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E. H. BOURQUARD ASSOCIATES, INC.

WATER RESOURCES ENGINEERING

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FLOOD CONTROL PROJECTS
DAMS & RESERVOIRS
DRAINAGE-STORMWATER
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TELEPHONE (717) 238-9505

September 9, 1982

Mr. Rex Wescott,
U. S. Nuclear Regulatory Commission,
7920 Norfolk Ave.,
Bethesda, MD 20014.

Re: Bradshaw Reservoir and Pumping Station

Dear Mr. Westcott:

In accordance with our phone conversation today, enclosed are the following:

1. Bradshaw Reservoir Specification Section 02220, Earth Fill.

2. Soil Testing for Engineers, Lambe, Chapter VI, Permeability Test.

Item 1 specifies the requirements the Contractor must follow in construction of the earthen dam and impervious liner. As we discussed, there will be no specific permeability requirement for the Contractor to meet. However, the material as specified should provide a maximum in place permeability of 0.000005 cm/sec. In the unlikely event the material would exceed this permeability, bentonite would be incorporated into the liner material as necessary to reduce the permeability. Since the need for bentonite is unlikely, it has not been included in the Specification. If needed, the additional work would be carried out under a Contract change order.

Item 2 describes the permeability test procedure. Tests will be conducted at the Contractor's proposed off-site borrow area on the natural undisturbed material and also on the liner after compaction. Undisturbed samples will be taken at both locations. The variable head method will be used.

Sincerely yours,

Robert Bourquard

Robert H. Bourquard

B001

RHB/bs
Encl. As Noted
c.c. Dave Morad, PECO w/encl.

SEP 9 1982

SECTION 02220

EARTH FILL

PART 1 GENERAL

1.01 WORK INCLUDED

A. Construct earth embankments and other earth fills required by the Drawings and Specifications.

1.02 RELATED WORK

A. Section 02210: Salvaging and Spreading Topsoil.

B. Section 02211: Excavation.

C. Section 02265: Water for Construction.

D. Section 02266: Removal of Water

1.03 REFERENCES

A. ASTM D698 - Moisture Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5 lb. (2.49 kg) Rammer and 12-in. (305 mm) Drop.

PART 2 PRODUCTS

2.01 MATERIALS

A. On-site Borrow Areas.
   1. Obtain suitable fill material from required excavations and designated borrow areas.

B. Off-site Borrow Areas.
   1. Locate a suitable off-site borrow area(s), notify the Owner of its location and arrange for entry to the site for testing and sampling. Provide the necessary excavating equipment and labor to permit the procurement of soil samples considered representative of the borrow material proposed for use. The Owner will perform all field and laboratory testing of the material to determine its suitability for construction of the required fills.
   2. If in the judgment of the Owner the material is suitable, the Contractor shall furnish such material from the approved off-site borrow area(s). In the event the material is not suitable, the Contractor shall locate an additional borrow area(s) and the process repeated until an approved borrow area(s) is located.

3. It shall be the Contractor's responsibility to locate the borrow area(s), purchase the material, furnish such material to the site, place and compact such material in accordance with the specifications and meet all governmental requirements.

4. Impervious Material.

   a. Off-site borrow material for reservoir embankment or impervious liner construction shall be suitable impervious, inorganic, fill consisting of uniformly-graded silty clays and clayey silts with the amount of friable rock fragments not exceeding more than 20 percent of the total mass, but averaging eight (8) percent or less. Soils classified as impervious fill shall contain at least 65 percent, by weight, of material finer than the No. 200 mesh sieve with the average percent passing the No. 200 mesh sieve being at least 80 percent. All soils shall be classify as ML, CL or ML-CL types according to the Unified Soil Classification System (USCS). They shall not have a liquid limit (LL) exceeding 50 and shall have plasticity indices (PI) ranging from at least two (2) to a maximum of 22. No cobbles, boulders or otherwise durable rock fragments having a maximum dimension in excess of four (4) inches shall be included in the impervious fill. In addition, the impervious fill materials, when subjected to the Standard Compaction Test, ASTM Designation 698, latest edition, shall indicate a maximum dry density at the optimum moisture content of at least 107.0 p.c.f. (pounds per cubic foot). All fill materials, regardless of type or source shall be free of topsoil, wood, lumber, roots, grass, rubbish, metal, organic content, or other deleterious material.

   b. All "off-site" material proposed for use as impervious fill shall require demonstration of suitability by grain size distribution, plasticity and compaction tests, the results of which must be first approved by the Owner.

   c. If during the excavation and hauling of the impervious fill from the approved borrow pit it becomes apparent that the appearance and characteristics of the fill material change to an extent readily noticeable by visual inspection, a complete classification and new compaction control curve will be obtained. If such additional testing indicates the material does not meet the previously approved kind, a new source shall be immediately located by the Contractor which shall be tested and approved prior to the material being hauled to the project site.

C. Fill Material.
   1. The selection, blending, routing and disposition of materials is subject to the Owner approval.
   2. Material to be free from sod, brush, roots and rock particles larger than 3 inches.

PART 3 EXECUTION

3.01 FOUNDATION PREPARATION

A. Strip foundations to remove vegetation, topsoil and other unsuitable materials.

B. Grade foundation surface to remove irregularities and scarifice parallel to the axis of the fill to a minimum depth of 2 inches. Control the moisture content of the loosened material as specified for the earth fill.

C. Compact and bond the first layer of earth fill with the surface materials of the foundation.

D. Clear loose material from rock foundations by hand or other effective means. Remove standing water from rock foundations before placing fill.

3.02 PLACEMENT OF FILL

A. Complete the required excavation and foundation preparation prior to placement of fill.

B. Do not place fill on a frozen surface nor incorporate snow, ice or frozen material in the fill.

C. Place fill in approximately horizontal layers not more than 8 inches before compaction.

D. Uniformly spread materials in piles or windrows to not more than 8 inches in uncompacted thickness before compaction.

E. Spread material to be hand compacted or compacted by manual directed power tampers in layers not more than 4 inches thick before compaction.

