METHOD 9100

SATURATED HYDRAULIC CONDUCTIVITY,
SATURATED LEACHATE CONDUCTIVITY, AND
INTRINSIC PERMEABILITY

1.0 INTRODUCTION

1.1 Scope and Application: This section presents methods available to hydrogeologists and and geotechnical engineers for determining the saturated hydraulic conductivity of earth materials and conductivity of soil liners to leachate, as outlined by the Part 264 permitting rules for hazardous-waste disposal facilities. In addition, a general technique to determine intrinsic permeability is provided. A cross reference between the applicable part of the RCRA Guidance Documents and associated Part 264 Standards and these test methods is provided by Table A.

1.1.1 Part 264 Subpart F establishes standards for ground water quality monitoring and environmental performance. To demonstrate compliance with these standards, a permit applicant must have knowledge of certain aspects of the hydrogeology at the disposal facility, such as hydraulic conductivity, in order to determine the compliance point and monitoring well locations and in order to develop remedial action plans when necessary.

1.1.2 In this report, the laboratory and field methods that are considered the most appropriate to meeting the requirements of Part 264 are given in sufficient detail to provide an experienced hydrogeologist or geotechnical engineer with the methodology required to conduct the tests. Additional laboratory and field methods that may be applicable under certain conditions are included by providing references to standard texts and scientific journals.

1.1.3 Included in this report are descriptions of field methods considered appropriate for estimating saturated hydraulic conductivity by single well or borehole tests. The determination of hydraulic conductivity by pumping or injection tests is not included because the latter are considered appropriate for well field design purposes but may not be appropriate for economically evaluating hydraulic conductivity for the purposes set forth in Part 264 Subpart F.

1.1.4 EPA is not including methods for determining unsaturated hydraulic conductivity at this time because the Part 264 permitting standards do not require such determinations.

1.2 Definitions: This section provides definitions of terms used in the remainder of this report. These definitions are taken from U.S. Government publications when possible.

TABLE A

HYDRAULIC AND LINER CONDUCTIVITY DETERMINATION
METHODS FOR SURFACE IMPOUNDMENT,
WASTE PILE, AND LANDFILL COMPONENTS, AS CITED
IN RCRA GUIDANCE DOCUMENTS AND DESCRIBED IN SW-846

Surface Impoundments	Guidance Cite[1] Associated Regulation	Corresponding SW-846 Section
Soil liner hydraulic conductivity	Guidance section D(2)(b)(1) and D(2)(c)(1)/Section 264.221(a),(b)	2.0
Soil liner leachate conductivity	Guidance section D(2)(b)(2) and D(2)(c)(2)	2.11
Leak detection	Guidance section C(2)(a)/ Section 264.222	2.0
Final cover drain layer	Guidance section E(2)(d)(1) Section 264.228	2.0
Final cover low permeability layer	Guidance section E(2)(e)(2)(A)/ Section 264.228	2.0
General hydrogeologic site investigation	264 subpart F	3.0

[1] RCRA Guidance Document: Surface Impoundments, Liner Systems, Final Cover, and Freeboard Control. Issued July, 1982.

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TABLE A (continued)

Waste Piles	Guidance Cite[2] Associated Regulation	Corresponding SW-846 Section
Soil liner hydraulic conductivity	Guidance section D(2)(b)(i) and D(2)(c)(i)/ Section 264.251(a)(1)	2.0
Soil liner leachate conductivity	Guidance section D(2)(b)(ii) and D(2)(c)(ii)	2.11
Leak detection system	Guidance section C(2)(a)/ Section 264.252(a)	2.0
Leachate collection and renewal system	Guidance section C(2)(a)/ Section 264.251(a)(2)	2.0
General hydrogeologic site investigation	264 subpart F	3.0

[2] RCRA Guidance Document: Waste Pile Design, Liner Systems. Issued July, 1982.

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TABLE A (continued)

Landfills	Guidance Cite[3] Associated Regulation	Corresponding SW-846 Section
Soil liner hydraulic conductivity	Guidance section D(2)(b)(1)/ Section 264.301(a)(1)	2.0
Soil liner leachate conductivity	Guidance section D(2)(b)(2)	2.11
Leak detection system	Guidance section C(2)(a)/ Section 264.302(a)(3)	2.0
Leachate collection and removal system	Guidance section C(2)(a)/ Section 264.301(a)(2)	2.0
Final cover drain layer	Guidance section E(2)(d)(1)/ Section 264.310(a)(b)	2.0
Final cover low permeability layer	Guidance section E(2)(e)(2)(A) Section 264.310(a)(b)	2.0
General hydrogeologic site investigation	264 subpart F	3.0

[3] RCRA Guidance Document: Landfill Design, Liner Systems and Final Cover. Issued July, 1982.

1.2.1 **Units:** This report uses consistent units in all equations. The symbols used are:

Length = L,
Mass = M, and
Time = T.

1.2.2 **Fluid potential or head** (h): A measure of the potential energy required to move fluid from a point in the porous medium to a reference point. For virtually all situations expected to be found in disposal sites and in ground water systems, h is defined by the following equation:

$$h = h_p + h_z \qquad (1)$$

where:

h is the total fluid potential, expressed as a height of fluid above a reference datum, L;

h_p, the pressure potential caused by the weight of fluid above the point in question, L, is defined by $h_p = P/\rho g$,

where:

P is the fluid pressure at the point in question, $ML^{-1}T^{-2}$,

ρ is the fluid density at the prevailing temperature, ML^{-3}, and

g is the acceleration of gravity, LT^{-2}; and

h_z is the height of the point in question above the reference datum, L.

By knowing h_p and h_z at two points along a flow path and by knowing the distance between these points, the fluid potential gradient can be determined.

1.2.3 **Hydraulic potential or head:** The fluid potential when water is the fluid.

1.2.4 **Hydraulic conductivity:** The fluid potential when water is the fluid. The generic term, fluid conductivity, is discussed below in 1.2.5.

1.2.5 **Fluid conductivity** (K): Defined as the volume of fluid at the prevailing density and dynamic viscosity that will move in a unit time under a unit fluid potential gradient through a unit area measured at right angles to the direction of flow. It is a property of both the fluid and the porous medium as shown by the following equation:

$$K = \frac{k\rho g}{u} ; \qquad (2)$$

where:

K is the fluid conductivity, LT^{-1};

k is the <u>intrinsic permeability</u>, a property of the porous medium alone, L^2; and

u is the dynamic viscosity of the fluid at the prevailing temperature, $ML^{-1}T^{-1}$.

