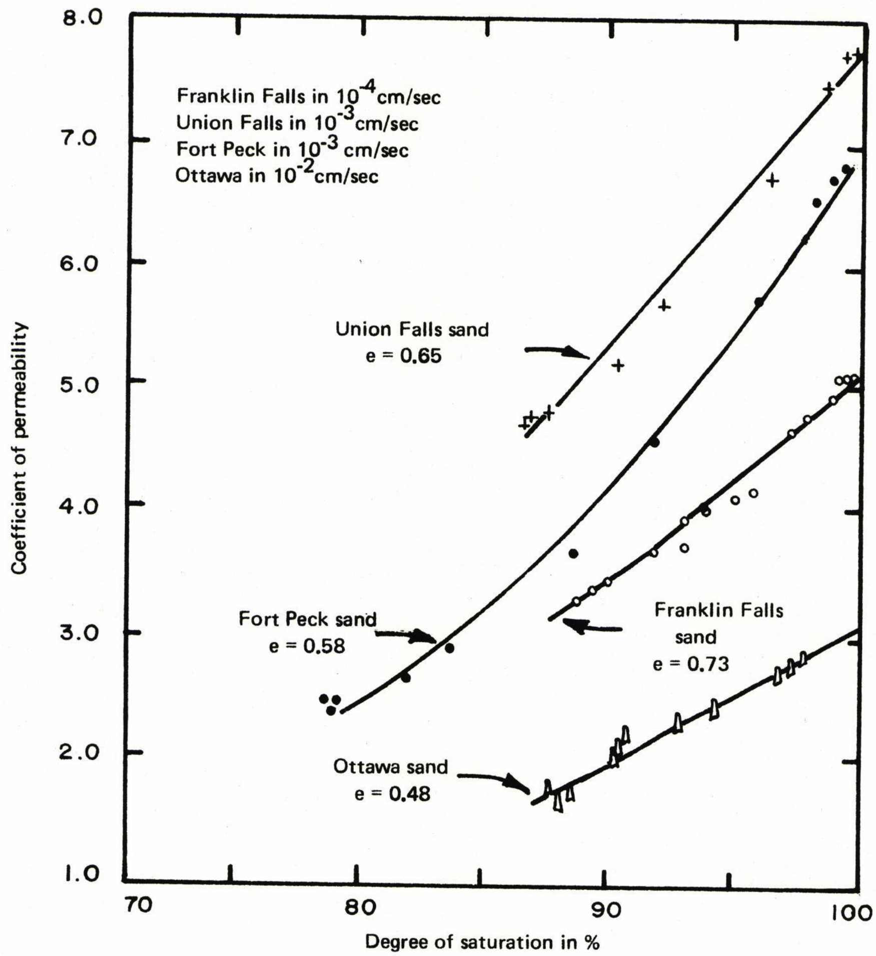


Figure 8. Permeability versus degree of saturation for various sands (from Lambe 1960, P. 53).

water is important in relation to permeability in fine-grained soils because the thickness of the absorbed layers reduces the effective size of the pores in fine-grained soil. The absorbed layers tend to retard the flow of water through the soil and reduces the permeability. The thickness of the absorbed layer is influenced by the concentration of ions in the soil water and the type of cation in the absorbed layer.

MEASUREMENTS OF PERMEABILITY

The permeability of soil can be measured in either the laboratory or the field. A laboratory test is satisfactory for material used in construction. Field tests are best for in-situ soils. Laboratory tests are much easier to make than field tests. Because it is difficult to obtain representative undisturbed samples from the field, in-situ tests are needed.

Laboratory Tests

There are two methods to determine permeability in the laboratory: the direct method and indirect method.

Direct Methods

Direct methods consist of Constant Head Permeameter (Figure 9) and Falling or Variable Head Permeameter (Figure 10). They are used for soils with permeabilities down to about 10^{-7} cm/sec.

Constant Head Permeameter. The constant head permeameter is widely used for coarse-grained soil with a coefficient of permeability greater than 10^{-4} cm/sec (Figure 3). Fine-grained soils are difficult to measure accurately because Q, the quantity of water flow through the sample, is small.

To run a constant head test, a sample of material is placed in a cylindrical container with a continuous supply of water under a fixed total head difference. The water that passes through the sample in a given time is collected and the amount is determined.

From Equation 2-2

$$v = ki$$

Then

$$k = \frac{v}{i}$$

Since

$$v = \frac{Q}{At}$$

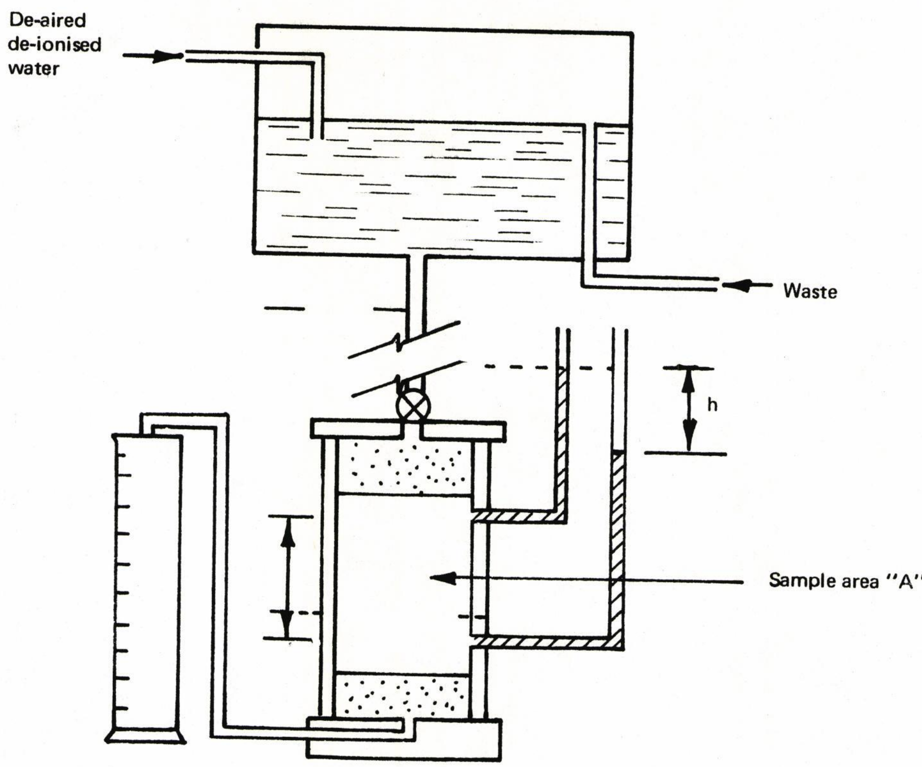


Figure 9. A constant head permeameter. (From Scott, 1969, P. 68).

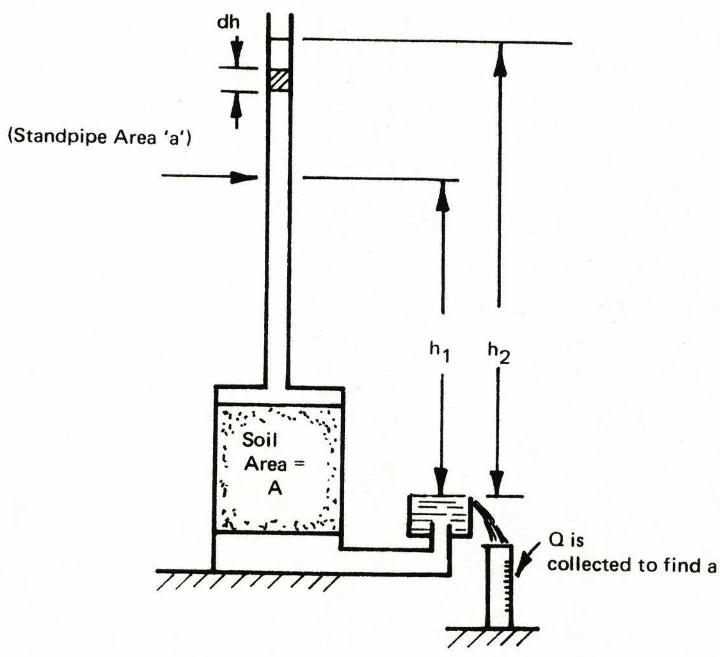


Figure 10. A falling head permeameter. (From Bowles, 1968, P. 35).

Therefore

$$k = \frac{Q}{iAk} \quad \text{or} \quad \frac{QL}{hAt} \qquad \text{(Eq. 2-8)}$$

Where Q = amount of water through the specimen

L = length of sample (between the two piezometers)

A = cross-sectional area of the sample

h = head

t = time

i = hydraulic gradient $= \frac{h}{L}$

Falling Head Permeameter. The falling head permeameter is used for soils with low permeability like fine sand or clays with k values between 10^{-4} and 10^{-7} cm/sec.

To run a falling head test, a soil sample is set up below a vertical standpipe. Water is permitted to run through the soil driven by the head in the standpipe. As the water flows through the soil, the level in the standpipe diminishes and the head of water is not held constant.

Calculation of the coefficient of permeability for the falling head permeameter is as follows (Scott, 1969, pp. 66-67):

Let the level drop - dh in time dt. Then the rate of flow

$$q = -a\frac{dh}{dt} = A\,k\,\frac{h}{L}$$

Therefore

$$-\frac{dh}{h} = \frac{Ak}{aL} \cdot dt$$

Integrating

$$\int_{h_1}^{h_2} -\frac{dh}{h} = \frac{Ak}{aL}\int_{t_1}^{t_2} dt$$

$$\mathrm{Ih}\,\frac{h_1}{h_2} = \frac{Ak}{aL}(t_2 - t_1)$$

$$k = \frac{aL}{A(t_2 - t_1)} \ln \frac{h_1}{h_2}$$

$$k = 2.303 \frac{aL}{A(t_2 - t_1)} \log \frac{h_1}{h_2} \qquad \text{(Eq. 2-9)}$$

where a = cross-sectional area of the standpipe

L = length of soil sample

A = cross-sectional area of the permeameter

t_1 = initial time the water level in the standpipe is at h_1

t_2 = final time the water level in the standpipe is at h_2

h_1, h_2 = the heads between which the permeability is determined

Indirect Methods

Indirect methods are applicable for permeability below about 10^7 cm/sec.

Consolidation Test. Based on Terzaghi's theory, the coefficient of consolidation is defined (Wu, 1966, pp. 103)

$$Cv = \frac{k}{\gamma w\ Mv} \qquad \text{(Eq. 2-10)}$$

where C_v = coefficient of consolidation

k = coefficient of permeability

γw = unit weight of water

M_v = coefficient of volume change and is defined is defined as the change in volume, per unit volume, per unit change of effective stress

$$Mv = \frac{1}{V} \frac{dv}{dp}$$

Since the change in total volume equals the change in the volume of the voids

$$\frac{dv}{v} = \frac{de}{1+e} \qquad \text{(Eq. 2-11)}$$

Therefore

$$M_v = \frac{1}{1+e} \frac{de}{dp} \qquad \text{(Eq. 2-12)}$$

$$= \frac{1}{1+e} Av \qquad \text{(Eq. 2-13)}$$

where A_v = Coefficient of compressibility

$= \frac{de}{dp}$

Therefore

$$k = C_v\ \gamma w\ Mv \qquad \text{(Eq. 2-14)}$$

$$= \frac{Cv\ Av\ \gamma w}{1+e} \qquad \text{(Eq. 2-15)}$$

where k = coefficient of permeability

C_v = coefficient of consolidation

A_v = coefficient of compressibility

γw = unit weight of water

e = void ratio

With the data from the consolidation test, we compute e, C_v, A_v from which we calculate the value of k, coefficient of permeability.

The coefficient of consolidation, C_v, is computed from one of the following equations:

1. Square Root Fitting Method (Lambe, 1960, pp. 82)

$$C_v = \frac{0.848 \ H^2}{t_{90}} \qquad \text{(Eq. 2-17)}$$

2. Log Fitting Method (Lambe, 1960, pp. 82)

$$C_v = \frac{0.197 \ H^2}{t_{50}} \qquad \text{(Eq. 2-18)}$$

where H = average length of the drainage path for the load increment

t_{90} = time for 90% of primary compression

t_{50} = time for 50% of primary compression

The coefficient of compressibility, A_v, is the slope of the pressure-void ratio plot. Since the log p versus e curve is usually plotted rather than the p versus e curve, A_v can be found from C_c by

$$C_c = -\frac{de}{d(\log_{10} p)} \qquad \text{(Eq. 2-19)}$$

$$A_v = \frac{0.435 \ C_c}{p} \qquad \text{(Eq. 2-20)}$$

where C_c = compression index

p = average pressure for the increment in the test

e = void ratio

Dissipation Test. The sample is set up in the triaxial cell instead of the consolidation ring. From experimental data, e, C_v, A_v and k can be calculated as in the method of the consolidation test.

Field Tests

Determination of the soils permeability at the site can be made by pumping water from wells and observing the rate of fall of the water surface in the well and in adjacent wells. The method of finding the permeability depends upon whether one encounters equilibrium or non-equilibrium well conditions.

Equilibrium Well Condition

There are two basic flow conditions for well discharge: Unconfined flow (water-table condition) and confined flow (artesian condition).

In computing the permeability for unconfined flow (Figure 11) and confined flow (Figure 12), the following assumptions must be made (Johnson, 1966, p. 104):

1. The aquifer is homogenous and isotropic
2. The water-bearing materials have uniform permeability within the radius of influence of the well
3. The aquifer is not stratified
4. For unconfined flow, the saturated thickness is constant before pumping starts; for confined flow, the aquifer thickness is constant
5. The pumping well is 100 percent efficient
6. The pumping well penetrates to the bottom of the aquifer
7. Neither the water table nor piezometric surface has any slope; both are horizontal surfaces
8. Laminar flow exists throughout the aquifer and within the radius of influence of the well
9. The cone of depression has reached equilibrium so that both drawn-down and radius of influence of the well do not change with continued time of pumping at a given rate

Unconfined Flow

For steady condition, the flow towards the well must be the same at all radii. Permeability for steady state is:

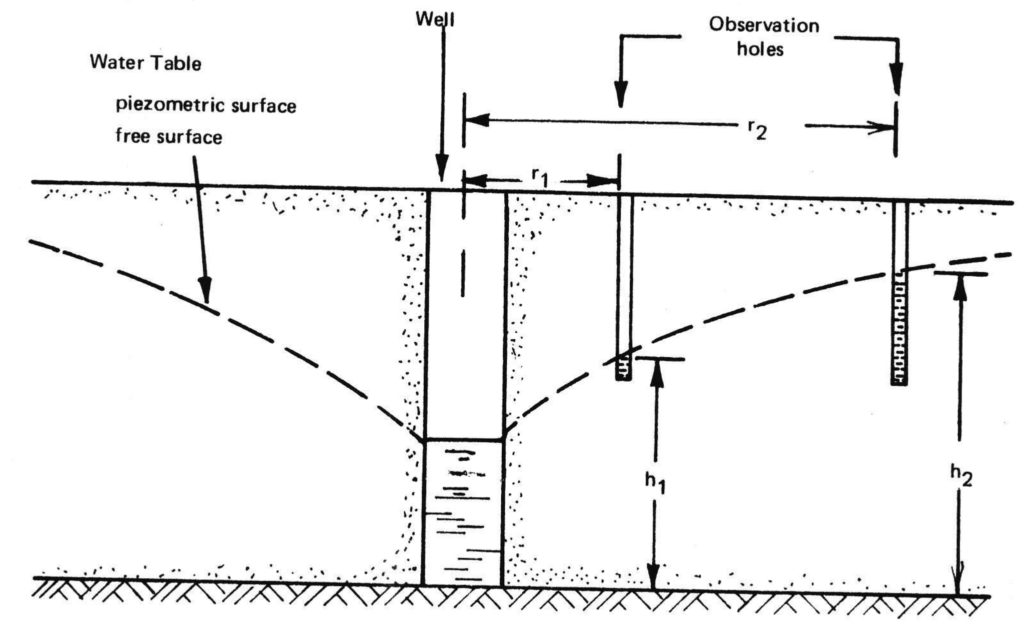


Figure 11. Pumping Test - unconfine flow (from Scott, 1969, P. 70).

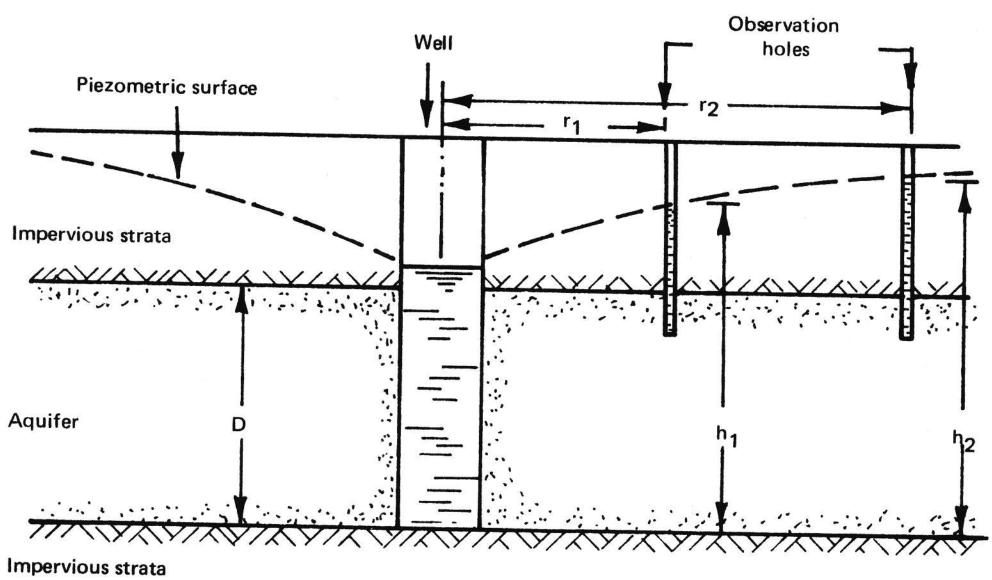


Figure 12. Pumping Test - confine flow (from Scott, 1969, P. 72).

$$k = \frac{q}{\pi} \quad \frac{\ln \frac{\gamma_2}{\gamma_1}}{(h_2^2 - h_1^2)} \qquad \text{(Eq. 2-21)}$$

where k = coefficient of permeability

q = rate of flow

γ_1, γ_2 = the distance of well from the observation hole 1 & 2 respectively

h_1, h_2 = the piezometric level at observation hole 1 & 2 respectively

Confined Flow

A well is sunk through an aquifer confined at both top and bottom by impervious strata. In this case, there is no free surface, and the piezometric level is above the top of the aquifer (Scott, 1969, pp. 72).

Then for steady flow, we get

$$k = \frac{q}{2\pi D} \quad \frac{\ln \frac{r_2}{r_1}}{(h_2 - h_1)} \qquad \text{(Eq. 2-22)}$$

where k = coefficient of permeability

D = thickness of aquifer

r_1, r_2 = the distance of well from observation hole 1 & 2 respectively

h_1, h_2 = the piezometric level at observation hole 1 & 2 respectively

Non-Equilibrium Well Condition

Steady state seldom exists in nature because months or years may be required to attain a steady state. Because of the time required to attain a steady state, pumping tests are usually run under transient conditions and solved by the Theis analysis.

The Theis formula was the first non-equilibrium well formula to take account of the effect of time of pumping on well yield. By use of the formula, the drawdown can be predicted at any time after pumping begins and the transmissibility and average permeability can be determined from the early stages of a pumping test.

The Theis formula is based on the following assumptions (Johnson, 1966, pp. 108):

1. The water-bearing formation is uniform in character and permeability in both horizontal and vertical directions.
2. The formation has uniform thickness.
3. The formation has infinite areal extent.
4. The formation receives no recharge from any source.
5. The pumped well penetrates and receives water from the full thickness of the water-bearing formation.
6. The water removed from storage is discharged instantaneously with lowering of the head.

$$S = \frac{114.6\ Q}{T} \quad W(u) \qquad \text{(Eq. 2-23)}$$

where S = drawdown, in ft., at any point in the vicinity of a well discharging at a constant rate

Q = pumping rate, in gpm

t = coefficient of transmissibility of tne aquifer, in gpd per ft.

W(u) = called well function of U, can be evaluated from the series

$$W(U) = -0.5772 - \ln U + U - \frac{U^3}{2.21} + \frac{U^3}{3.31} \qquad \text{(Eq. 2-24)}$$

Values of W(u) for various values of u are given in Table 1.

$$U = \frac{1.87\ r^2\ S}{T\ t} \qquad \text{(Eq. 2-25)}$$

where r = distance, in ft. from center of pumped well to point where drawdown is measured

S = coefficient of storage, dimensionless

T = coefficient of transmissibility, in gpd in ft.

t = time since pumping started, in days

In order to determine coefficient of permeability, the coefficient of transmissibility has to be evaluated from equation 2-23.

T = kD

where T = coefficient of transmissibility

k = coefficient of permeability

D = thickness of aquifer

Table 1. Values of W(u) for various values of u. (From Linsley, Kohler, Paulhus, 1975, pp. 208)

u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
$\times 1$	0.219	0.049	0.013	0.0038	0.00114	0.00036	0.00012	0.000038	0.000012
$\times 10^{-1}$	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
$\times 10^{-2}$	4.04	3.35	2.96	2.68	2.48	2.30	2.15	2.03	1.94
$\times 10^{-3}$	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
$\times 10^{-4}$	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
$\times 10^{-5}$	10.95	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
$\times 10^{-6}$	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
$\times 10^{-7}$	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
$\times 10^{-8}$	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
$\times 10^{-9}$	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
$\times 10^{-10}$	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
$\times 10^{-11}$	24.75	24.06	23.65	23.36	23.14	22.95	22.81	22.67	22.55
$\times 10^{-12}$	27.05	26.36	25.95	25.66	25.44	25.26	25.11	24.97	24.86
$\times 10^{-13}$	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
$\times 10^{-14}$	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
$\times 10^{-15}$	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

The method used to find the coefficient of transmissibility involves matching a curve plotted from specific pumping test data with what is called a type curve (Figure 13). The type curve is prepared by plotting values of W(u) against 1/u on graph paper with logarithmic scales. The field test data are plotted with drawdown on the vertical axis and time since pumping started on the horizontal axis. This graph is superimposed on the type curve. Once a good matching position is found, a match point is selected. So, W(u) and S can be found from the match point on the graphs. The pumping rate, Q, is constant for a given pumping test. By applying Equation 2-23, the coefficient of transmissibility, T, can be determined.