F. Adjacent to structures, place fill in such a manner to prevent damage to the structures and to allow the structures to assume the loads from the fill gradually and uniformly. Increase the height of the fill at the same rate on all sides of the structure. Do not place fill against structures before the time interval listed below.

Structure	Time Interval
1. Retaining walls	14 days
2. Walls backfilled on both sides simultaneously	7 days
3. Conduits and spillway risers, cast in place (with inside forms in place)	7 days
4. Spillway risers (inside forms removed)	14 days
5. Conduits, precast, cradled	2 days
6. Conduits, precast, bedded	1 day
7. Antiseep collars	3 days

G. Place fill for earth fill dams, levees and other structures designed to restrain the movement of water in accordance with the following requirements:

1. Place fill so that the distribution of materials throughout each zone is essentially uniform. Fill to be free from lenses, pockets, streaks, or layers of material differing substantially in texture or gradation from surrounding material.
2. If the surface of any layer becomes too hard and smooth for proper bond with the succeeding layer, scarifice it parallel to the axis of the fill to a depth of not less than 2 inches before the next layer is placed.
3. Maintain the top surface of embankments approximately level during construction. Provide a crown or cross-slope of not less than 2 percent to insure effective drainage. If the Drawings or Specifications require or the Owner directs that fill be placed at a higher level in one part of an embankment than another, maintain the top surface of each part as specified above.
4. Construct dam embankments in continuous layers the entire length. Openings may be provided to facilitate construction or to allow the passage of stream flow.
5. If an embankment is built at different levels, provide a maximum slope of 3 to 1 at their junction. °Strip the bonding surface of the higher embankment of all loose material and scarifice, moisten the soil and recompact when new fill is placed against it to insure a good bond between the two fills and to obtain the specified moisture content and de in the junction.

3.03 CONTROL OF MOISTURE CONTENT

A. Moisture content of the material at the time of compaction to be not more than 3 percentage points above or one percent below the optimum moisture content. Soils containing free water or soils having moisture contents greater than a moisture content midway between the liquid and plastic limits for the material are considered too wet for placement in the embankment. If they are used, dry prior to placement. Accelerate drying action by discing, harrowing, or manipulating to the extent necessary to reduce the moisture content to within the specified limits. When the material is more than one percentage point below optimum, wet the material by sprinkling uniformly, and disc or harrow to obtain uniform distribution of the moisture content to within the specified limits.

B. Scarifice and dry or moisten the previously placed layers when necessary to produce a suitable bond for the succeeding layer.

3.04 COMPACTION

A. When the moisture content and condition of the layer is satisfactory, compact by tamping rollers to a density of at least 95% of the maximum density as determined by ASTM D698.

B. Tamping rollers to consist of one or more heavy duty double drum units with a drum diameter of not less than 60 inches. The drums to be capable of being ballasted. Each drum to have staggered feet uniformly spaced over the cylindrical surface such as to provide approximately three tamping feet for each two square feet of drum surface with the distance between the feet equal to or greater than 9 inches. The tamping feet to be 8 to 10 inches in clear projection from the cylindrical surface of the roller and to have a face area of not less than 6 nor more than 10 square inches. The roller to be equipped with cleaning fingers, so designed and attached as to prevent the accumulation of material between the tamping feet. The weight of the roller to be not less than 4,000 pounds per foot of linear drum length ballasted, and not more than 3,250 pounds per foot of drum length empty. The loading to be such as to obtain the specified compaction. The roller to be pulled by a crawler-type tractor of sufficient power to operate the roller at a speed of approximately 3½ mph.

C. Use power driven hand tampers, vibratory or other satisfactory tampers to tamp around structures or other locations where larger rollers cannot satisfactorily compact the material.

D. Compaction rollers of other designs may be used after approval by the Owner provided the requirements for compaction and other specified requirements are met.

3.05 REMOVAL AND PLACEMENT OF DEFECTIVE FILL

A. Remove or rework fill placed at densities lower than the specified minimum density or at moisture contents outside the specified acceptable range.

3.06 TESTING

A. During the course of the work, the Owner will perform such tests as are required to identify materials, to determine compaction characteristics, to determine moisture content, to determine permeability and to determine density of fill in place. These tests performed by the Owner will be used to verify that the fills conform to the requirements of the Specifications. Such tests are not intended to provide the Contractor with the information required by him for the proper execution of the work and their performance shall not relieve the Contractor of the necessity to perform tests for that purpose.

END OF SECTION

SOIL TESTING

for Engineers

T. WILLIAM LAMBE

The Massachusetts Institute of Technology

NEW YORK · JOHN WILEY & SONS, INC.
LONDON · CHAPMAN & HALL, LIMITED

Fifth printing, February, 1958

PRINTED IN THE UNITED STATES OF AMERICA

CHAPTER

VI

Permeability Test

Introduction

A hundred years ago, Darcy showed experimentally that the rate of water q flowing through soil of cross-sectional area A was proportional to the imposed gradient i or

$$\frac{q}{A} \sim i \qquad q = kiA$$

The coefficient of proportionality k has been called "Darcy's coefficient of permeability" or "coefficient of permeability" or "permeability."[1] Thus permeability is a soil property which indicates the ease with which water[2] will flow through the soil.

Permeability enters all problems involving flow of water through soils, such as seepage under dams, the squeezing out of water from a soil by the application of a load, and drainage of subgrades, dams, and backfills. As will be discussed in later chapters, the effective strength of a soil is often indirectly controlled by its permeability.

Permeability depends on a number of factors. The main ones are:

1. *The size of the soil grains.* As pointed out on page 30, permeability appears to be proportional to the square of an effective grain size. This proportionality is due to the fact that the pore size, which is the primary variable, is related to particle size.

2. *The properties of the pore fluid.* The only important variable of water is viscosity, which in turn is sensitive to changes in temperature. Equation VI-3 expresses the relationship between viscosity and permeability.

3. *The void ratio of the soil.* The major influence of void ratio on permeability is discussed later in this chapter.

4. *The shapes and arrangement of pores.* Although permeability depends on the shapes and arrangement of pores, this dependency is difficult to express mathematically.