The fluid conductivity of a porous material is also defined by Darcy's law, which states that the fluid flux (q) through a porous medium is proportional to the first power of the fluid potential across the unit area:

$$q = \frac{Q}{A} = -KI \qquad (3)$$

where:

q = the specific fluid flux, LT^{-1},

Q is the volumetric fluid flux, L^3T^{-1},

A is the cross-sectional area, L^2, and

I is the fluid potential gradient, L^0.

Darcy's law provides the basis for all methods used to determine hydraulic conductivity in this report. The range of validity of Darcy's law is discussed in Section 1.5 (Lohman, 1972).

1.2.6 **Leachate conductivity:** The fluid conductivity when leachate is the fluid.

1.2.7 **Aquifer:** A geologic formation, group of formations, or part of a formation capable of yielding a significant amount of ground water to wells or springs (40 CFR 260.10).

1.2.8 **Confining layer:** By strict definition, a body of impermeable material stratigraphically adjacent to one or more aquifers. In nature, however, its hydraulic conductivity may range from nearly zero to some value distinctly lower than that of the aquifer. Its conductivity relative to that of the aquifer it confines should be specified or indicated by a suitable modifier, such as "slightly permeable" or "moderately permeable" (Lohman, 1972).

1.2.9 **Transmissivity**, T [L^2, T^{-1}]: The rate at which water of the prevailing kinematic viscosity is transmitted through a unit width of the aquifer under a unit hydraulic gradient. Although spoken of as a property of the aquifer, the term also includes the saturated thickness of the aquifer and the properties of the fluid. It is equal to an integration of the hydraulic conductivities across the saturated part of the aquifer perpendicular to the flow paths (Lohman, 1972).

1.3 Temperature and viscosity corrections: By using Equation (2), corrections to conditions different from those prevailing during the test can be made. Two types of corrections can commonly be made: a correction for a temperature that varies from the test temperature, and a correction for fluids other than that used for the test. The temperature correction is defined by:

$$K_f = \frac{K_t \; u_t \; \rho_f}{u_f \; \rho_t} \qquad (4)$$

where:

the subscript f refers to field conditions, and

the subscript t refers to test conditions.

Most temperature corrections are necessary because of the dependence of viscosity on temperature. Fluid density variations caused by temperature changes are usually very small for most liquids. The temperature correction for water can be significant. Equation (4) can also be used to determine hydraulic conductivity if fluids other than water are used. It is assumed, however, when using Equation (4) that the fluids used do not alter the intrinsic permeability of the porous medium during the test. Experimental evidence shows that this alteration does occur with a wide range of organic solvents (Anderson and Brown, 1981). Consequently, it is recommended that tests be run using fluids, such as leachates, that might occur at each particular site. Special considerations for using non-aqueous fluids are given in Section 3.3 of this report.

1.4 Intrinsic permeability (k): Rearrangement of Equation 2 results in a definition of intrinsic permeability:

$$k = \frac{Ku}{\rho g} \; . \qquad (5)$$

Since this is a property of the medium alone, if fluid properties change, the fluid conductivity must also change to keep the intrinsic permeability a constant. By using measured fluid conductivity, and values of viscosity and density for the fluid at the test temperature, intrinsic permeability can be determined.

1.5 Range of validity of Darcy's law: Determination of fluid conductivities using both laboratory and field methods requires assuming the validity of Darcy's law. Experimental evidence has shown that deviations from the linear dependence of fluid flux on potential gradient exist for both extremely low and extremely high gradients (Hillel, 1971; Freeze and Cherry, 1979). The lower limits are the result of the existence of threshold gradients required to initiate flow (Swartzendruber, 1962). The upper limits to the validity of Darcy's law can be estimated by the requirements that the Reynolds number, Re, in most cases be kept below 10 (Bear, 1972). The Reynolds number is defined by:

$$Re = \frac{\rho q d}{u} \tag{6}$$

where:

d is some characteristic dimension of the system, often represented by the median grain size diameter, D_{50}, (Bouwer, 1978), and

q is the fluid flux per unit area, LT^{-1}.

For most field situations, the Reynolds number is less than one, and Darcy's law is valid. However, for laboratory tests it may be possible to exceed the range of validity by the imposition of high potential gradients. A rough check on acceptable gradients can be made by substituting Darcy's law in Equation (6) and using an upper limit of 10 for Re:

$$I \leq \frac{10u}{\rho K D_{50}} \tag{7}$$

where:

K is the approximate value of fluid conductivity determined at gradient I.

A more correct check on the validity of Darcy's law or the range of gradients used to determine fluid conductivity is performed by measuring the conductivity at three different gradients. If a plot of fluid flux versus gradient is linear, Darcy's law can be considered to be valid for the test conditions.

1.6 Method Classification: This report classifies methods of determining fluid conductivity into two divisions: laboratory and field methods. Ideally, and whenever possible, compliance with Part 264 disposal facility requirements should be evaluated by using field methods that test the materials under in-situ conditions. Field methods can usually provide more representative values than laboratory methods because they test a larger volume of material, thus integrating the effects of macrostructure and heterogeneities. However, field methods presently available to determine the conductivity of compacted fine-grained materials in reasonable times require the tested interval to be below a water table or to be fairly thick, or require excavation of the material to be tested at some point in the test. The integrity of liners and covers should not be compromised by the installation of boreholes or piezometers required for the tests. These restrictions generally lead to the requirement that the fluid conductivity of liner and cover materials must be determined in the laboratory. The transfer value of laboratory data to field conditions can be maximized for liners and covers because it is possible to reconstruct relatively accurately the desired field conditions in the laboratory. However, field conditions that would alter the values determined in the laboratory need to be addressed in permit applications. These conditions include those that would increase conductivity by the formation of microcracks and channels by repeated wetting and drying, and by the penetration of roots.

1.6.1 **Laboratory methods** are categorized in Section 2.0 by the methods used to apply the fluid potential gradient across the sample. The discussion of the theory, measurement, and computations for tests run under constant and falling-head conditions is followed by a detailed discussion of tests using specific types of laboratory apparatus and the applicability of these tests to remolded compacted, fine-grained uncompacted, and coarse-grained porous media. Section 2.3 provides a discussion of the special considerations for conducting laboratory tests using non-aqueous permeants. Section 2.10 gives a discussion of the sources of error and guidance for establishing the precision of laboratory tests. Laboratory methods may be necessary to measure vertical fluid conductivity. Values from field tests reflect effects of horizontal and vertical conductivity.