FLY ASH

Permeability of Fly Ash

Fly ash can be effectively and economically used as a fill material to construct stable embankments for land reclamation (DiGioria & Nuzzo, 1972, pp. 5) and highway embankments (Kawan, Smith, et. al., 1975, pp. 44; Faber & DiGioia, 1976, pp. 19). Fly ash can also be used as a soil stabilizer (Thornton & Parker, 1976. pp. 76) or as an additive in lime stabilization treatments (Chu, Davidson, et. al. 1955, pp. 102; Mateos & Davidson, 1962, pp. 63).

Although much research has been done on the use of fly ash as a stabilizing agent, little is known about the permeability of fly ash and soil-fly ash mixtures. Permeability affects the potential for frost heave, durability, leaching, and runoff of soils. Therefore, the coefficient of permeability plays an important role in the other physical properties of fly ash.

The coefficient of permeability for fly ash depends upon its degree of compaction and the pozzolanic activity. The coefficient of permeability for some fresh U.S. fly ashes was found to range from $1x10^{-4}$ to $5x10^{-4}$ cm/sec (Faber & DiGioia, 1976, pp. 9).

Lin measured the permeability of fly ash from Tranton Channel Plant, Michigan. A theoretical curve was calculated according to the Kozeny-Carman relationship (Figure 14). The good correlation may be due to the spherical shape of particles, on which the shape factor of the Kozeny-Carman equation is based. The curve showed that the permeability of this particular Michigan fly ash is about $6x10^{-5}$ cm/sec at modified AASHO maximum dry density.

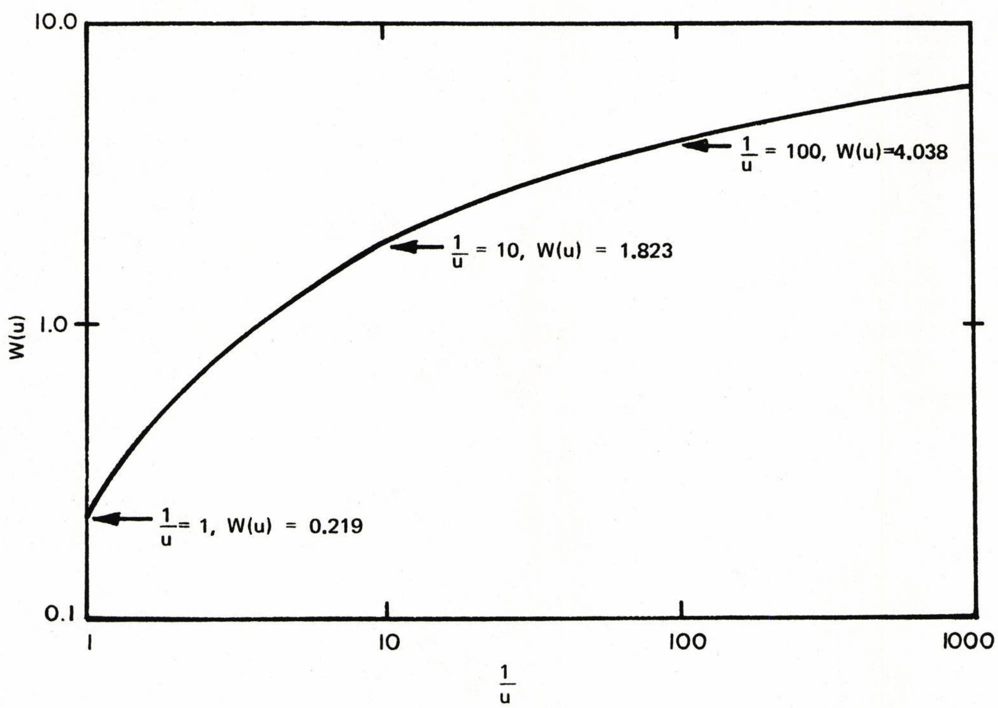


Figure 13. Type curve for use in graphical solution of Theis non-equilibrium formula (From Johnson, 1966, p. 110).

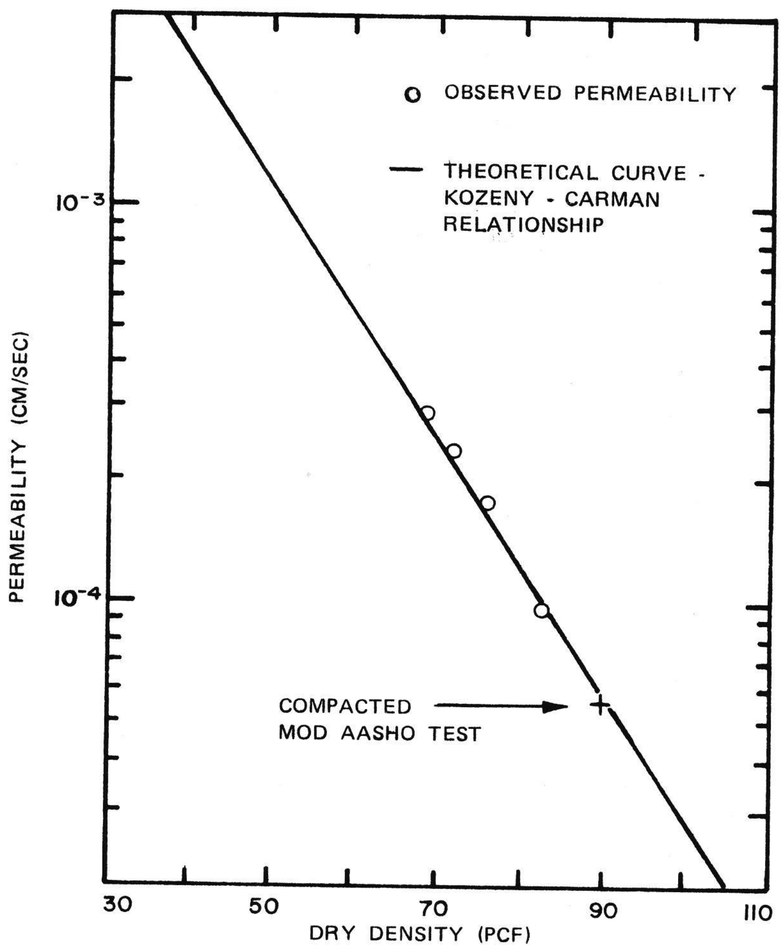


Figure 14 . Variation in permeability of fly ash with dry density, Trenton Channel Plant (from Gray & Lin, 1972, P. 373).

The measured coefficient of some typical Western Pennsylvania fly ash ia about $3x10^{-4}$ cm/sec under a hydraulic gradient of 1.0 (DiGioia & Nuzzo, 1972, pp. 78). This permeability is equivalent to a well graded silty sand (Figure 3) and this particular fly ash is relatively self-draining.

The laboratory determination of permeability of British fly ash compacted at 95% to 100% of the proctor maximum dry density was $5x10^{-7}$cm/sec to $8x10^{-5}$ cm/sec (Gray & Lin, 1972, pp. 371). These values correspond to drainage characteristics ranging from practically impervious to medium (Figure 3). These low results may be because British fly ash possess self-hardening properties which causes the fly ash particles to become cemented or partially cemented and renders the fly ash fill less permeable.

The coefficient of permeability was also measured on compacted lime-sulfate waste-fly ash-soil mixtures which were cured for 7 days at 22.8° C (Table 2) (Kawan, 1975). The Calcite lime - Sulfate waste - Albright fly ash combination specimens carcked during saturation. For Dolomitic lime - Sulfate waste - Amax fly ash, a coefficient of permeability of 3.7 x 10^{-5} cm/sec was measured (Kawam, Smith, et. al. 1975, pp. 25). The value is quite low and is indicative of a fairly impermeable material. The coefficient of permeability of lime-sulfate waste-fly ash-clayey soil mixtures are about 10^{-5} cm/sec. The coefficient of permeability of lime-sulfate waste-fly ash-sandy soil mixtures are from 10^{-6} to 10^{-8} cm/sec. The sandy soil is less pervious than the clayey soil. The difference in permeability between the two soils is most certainly due to the large volume change (expansion) which took place in the clayey soil specimens during both curing and subsequent saturation (Kawam, Smith, et. al. 1975, p. 37).

Effect of Additives on Permeability of Fly Ash

An addition of lime to a clay reduced the permeability from about 10^{-5} cm/sec to 10^{-6} cm/sec (Fossberg in Sutherland & Gaskin, 1967, pp. 30).

Almost all soils at the required cement content have an extremely low coefficient of permeability, usually less than 1 x 10^{-6} cm/sec (Figure 15, & 16, P.C.A., 1975, pp. 5). Permeabilities below 10^{-6} cm/sec are impervious for all practical purposes (Figure 3). Silt and clay soils stabilized with cement have ever lower permeability coefficients (P.C.A., 1975, pp. 12).

An addition of 10 percent lime or cement to fly ash can reduce the coefficient of permeability by a factor as high as 10 (Faber & DiGioia. 1967, pp. 9). Lime and cement can

Table 2. Results of permeability tests. (From Kawam, et. al. 1975, pp 41)

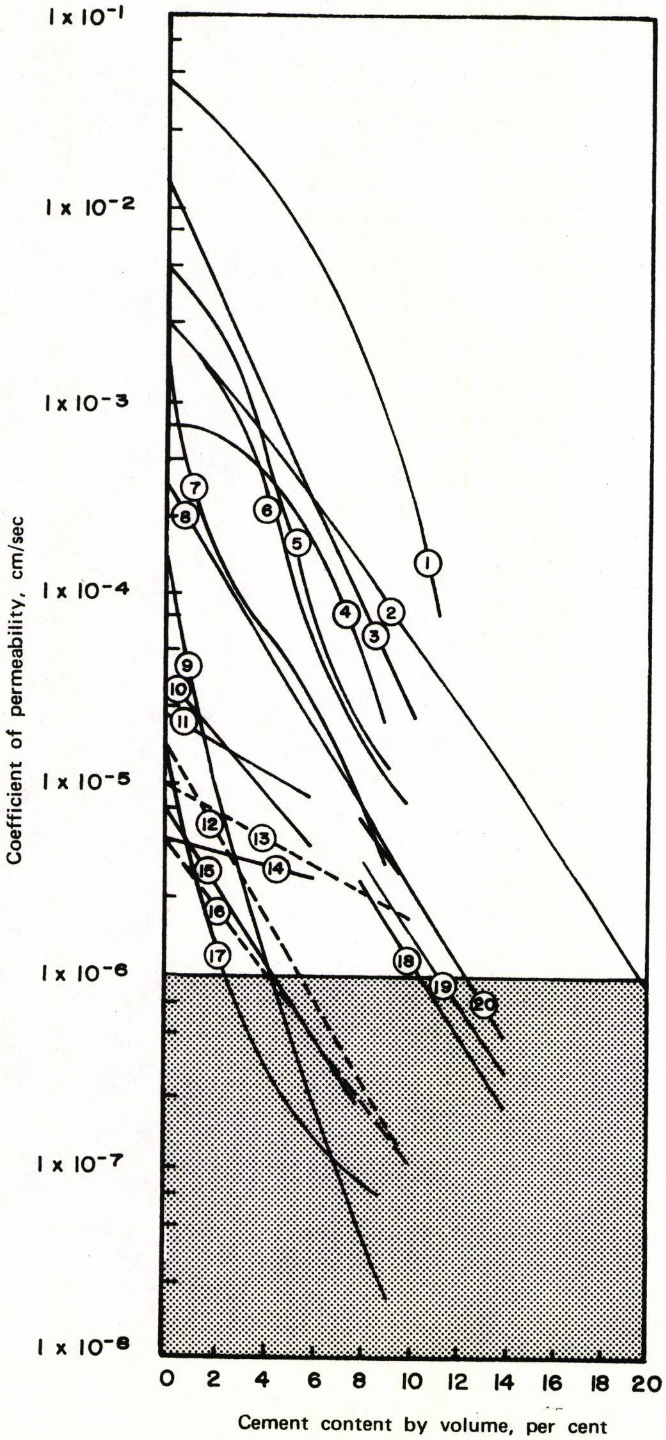
No. of Samples	Formulation[a]	Coefficient of Permeability after seven - day cure (20 C) (cm/sec)
1	C-3-0	3.70×10^{-5}
2	C-3-30 S_D	3.37×10^{-5}
2	C-3-50 S_D	3.58×10^{-5}
1	C-3-70 S_D	1.13×10^{-5}
1	C-3-30 S_L	1.84×10^{-6}
	C-3-50 S_L	So impervious was unable to get water through sample.
1	C-3-70 S_L	6.89×10^{-8}
2	B-2-0	Both samples cracked during saturation - no test.
1	B-2-30 S_D	4.23×10^{-5}
1	B-2-50 S_D	Sample cracked during curing - no test.
	B-2-70 S_D	Sample cracked during curing - no test.
1	B-2-30 S_L	1.27×10^{-6}
1	B-2-50 S_L	5.88×10^{-8}

[a]C-3 formulation: 8 percent Dolomitic lime, 16 percent Sulfate waste, 76 percent Amax Fly ash.

B-2 formulation: 5 percent Calcitic lime; 10 percent Sulfate wate, 85 percent Albright fly ahs.

S_D Danville, Virginia clayey Soil

S_L Ladyamith, Virginia Sandy Soil



NOTE: 1, 6, 17 Coarse Sand
2, 3, 4, 7, 13 Sand
5, 8, 9, 10, 11, 14, 18, 19, 20 Fine Sand
12 Loamy Sand
15 Sandy Loam
16 Loamy Fine Sand

Figure 15. Permeability of cement-treated soils (from Portland Cement Association, 1975, P. 5).

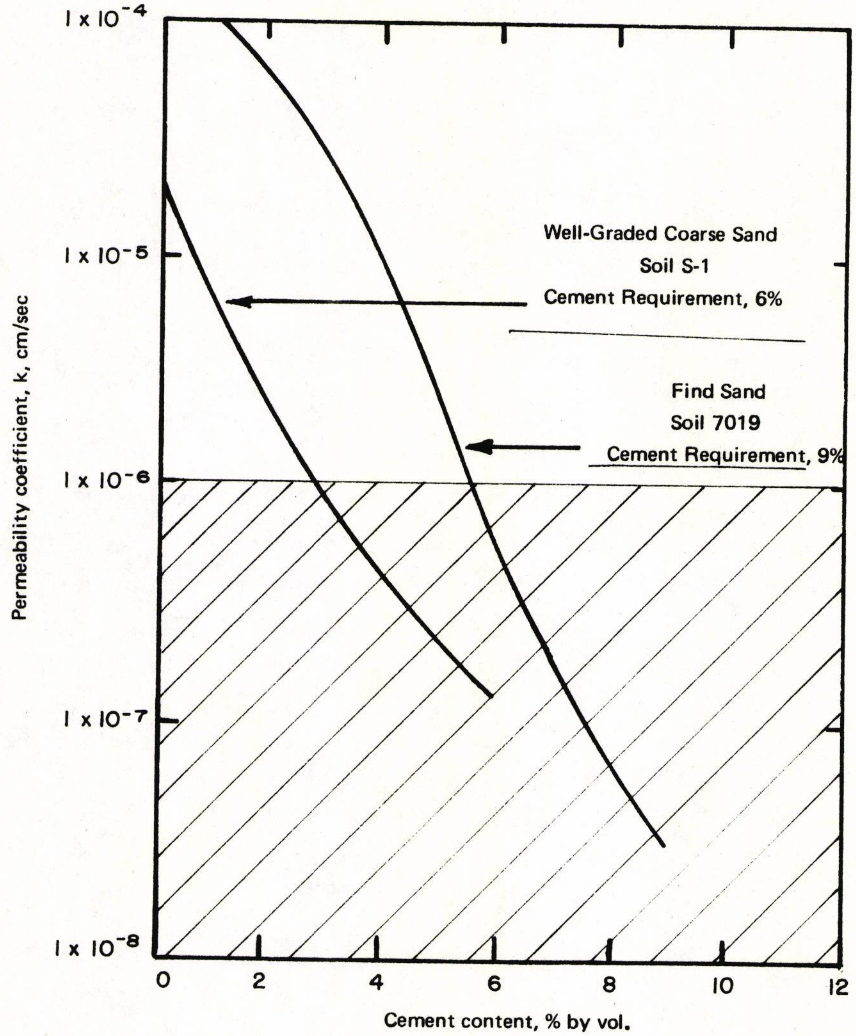


Figure 16. Effect of cement content on permeability for cement-treated subbase materials (Portland Cement Association, 1975, P. 12).

reduce the permeability of British fly ash (Sutherland & Gaskin, 1969, pp. 75) and Michigan fly ash (Gray & Lin, 1972, pp. 378). But the lime is not as effective as cement in reducing the coefficient of permeability in Michigan fly ash (Figure 17).

Relationships Between Frost Heave and Permeability of Fly Ash

Based on the mechanism of frost heave, permeability also affects the rate of water movement to the freezing front in response to the suction and formation of ice lenses within a fine grained soil. Therefore permeability may have an important effect on how much the soil frost heaves.

Frost susceptibility hinders widespread use of fly ash in load-bearing fill. Fly ash, predominantly silt size material, is highly frost susceptible (Figure 18, Gray & Lin, 1972, pp. 373). Fly ash does not usually contain a sufficient amount of free lime to provide age hardening reactions which limit frost heave.

Nine sources of fly ashes were investigated for frost heave by Road Research Laboratory. Four of the fly ashes heaved considerably. One was marginally frost susceptible. The remaining four were satisfactory (Croney and Jacobs in Sutherland, Finlay & Cram. 1968, pp. 7).

The permeability of fly ash decreases as the frost heave decreases (Figure 19, Sutherland & Gaskin, 1970, pp. 75). Sutherland and Gaskin studied four different kinds of British fly ash by means of a constant head permeameter. The permeability of three of the ashes dropped to about 4 X 10^{-6} cm/sec. the ashes became non-frost susceptible according to the R.R.L. (Road Research Laboratory) Frost Test criterion. For the fourth ash, the Barony ash, the corresponding limit of permeability was 4 x 10^{-7} cm/sec. Therefore, no one value of permeability could be applied to all four British fly ashes in order to limit the frost heave to the amount that would allow them to be classified as nonfrost susceptible (Sutherland & Gaskin, 1970, pp. 74-75).

Effects of Additives on Frost Heave of Fly Ash

Cement reduces the frost susceptibility of fly ash. An addition of 2 to 4 percent cement to frost susceptible ashes reduced the heaving and made the fly ash non-frost susceptible (Croney and Jacobs in Sutherland, Finlay & Cram, 1968, pp. 7). An addition of 10 percent cement also made a fly ash non-frost susceptible based on one British fly ash but stabilization was not based on the Road Research Laboratory Frost Test criterion

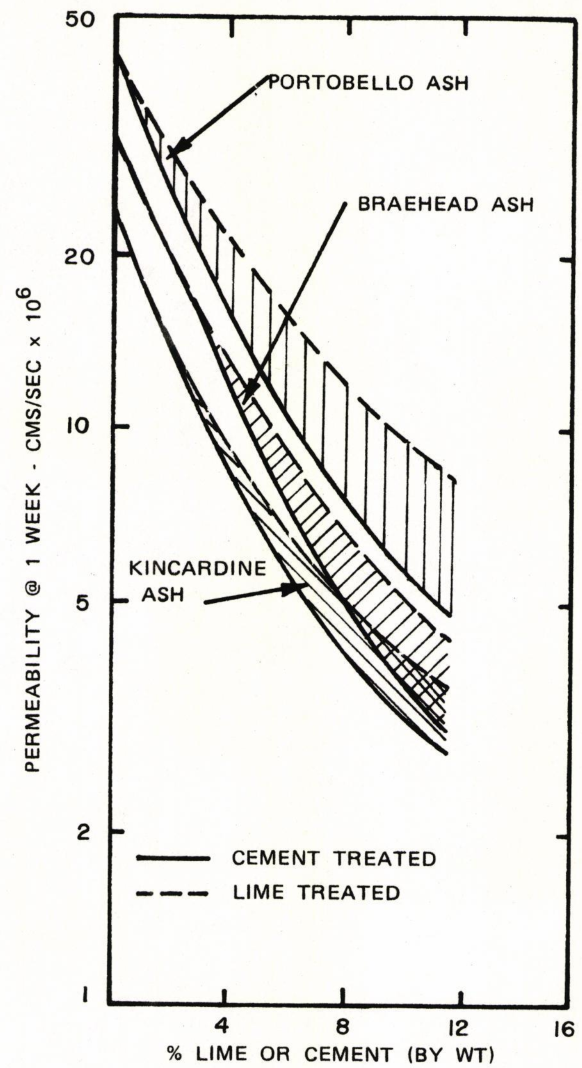


Figure 17. Effect of Lime and Cement additions on permeability of some compacted, British Fly Ashes (from Gray & Lin, 1972, P. 377).

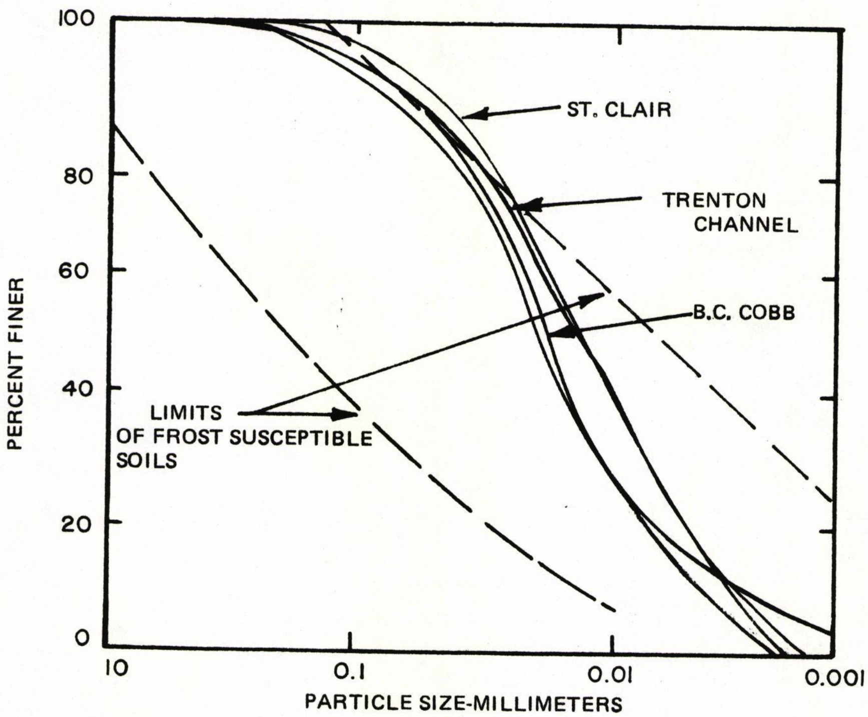


Figure 18. Grain size distribution of some typical Michigan Fly Ashes. (from Gray & Lin, 1972, P. 366).