5. *The degree of saturation.* An increase in the degree of saturation of a soil causes an increase in permeability. This effect is illustrated by Fig VI-1.

For testing sands and silts, the normal procedure is first to determine, by laboratory tests on disturbed samples, the relationship of void ratio to permeability. After obtaining the in situ void ratio of the soil, we can predict the in situ permeability by using the void ratio-permeability curve determined in the laboratory. This procedure is the most feasible one because of the difficulty of obtaining undisturbed samples of cohesionless soils. It should be remembered, however, that many soils have widely different[3] permeabilities along the stratification and perpendicular to it, and, therefore, the results obtained on disturbed samples may be of little real significance. The permeability of an undisturbed sample of clay can be determined directly at several different void ratios while running a consolidation test, as described in Chapter IX.

At least four laboratory methods of measuring the permeability of a soil are available. The variable

[1] The three terms are used interchangeably, even though the use here of "coefficient" may be questioned. The coefficient is not dimensionless, but has the units of velocity.

[2] The soil engineer rarely deals with pore fluids other than water. However, the permeability of a soil can also be obtained for fluids such as oil.

[3] Very frequently the permeability along the stratification is five to fifty times as large as that across it.

head and constant head tests are presented in this chapter. The capillarity method is presented in Chapter VIII; the use of consolidation test data to compute permeability is discussed in Chapter IX. The variable head test is normally more convenient for cohesionless soils than the constant head test because of the simpler instrumentation. There are conditions, however, under which the constant head test is preferable: for example, for the tests on partially

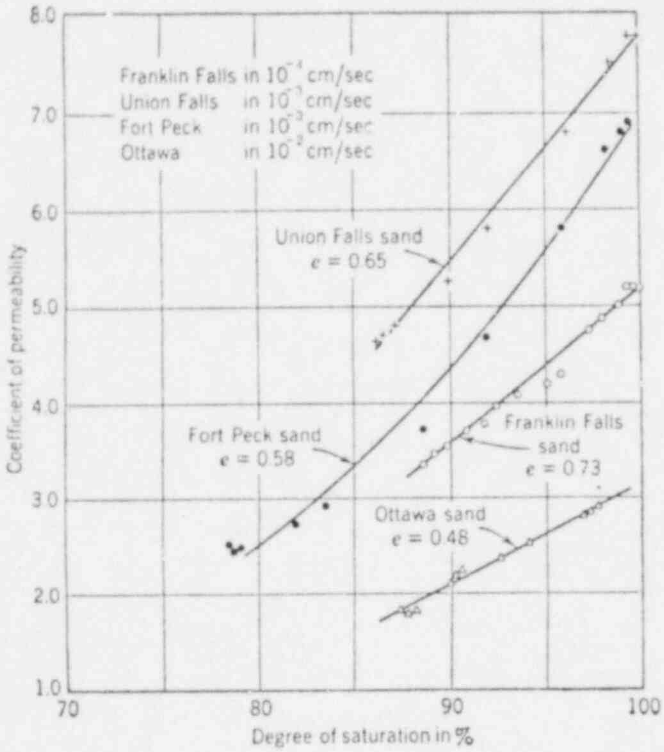

FIGURE VI-1. Permeability versus degree of saturation for various sands. (Data from reference VI-6.)

saturated soils (discussed in Chapter VIII) and for direct permeability determinations in conjuncton with consolidation tests (discussed in Chapter IX) on certain soils.

Apparatus and Supplies [4]

Variable Head Test

Special

1. Permeameter tube [5]
   (*a*) Two screens
   (*b*) Two rubber stoppers
   (*c*) Spring
2. Standpipe
3. Deairing and saturating device
4. Support frame and clamps

General

1. Wooden hammer
2. Bell jar for constant head chamber
3. Supply of distilled, deaired water
4. Vacuum supply
5. Balance (0.1 g sensitivity)
6. Drying oven
7. Desiccator [6]
8. Scale
9. Thermometer (0.1° sensitivity)
10. Stop clock
11. Rubber tubing
12. Evaporating dish
13. Funnel
14. Pinch clamps

Figure VI-2 is a diagrammatic sketch of a variable head test setup which has proved satisfactory. In the laboratory, the parts can be permanently mounted to a panel or simply held to a support frame by clamps. The use of a transparent material, such as lucite, for the permeameter and water chamber is highly desirable, because it facilitates the measurement of the length of soil sample, L, and aids the detection of any air bubbles or movement of soil fines during the test. Likewise the water level in a transparent water chamber can be observed. The measuring of the soil length can be further facilitated by the cementing of graph paper strips, with units of length marked on them, to the outside of the permeameter. It is good policy to number each permeameter and standpipe, and mark on each its cross-sectional area. The bottom screen in the permeameter should be attached by some type of inside wedge and not screws, since screw holes are a possible source of leaks when the permeameter is evacuated.

The tubing should be either metal, high-pressure rubber (see Fig. VI-2), or some other material which can resist the applied vacuum. If low-pressure tubing is used between the standpipe and the permeameter, it will decrease in diameter as the hydrostatic pressure decreases because of a lowering of the water level in the standpipe. To prevent errors from such volume changes, the amount of tubing in this connection should be kept to a minimum. Water traps in the line preceding the manometers are desirable to prevent water from flowing into the manometers during the saturating process.

[4] The apparatus for this test is described in more detail than for some of the other tests because it is more often constructed in the soils laboratory from stock materials.

[5] The desirable size of a permeameter depends on the soil to be tested. Permeameters in the neighborhood of 4 cm in diameter and 30 cm long have been found satisfactory for many soils. See page 58.

[6] A desiccator may not be needed. See page 10.

The choice of standpipe size should be made with regard to the soil to be tested. For a coarse sand, a standpipe whose diameter is approximately equal to that of the permeameter is usually satisfactory. On the other hand, fine silts may necessitate a standpipe whose diameter is one-tenth or less of the permeameter diameter.

Constant Head Test. There are several items needed for the constant head test in addition to those listed for the variable head test. The additional items depend on the type of setup used.