1.6.2 **Field methods** are discussed in Section 3.0 and are limited to those requiring a single bore hole or piezometer. Methods requiring multiple bore holes or piezometers and areal methods are included by reference. Because of the difficulties in determining fluid conductivity of in-place liner and cap materials under field conditions without damaging their integrity, the use of field methods for fine-grained materials will be generally restricted to naturally occurring materials that may serve as a barrier to fluid movement. Additional field methods are referenced that allow determination of saturated hydraulic conductivity of the unsaturated materials above the shallowest water table. General methods for fractured media are given in Section 3.8. A discussion of the important considerations in well installation, construction, and development is included as an introduction to Section 3.0.

2.0 LABORATORY METHODS

2.1 Sample collection for laboratory method: To assure that a reasonable assessment is made of field conditions at a disposal site, a site investigation plan should be developed to direct sampling and analysis. This plan generally requires$the professional judgement of an experienced hydrogeologist or geotechnical engineer. General guidance is provided for plan development in the Guidance Manual for Preparation of a Part 264 land Disposal Facility Permit Application (EPA, in press). The points listed below should be followed:

- o The hydraulic conductivity of a soil liner should be determined either from samples that are processed to simulate the actual liner, or from an undisturbed sample of the complete liner.

- o To obtain undisturbed samples, the thin-walled tube sampling method (ASTM Method # D1587-74) or a similar method may be used. Samples representative of each lift of the liner should be obtained, and used in the analyses. If actual undisturbed samples are not used, the soil used in liner construction must be processed to represent accurately the liner's initial water content and bulk density. The method described in Section 2.7.3 or ASTM Method #D698-70 (ASTM, 1978) can be used for this purpose.

- o For purpose of the general site investigation, the general techniques presented in ASTM method #D420-69 (ASTM, 1978) should be followed. This reference establishes practices for soil and rock investigation and sampling, and incorporates various detailed ASTM procedures for investigation, sampling, and material classification.

2.2 Constant-head methods: The constant-head method is the simplest method of determining hydraulic conductivity of saturated soil samples. The concept of the constant-head method is schematically illustrated in Figure 1. The inflow of fluid is maintained at a constant head (h) above a datum and outflow (Q) is measured as a function of time (t). Using Darcy's law, the hydraulic conductivity can be determined using the following equation after the outflow rate has become constant:

$$K = QL/hA, \tag{8}$$

where:

K = hydraulic conductivity, LT^{-1};

L = length of sample, L;

A = cross-sectional area of sample, L^2;

Q = outflow rate, L^3T^{-1}; and

h = fluid head difference across the sample, L.

Constant-head methods should be restricted to tests on media having high fluid conductivity.

2.3 Falling-head methods: A schematic diagram of the apparatus for the falling-head method is shown in Figure 2. The head of inflow fluid decreases from h_1 to h_2 as a function of time (t) in a standpipe directly connected to the specimen. The fluid head at the outflow is maintained constant. The quantity of outflow can be measured as well as the quantity of inflow. For the setup shown in Figure 2a, the hydraulic conductivity can be determined using the following equation:

$$K = \frac{2.3\ aL}{At}\log_{10}\frac{h_0}{h_1}\ , \tag{9}$$

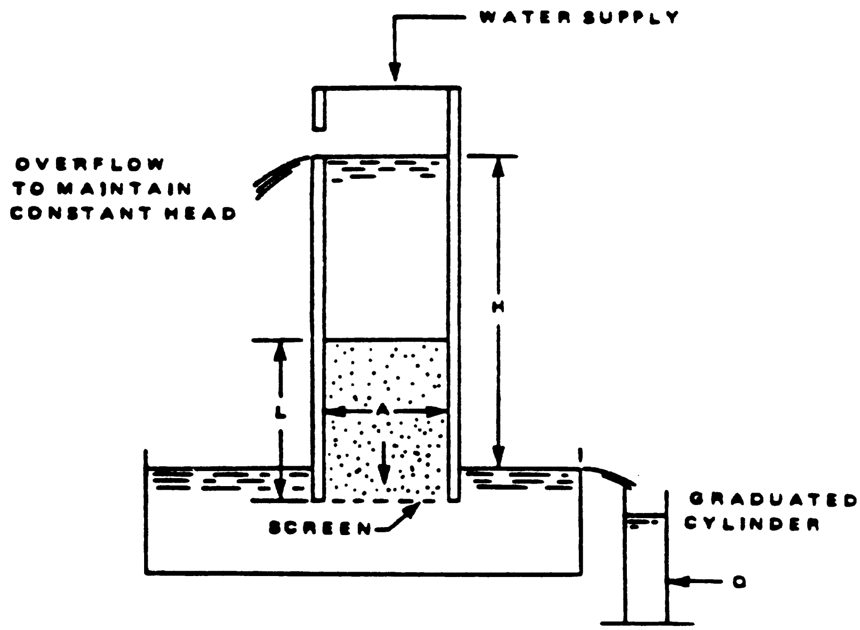

Figure 1.--Principle of the constant head method

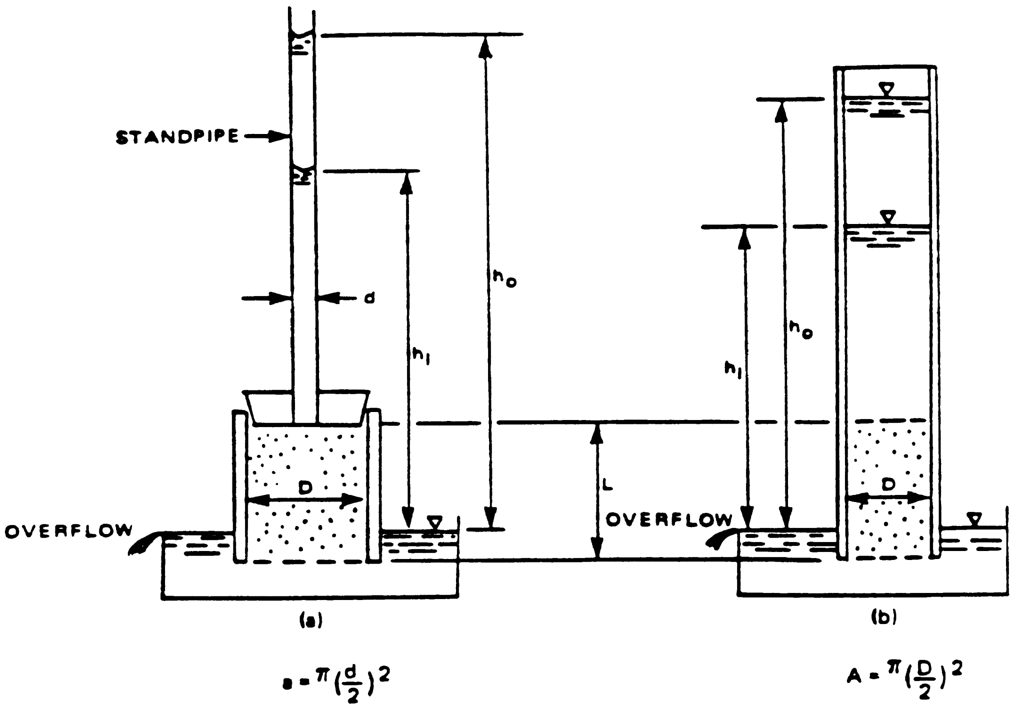

Figure 2.--Principle of the falling head method using a small (a) and large (b) standpipe.

where:

a = the cross-sectional area of the standpipe, L^2;

A = the cross-sectional area of the specimen, L^2;

L = the length of the specimen, L; and

t = elapsed time from t_1 to t_2, T.