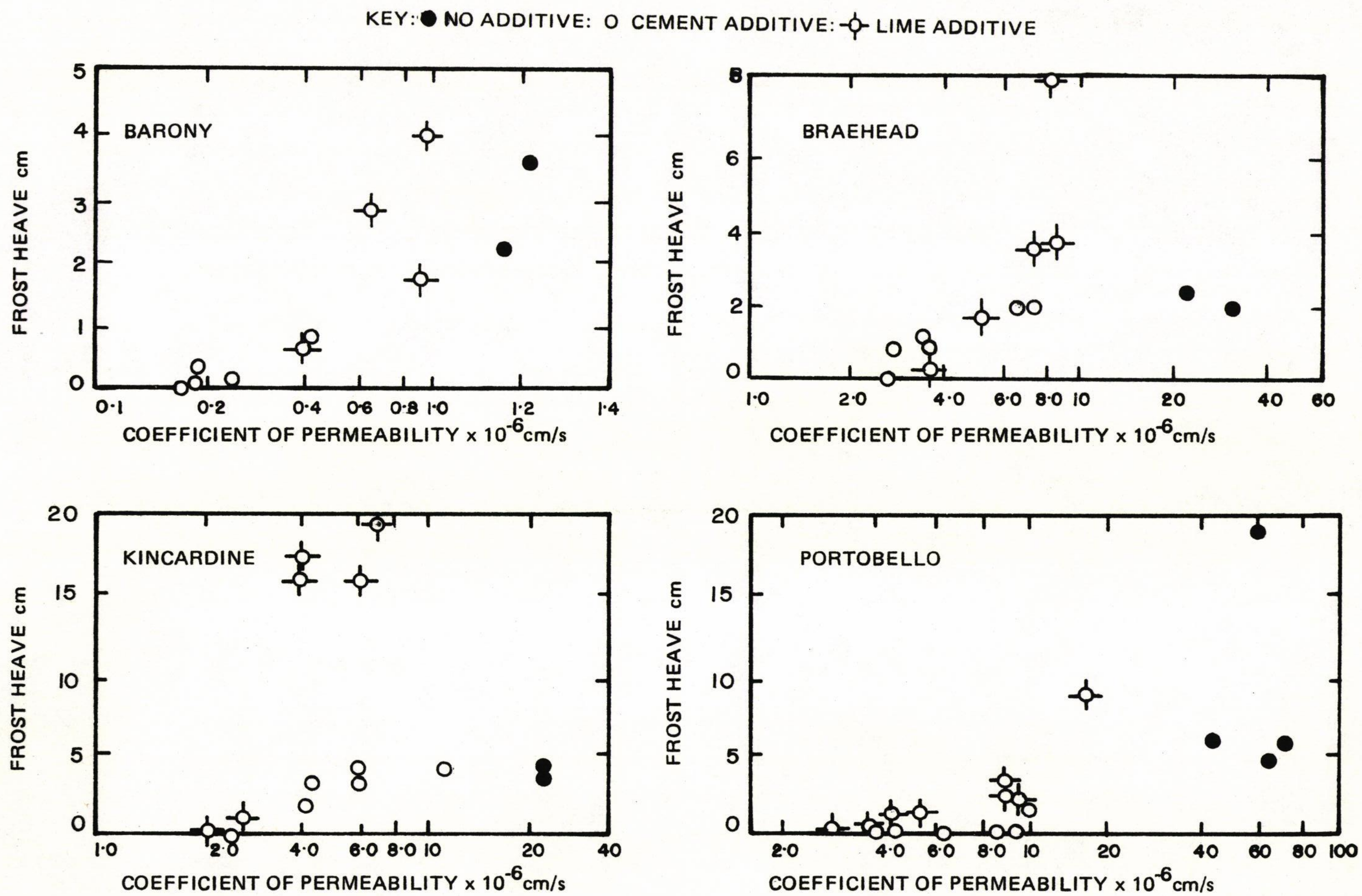


Figure 19. Plots of frost heave versus coefficient of permeability (from Sutherland, Finlay & Cram, 1970, P.75).

(Raymond and Smith in Sutherland, Finlay & Cram, 1968, pp. 8). The heaving of two Michigan fly ashes were also reduced by addition 5 and 10 percent cement (Figure 20).

Lime also reduces the frost heaving of fly ash. The use of between 6 and 15 percent of calcitic hydrated lime to reduce the frost susceptibility of four British fly ashes was recommended by Sutherland and Gaskin (1970, pp. 78). An addition of 10 percent of lime can reduce heaving of two Michigan fly ashes (Figure 20) (Gray & Lin, 1972).

Cement and Lime can reduce frost heave of fly ash because both cement and lime fill the pores and aggregate the particles of fly ash. Therefore, the flow of unfrozen water to the ice front is restricted and the heaving process is retarded. Lime is not as effective as in reducing frost heave (Gray & Lin, 1972, pp. 378). Although cement stabilized fly ashes are stronger than lime at early stages, the difference is generally eliminated within three months for most fly ashes (Sutherland, Finlay and Cram, 1968, pp. 7).

Relationship Between Heaving Pressure and Permeability of Fly Ash

If a frost susceptible soil is frozen and restrained from heaving, it exerts a pressure known as the heaving pressure. Heaving pressure can be large enough to lift foundations and damage structures.

As the permeability of British fly ash decreases, the heaving pressure increases (Figure 21, Sutherland & Gaskin, 1967, pp. 33). The heaving pressure is equal to the induced tension in the porewater (Everett in Sutherland & Gaskin, 1967, pp. 31). As the interconnecting pores decreases, the induced tension in the porewater increases (Penner and Williams in Sutherland & Gaskin, 1967, pp. 31). Since the permeability decreases as the size of the interconnecting pores decreases, heaving pressure increases as permeability decreases.

Relationship Between Compressive Strength and Permeability of Fly Ash

The compressive strength of British fly ash increases as the permeability decreases indicating that for a particular fly ash increasing amounts of additive produce a decrease in permeability (Figure 22, 23, 24 and 25). Sutherland measured the unconfined compressive strength of fly ash after 250 hours curing. The increase in compressive strength was due to the increased percent of cement and lime additive. No general linear relationship could be made between the compressive strength and permeability of the four British fly ashes.

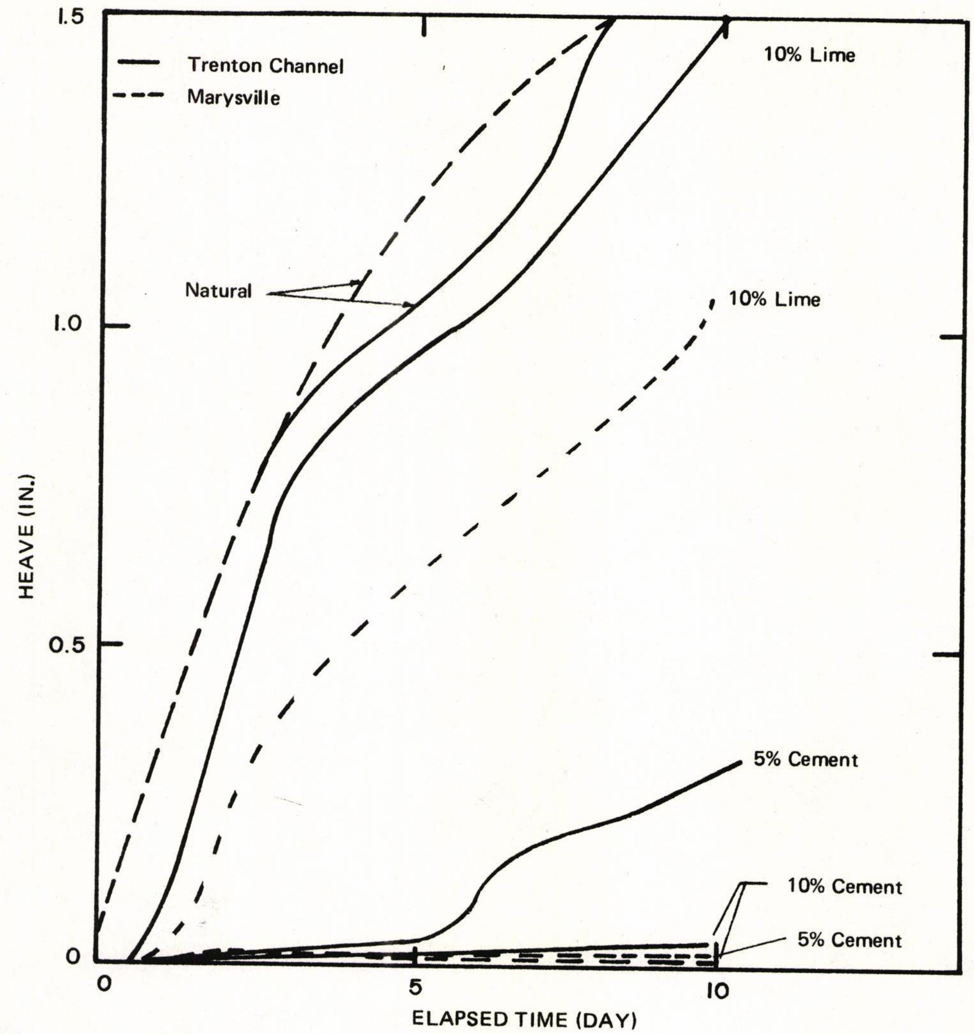


Figure 20. Effect of Cement and Lime additions on Frost Heave of two compacted Michigan Fly Ashes. (from Gray & Lin, 1972, P. 376).

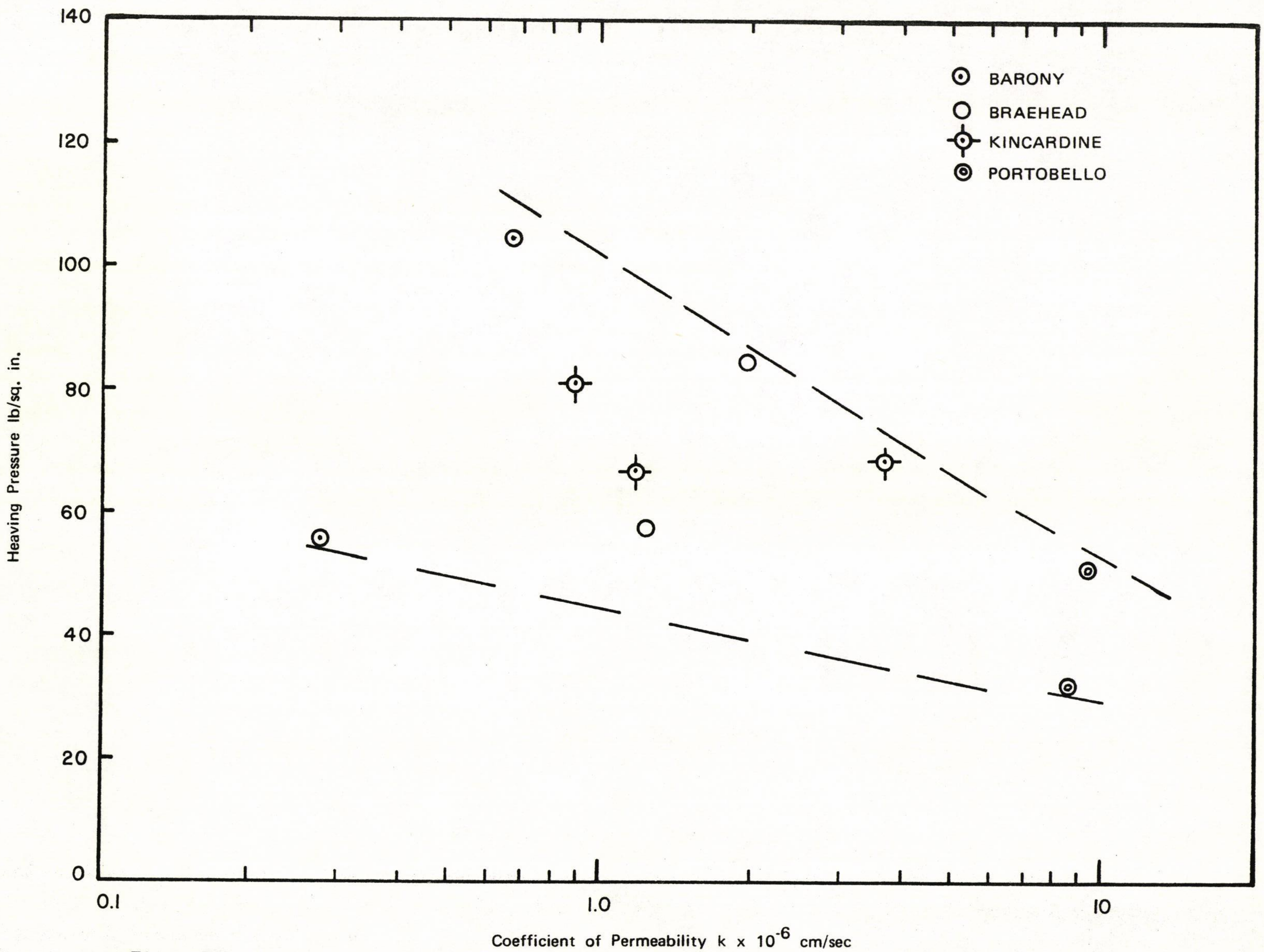


Figure 21. Heaving Pressure versus Coefficient of Permeability (from Sutherland & Gaskin, 1967, Fig. No. 9.7).

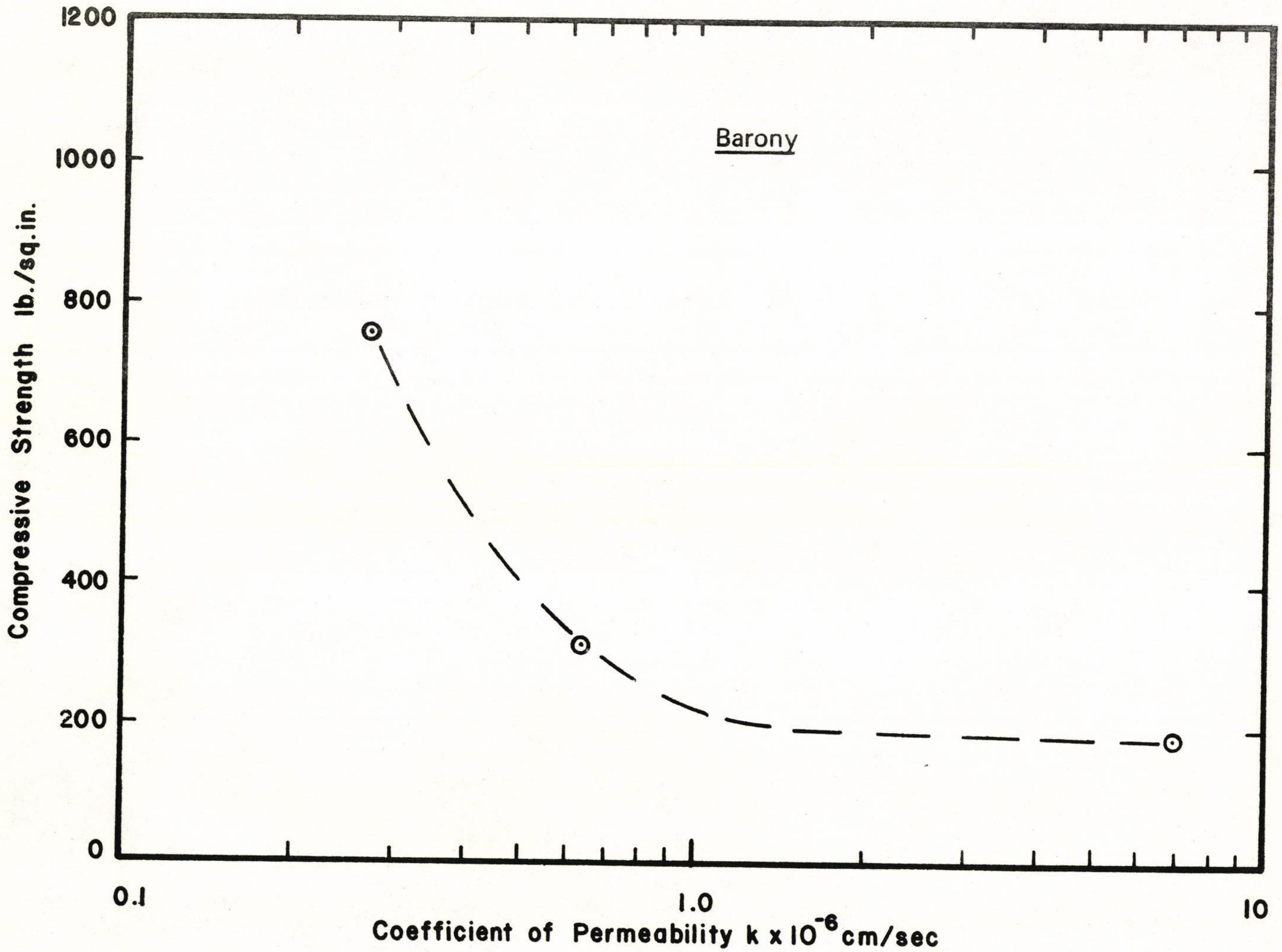


Figure 22. Compressive Strength versus Coefficient of Permeability (From Sutherland & Gaskin, 1967, Fig. No. 9.3).

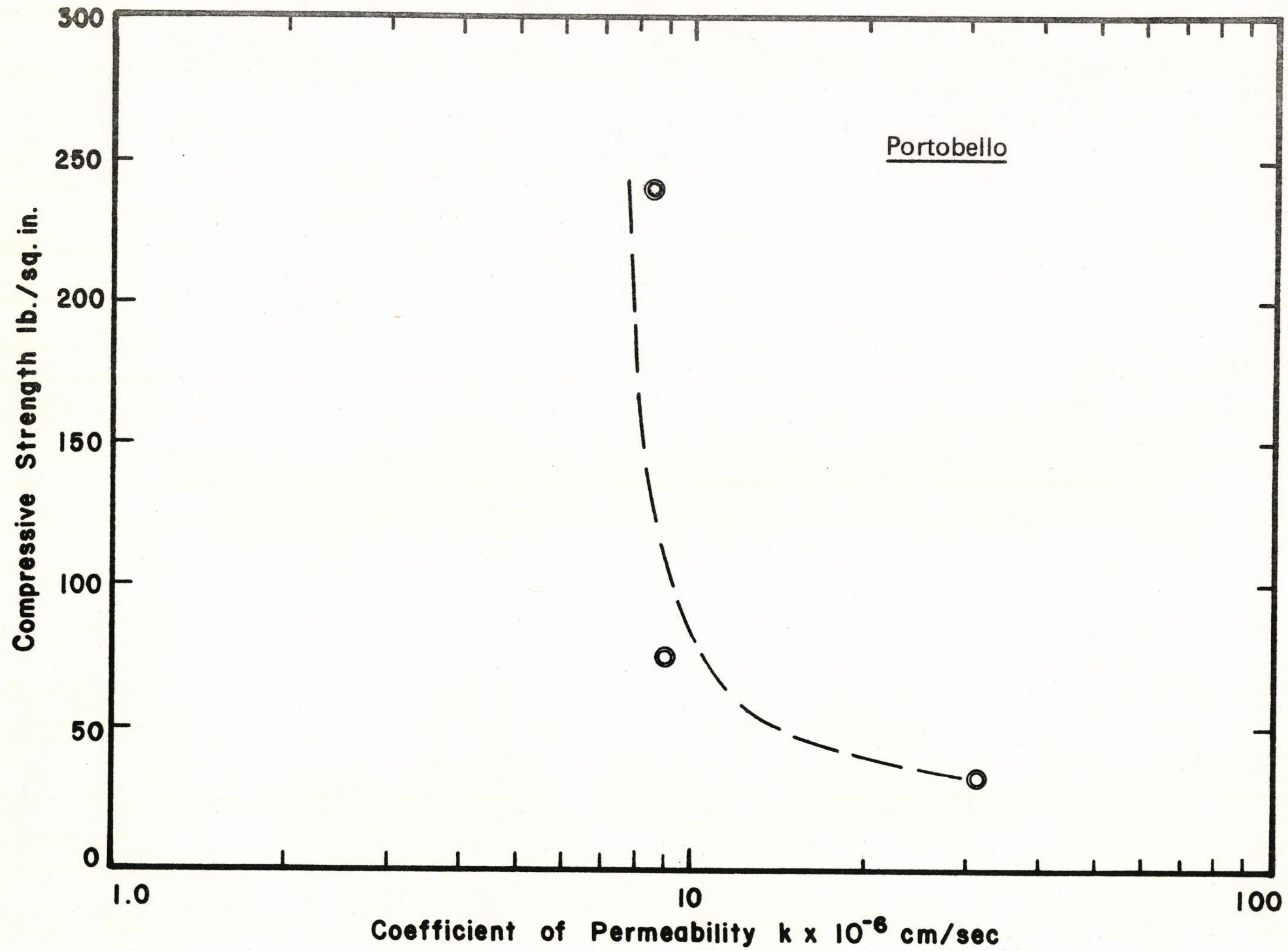


Figure 23. Compressive Strength versus Coefficient of Permeability (From Sutherland & Gaskin, 1967, Fig. No. 9.6).