In Fig. VI-3 are shown diagrammatically two test setups for running the constant head test. Although the one on the left is simpler, it should be used only for soils of high permeability. This limitation is due to the fact that, if the soil is relatively impermeable, the rate of flow is low, and thus the loss of water by evaporation can become an important consideration.

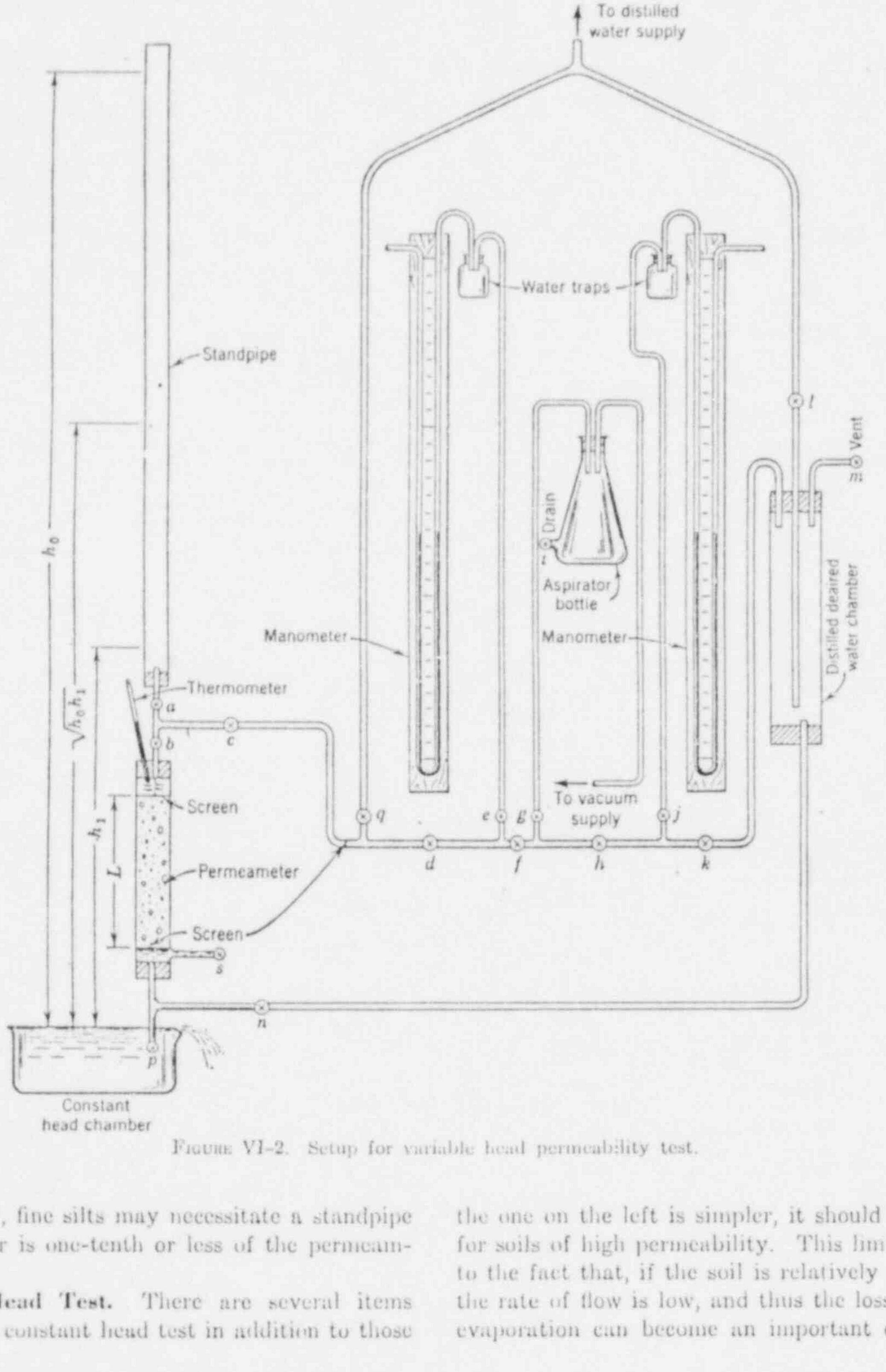

FIGURE VI-2. Setup for variable head permeability test.

The balloons (Fig. VI–3b) furnish a convenient means of preventing evaporation. If the air inside them is allowed to become saturated with water vapor prior to testing, no evaporation will occur during the test (unless the atmospheric pressure or temperature changes). The balloons should be kept very loose so that the pressure in them will be essentially atmospheric.

If the diameter of the water supply bottle (Fig. VI–3b) is large relative to the diameter of the permeameter, the value of h can usually be considered constant for a test. The water level in the bottle should be recorded at the start and completion of the test to check the degree of validity of this assumption. The use of a bottle for the water supply has two advantages; it is a convenient means of storing water between tests, and it easily permits pressure to be applied to the water to obtain the additional head sometimes needed for testing impermeable soils.

Recommended Procedure [7]

The detailed procedures described below are for soils which are cohesionless; permeability determinations on fine-grained soils are discussed in Chapter IX.