For the setup in Figure 2b, the term a/A in Equation (9) is replaced by 1.0. Generally, falling-head methods are applicable to fine-grained soils because the testing time can be accelerated.

2.4 General test considerations:

2.4.1 **Fluid supplies to be used:** For determining hydraulic conductivity and leachate conductivity, the supplies of permeant fluid used should be de-aired. Air coming out of solution in the sample can significantly reduce the measured fluid conductivity. Deairing can be achieved by boiling the water supply under a vacuum, bubbling helium gas through the supply, or both.

2.4.1.1 Significant reductions in hydraulic conductivity can also occur in the growth and multiplication of microorganisms present in the sample. If it is desirable to prevent such growth, a bactericide or fungicide, such as 2000 ppm formaldehyde or 1000 ppm phenol (Olsen and Daniel, 1981), can be added to the fluid supply.

2.4.1.1 Fluid used for determining hydraulic conductivity in the laboratory should never be distilled water. Native ground water from the aquifer underlying the sampled area or water prepared to simulate the native ground water chemistry should be used.

2.4.2 **Pressure and Fluid Potential Measurement:** The equations in this report are all dimensionally correct; that is, any consistent set of units may be used for length, mass, and time. Consequently, measurements of pressure and/or fluid potential using pressure gages and manometers must be reduced to the consistent units used before applying either Equation 8 or 9. Pressures or potentials should be measured to within a few tenths of one percent of the gradient applied across the sample.

2.5 Constant-head test with conventional permeameter:

2.5.1 **Applicability:** This method covers the determination of the hydraulic conductivity of soils by a constant-head method using a conventional permeameter. This method is recommended for disturbed coarse-grained soils. If this method is to be used for fine-grained soils, the testing time may be prohibitively long. This method was taken from the Engineering and Design, Laboratory Soils Testing Manual (U.S. Army, 1980). It parallels ASTM Method D2434-68 (ASTM,1978). The ASTM method gives extensive discussion of sample preparation and applicability and should be reviewed before conducting constant-head tests. Lambe (1951) provides additional information on sample preparation and equipment procedures.

2.5.2 **Apparatus:** The apparatus is shown schematically in Figure 3. It consists of the following:

1. A permeameter cylinder having a diameter at least 8 times the diameter of the largest particle of the material to be tested;
2. Constant-head filter tank;
3. Perforated metal disks and circular wire to support the sample;
4. Filter materials such as Ottawa sand, coarse sand, and gravel of various gradations;
5. Manometers connected to the top and bottom of the sample;
6. Graduated cylinder, 100-mL capacity;
7. Thermometer;
8. Stop watch;
9. Deaired water;
10. Balance sensitive to 0.1 gram; and
11. Drying oven.

2.5.3 **Sample preparation:**

1. Oven-dry the sample. Allow it to cool, and weigh to the nearest 0.1 g. Record the oven-dry weight of material. The amount of material should be sufficient to provide a specimen in the permeameter having a minimum length of about one to two times the diameter of the specimen.
2. Place a wire screen, with openings small enough to retain the specimen, over a perforated disk near the bottom of the permeameter above the inlet. The screen opening should be approximately equal to the 10 percent size of the specimen.
3. Allow deaired water to enter the water inlet of the permeameter to a height of about 1/2 in. above the bottom of the screen, taking care that no air bubbles are trapped under the screen.

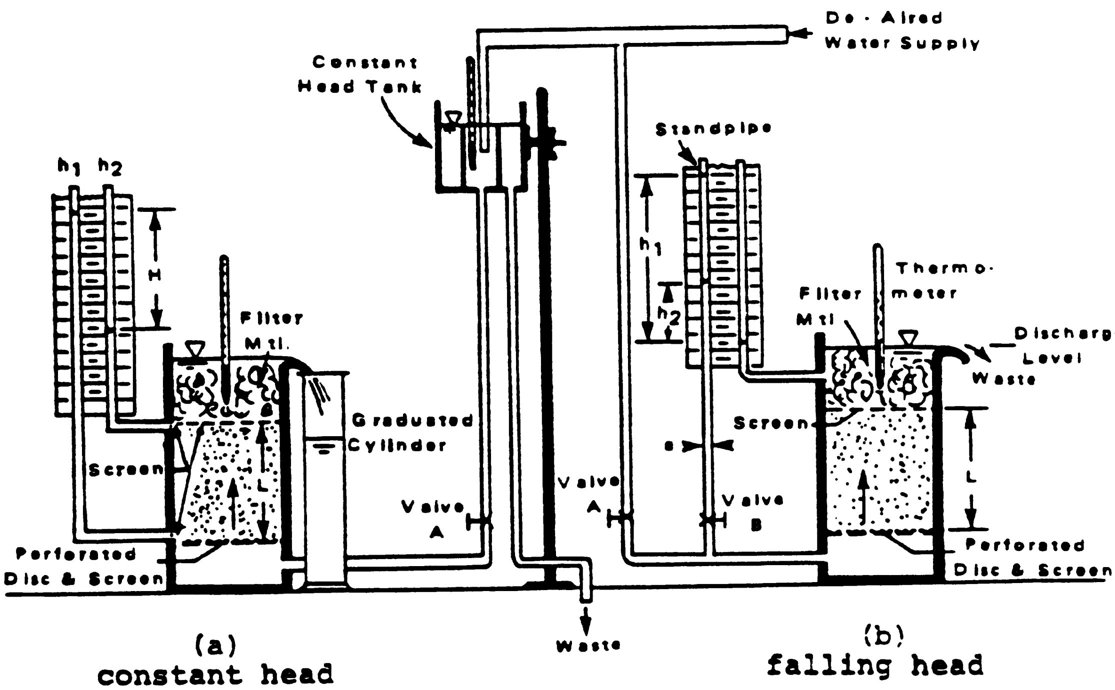

Figure 3.--Apparatus setup for the constant head (a) and falling head (b) methods.