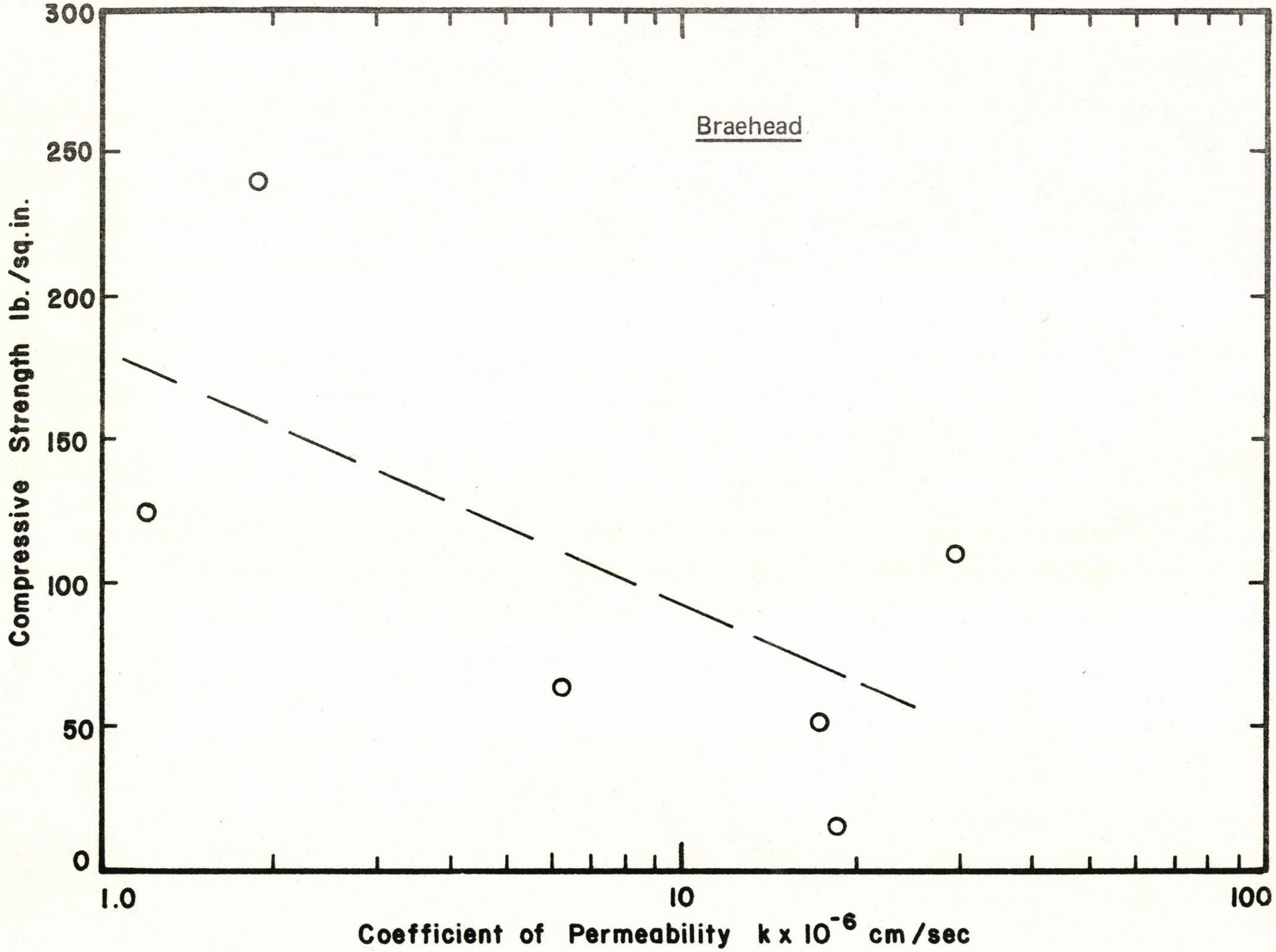


Figure 24. Compressive Strength versus Coefficient of Permeability (From Sutherland & Gaskin, 1967, Fig. No. 9.4).

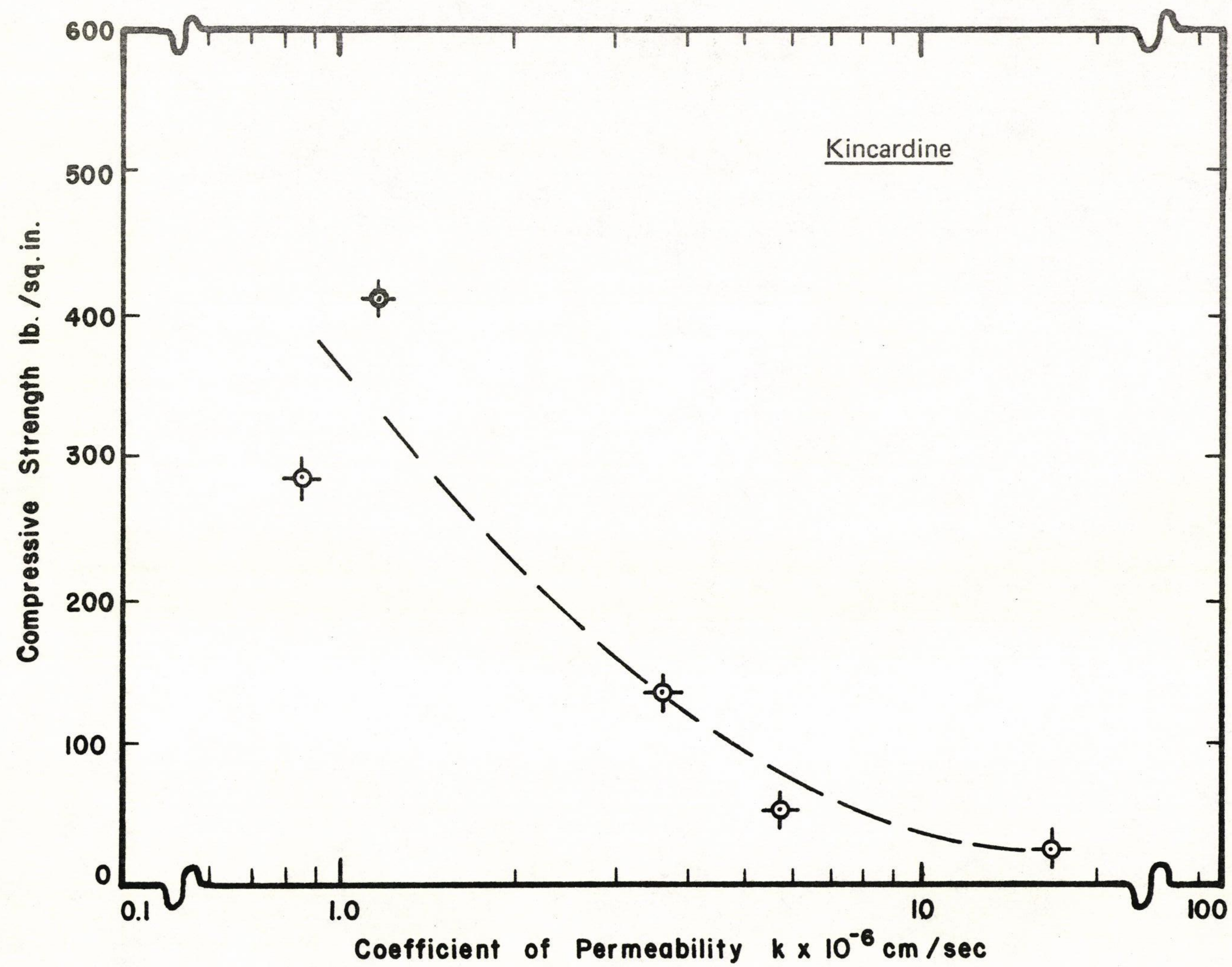


Figure 25. Compressive Strength versus Coefficient of Permeability (From Sutherland & Gaskin, 1967, Fig. No. 9.5).

Relationship Between Strength and Frost Heave of Fly Ash

Unconfined compression strength is an effective indicator of freeze-thaw resistance of soil-lime mixtures (Lime-Fly Ash Committee of Transportation Research Board, 1976, p. 11). Soil-lime samples with high unconfined strength, e.g. 300 psi, retained most of their strength. Average strength decreases for typical mixtures were 6.2 N/cm^2/cycle (9 psi/cycle) for sample cured 48 hours at 49^{o}C (120^{o}F) (Dempsey and Thompson, 1968).

Tensile strength testing is also a good replacement for the freezing-thawing test. Failure of stabilized materials in the field due to freezing-thaw action or instability can be related to tensile strength insufficient to sustain the induced tensile strain produced by freeze-thaw action (Cumberledge, Hoffman & Bhajandas, 1976, p. 22). The tensile strength of stabilized material must be overcome for an ice lens to begin to form.

As the tensile strength of British fly ash increases, the frost heave decreases (Sutherland & Gaskin, 1969, pp. 74). The tensile strength was measured by the split cylinder test.

Environmental Effects of Fly Ash

Fly ash usually contains constituents which, if released into water, could cause water pollution. A study of the fly ash used in this investigation was conducted by Reed (1976, P. 50 and 110) to determine the effect of the ash on the following water quality parameters: pH, alkalinity, hardness, ammonia, nitrate, phosphorous, sulfate, silica, aluminum, cadmium, calcium, copper, iron, lead, magnesium, manganese, nickel and zinc. The only constituents that were found to be potential water quality problems were high pH, alkalinity and hardness.

A second study (Burnett, 1975) showed that the maximum values of pH, alkalinity and hardness in the effluent from permeameters was 11.2, 580 mg/l and 640 mg/l respectively.

THE LABORATORY INVESTIGATION

INTRODUCTION

Two soil-fly ash mixtures were tested for permeability in the laboratory investigation; clay-fly ash mixtures and sand-fly ash mixtures. The high calcium fly ash used was produced from Wyoming coal. The physical and chemical properties of clay, sand and fly ash were determined in an early study (Highway Research Project 43) (Thornton and Parker, 1975, p. 31-39).

MATERIALS USED IN THE INVESTIGATION

Fly Ash

The fly ash used in this study was collected by a Research Cottrell electrostatic precipitator from a 350 megawatt Combustion Engineering boiler at the Public Service Company power station in Pueblo, Colorado. The coal, obtained from Roland and Smith seams in Campbell County, Wyoming was pulverized to pass the No. 200 mesh and then injected into the tangential burner boiler. The Pueblo fly ash has a light cream color. Photomicrographs of the fly ash show the particles to be spherical in shape. The chemical and physical properties of the fly ash are shown in Table 3, and a grain size distribution curve is presented in Figure 26.

Soils

Two soils were extensively tested in the laboratory investigation. Soil No. 1 was a clay from section 24, Township 4 South, Range 17 West of the fifth Principal Meridian in Hot Springs County, Arkansas. The clay was taken from the clay pit of the Acme Brick Company plant just east of Malvern on U.S. Highway 270. The clay is part of the Wilcox Formation (Williams and Plummer, 1951, p. 1-35). An x-ray diffraction analysis of the clay determined the predominant clay mineral to be kaolinite. The properties of the clay are given in Table 4. The clay, dark gray in color, is quite high in organic content (11.4%).

The second soil tested was a light brown, fine sand from Section 20, Township 4 South, Range 11 West of the Fifth Principal Meridian in Grant County, Arkansas. The sampling site is approximately 7 miles southwest of the site of the proposed coal-fired power station near Redfield. According to the Soil Conservation Service, General Soil Map

Table 3. Properties of Fly Ash. (From Thornton, Parker & White, 1975, p. 4).

Chemical Analysis of the Fly Ash.[a]

	Chemical Composition, % by weight
SiO_2	34.0
Al_2O_3	13.0
Fe_2O_3	6.0
CaO	20.0
MgO	6.0
K_2O	0.8
Na_2O	2.8
SO_3	13.7
TiO_2	1.0
Undetermined	2.7
	100.0

Physical Properties of the Fly Ash.[b]

Loss on Ignition	0.0%
pH	11.2
Water Soluble Fraction	1.0%
Pozzolanic Activity Index	1074.3 psi
Specific Gravity	2.75
Minimum Density	62.2 pcf
Maximum Density (Modified Proctor)	118.0 pcf
Optimum Moisture Content	9.0%
% Passing #40 Sieve	99.5%
% Passing #100 Sieve	98.0%
% Passing #200 Sieve	94.0%
% Passing #325 Sieve	86.6%

[a] Determined by Sargent and Lundy, Engineers, Chicago.

[b] Determined in the University of Arkansas Soils Laboratory.

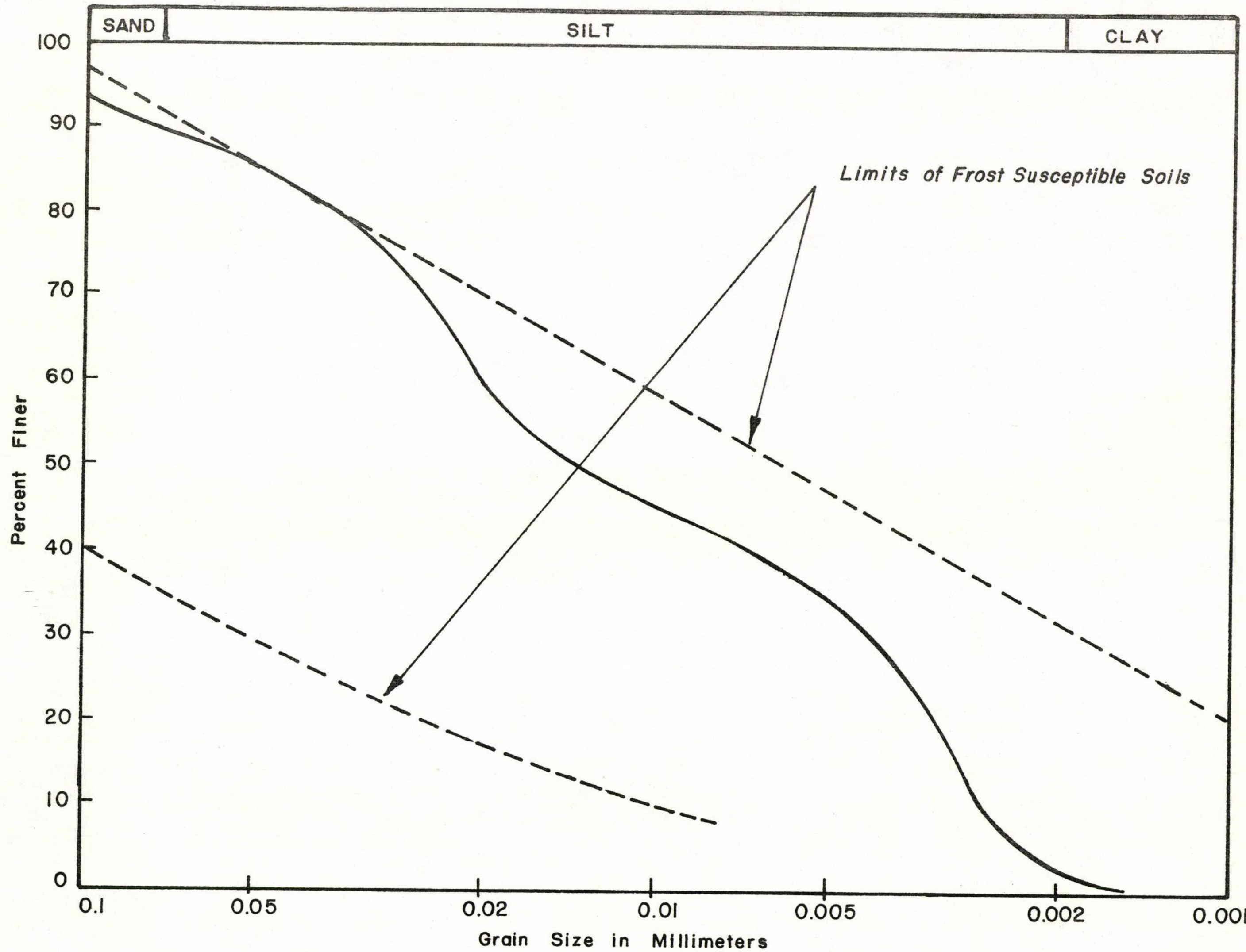


Figure 26. Grain Size Distribution Curve of Fly Ash.

Table 4. Properties of Soil #1 (Clay)*

Percent Silt	41.0%
Percent Clay	59.0%
Liquid Limit	54%
Plastic Limit	35%
Shrinkage Limit	19%
Plasticity Index	19%
Specific Gravity	2.62
pH	3.9
Organic Content[a]	14.9%
Organic Content[b]	11.4%
Predominant Clay Mineral	Kaolinite
Modified Proctor Density	97.5%
Optimum Moisture Content	20.0%
Unified Classification	OH
AASHTO Classification	A-7-5 (14)

*Thornton, Parker & White, 1975, p. 6.

[a]Determined in accordance with ASTM D 2974-71

[b]Determined in accordance with procedures outlined by Arman and Munfakh (55).

of Grant County, the soil is part of the Angie-Sacul Association. The properties of the sand are shown in Table 5. Grain size distribution curves for both the clay and the fine sand are shown in Figure 27.

PREPARATION OF MATERIALS

The fly ash was in the dry state when received and was placed in barrels to protect the contents from moisture in the air.

The soils were sampled by hand in the field and carried in large sample bags to the laboratory. The samples were placed in large pans and allowed to dry in ovens at temperatures not exceeding 140°F.

After drying, the sand could be easily crumbled by hand or by means of a laboratory jaw crusher. The clay developed hard lumps upon drying. The lumps were broken into smaller lumps by means of the jaw crusher. These small lumps of clay were further pulverized by placing the soil in a laboratory disc type material grinder. Care was taken so that soil would not be ground so fine as to excessively disturb the structure. The clay was pulverized to the extent that all the soil passed the No. 10 sieve. After preparation, both the soils were placed in large covered cans for storage.

Mixing the Soil-Fly Ash Mixtures

Two methods of mixing the soil-fly ash mixtures were used throughout the laboratory investigation.

Method one - The constituents were proportioned and dry mixed in a Hobart 1/8 H.P. mixer for three minutes.

Method two - The constituents were proportioned and dry mixed by hand. The mixture was then dry mixed in a Hobart 1/8 H.P. mixer for one minute. Next, a predetermined quantity of water was added and the mixture was stirred in the mixer for one minute. The sides of the bowls were scraped clean by hand and then mixing was continued for an additional one-half minute.

Compacting the Soil-Fly Ash Mixture

Two methods of compacting were used throughout this laboratory investigation.

Method One - The soil-fly ash-water mixtures were compacted with Standard or Modified Proctor compactive effort in accordance with ASTM D 698-70 or ASTM D

Table 5. Properties of Soil #2(Sand)*

Percent Sand	92.9%
Percent Silt	1.8%
Percent Clay	5.3%
Liquid Limit	NP
Plastic Limit	NP
Plasticity Index	NP
Permeability at 68% relative density	3.3×10^{-3} cm/sec
Specific Gravity	2.67
pH	4.3
Organic Content	0.75%
Minimum Density	76 pcf
Maximum Density (Modified Proctor)	99 pcf
Optimum Moisture Content	8.0%
Predominant Clay Mineral	ND
Unified Classification	SP-SMu
AASHTO Classification	A-3

* Thornton, Parker & White, 1975, p. 6.

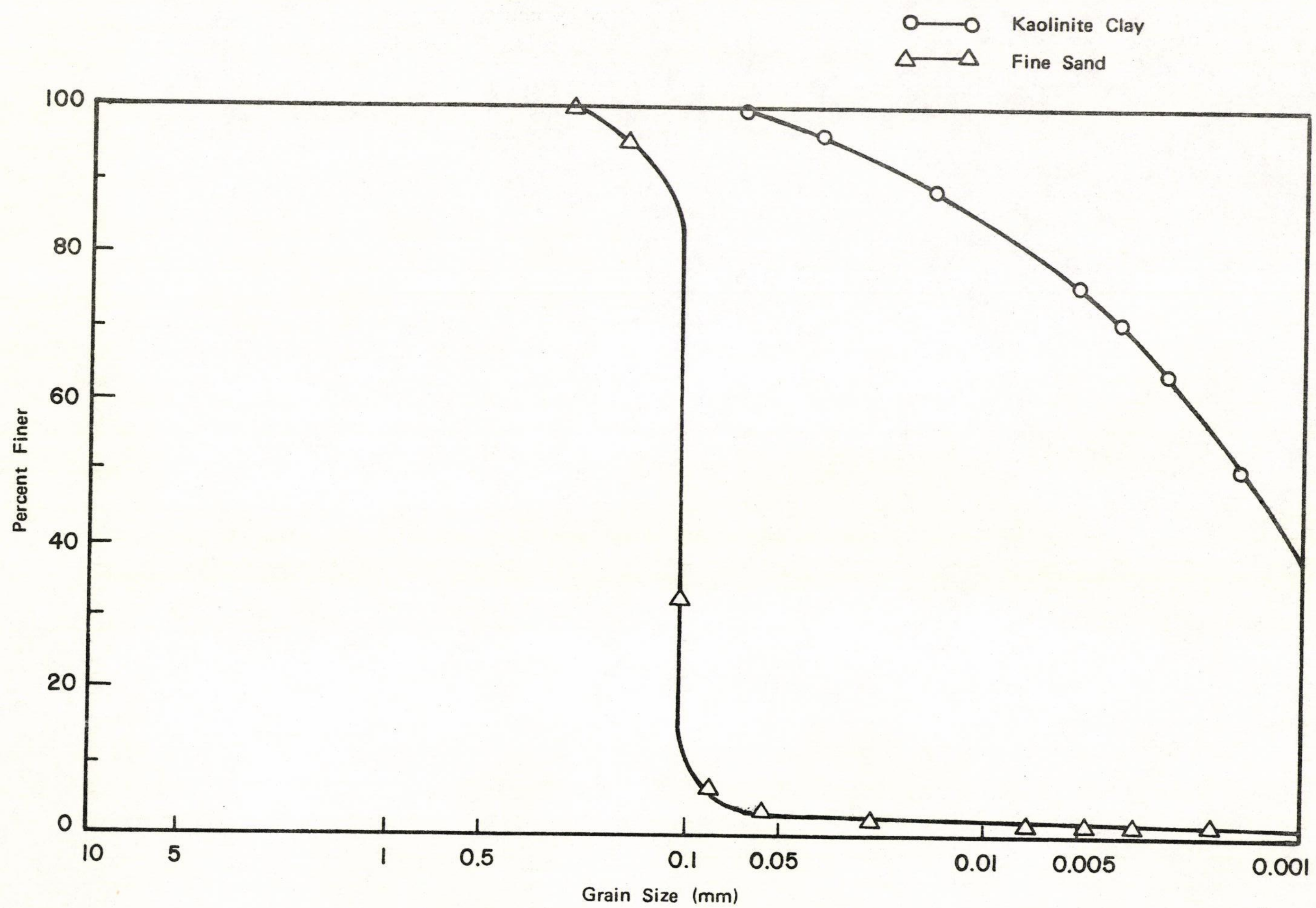


Figure 27. Grain Size Distribution of Clay and Sand (from Thornton & Parker, 1975, P. 39).