Variable Head Test

1. Measure the inside diameter of the standpipe and permeameter.

2. Obtain to 0.1 g the weight of the empty permeameter plus screens, stoppers, and spring.

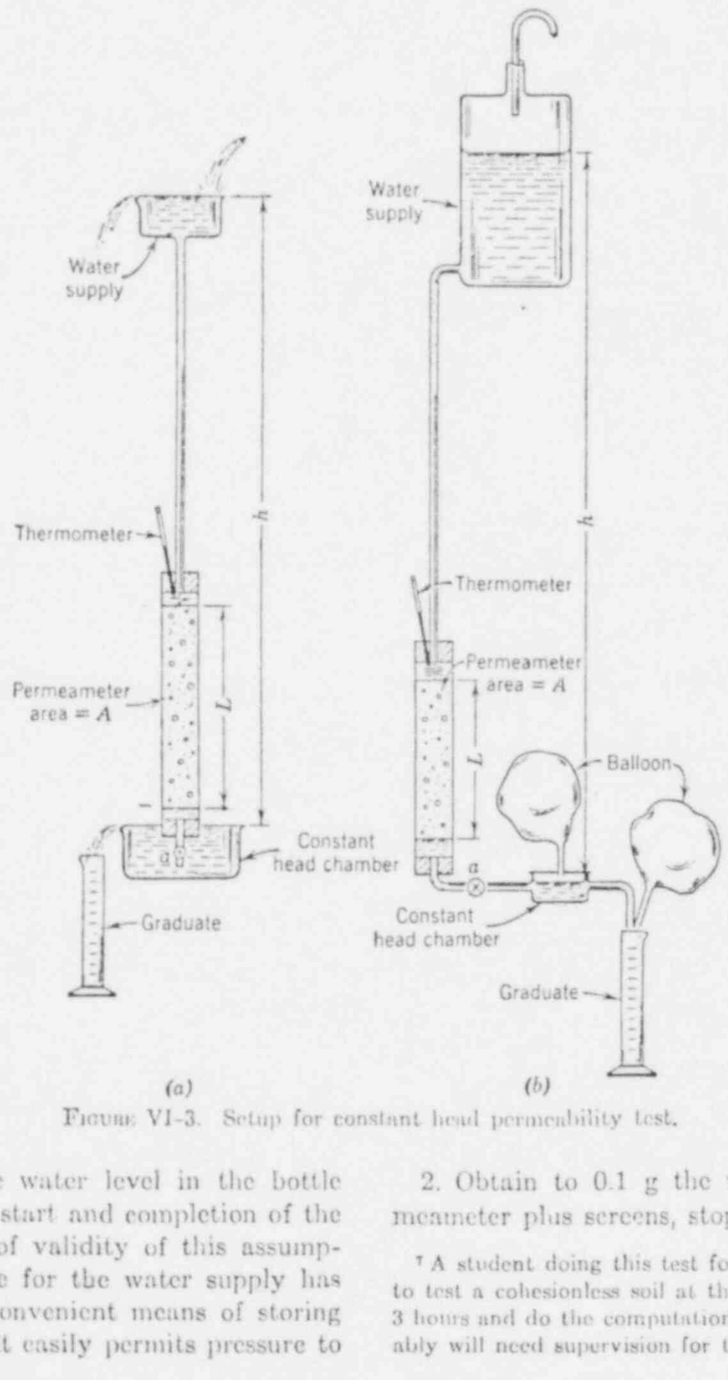

FIGURE VI-3. Setup for constant head permeability test.

[7] A student doing this test for the first time should be able to test a cohesionless soil at three or four void ratios in 2 to 3 hours and do the computations in about an hour. He probably will need supervision for the first part of the test.

3. Load the permeameter with dry soil[8] to a loose, uniform density by pouring the soil in.[9]

4. Place the top screen, spring, and two stoppers in the tube. The spring should be compressed so that it will apply a pressure to the soil and help keep it in place when it is saturated.

5. Weigh the filled permeameter; the difference between the two weights is the amount of soil used.

6. Place the filled permeameter in position for testing as shown in Fig. VI-2.

7. Evacuate the sample to an absolute pressure of only a few centimeters of Hg by the following method:

(*a*) Close all valves shown in Fig. VI-2.

(*b*) Open valves g, h, j, k, f, e, d, c, and b.

8. After waiting some 10 to 15 minutes for the removal of air, saturate the soil by the following method:

(*a*) Close valves f, g, and h.

(*b*) Open valve n. The water will enter the soil because of the capillary attraction aided by the difference in elevation between water chamber and permeameter. If more head difference is needed, it can be obtained by slightly opening vent m. The difference in readings of the two manometers will indicate the additional pressure head that is thus obtained.

(*c*) Allow the water to saturate the sample and rise up to valve b, then close n.

(*d*) Release the vacuum on the sample by first closing k and d, and then slowly opening q and m.

(*e*) Any air bubble in the permeameter above the soil should be removed by slightly opening the upper stopper while applying water through q, with d closed. Any bubble in the bottom should be removed through s, while applying water through n with m open.

9. Measure the length of sample L and locate and measure the heads h_0 and h_1. The top limit of h_0 is selected at the upper end of the standpipe; h_1 a few centimeters above the lower end of the standpipe; the head $\sqrt{h_0h_1}$ should be marked on the standpipe.

10. With valves n and d closed, fill the standpipe with distilled, deaired water to an elevation which is a few centimeters above h_0 by opening valves q, c, and a. Close valve c; leave a open.

11. Check to see that there is no air in the line between the standpipe and permeameter up to valve c, as well as in the line from the permeameter into the constand head chamber.

12. Begin the test by opening valve p; start the timer as the water level falls to h_0 and record the elapsed times when the water level reaches $\sqrt{h_0h_1}$ and h_1. Stop the flow after the level passes h_1 by closing p.

13. Obtain temperature readings at the head water end of the sample and in the constant head chamber.

14. Compare the elapsed time required for fall from h_0 to $\sqrt{h_0h_1}$ with that for $\sqrt{h_0h_1}$ to h_1.[10] If these times do not agree within 2% or 3%, refill and rerun.[11]

15. When a good run has been obtained, decrease the void ratio by tapping the side of the permeameter with the wooden hammer.

16. Remeasure the sample length and obtain time observations for the falling head in the standpipe as was done for the previous void ratio.

Constant Head Test

1. Place the soil in a measured permeameter, weigh, and saturate as in the variable head test (steps 1-8).

2. Measure the value of the head, h, and specimen length, L.

3. Start flow by opening valve a (see Fig. VI-3).

4. After allowing a few minutes for equilibrium conditions to be reached, obtain graduate and time readings.

5. After a sufficient amount of water has collected in the graduate for a satisfactory measure of its volume, take graduate and time observations. Subtract the graduate and time readings obtained in step 4 from the respective values obtained in this step to give Q and t for Eq. VI-2.