4. Mix the material thoroughly and place in the permeameter to avoid segregation. The material should be dropped just at the water surface, keeping the water surface about 1/2 in. above the top of the soil during placement. A funnel or a spoon is convenient for this purpose.

5. The placement procedure outlined above will result in a saturated specimen of uniform density although in a relatively loose condition. To produce a higher density in the specimen, the sides of the permeameter containing the soil sample are tapped uniformly along its circumference and length with a rubber mallet to produce an increase in density; however, extreme caution should be exercised so that fines are not put into suspension and segregated within the sample. As an alternative to this procedure, the specimen may be placed using an appropriate sized funnel or spoon. Compacting the specimen in layers is not recommended, as a film of dust which might affect the permeability results may be formed at the surface of the compacted layer. After placement, apply a vacuum to the top of the specimen and permit water to enter the evacuated specimen through the base of the permeameter.

6. After the specimen has been placed, weigh the excess material, if any, and the container. The specimen weight is the difference between the original weight of sample and the weight of the excess material. Care must be taken so that no material is lost during placement of the specimen. If there is evidence that material has been lost, oven-dry the specimen and weigh after the test as a check.

7. Level the top of the specimen, cover with a wire screen similar to that used at the base, and fill the remainder of the permeameter with a filter material.

8. Measure the length of the specimen, inside diameter of the permeameter, and distance between the centers of the manometer tubes (L) where they enter the permeameter.

2.5.4 **Test procedure:**

1. Adjust the height of the constant-head tank to obtain the desired hydraulic gradient. The hydraulic gradient should be selected so that the flow through the specimen is laminar. Hydraulic gradients ranging from 0.2 to 0.5 are recommended. Too high a hydraulic gradient may cause turbulent flow and also result in piping of soils. In general, coarser soils require lower hydraulic gradients. See Section 1.5 for further discussion of excessive gradients.

2. Open valve A (see Figure 3a) and record the initial piezometer readings after the flow has become stable. Exercise care in building up heads in the permeameter so that the specimen is not disturbed.

3. After allowing a few minutes for equilibrium conditions to be reached, measure by means of a graduated cylinder the quantity of discharge corresponding to a given time interval. Measure the piezometric heads (h_1 and h_2) and the water temperature in the permeameter.

4. Record the quantity of flow, piezometer readings, water temperature, and the time interval during which the quantity of flow was measured.

2.5.5 **Calculations:** By plotting the accumulated quantity of outflow versus time on rectangular coordinate paper, the slope of the linear portion of the curve can be determined, and the hydraulic conductivity can be calculated using Equation (8). The value of h in Equation (8) is the difference between h_1 and h_2.

2.6 Falling-head test with conventional permeameter:

2.6.1 **Applicability:** The falling-head test can be used for all soil types, but is usually most widely applicable to materials having low permeability. Compacted, remolded, fine-grained soils can be tested with this method. This method presented is taken from the Engineering and Design, Laboratory Soils Testing Manual (U.S. Army, 1980).

2.6.2 **Apparatus:** The schematic diagram of the falling-head permeameter is shown in Figure 3b. The permeameter consists of the following equipment:

1. Permeameter cylinder, a transparent acrylic cylinder having a diameter at least 8 times the diameter of the largest particles;

2. Porous disk;

3. Wire screen;

4. Filter materials;

5. Manometer;

6. Timing device; and

2.6.3 **Sample Preparation:** Sample preparation for coarse-grained soils is similar to that described previously in Section 2.4.3. For fine-grained soils, samples are compacted to the desired density using methods described in ASTM Method D698-70.

2.6.4 **Test Procedure:**

1. Measure and record the height of the specimen, L, and the cross-sectional area of the specimen, A.

2. With valve B open (see Figure 3b), crack valve A, and slowly bring the water level up to the discharge level of the permeameter.

3. Raise the head of water in the standpipe above the discharge level of the permeameter. The difference in head should not result in an excessively high hydraulic gradient during the test. Close valves A and B.

4. Begin the test by opening valve B. Start the timer. As the water flows through the specimen, measure and record the height of water in the standpipe above the discharge level, h_1, at time t_1, and the height of water above the discharge level, h_2 at time t_2.

2.6.5 **Calculations:** From the test data, plot the logarithm of head versus time on rectangular coordinate paper, or use semi-log paper. The slope of the linear part of the curve is used to determine $\log_{10}(h_1/h_2)/t$. Calculate the hydraulic conductivity using Equation (9).

2.7 Modified compaction permeameter method:

2.7.1 **Applicability:** This method can be used to determine the hydraulic conductivity of a wide range of materials. The method is generally used for remolded fine-grained soils. The method is generally used under constant-head conditions. The method was taken from Anderson and Brown, 1981, and EPA (1980). It should be noted that this method method of Section 2.9.

2.7.2 **Apparatus:** The apparatus is shown in Figure 4 and consists of equipment and accessories as follows:

1. Soil chamber, a compaction mold having a diameter 8 times larger than the diameter of the largest particles (typically, ASTM standard mold, Number CN405, is used);

2. Fluid chamber, a compaction mold sleeve having the same diameter as the soil chamber;

3. 2-kg hammer;