1557-70 in a compacting permeameter. A Rainhart automatic laboratory compaction apparatus equipped with a sector-faced tamper was used. All mixtures were compacted immediately after mixing. Compaction being completed within 5 minutes of mixing.

Method Two - The soil-fly ash dried mixtures were compacted in accordance with ASTM D 2434-68 in a clear plastic permeameter. A sliding tamper with a tamping foot 51 mm in diameter, and a rod for sliding weights of 100g to 1 kg, having an adjustable height of drop from 102 mm to 254 mm, were used (Figure 28).

Saturation of Soil - Fly Ash Specimen

Plastic (ASTM) permeameters: The sand-fly ash specimen was evacuated from the bottom upward under 5 in. Hg vacuum pressure for 20 minutes to remove air adhering to soil particles and from the voids. The clay-fly ash specimen were evacuated from the bottom upward under 25 in. Hg. vacuum pressure for 12 hours.

Compacting permeameters: The sand-fly ash specimen were evacuated from the top downward under 5 in. Hg. vacuum pressure for 20 minutes. Clay-fly ash specimen were evacuated from top downward under 25 in. Hg. vacuum for 48 hours.

PERMEABILITY TESTING

The apparatus for measuring the permeability of soil-fly ash mixtures was set up based on ASTM-D 2434-68 (Figure 29). The apparatus was modified so that both constant head test and falling head test could be run.

The low permeability of soil-fly ash mixtures was most suitable for the falling head test. Therefore, the falling head test was used throughout this laboratory investigation for the plastic permeameter (Figure 30) and the compacting permeameter.

The specimens in the plastic permeameters were prepared for running permeability tests in accordance with ASTM-D 2434-68 as follows:

1. A plastic collar and a plastic plate were placed at the bottom of plastic permeater for supporting the sample.
2. A wire screen and a filter paper were put on the top of plastic plate.
3. The soil-fly ash mixtures were placed and compacted.
4. The sample was covered with a filter paper, a wire screen and a spring.

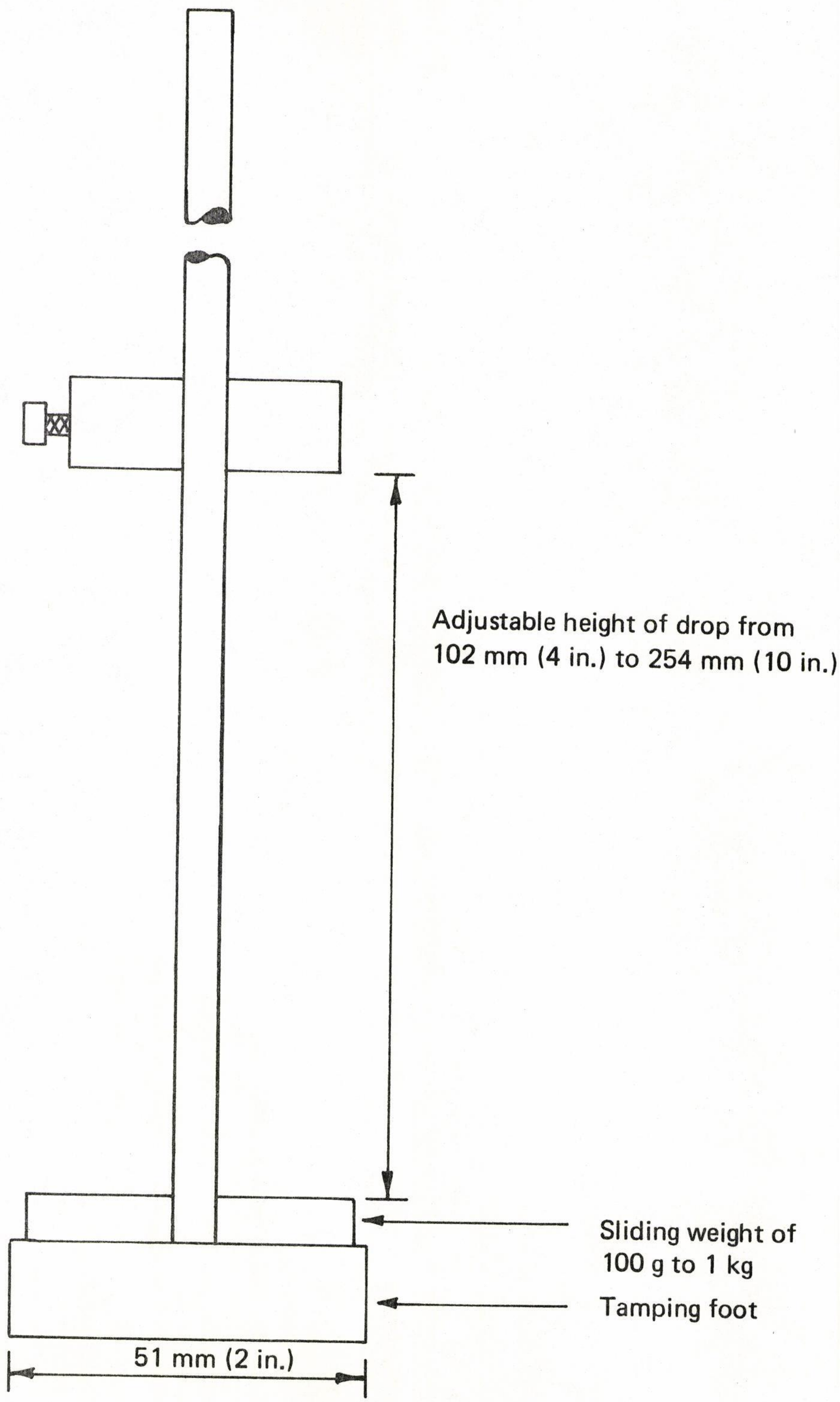


Figure 28. Schematic diagram of ASTM Compacting Device.

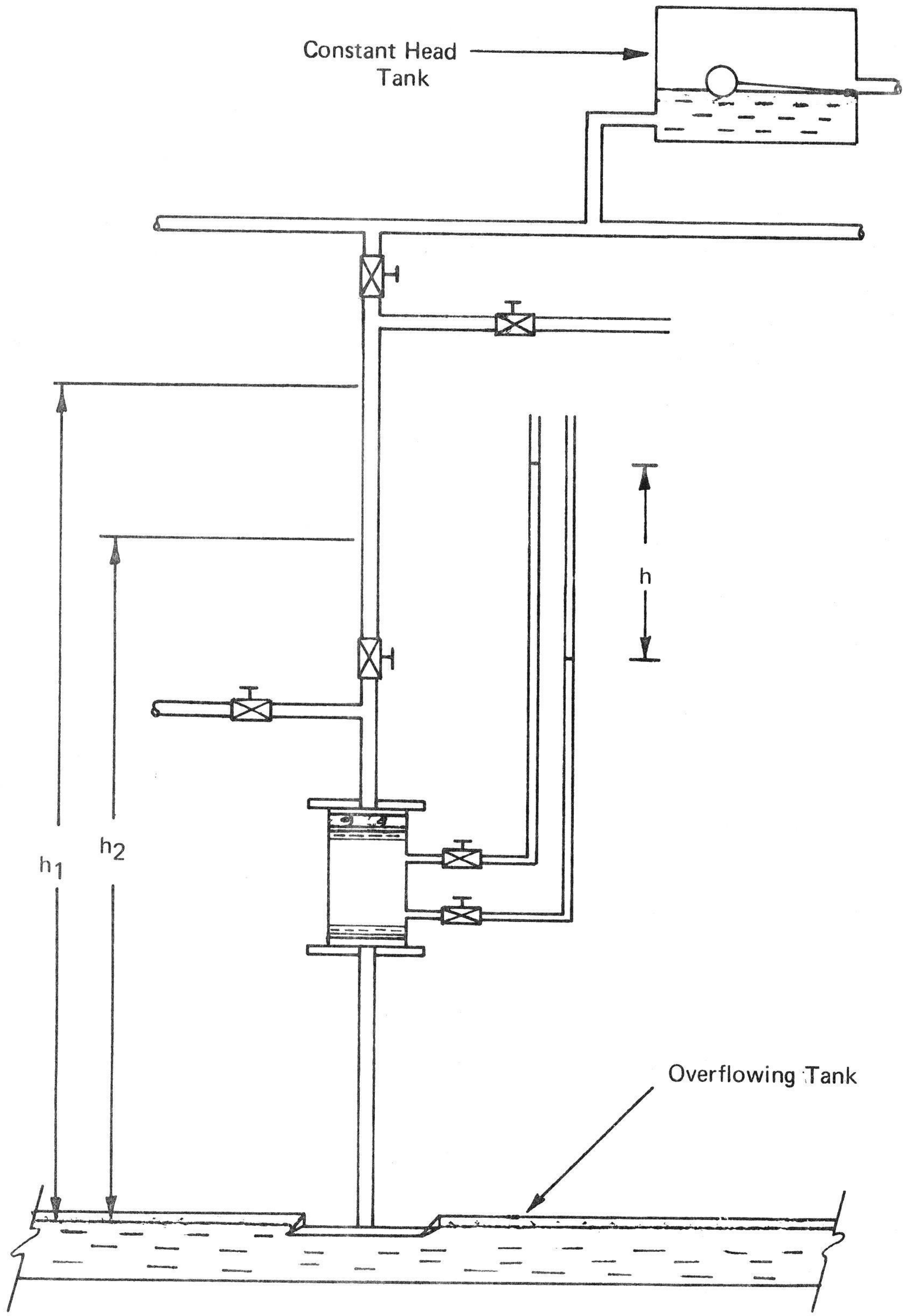


Figure 29. Schematic diagram of apparatus measuring the permeability of soil-fly ash mixtures.

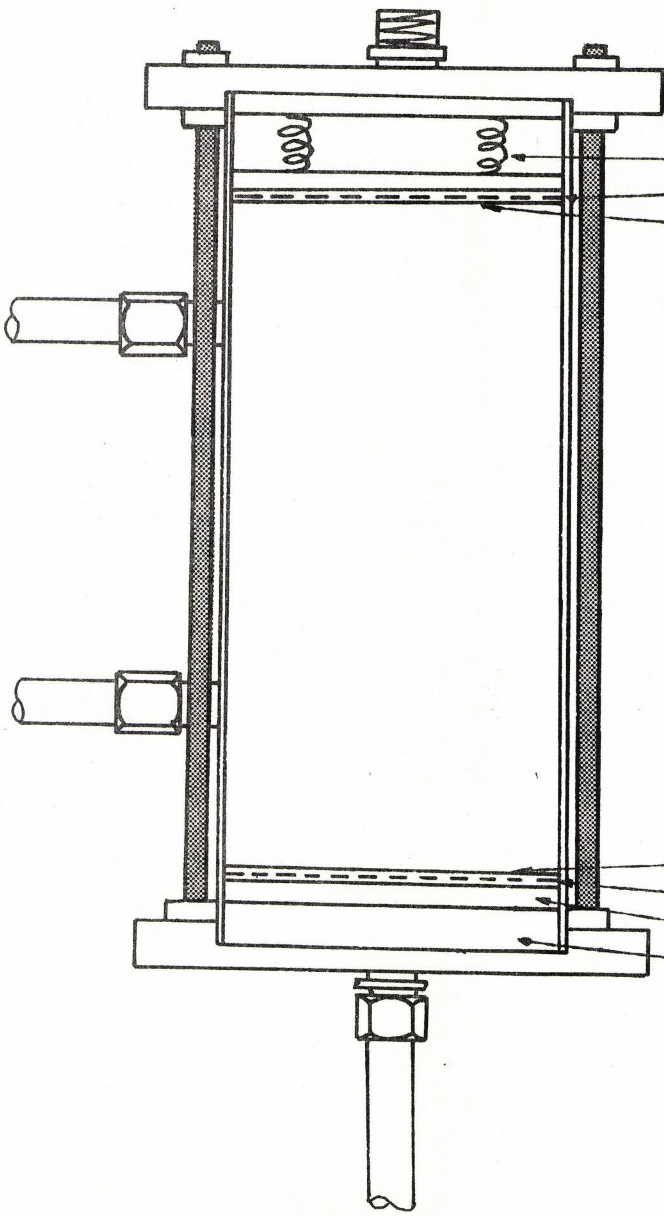


Figure 30. Schematic diagram of plastic permeameter.

5. The spring was compressed so that it would apply a pressure to soil-fly ash mixtures and help to keep the soil-fly ash sample in place during saturation.
6. The top of permeameter was positioned.
7. The diameter of standpipe and permeameter and the length of sample were measured.
8. The weight of the empty permeameter and the weight of full soil-fly ash mixtures permeameter were also recorded.

The specimens in compacting permeameter were prepared for running permeability tests as follows:

1. The soil-fly ash mixtures were poured into the compaction mold permeameter and compacted by Rainhart automatic laboratory compaction apparatus.
2. A plastic plate, a wire screen and a filter paper were inserted into the base of permeameter.
3. The compaction mold was fixed in the permeameter.
4. A wire screen and filter paper was placed on the top of compaction mold.
5. The collar and the top of permeameter were positioned.
6. The diameter of standpipe and compaction mold and the length of sample were measured.
7. The wieght of the empty permeameter and the weight of the permeameter with sample were also recorded.

To run a falling head test, a soil-fly ash sample was set up below a vertical standpipe. The sample was saturated by applying vacuum pressure to remove air bubbles. The apparatus was then filled with water to a convenient mark on the standpipe, say h_1, which was the height above the water level of the overflow tank. Water was permitted to run through the soil driven by the head in the standpipe until the level in the standpipe dropped to a second mark, say h_2, which was also the height above the water level of the overflow tank. The time for this fall was recorded.

The coefficient of permeability was calculated by applying Equation 2-9.

$$k = 2.303 \left[\frac{aL}{A\,(t_2 - t_1)} \log \frac{h_1}{h_2} \right]$$

where a = cross-sectional area of the standpipe

L = length of soil sample

A = cross-sectional area of the permeameter

t_1 = initial time the water level in the standpipe is at h_1

t_2 = final time the water level in the standpipd is at h_2

h_1, h_2 = the heads between which the permeability is determined

TEST RESULTS

The coefficient of permeability of soil-fly ash mixtures was determined by the falling head test at room temperature. Conversion was made to the standard temperature, 20°C, by the chart in Figure 4. Each point in the figures of permeability was the average of two or more determinations.

The falling head test was used throughout this study because the permeabilities measured were generally low and therefore in the range where the falling head test was most appropriate.

However, a comparison between falling and constant head permeability was made for the sand sample. The falling head test indicated a permeability of 3.3×10^{-3} cm/sec, while the constant head test indicated 3.8×10^{-3} cm/sec. The difference in permeabilities is small and probably due to the resistance to flow caused by the filter paper used in the falling head test.

The filter paper used in the study had a permeability of 1.9×10^{-2} cm/sec over a length of 15.4 cm. Because of the small effect, permeabilities reported include losses through the filter paper.

A 1/2 inch porous stone, which is often used in the permeability test, had a permeability of only 1.5×10^{-2} cm/sec., however, stones were not used in this study because of the possibility of the fly ash clogging the stone.

The hydraulic heads used in permeability testing during this study ranged from 7.5 to 13.5. To test the validity of Darcy's law at three hydraulic gradients, test were run at different gradients (Figure 31). Since no change in permeability was observed with change in gradient, Darcy's law was assumed to be valid under the conditions of this study.

SOIL-FLY ASH MIXTURES COMPACTED DRY BY ASTM-D 2434-68 STANDARD

Effect of Fly Ash on Density of Soil

The dry density of sand-fly ash mixtures increased as the percentage of fly ash increased (Figure 32). The sand-fly ash mixtures were compacted dry by an ASTM-D 2434-68 sliding tamper and a rod with a weight of 100 g. and a height of drop of 102 mm. As the percentage of fly ash increased in sand-fly ash samples, the more difficult the sample was to compact and the mixtures tended to be thrown out of the mold during compaction.

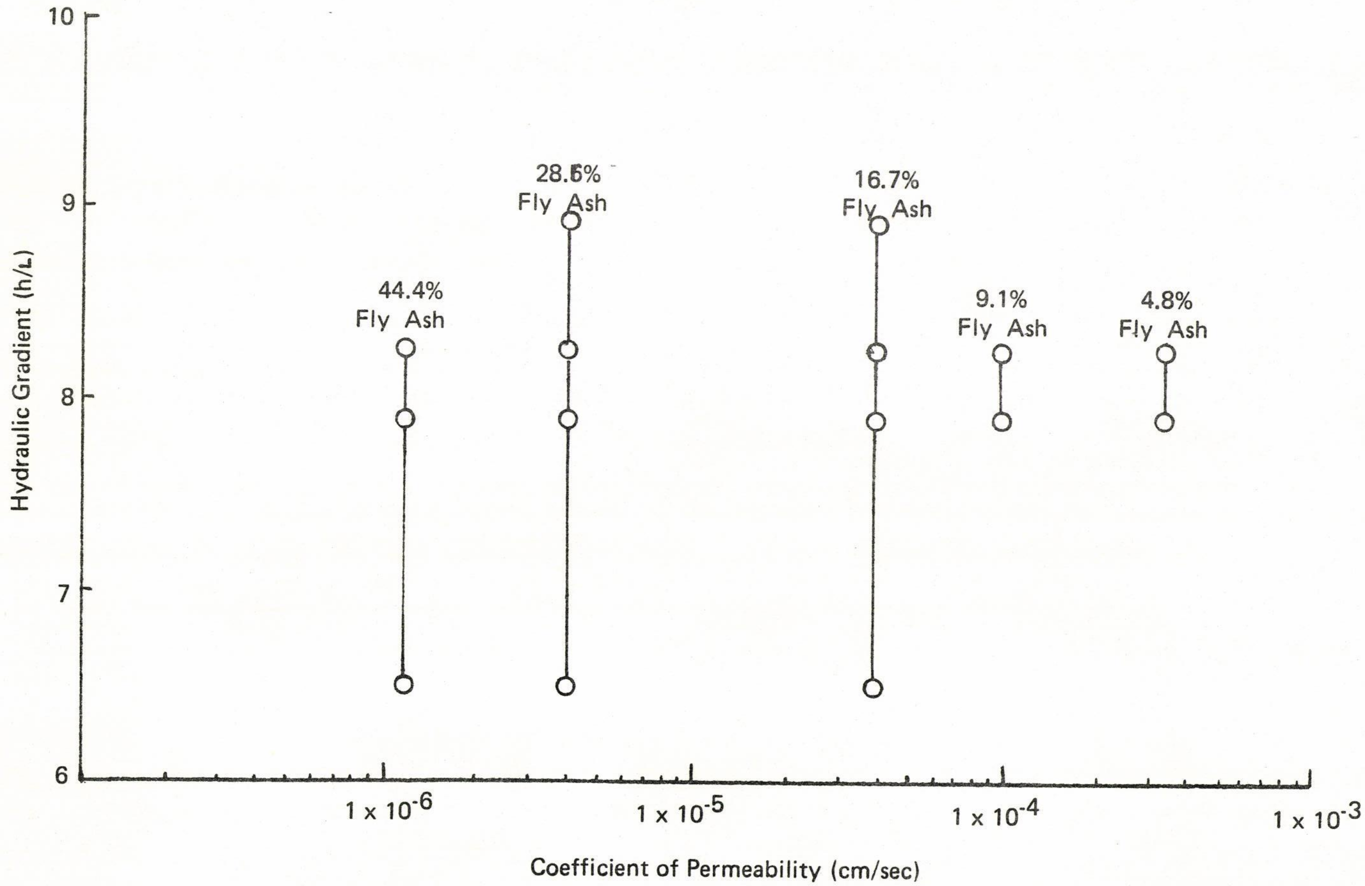


Figure 31. Hydraulic Head versus Coefficient of Permeability for sand fly-ash mixtures.

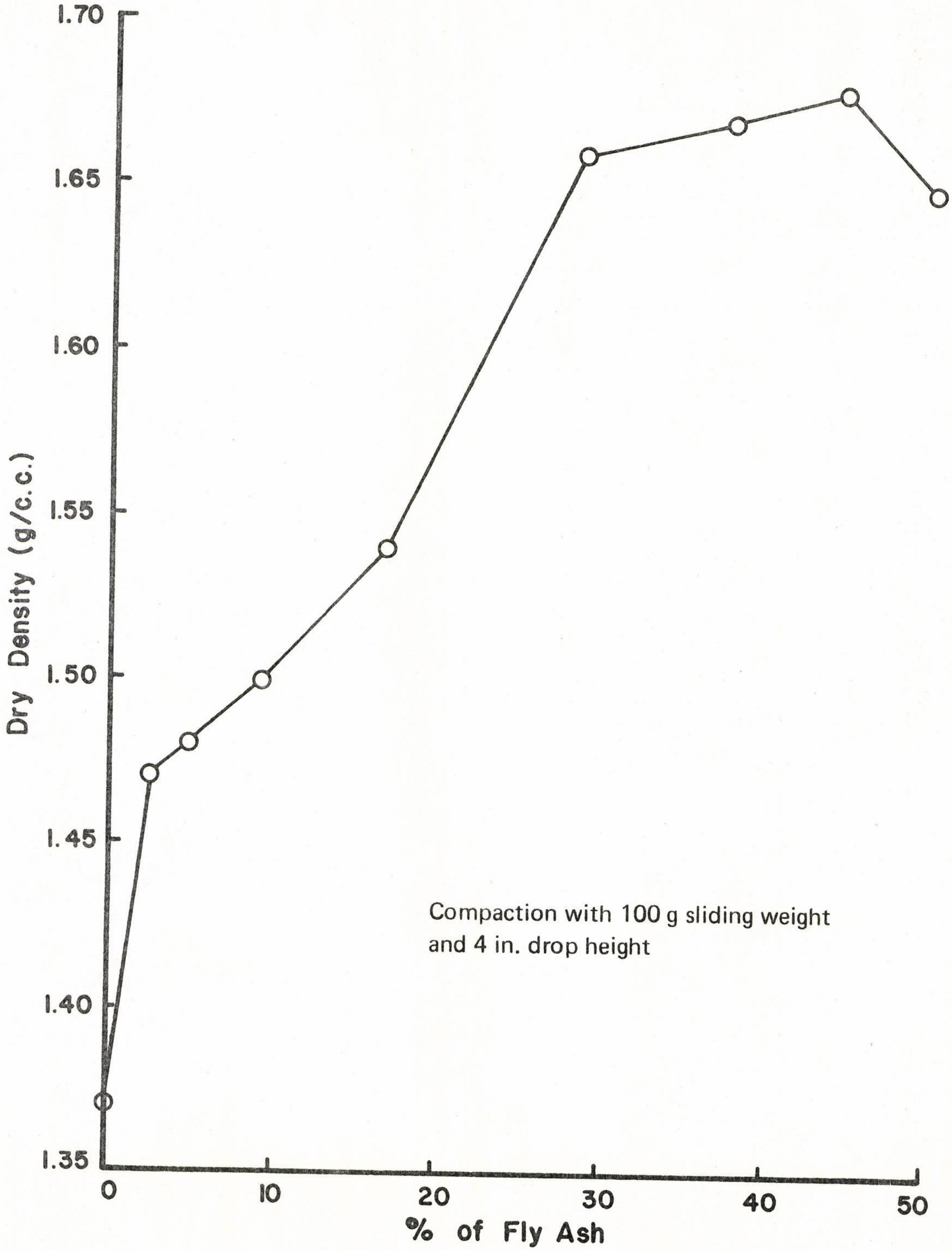


Figure 32. Effect of Fly Ash on dry density of sand.