6. Record the temperature of the water every few minutes.

7. Change the void ratio of the soil as was done in the variable head test, and take another series of graduate and time readings. Measure the specimen length at each void ratio.

Discussion of Procedure

Degree of Saturation. In the preceding procedure, an attempt was made to get the soil completely satu-

[8] See page 58 for a discussion of the maximum grain size which should be used.

[9] Pouring the soil into the permeameter tends to cause segregation. Segregation can be minimized by placing the soil with a small can tied to strings in such a way that it can be lowered into the permeameter and then emptied.

[10] Since

$$\frac{h_0}{\sqrt{h_0h_1}} = \frac{\sqrt{h_0h_1}}{h_1}$$

the elapsed times should be equal because the other terms in Eq. VI-1 are constant for any given run. A lack of agreement here could be due to leaks, incomplete saturation, movement of fines, foreign matter in water, or water not sufficiently deaired.

[11] Even though the times for the two decrements are in agreement, it is a good policy to make a check run (see Numerical Example).

rated because the permeability of an "almost saturated" soil may be considerably different from its saturated[12] value. Figure VI-1 illustrates this point. To obtain a high degree of saturation, use a vacuum approaching absolute zero. For example, Fig. VI-4 shows the relationship between the degree of vacuum for evacuating a certain fine sand and the resulting degree of saturation. In this case an applied vacuum of at least 27 or 28 in. of mercury was necessary to get a high degree of saturation.

The water used for saturating the soil should be almost completely deaired, because if there is much

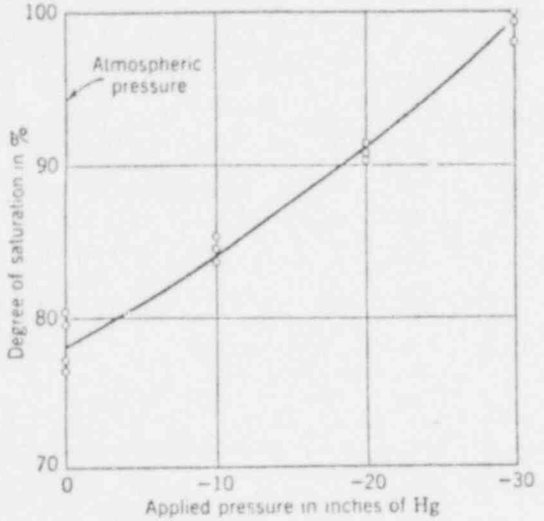

FIGURE VI-4. (From reference VI-4.)

air dissolved in the water, most of it will be brought out of solution by the high vacuum used for the saturating process of step 7 (see page 56). The deairing of the saturating water, however, presents no problems in the apparatus shown in Fig. VI-2. In fact, the procedure described in step 7 applies a vacuum to the water in the "distilled deaired water chamber" from which the saturating water is drawn. A vacuum can be kept on the water in this chamber when the apparatus is not in use.

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 to collect in the tubing or to collect in the soil, thus decreasing its degree of saturation. If water saturated with air were used, a rise in temperature or a decrease in pressure in the water would have to be prevented as it flowed from its storage supply through the soil. This is because the solubility of air is proportional to the pressure of the air above the water for small pressures (Henry's law, VI-3) and decreases with temperature as shown by Fig. VI-5. The solubility of air in water may be altered by other changes in the water as it flows through the soil; for example, the dissolving of any soluble salts from the soil.

To prevent any air from coming out of solution, two procedures are recommended. First, keep the temperature of the water a few degrees warmer than the soil and tubing. If this is done, the water will cool as it flows, thus slightly increasing its capacity for dissolving air. This procedure is known as "maintaining a favorable temperature gradient." Second, use water which has less than its capacity of air dissolved in it; such water is commonly called "deaired" water.

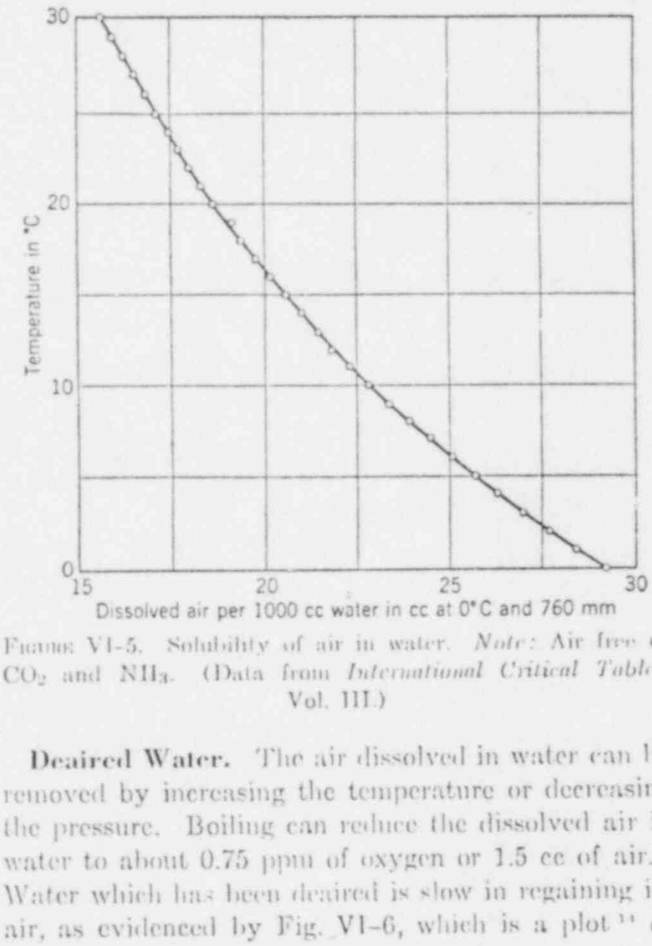

FIGURE VI-5. Solubility of air in water. *Note:* Air free of CO_2 and NH_3. (Data from *International Critical Tables*, Vol. III.)