4. Rubber rings used for sealing purposes;

5. A coarse porous stone having higher permeability than the tested sample;

6. Regulated source of compressed air; and

7. Pressure gage or manometer to determine the pressure on the fluid chamber.

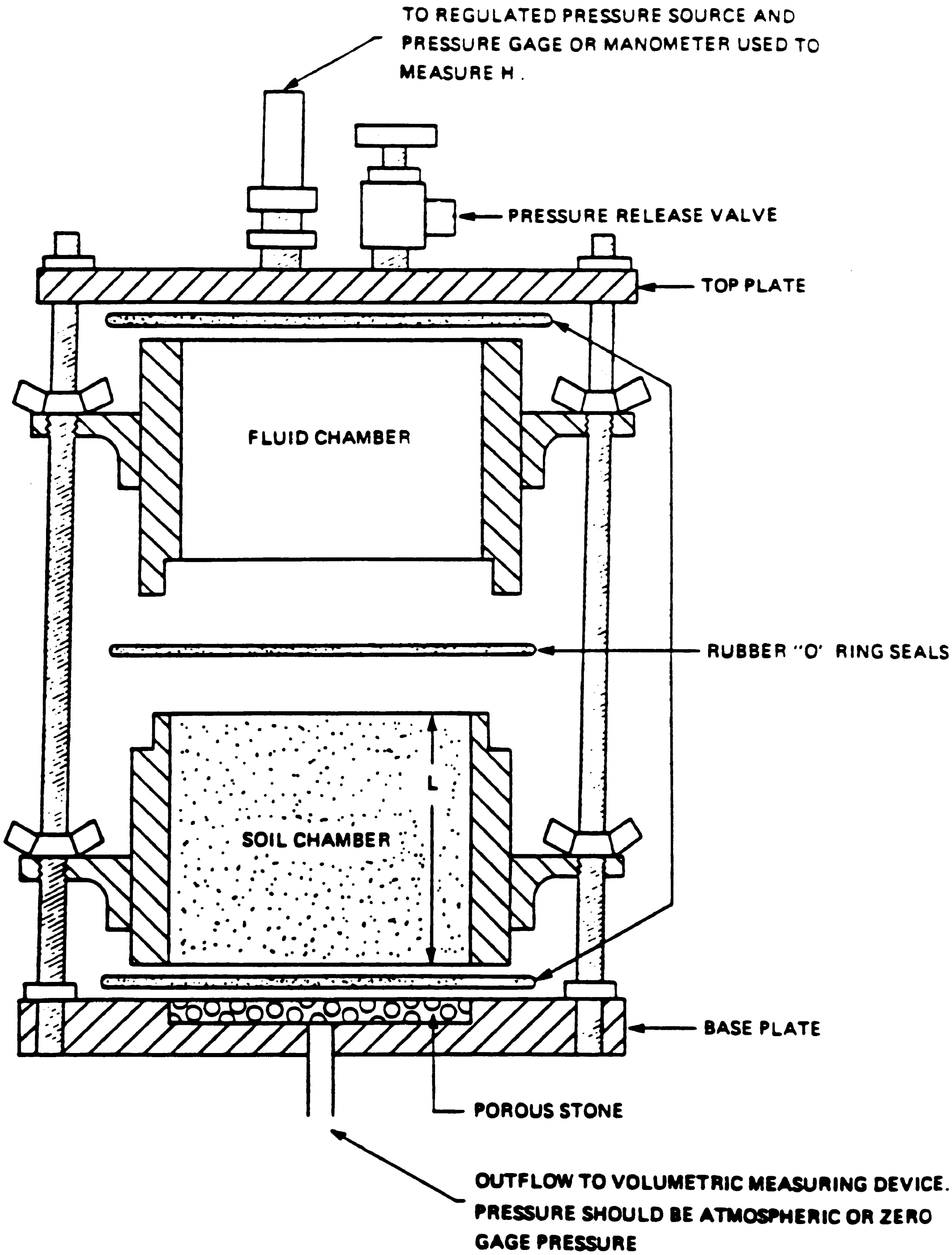

Figure 4.--Modified compaction permeameter.
Note: h in Equation 8 is the difference between the regulated inflow pressure and the outflow pressure. Source: Anderson and Brown, 1981.

2.7.3 **Sample preparation:**

1. Obtain sufficient representative soil sample. Air dry the sample at room temperature. Do not oven dry.

2. Thoroughly mix the selected representative sample with water to obtain a desired moisture content.

3. Compact the sample to the desired density within the mold using the method described as part of ASTM Method D698-70.

4. Level the surface of the compacted sample with straight edge, weigh and determine the density of the sample.

5. Measure the length and diameter of the sample.

6. Assemble the apparatus, make sure that there are no leaks, and then connect the pressure line to the apparatus.

2.7.4 **Test procedure:**

1. Place sufficient volume of water in the fluid chamber above the soil chamber.

2. Apply air pressure <u>gradually</u> to flush water through the sample until no air bubbles in the outflow are observed. For fine-grained soils, the saturation may take several hours to several days, depending on the applied pressure.

3. After the sample is saturated, measure and record the quantity of outflow versus time.

4. Record the pressure reading (h) on the top of the fluid chamber when each reading is made.

5. Plot the accumulated quantity of outflow versus time on rectangular coordinate paper.

6. Stop taking readings as soon as the linear position of the curve is defined.

2.7.5 **Calculations:** The hydraulic conductivity can be calculated using Equation (8).

2.8 <u>Triaxial-cell method with back pressure</u>:

2.8.1 **Applicability:** This method is applicable for all soil types, but especially for fine-grained, compacted, cohesive soils in which full fluid saturation of the sample is difficult to achieve. Normally, the test is run under constant-head conditions.

2.8.2 **Apparatus:** The apparatus is similar to conventional triaxial apparatus. The schematic diagram of this apparatus is shown in Figure 5.

2.8.3 **Sample preparation:** Disturbed or undisturbed samples can be tested. Undisturbed samples must be trimmed to the diameter of the top cap and base of the triaxial cell. Disturbed samples should be prepared in the mold using either kneading compaction for fine-grained soils, or by the pouring and vibrating method for coarse-grained soils, as discussed in Section 2.5.3.

2.8.4 **Test procedure:**

1. Measure the dimensions and weight of the prepared sample.

2. Place one of the prepared specimens on the base.

3. Place a rubber membrane in a membrane stretcher, turn both ends of the membrane over the ends of the stretcher, and apply a vacuum to the stretcher. Carefully lower the stretcher and membrane over the specimen. Place the specimen and release the vacuum on the membrane stretcher. Turn the ends of the membrane down around the base and up around the specimen cap and fasten the ends with O-rings.

4. Assemble the triaxial chamber and place it in position in the loading device. Connect the tube from the pressure reservoir to the base of the triaxial chamber. With valve C (see Figure 5) on the pressure reservoir closed and valves A and B open, increase the pressure inside the reservoir, and allow the pressure fluid to fill the triaxial chamber. Allow a few drops of the pressure fluid to escape through the vent valve (valve B) to insure complete filling of the chamber with fluid. Close valve A and the vent valve.

5. Place saturated filter paper disks having the same diameter as that of the specimen between the specimen and the base and cap; these disks will also facilitate removal of the specimen after the test. The drainage lines and the porous inserts should be completely saturated with deaired water. The drainage lines should be as short as possible and made of thick-walled, small-bore tubing to insure minimum elastic changes in volume due to changes in pressure. Valves in the drainage lines (valves E, F, and G in Figure 5) should preferably be of a type which will cause no discernible change of internal volume when operated. While mounting the specimen in the compression chamber, care should be exercised to avoid entrapping any air beneath the membrane or between the specimen and the base and cap.