The dry density of 100% sand was 1.37 g/cc. The dry density of sand-fly ash mixtures was increased up to 1.68 g/cc for 44.4% fly ash and 55.6% sand samples. Part of the increase in the dry density of mixtures is due to a higher S.G. of fly ash, 2.75, compared to sand's 2.67. But for 50% fly ash-sand mixtures, the dry density dropped down to 1.65 g/cc. Dynamic compaction was not effective with samples with 50% or more fly ash.

The dry density of clay-fly ash increased as the percentage of fly ash increased (Figure 33). The clay-fly ash mixtures were compacted dry by the same method as the sand-fly ash mixtures. The dry density of mixtures increased from 1.09 g/cc (0% fly ash) to 1.33 g/cc (50% fly ash). Part of the increase in dry density of clay-fly ash mixtures is due to a higher S.G. of fly ash, 2.75, vs. clay's 2.62.

For the 100% fly ash sample compacted dry, the dry density was only 1.28 g/cc, which is lower than the dry density of sand (1.37 g/cc), but higher than the dry density of clay (1.09 g/cc).

Effect of Fly Ash on Permeability of Soil

As the percentage of fly ash increased, the coefficient of permeability of sand-fly ash mixtures decreased (Figure 34). The coefficient of permeability of sand-fly ash mixtures decreased from 3.3×10^{-3} cm/sec to 1.5×10^{-6} cm/sec, as the percent of fly ash increased up to 50%. The self-hardening process and chemical reaction of fly ash cemented the sand particles and may have to reduce permeability.

As the percent of fly ash increased, the coefficient of permeability of clay-fly ash decreased (Figure 35). The clay-fly ash mixtures were compacted dry by the ASTM-D 2434-08 method. The coefficient of permeability of clay-fly ash mixtures decreased from 8.0×10^{-6} cm/sec (0% fly ash) to 2.0×10^{-6} cm/sec (50% fly ash). The fly ash was less effective in reducing the permeability of clay than sand.

For 100% fly ash sample compacted dry, vertical cracks formed (Figure 36) creating secondary permeability. The vertical cracks were due to the chemical reaction and volume change of fly ash. The measured coefficient of permeability of fly ash under this condition was found to be about 1.2×10^{-3} cm/sec, only a little lower than value of pure sand (3.3×10^{-3} cm/sec).

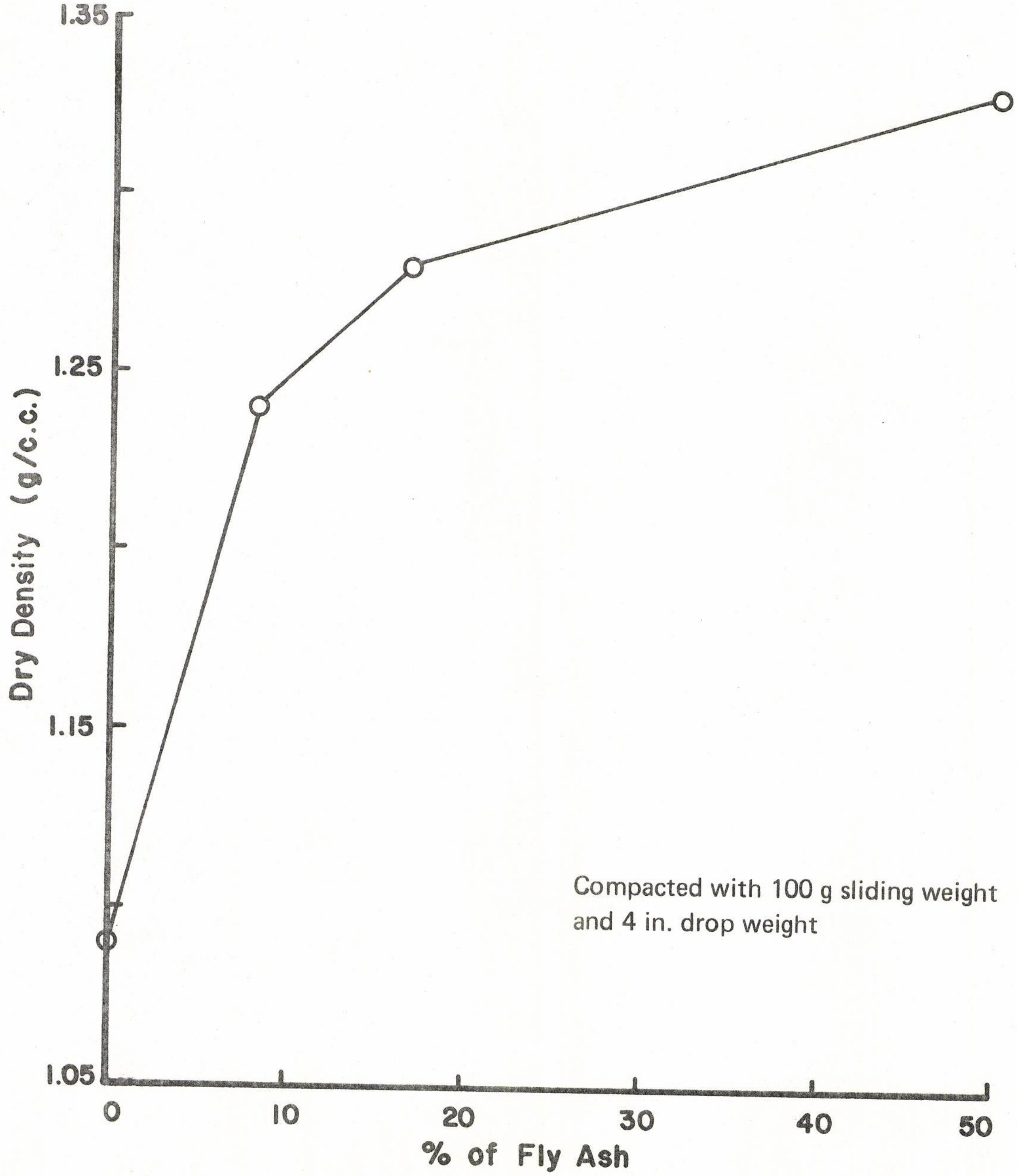


Figure 33. Effect of Fly Ash on dry density of clay.

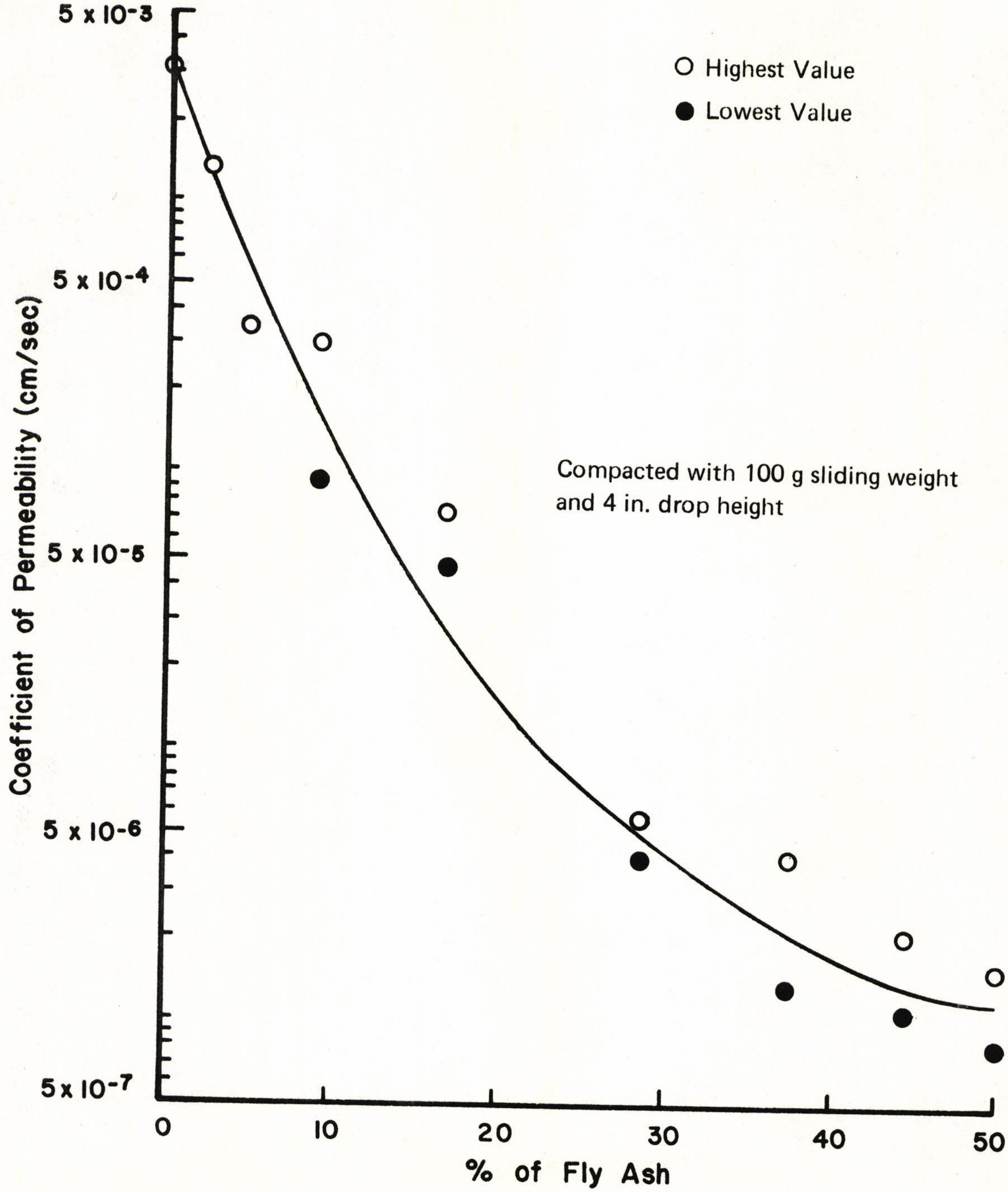


Figure 34. Effect of Fly Ash on permeability of sand.

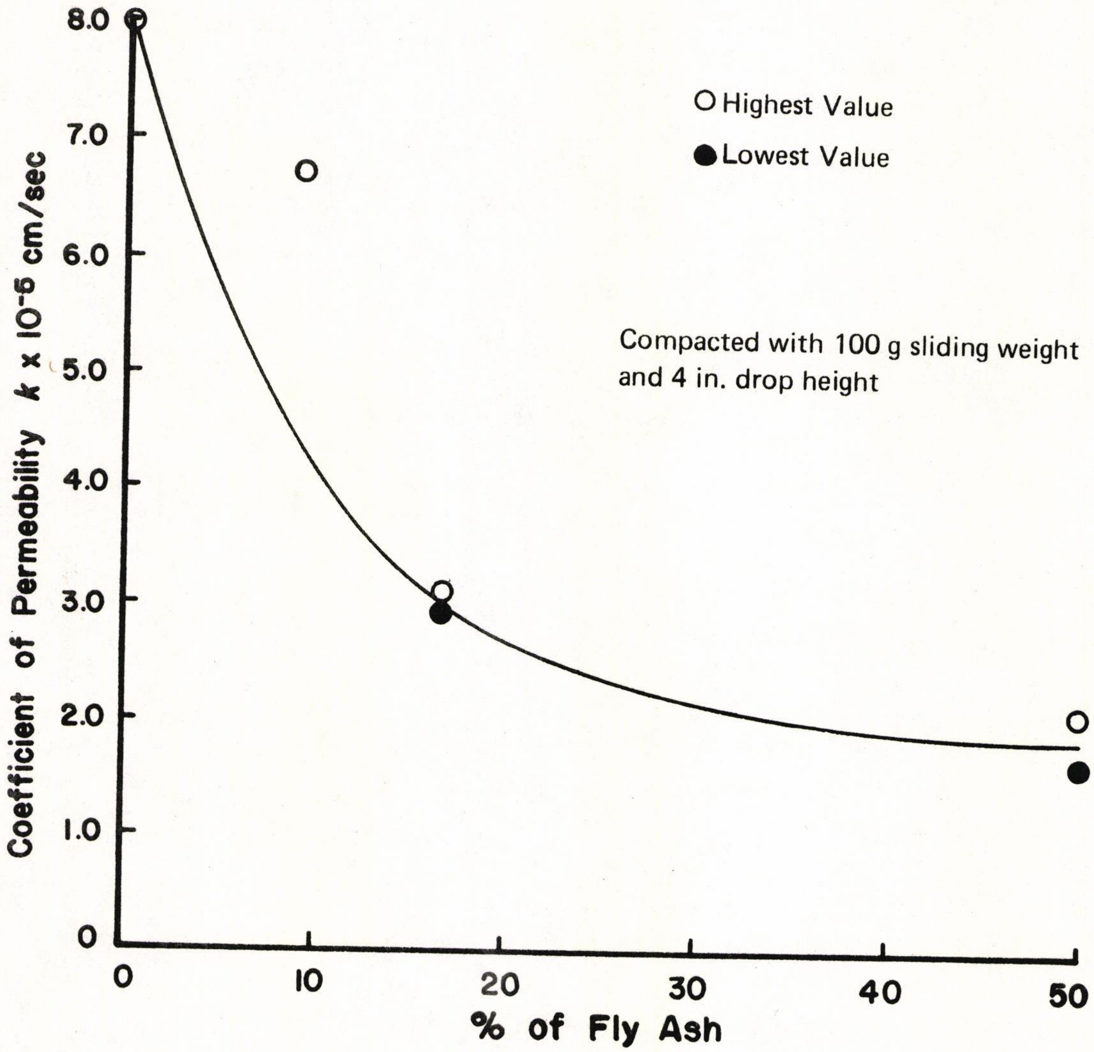


Figure 35. Effect of Fly Ash on permeability of clay.

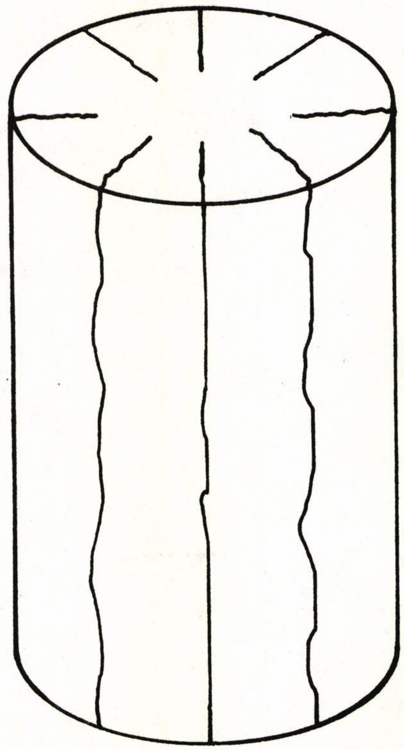


Figure 36. Cracks pattern which developed in 100% Fly Ash sample during saturation.

Effect of Time on Permeability

The coefficient of permeability was found to change with time. Sand-fly ash samples were kept in permeameters in a saturated condition without water flowing and tested periodically for up to 13 days (Figure 37). In general, the coefficient of permeability decreased to a constant value for each of the fly ash mixtures used. The decrease in permeability with time was probably due to chemical reactions between the fly ash and sand.

A second extended time study was conducted using a 16.7% sand-fly ash sample compacted dry by an ASTM-D 2434-68 tamper (100g weight at drop height of 102 mm) (Figure 38).

For the first 23 days, water only flowed through the permeameter during the actual testing and between 23 and 80 days, the water was allowed to flow continuously.

During the first 23 days, the coefficient of permeability decreased from an initial 7.5 x 10^{-5} cm/sec to a constant 3.5 x 10^{-5} cm/sec. After continuous water flow was started at 23 days, the permeabilities increased to approximately 1.5 x 10^{-4} cm/sec and remained relatively constant for the remainder of the test period.

Effect of Compactive Effort on Soil-Fly Ash Mixtures

As relative density of sand-fly ash mixtures increased, the coefficient of permeability decreased (Figure 39). While the porosity of sand-fly ash mixtures increased, the coefficient of permeability increased (Figure 40). The 20% fly ash-sand mixtures were compacted dry (ASTM) by using different drop weights and various drop heights. The coefficient of permeability decreased from 2.5 x 10^{-5} cm/sec (Dr = 58.3%, n = 0.41) to 1.4 x 10^{-5} cm/sec (dr - 80.8%, n - 0.35). Even when the 500 g drop weight and 254 mm drop height was used, no more than 81% relative density could be achieved. Moreover, the mixtures could not be compacted by the standard proctor method because the mixtures spread out during compaction.

As the dry density of clay-fly ash mixtures increased, the coefficient of permeability decreased (Figure 41). The dry density of ASTM compacted 20% fly ash clay mixtures increased only from 1.24 g/cc to 1.33 g/cc even though compaction effort increased 40 fold from a 100 g weight falling 2 inches to a 1000 g weight falling 8 inches. The coefficient of permeability of clay-fly ash mixtures decreased from 3.4 x 10^{-6} cm/sec to 2.1 x 10^{-6} cm/sec while the dry density increased from 1.24 g/cc to 1.33 g/cc.

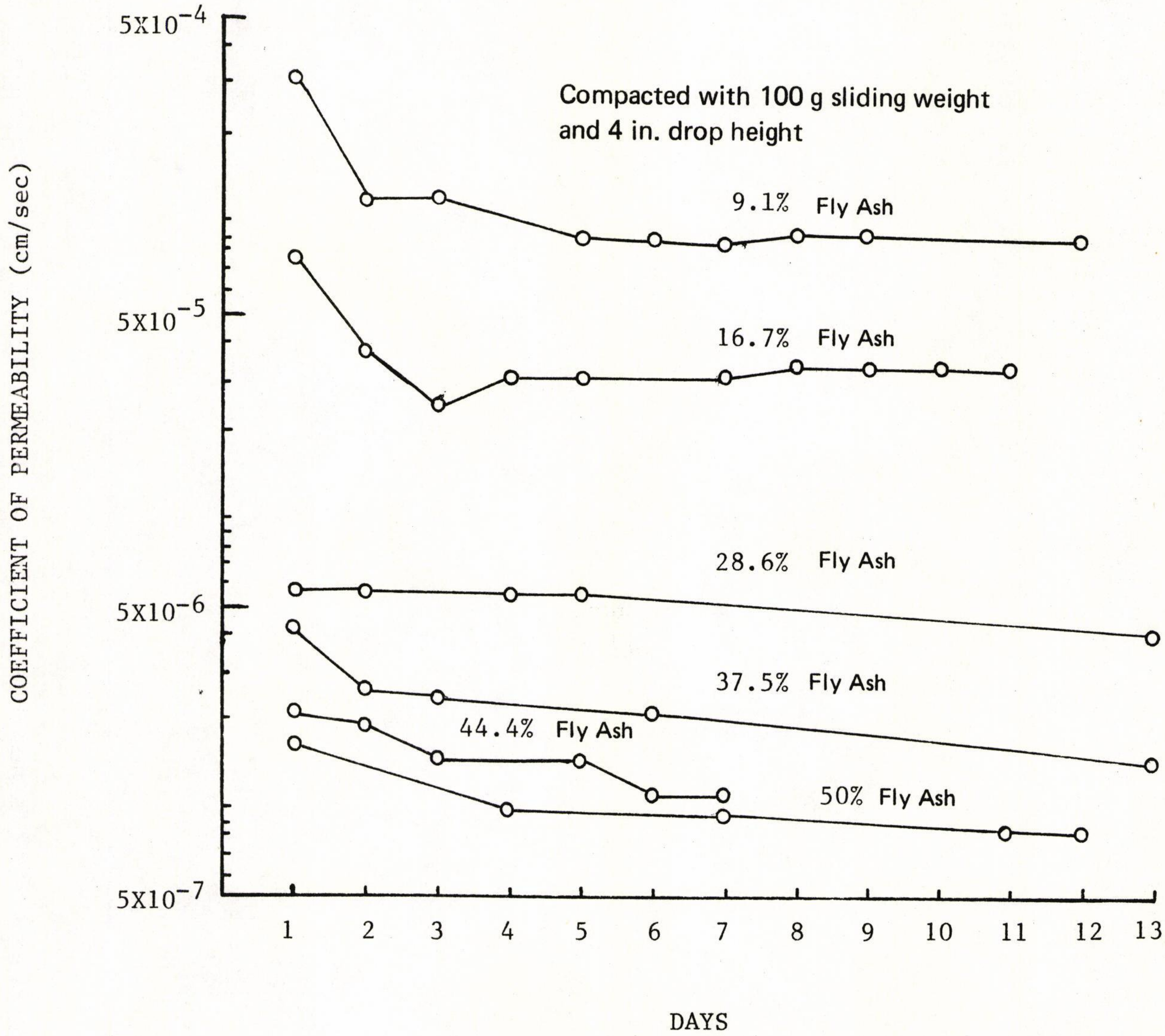


Figure 37. Permeability - time characteristics for sand-fly ash mixtures.