Deaired Water. The air dissolved in water can be removed by increasing the temperature or decreasing the pressure. Boiling can reduce the dissolved air in water to about 0.75 ppm of oxygen or 1.5 cc of air.[13] Water which has been deaired is slow in regaining its air, as evidenced by Fig. VI-6, which is a plot[14] of

[12] As discussed in Chapter VIII, natural soils do not necessarily exist in a saturated state. Careful control of the degree of saturation, however, is required in order to obtain test data which can be reproduced. Also, the permeability of a soil when saturated is a limiting value and, therefore, is of importance (see Chapter VIII). Unfortunately, there are permeability test procedures in use which do not control, or even measure, the degree of saturation.

[13] One ppm of oxygen in air dissolved in water corresponds approximately to 2.0 cc of air at 760 mm pressure and 0° C per 1000 cc of water.

[14] This is a plot of data from a research project in the Hydraulics Laboratory at M.I.T. The data were obtained by the mercury-dropping electrode system; the readings were taken at

oxygen pick-up against elapsed time for a vessel of deaired water whose surface was exposed to the air. Figure VI–6 shows that at the end of 13 days the water was only 60% saturated. More elaborate methods for deairing and storing water are available (VI–2), but they are not thought necessary for normal permeability testing. Boiled distilled water is satisfactory for most permeability testing for some time after

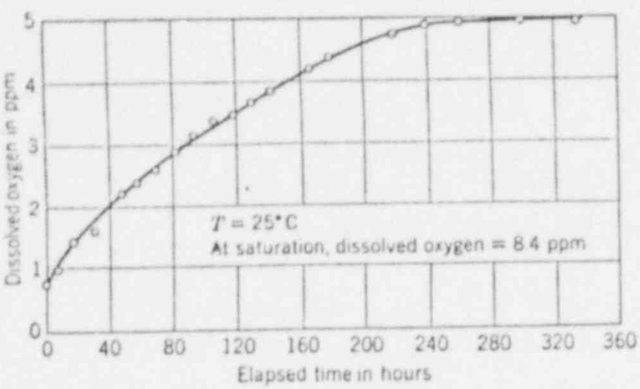

FIGURE VI–6. Pick-up of oxygen by water.

boiling. The water should not be agitated and should be covered to prevent the collection of foreign matter from the atmosphere. Water can easily be covered by stoppering the storage vessel and venting it with a tube whose end is pointing downward, as illustrated in Fig. VI–7. Figure VI–7 also shows the recommended manner to tap the water supply; the water at the bottom of the vessel tends to contain less dissolved air than that at the top.

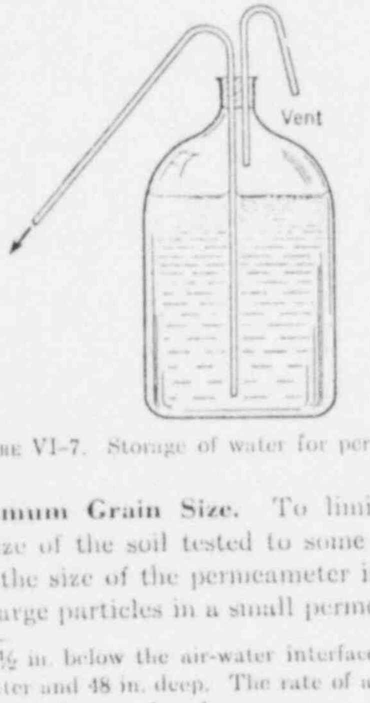

FIGURE VI–7. Storage of water for permeability tests.

Maximum Grain Size. To limit the maximum grain size of the soil tested to some reasonable fraction of the size of the permeameter is desirable. The use of large particles in a small permeameter increases the chance of large voids forming where the particles touch the wall of the permeameter. Keeping the ratio of the permeameter diameter to the diameter of the largest soil particle greater than about 15 or 20 has been found satisfactory. This limits the soil tested in the 4-cm permeameter suggested on page 53 to that passing a No. 8 or No. 10 sieve. A larger permeameter should be used to test a coarser soil.

If the soil tested is too coarse, the flow will be turbulent rather than laminar. Laminar flow is assumed in Darcy's law, by which Eqs. VI–1 and VI–2 are derived. For the normal test setup, laminar flow exists only in soils finer than coarse sands. The error appears small, however, in using Darcy's law on soils whose particles are a little larger than coarse sand.

Gradient Increase by Gas Pressure. To increase the rate of flow in the constant head testing of soils of low permeability, a gas pressure can be applied to the surface of the water supply. (When a pressure is used, it is advisable to cover the surface of the water supply with a membrane of some sort to reduce the amount of gas going into solution.) The head lost is then h (Fig. VI–3) plus the applied pressure changed to units of water head. Pressure is often employed for permeability determinations on consolidation specimens (Chapter IX).

Calculations

Variable Head Test

The coefficient of permeability k can be computed from

$$k = 2.3 \frac{aL}{A(t_1 - t_0)} \log_{10} \frac{h_0}{h_1} \qquad \text{(VI–1)}$$

in which a = cross-sectional area of the standpipe,
L = length of soil sample in permeameter,
A = cross-sectional area of the permeameter,
t_0 = time[15] when water in standpipe is at h_0,
t_1 = time when water in standpipe is at h_1,
h_0, h_1 = the heads between which the permeability is determined (see Fig. VI–2).

Constant Head Test

The coefficient of permeability k can be computed from

$$k = \frac{QL}{thA} \qquad \text{(VI–2)}$$

in which Q = total quantity of water which flowed through in elapsed time, t,
h = total head lost (see Fig. VI–3).

a point ½ in. below the air-water interface in a vessel 5½ in. in diameter and 48 in. deep. The rate of air pick-up is related to the ratio of exposed surface area over volume of the water.

[15] If the time is started at zero when the water in the standpipe is at h_0, then t_0 is equal to zero.