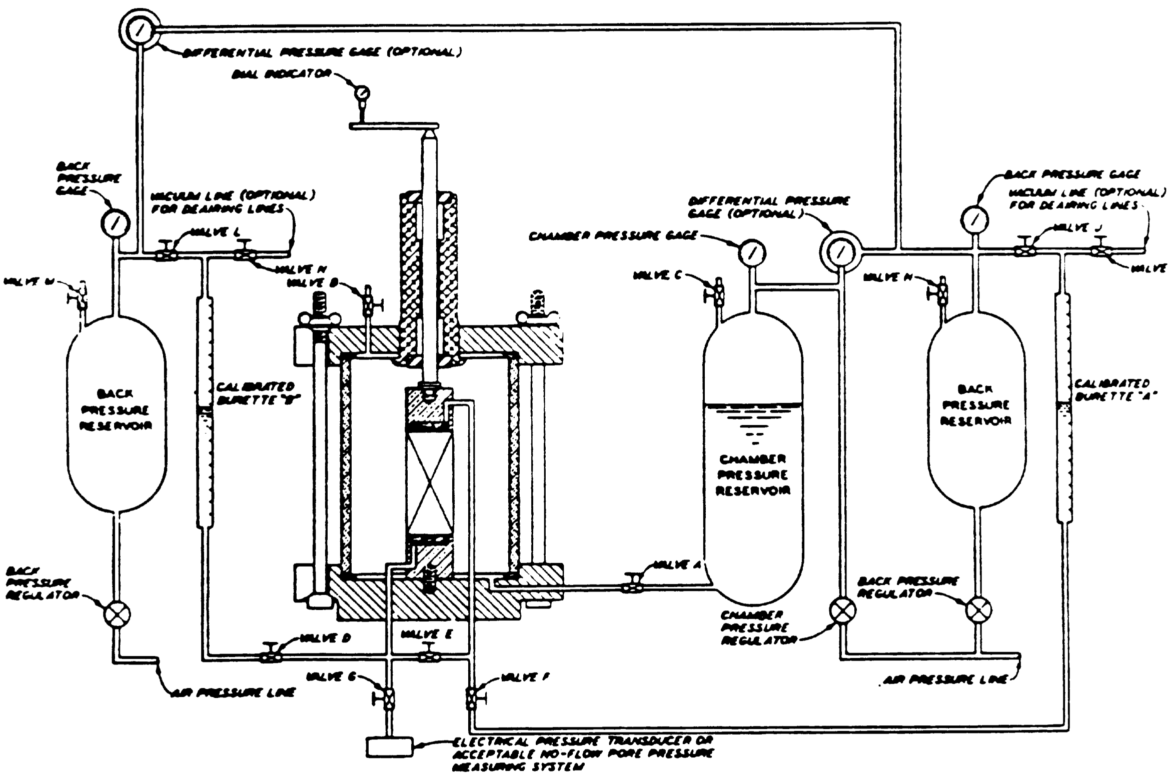

Figure 5.--Schematic diagram of typical triaxial compression apparatus for hydraulic conductivity tests with back pressure.
Source: U.S. Army Corps of Engineers, 1970

6. For ease and uniformity of saturation, as well as to allow volume changes during consolidation to be measured with the burette, specimens should be completely saturated before any appreciable consolidation is permitted; therefore, the difference between the chamber pressure and the back pressure should not exceed 5 psi during the saturation phase. To insure that a specimen is not prestressed during the saturation phase, the back pressure must be applied in small increments, with adequate time between increments to permit equalization of pore water pressure throughout the specimen.

7. With all valves closed, adjust the pressure regulators to a chamber pressure of about 7 psi and a back pressure of about 2 psi. Now open valve A to apply the preset pressure to the chamber fluid and simultaneously open valve F to apply the back pressure through the specimen cap. Immediately open valve G and read and record the pore pressure at the specimen base. When the measured pore pressure becomes essentially constant, close valves F and G and record the burette reading.

8. Using the technique described in Step 3, increase the chamber pressure and the back pressure in increments, maintaining the back pressure at about 5 psi less than the chamber pressure. The size of each increment might be 5, 10, or even 20 psi, depending on the compressibility of the soil specimen and the magnitude of the desired consolidation pressure. Open valve G and measure the pore pressure at the base immediately upon application of each increment of back pressure and observe the pore pressure until it becomes essentially constant. The time required for stabilization of the pore pressure may range from a few minutes to several hours depending on the permeability of the soil. Continue adding increments of chamber pressure and back pressure until, under any increment, the pore pressure reading equals the applied back pressure immediately upon opening valve G.

9. Verify the completeness of saturation by closing valve F and increasing the chamber pressure by about 5 psi. The specimen shall not be considered completely saturated unless the increase in pore pressure immediately equals the increase in chamber pressure.

10. When the specimen is completely saturated, increase the chamber pressure with the drainage valves closed to attain the desired effective consolidation pressure (chamber pressure minus back pressure). At zero elapsed time, open valves E and F.

11. Record time, dial indicator reading, and burette reading at elapsed times of 0, 15, and 30 sec, 1, 2, 4, 8, and 15 min, and 1, 2, 4, and 8 hr, etc. Plot the dial indicator readings and

burette readings on an arithmetic scale versus elapsed time on a log scale. When the consolidation curves indicate that primary consolidation is complete, close valves E and F.

12. Apply a pressure to burette B greater than that in burette A. The difference between the pressures in burettes B and A is equal to the head loss (h); h divided by the height of the specimen after consolidation (L) is the hydraulic gradient. The difference between the two pressures should be kept as small as practicable, consistent with the requirement that the rate of flow be large enough to make accurate measurements of the quantity of flow within a reasonable period of time. Because the difference in the two pressures may be very small in comparison to the pressures at the ends of the specimen, and because the head loss must be maintained constant throughout the test, the difference between the pressures within the burettes must be measured accurately; a differential pressure gage is very useful for this purpose. The difference between the elevations of the water within the burettes should also be considered (1 in. of water = 0.036 psi of pressure).

13. Open valves D and F. Record the burette readings at any zero elapsed time. Make readings of burettes A and B and of temperature at various elapsed times (the interval between successive readings depends upon the permeability of the soil and the dimensions of the specimen). Plot arithmetically the change in readings of both burettes versus time. Continue making readings until the two curves become parallel and straight over a sufficient length of time to determine accurately the rate of flow as indicated by the slope of the curves.

2.8.5 **Calculations:** The hydraulic conductivity can be calculated using Equation (8).

2.9 Pressure-chamber permeameter method:

2.9.1 **Applicability:** This method can be used to determine hydraulic conductivity of a wide range of soils. Undisturbed and disturbed samples can be tested under falling-head conditions using this method. This method is also applicable to both coarse- and fine-grained soils, including remolded, fine-grained materials.