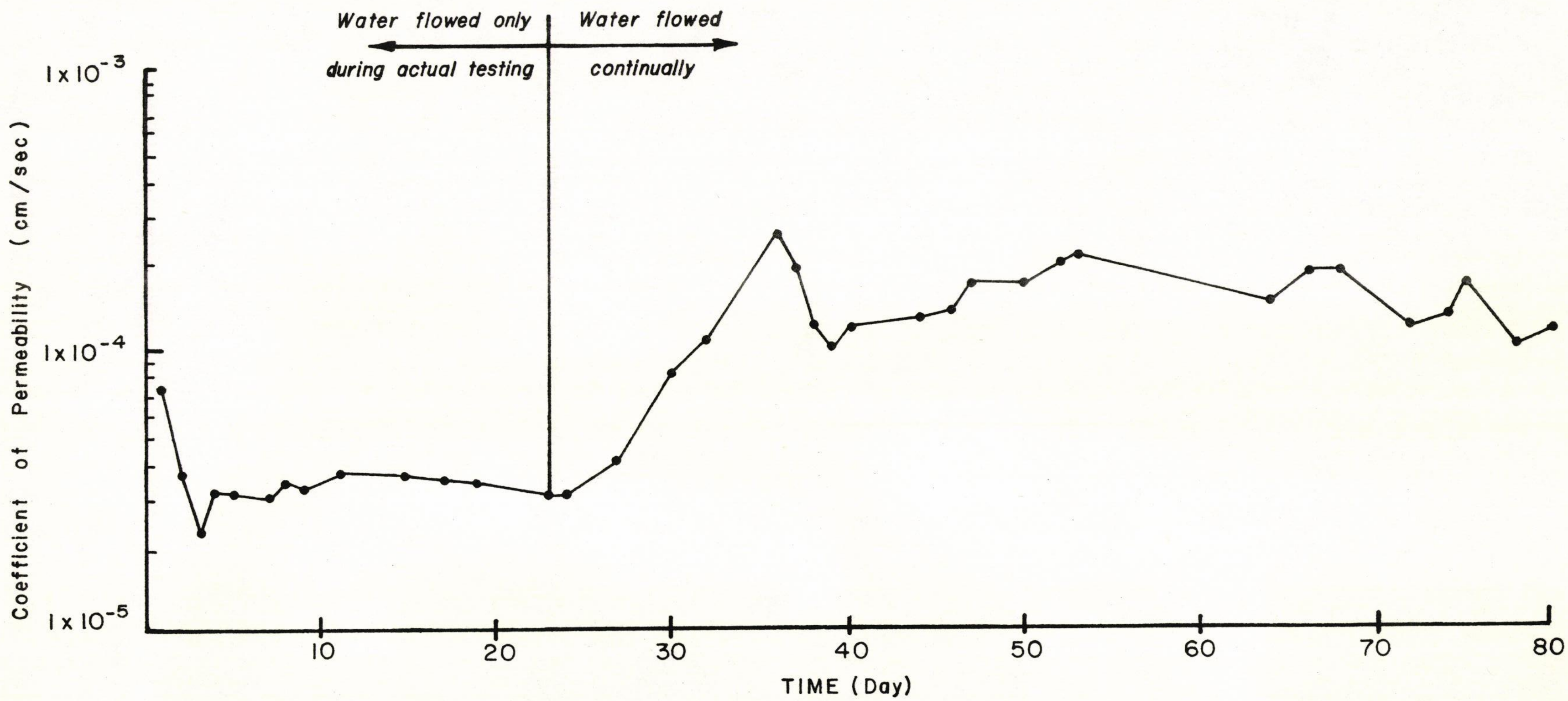


Figure 38. Permeability - Time characteristics for sand with 16.7% Fly Ash.

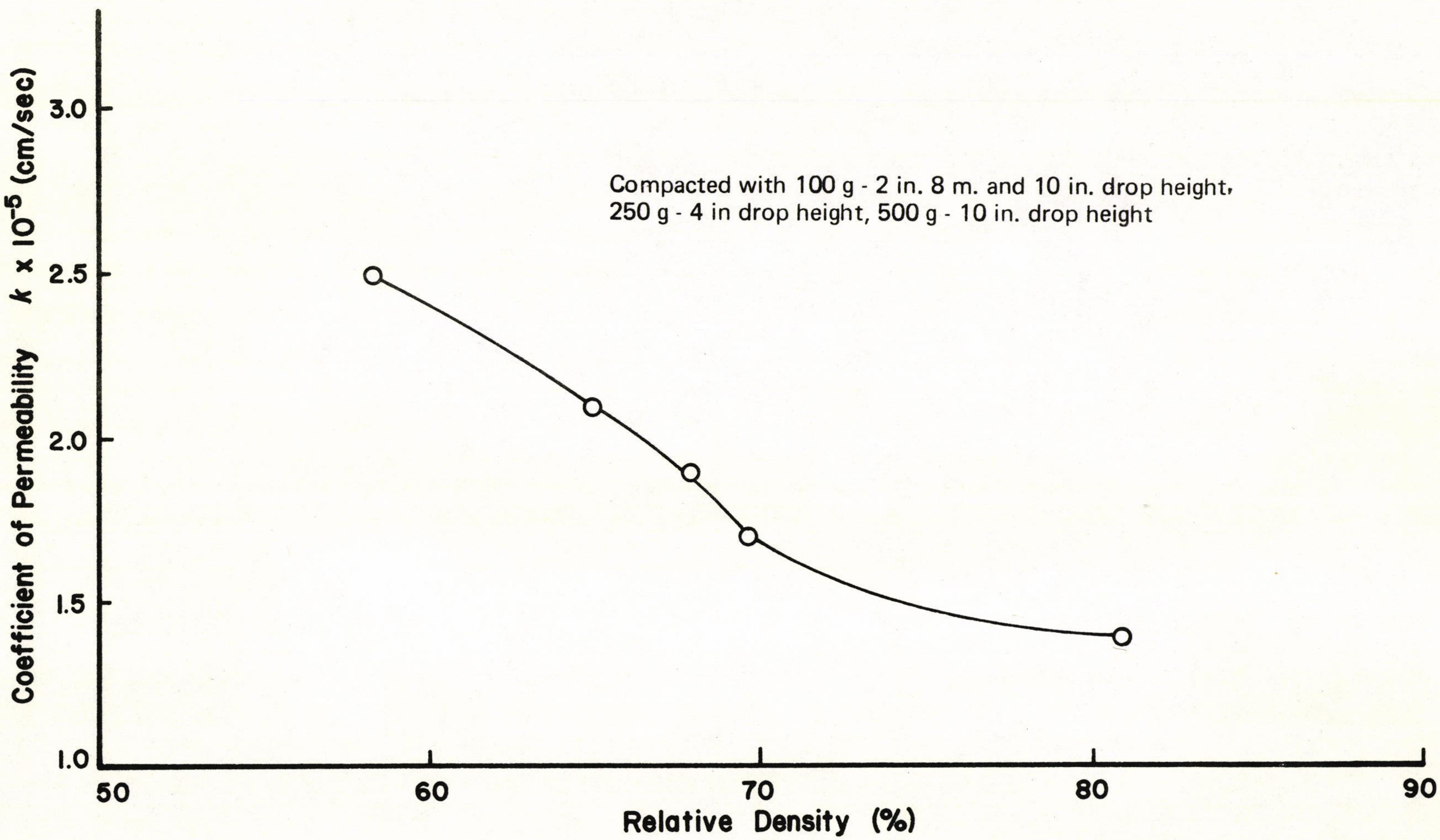


Figure 39. Coefficient of permeability versus relative density of sand with 20% fly ash.

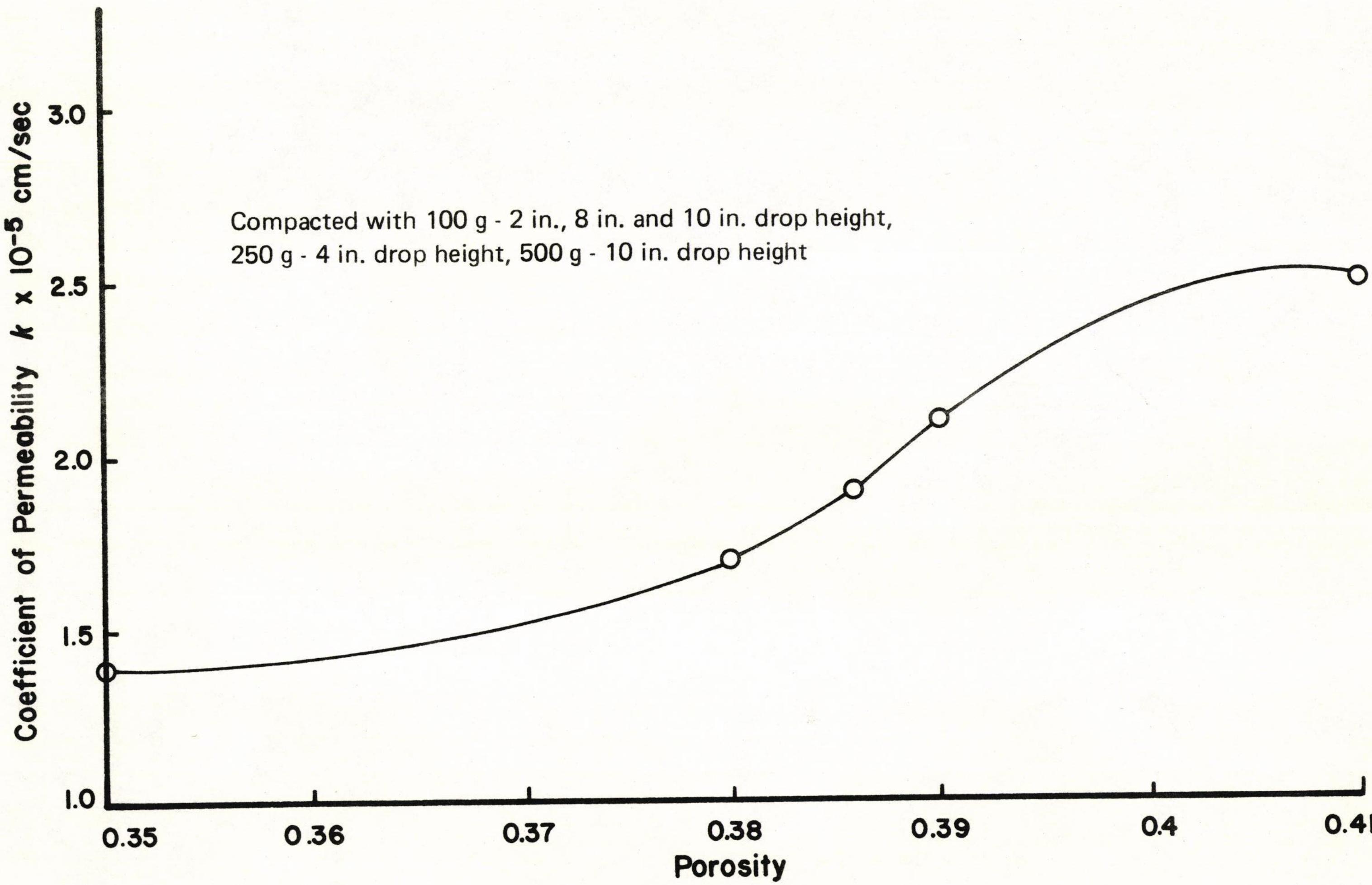


Figure 40. Coefficient of permeability versus porosity of sand with 20% fly ash.

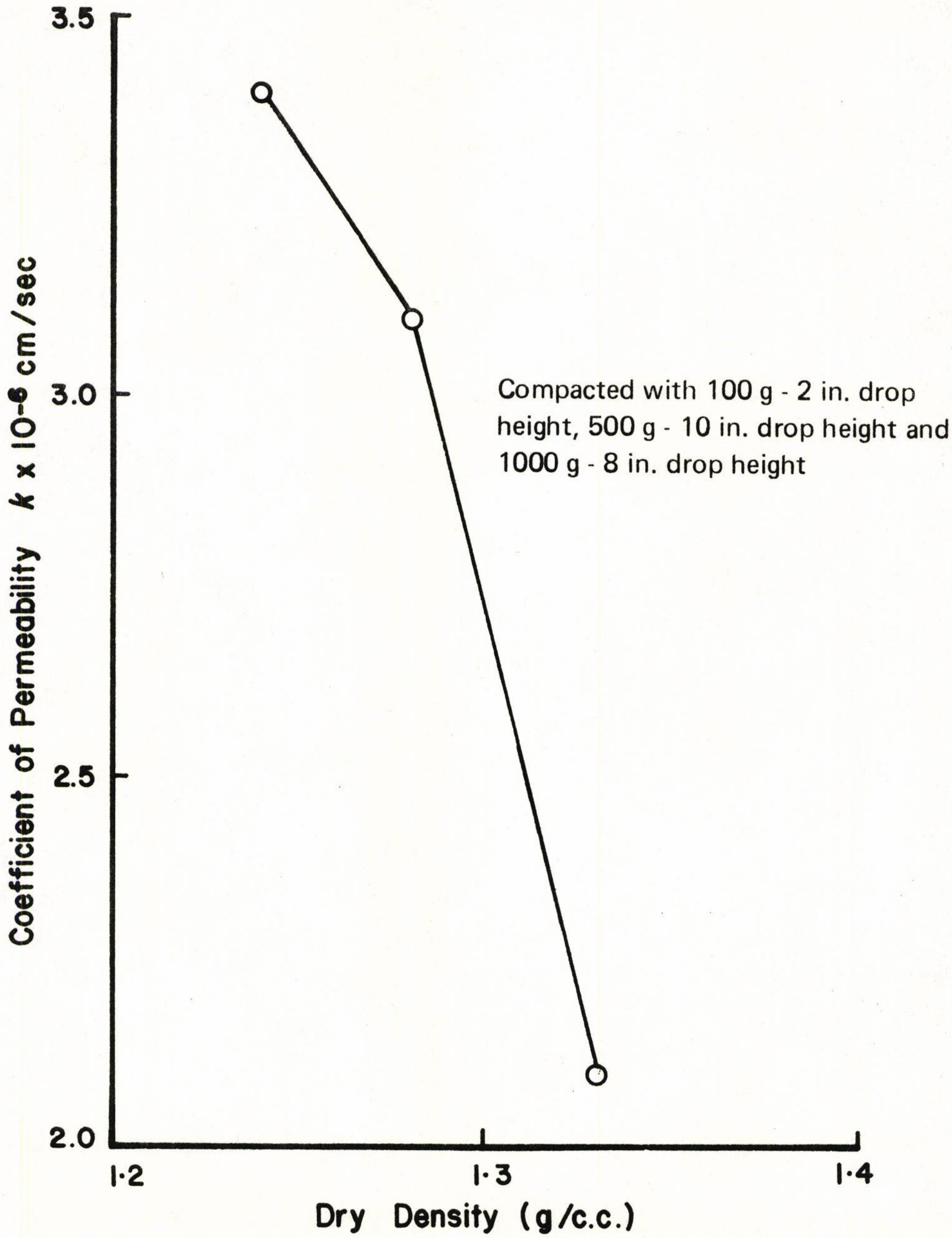


Figure 41. Coefficient of permeability versus dry density of clay with 20% fly ash.

SOIL-FLY ASH MIXTURES COMPACTED AT THE SOILS OPTIMUM MOISTURE CONTENT BY THE PROCTOR METHOD

Effect of Fly Ash on Dry Density of Soil

As the percentage of fly ash increased, the dry density of sand-fly ash mixtures increased (Figure 42). The sand-fly ash mixtures were compacted by standard proctor method at a water content of 8.5%. The dry density of sand increased from 1.53 g/cc (0% fly ash) to 1.67 g/cc (16.7% fly ash).

The dry density of modified proctor compacted sand at optimum water content (8.5%) increased from 1.57 g/cc (0% fly ash) to 1.89 g/cc (20% fly ash) (Figure 43).

The dry density of modified proctor compacted clay samples (water content 20%) increased only from 1.60 g/cc to 1.64 g/cc even though the percent of fly ash increased from 0 to 50.

Effect of Fly Ash on Proctor Compacted Permeability of Soils and Fly Ash

As the fly ash percent increased, the coefficient of permeability of sand-fly ash mixtures decreased (Figure 44). The sand-fly ash mixtures were compacted by standard proctor at a water content of 8.5%. The coefficient of permeability of compacted sand-fly ash mixtures decreased from 5.5×10^{-4} cm/sec (0% fly ash) to 3.5×10^{-6} cm/sec (16.7% fly ash).

The results with the modified proctor were similar to those with the standard proctor test (Figure 44, 45). The coefficient of permeability of modified proctor compacted sand-fly ash at optimum water content 8.0% of sand decreased from 4.6×10^{-4} cm/sec (0% fly ash) to 1.87×10^{-7} cm/sec (20% fly ash).

The coefficient of permeability of clay was so low that it could not be measured in the compaction permeameter. The clay was compacted by modified proctor at 20% water content. The coefficient of permeability of 50% clay-fly ash sample could not be determined. The sample was so impervious that water would not pass through the sample during saturation.

The coefficient of permeability of pure fly ash also could not be determined. When the fly ash sample was compacted by modified proctor at 8% optimum water content, the sample was so impervious that water would not pass through the sample from the bottom during saturation.

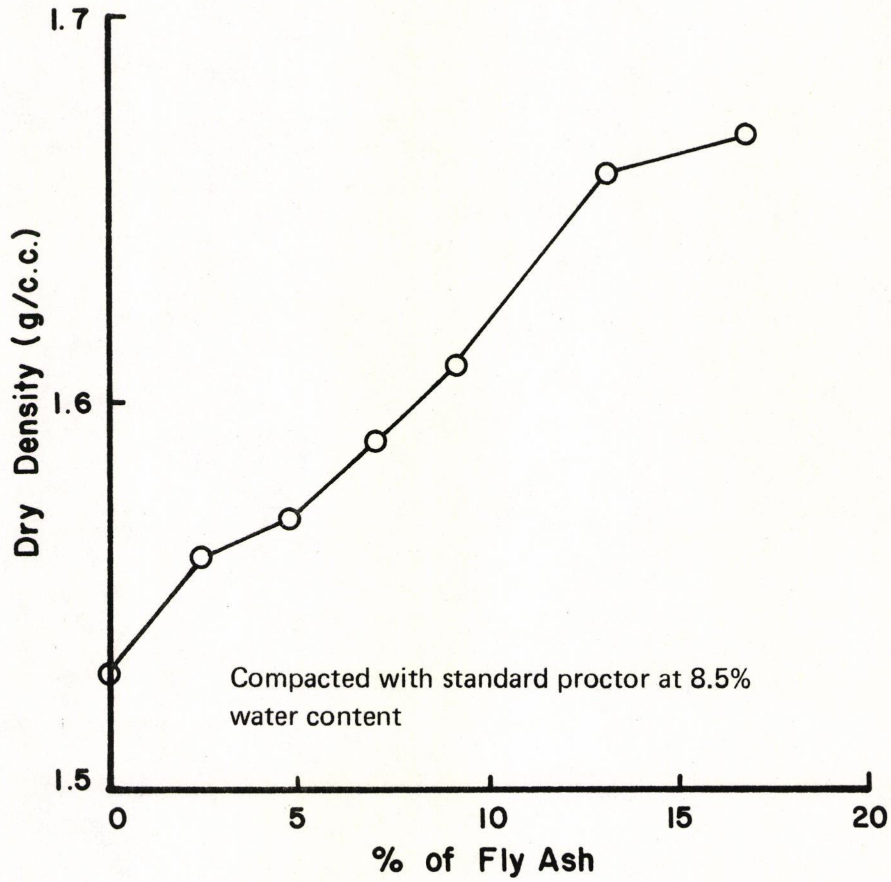


Figure 42. Effect of fly ash on dry density of sand.

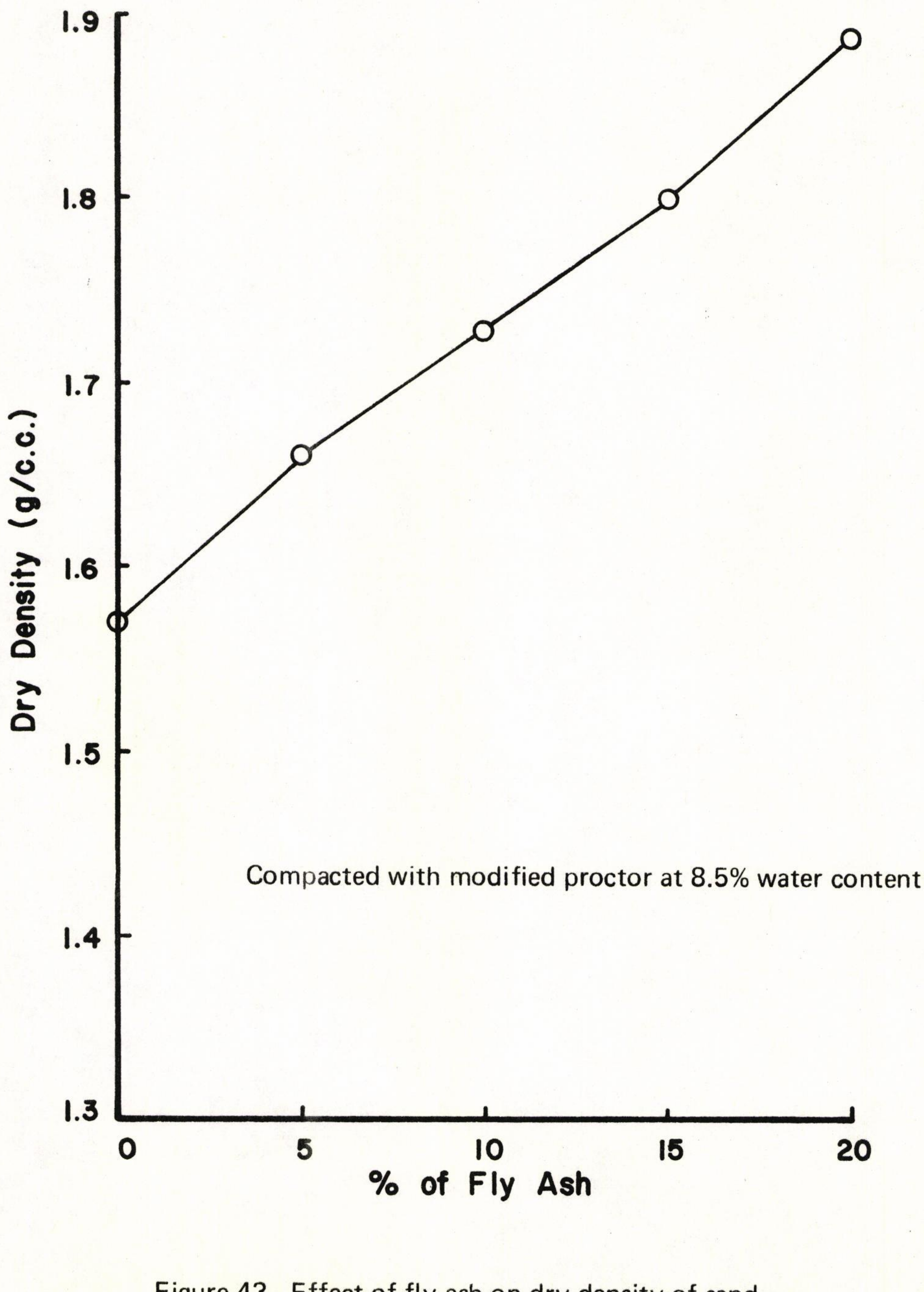


Figure 43. Effect of fly ash on dry density of sand.