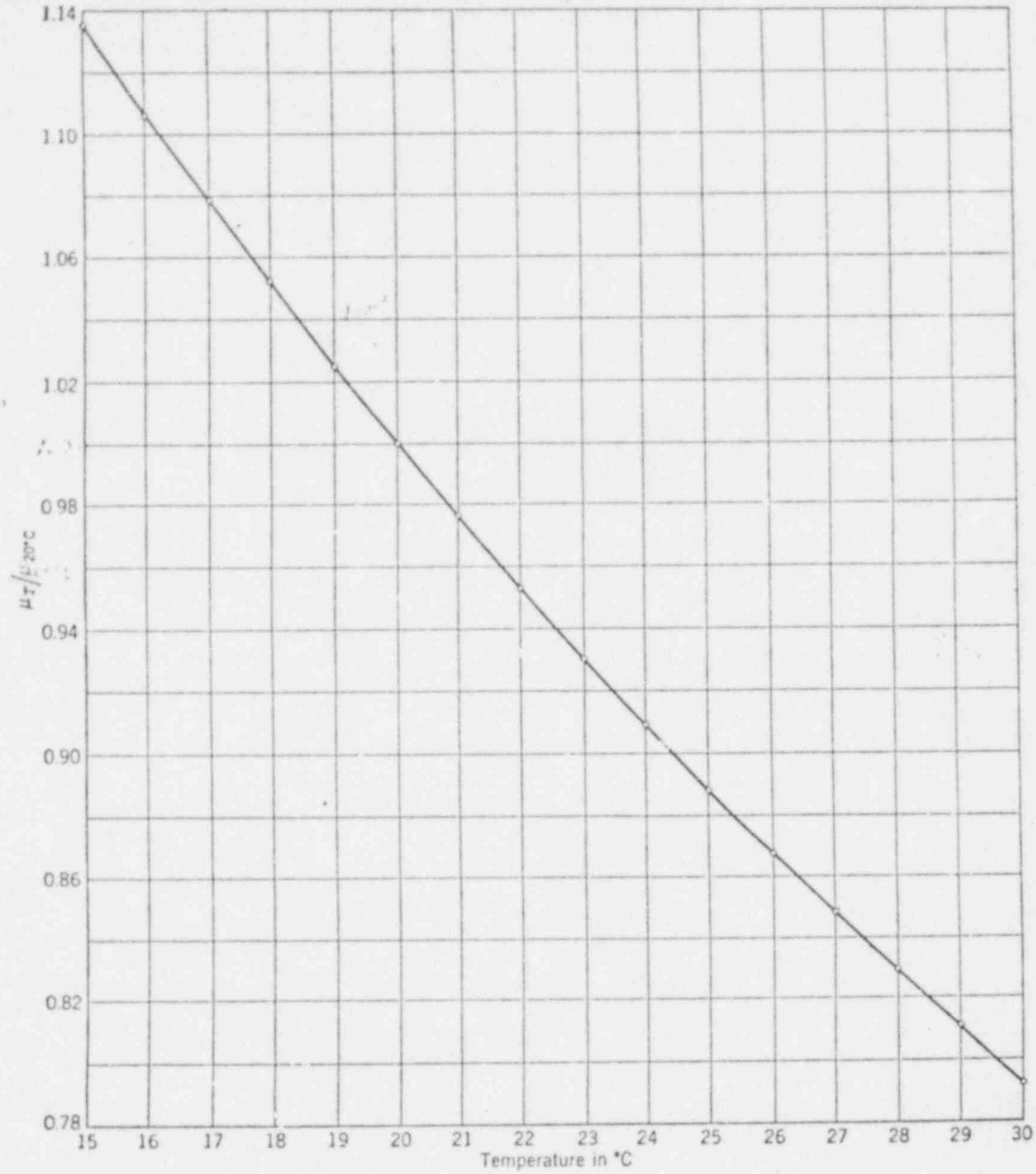

FIGURE VI-8. (Data from *International Critical Tables*, Vol. V.)

The permeability at temperature T, k_T, can be reduced to that at 20° C, $k_{20°C}$, by using

$$k_{20°C} = k_T \frac{\mu_T}{\mu_{20°C}} \qquad \text{(VI-3)}$$

in which $k_{20°C}$ = permeability at temperature 20° C,
k_T = permeability at temperature T,
μ_T = viscosity of water at temperature T (see Table A-3, p. 148),
$\mu_{20°C}$ = viscosity of water at temperature 20° C (see Table A-3, p. 148).

A plot of $\mu_T/\mu_{20°C}$ against temperature is given in Fig. VI-8.

Results

Method of Presentation. The results of a permeability test are usually presented in the form of a plot of some function of void ratio, e, against some function of permeability, $k_{20°C}$. Often two plots are made: k vs. $e^3/(1 + e)$, $e^2/(1 + e)$, and e^2 on one sheet and e vs. $\log k$. The best relationship of the above four is then used to present the results of the test (see discussion below).

Typical Values. The permeabilities of several soils are given in Fig. VI-1. A better indication of typical permeabilities can be obtained from the classification of soils based on their permeabilities which is given below (VI-5).

Degree of Permeability	*k in Centimeters per Second*
High	Over 10^{-1}
Medium	10^{-1} to 10^{-3}
Low	10^{-3} to 10^{-5}
Very low	10^{-5} to 10^{-7}
Practically impermeable	Less than 10^{-7}

A permeability of 1 μ per second (10^{-4} cm per second) is frequently used as the borderline between pervious and impervious soils. Thus a soil with a permeability less than 1 μ per second might be considered for a dam core or impervious blanket, whereas one with a permeability greater than 1 μ per second might be considered for a dam shell or pervious backfill.

Discussion. Both theoretically and experimentally there is more justification for $e^3/(1 + e)$ to be proportional to k than for either $e^2/(1 + e)$ or e^2 in the case of cohesionless soils. Laboratory tests on all types of soils have shown that a plot of void ratio versus log of permeability is usually close to a straight line.

Numerical Example

In the example on pages 61 and 62 are presented the results of a variable head permeability test on a well-graded, coarse sand, which was used for the shell of an earth dam. The plots of data in Fig. VI–9 show that $e^3/(1 + e)$ is almost proportional to k and that the e vs. log k curve is almost a straight line. According to the classification given under Typical Values, this soil would be called one of medium permeability.

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