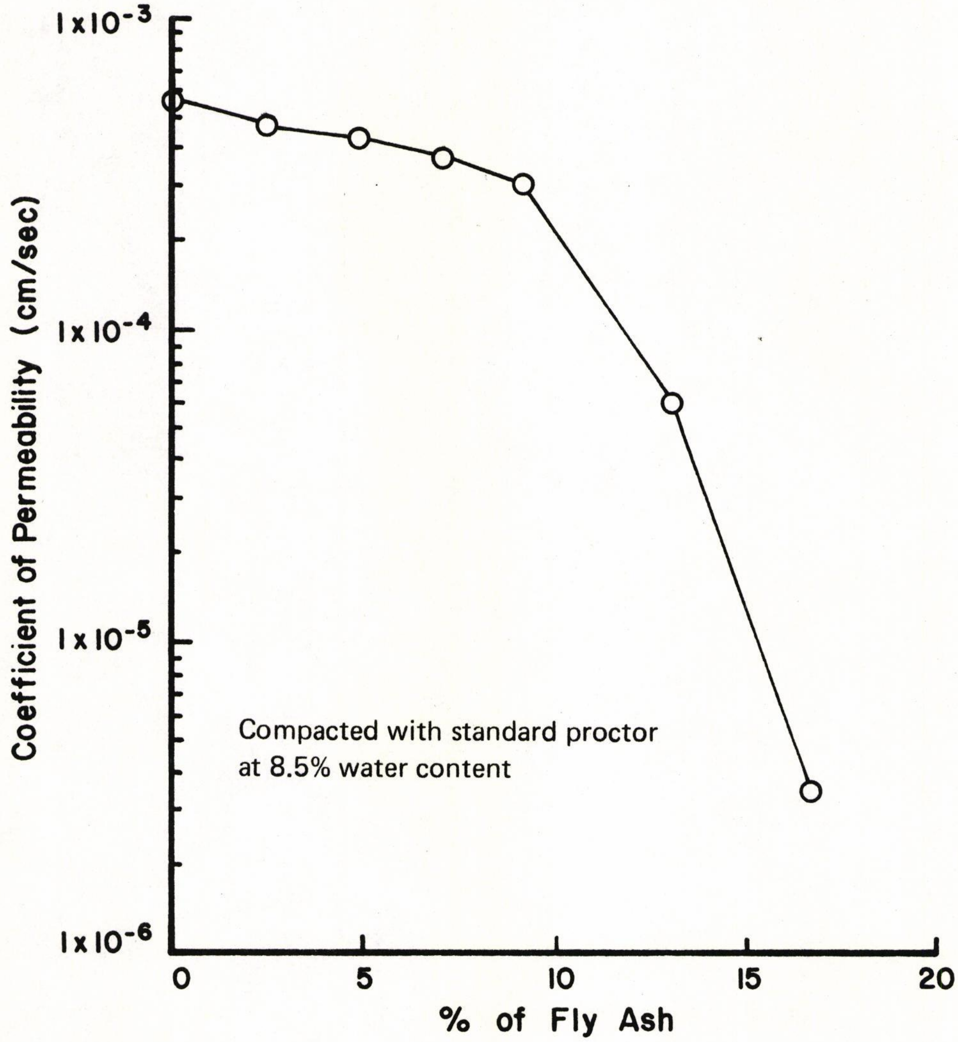


Figure 44. Effect of fly ash on permeability of sand.

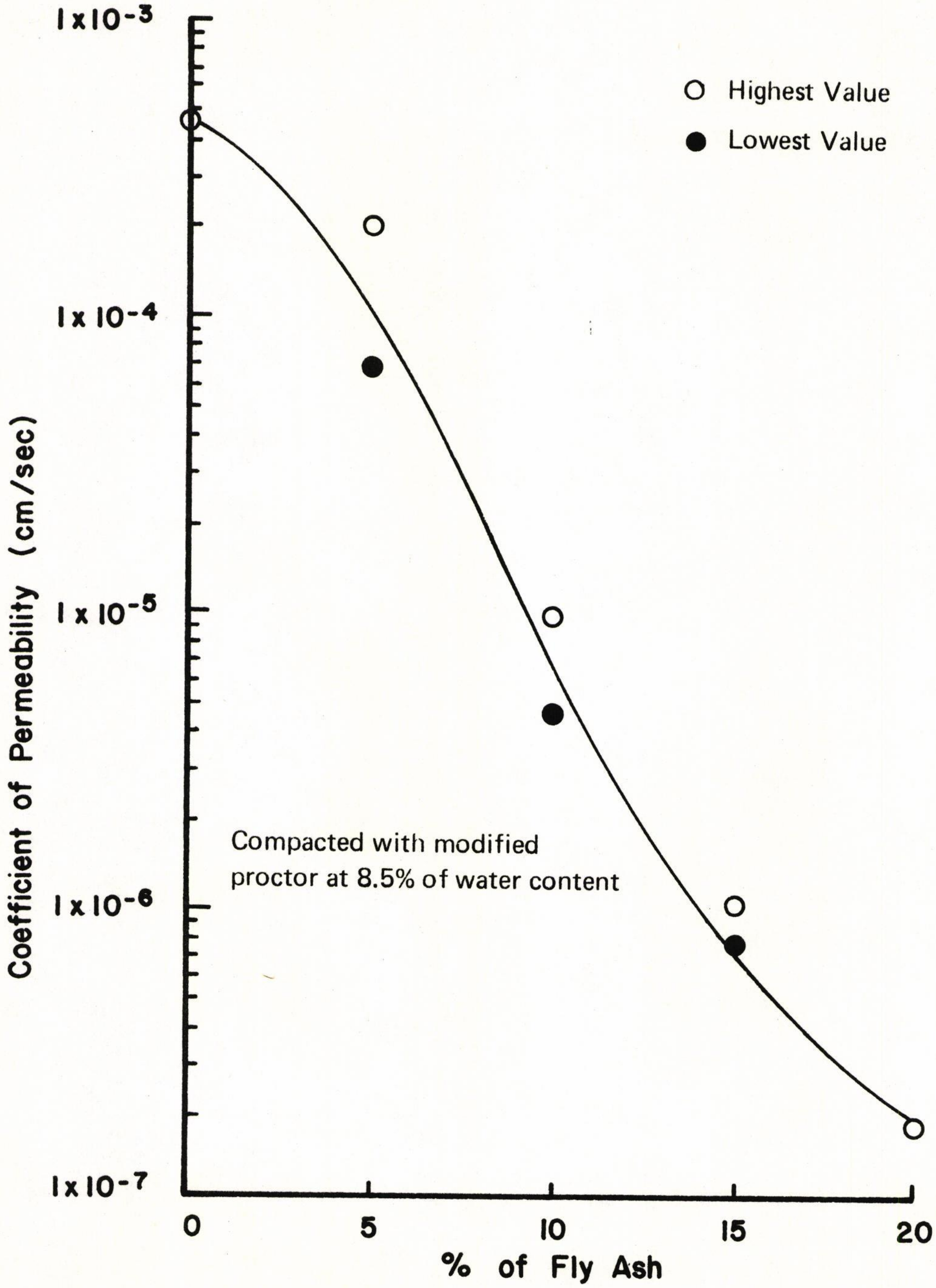


Figure 45. Effect of Fly Ash on permeability of sand.

PERMEABILITY OF FLY ASH - WATER SLURRY MIXTURES

The coefficient of permeability of five fly ash - water slurry mixtures was determined. The fly ash solution was mixed for one minute and poured into an ASTM-D 2434-68 (plastic) permeameter uncompacted. The coefficient of permeability was measured after 24 hours and 48 hours.

As the concentration of fly ash solution increased, the coefficient of permeability decreased (Figure 46). The coefficient of permeability of 0.25 g fly ash /ml H_2O solution was 5.8 x 10^{-5} cm/sec after 24 hours and 5.5 x 10^{-5} cm/sec after 48 hours. The coefficient of permeability of 4 g fly ash/ml H_2O solution was 3.2 x 10^{-6} cm/sec after 24 hours and 3.0 x 10^{-6} cm/sec after 48 hours. Generally, there is not much decrease in permeability between 24 hours and 48 hours for the concentrations of fly ash and water tested.

RELATIONSHIP BETWEEN COMPRESSIVE STRENGTH AND PERMEABILITY OF SAND-FLY ASH MIXTURES

As the coefficient of permeability of sand-fly ash mixtures increased, the unconfined compressive strength decreased (Figure 47). Two sand-fly ash samples were compacted by modified proctor at 8.0% water content. One sample was used for measuring permeability. Another sample was cured for 7 days and the unconfined compressive strength was measured. The unconfined compressive strength increased from 38.2 psi to 372.3 psi as the coefficient of permeability decreased from 1.2 x 10^{-4} cm/sec to 1.89 x 10^{-7} cm/sec.

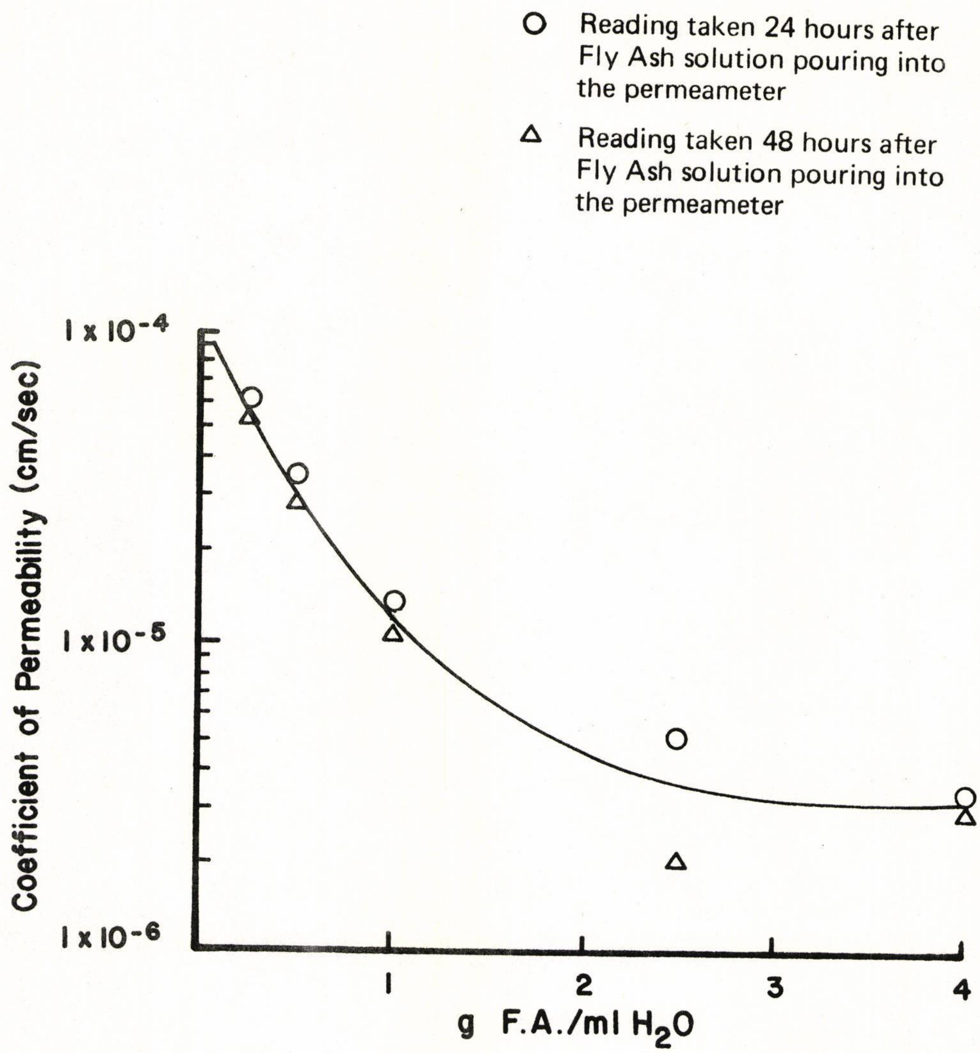


Figure 46. Permeability of Fly Ash - Water Mixtures.

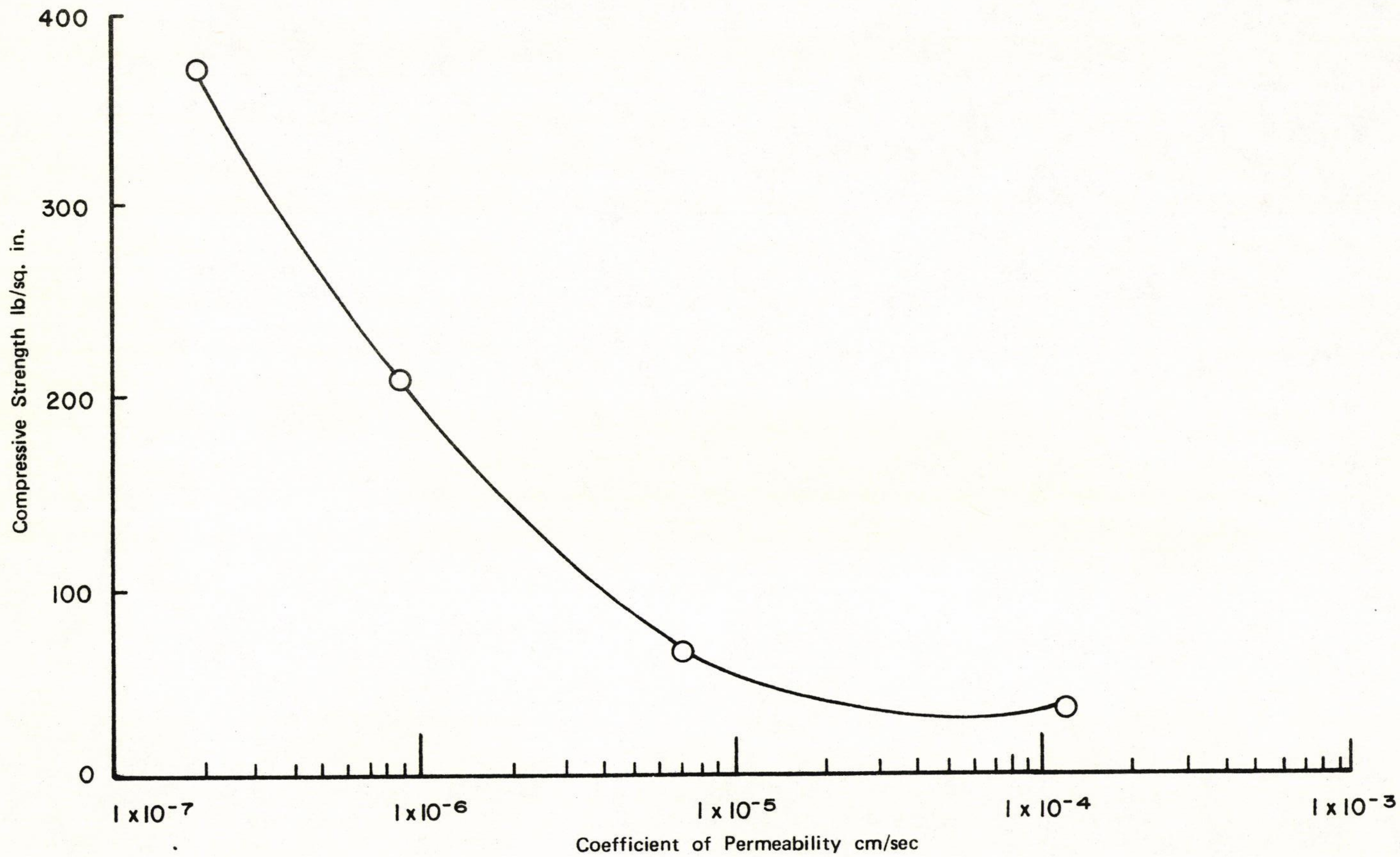


Figure 47. Compressive Strength Versus Coefficient of Permeability.

DISCUSSION

TESTING PROCEDURES

The falling head test procedure was used for this investigation because the coefficients of permeability measured were relatively low (from 10^{-4} to 10^{-6} cm/sec). Some error is encountered with the falling head test because of the headloss in the filter paper and wire screen used to contain the sample; however, this headloss was found to be small and thus the error was considered to be negligible.

The permeameters used in this study were not able to hold pressure, so a vacuum was used to saturate the samples. This procedure resulted in samples with saturation values ranging between 90 and 100% saturation. Incomplete saturation of samples causes the permeability to be less than it would be with 100% saturation. Although this factor introduces some error in the results, the magnitude of error is not large enough to invalidate the conclusion.

EFFECT OF FLY ASH ON DENSITY AND PERMEABILITY OF SOIL

The density increased and the coefficient of permeability decreased for both sand and clay when fly ash was added to the soil. Part of the increase in density of the mixtures was due to the relatively high specific gravity of the fly ash (2.75).

Adding 50% fly ash to the sand reduced the coefficient of permeability from an original greater than 1×10^{-4} cm/sec to less than 1×10^{-6} cm/sec.

The clay under investigation was already practically impermeable ($k = 8 \times 10^{-6}$ cm/sec). Adding 50% fly ash to clay compacted dry, reduced the coefficient of permeability even more ($k = 2 \times 10^{-6}$ cm/sec).

For 100% fly ash compacted dry, primary and secondary permeability were measured because of the formation of vertical cracks when water was added to saturate the sample.

When the fly ash was first mixed in a slurry form with water and then was allowed to cure for 24 hours in the permeameter, no cracks appeared and the coefficient of permeability was found to decrease with decreasing quantities of slurry water.

The coefficient of permeability of soil-fly ash mixtures was found to change with time. When the mixtures were kept exposed to water, the permeability generally decreased slightly to a constant value within a few days. However, when water was allowed to flow continuously through the soil-fly ash mixtures, the coefficient of permeability increased slightly to a constant value.

These changes with time can be a result of either chemical reactions between the soil and fly ash or a dissolving of constituents from the fly ash.

For a particular fly ash-soil mixture, an increase in the compactive effort resulted in an increase in the relative density and decrease in the porosity and coefficient of permeability.

ENVIRONMENTAL EFFECTS

The potential for water quality problems caused by the fly ash studies was found to be limited to a high pH, alkalinity and hardness. These are the same parameters that would be affected by lime. Therefore, it is felt that the use of fly ash to stabilize soils would present no more hazard to water quality than the use of lime.

IMPLICATIONS OF PERMEABILITY ON FROST HEAVE

The grain size distribution curve of the fly ash under investigation is within the limit of frost susceptible soils (Figure 47). No direct measurement of frost heave was made in this project but, because of grain size, the fly ash can be assumed to be frost susceptible.

The coefficient of permeability of sand and clay stabilized with 20% fly ash was 1.5×10^{-5} cm/sec and 2.5×10^{-6} cm/sec respectively. Permeability of sand with 20% fly ash when compacted in the modified proctor permeameter was 1.9×10^{-7} cm/sec. These low permeabilities do not indicate safety from frost heave, however, because no safe levels of permeability to prevent frost heave have been determined to date (1976).

IMPLICATIONS OF PERMEABILITY ON DURABILITY

Durability of construction materials is the resistance to the processes of weathering, erosion and traffic usage. Poor durability can be a problem both for natural and stabilized soils. Poor durability is also reflected in high maintenance costs.

In general, for cement stabilized soils, as cement content increased, durability to wet/dry cycles increases but permeability decreases (Ingles and Metcalf, 1973, pp. 107).

An addition of fly ash to sand can increase the durability of sand-fly ash mixtures and decrease permeability. The sand mixtures stabilized with 5% and 10% fly ash exhibit poor resistance to freeze-thaw and have 100% weight loss after 9 and 10 cycles respectively

(Thornton and Parker, 1975, pp. 61). The corresponding permeability of stabilized sand with 5% and 10% fly ash is 1×10^{-4} cm/sec and 6.5×10^{-6} cm/sec respectively. While the sand mixtures stabilized with 20% fly ash have better resistance to freeze-thaw a 15% weight loss after 20 cycles, the permeability is less at 1.87×10^{-7} cm/sec.

CONCLUSIONS

The following conclusions are based on the results of a study using a fly ash produced from Wyoming low sulfur coal and two Arkansas soils.

1. Addition of fly ash to clay or sand reduces the permeability. The fly ash was more effective in reducing the permeability of sand (permeability reduced three orders of magnitude at 50% fly ash) than in clay (reduced by a factor of 4 at 50% fly ash).
2. Permeability does not vary greatly with time. Variation of permeability with time was less than an order of magnitude, usually less than a factor of two. Change in permeability was most rapid the first three days with little change after three days.
3. Increased compactive effort increases density and reduces permeability in soils. However, reduction in permeability due to increased compactive effort are usually small.
4. The permeability of fly ash, placed in a slurry, varies between 10^{-4} cm/sec and 10^{-6} cm/sec depending on the amount of water in the slurry.
5. Fly ash placed dry, then saturated, developed shrinkage cracks which created secondary permeability.

APPENDIX I

Tests used to determine properties of soil-fly ash mixtures:

1. A.S.T.M. Standard Method of Test for permeability of granular soil D 2434-68

2. A.S.T.M. Standard Method of Test for Laboratory Determination of Moisture Content of Soil, D 2216-71

3. A.S.T.M. Standard Method of Test for Relative Density of Cohesionless Soil, D 2049-69

4. A.S.T.M. Standard Method of Test for Moisture-Density Relations of Soil, Using 10 lb. Rammer and 18 in.-drop, D 1557-70

5. A.S.T.M. Standard Method of Test for Moisture-Density Relations of Soil, Using 5.5 lb. Rammer and 12 in.-drop, D 698-70

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