2.9.2 **Apparatus:** The apparatus, shown in Figure 6, consists of

1. Pressure chamber;

2. Standpipe;

3. Specimen cap and base; and

4. Coarse porous plates.

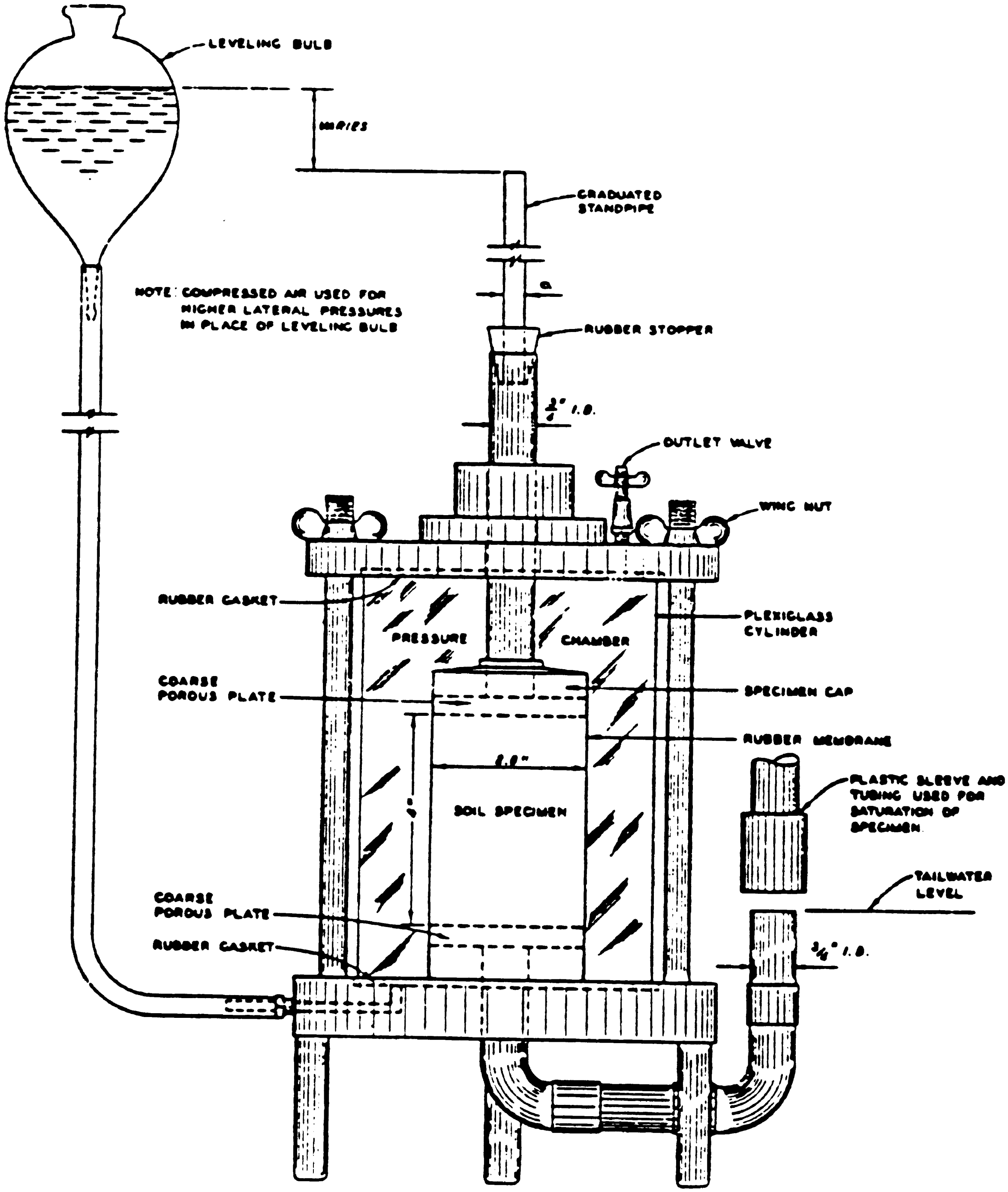

Figure 6.--Pressure chamber for hydraulic conductivity.
Source: U.S. Army Corps of Engineers, 1980.

The apparatus is capable of applying confining pressure to simulate field stress conditions.

2.9.3 **Sample preparation:** The sample preparation of disturbed and undisturbed conditions can be prepared in the chamber and enclosed within the rubber membrane, as discussed in Section 2.8.4.

2.9.4 **Test procedure:**

1. By adjusting the leveling bulb, a confining pressure is applied to the sample such that the stress conditions represent field conditions. For higher confining pressure, compressed air may be used.

2. Allow the sample to consolidate under the applied stress until the end of primary consolidation.

3. Flush water through the sample until no indication of air bubbles is observed. For higher head of water, compressed air may be used.

4. Adjust the head of water to attain a desired hydraulic gradient.

5. Measure and record the head drop in the standpipe along with elapsed time until the plot of logarithm of head versus time is linear for more than three consecutive readings.

2.9.5 **Calculations:** The hydraulic conductivity can be determined using Equation (9).

2.10 Sources of error for laboratory test for hydraulic conductivity: There are numerous potential sources of error in laboratory tests for hydraulic conductivity. Fixed-wall permeameters may have problems with sidewall leakage, causing higher values of hydraulic conductivity. Flexible-membrane permeameters may yield misleadingly low values for hydraulic conductivity when testing with a leachate that causes contraction and shrinkage cracks in the sample because the membrane shrinks with the sample. Table B summarizes some potential errors that can occur. Olsen and Daniel (1981) provide a more detailed explanation of sources of these errors and methods to minimize them. If the hydraulic conductivity does not fall within the expected range for the soil type, as given in Table C, the measurement should be repeated after checking the source of error in Table B.

TABLE B

SUMMARY OF PUBLISHED DATA ON POTENTIAL ERRORS
IN USING DATA FROM
LABORATORY PERMEABILITY TESTS ON SATURATED SOILS

	Source of Error (References)	Measured K Too Low or Too High?
1.	Voids formed in sample preparation (Olsen and Daniel, 1981).	High
2.	Smear zone formed during trimming (Olsen and Daniel, 1981).	Low
3.	Use of distilled water as a permeant (Fireman, 1944; and Wilkinson, 1969).	Low
4.	Air in sample (Johnson, 1954)	Low
5.	Growth of micro-organisms (Allison, 1947).	Low
6.	Use of excessive hydraulic gradient (Schwartzendruber, 1968; and Mitchell and Younger, 1967).	Low or High
7.	Use of temperature other than the test temperature.	Varies
8.	Ignoring volume change due to stress change, with no confining pressure used.	High
9.	Performing laboratory rather than in-situ tests (Olsen and Daniel, 1981).	Usually Low
10.	Impedance caused by the test apparatus, including the resistance of the screen or porous stone used to support the sample.	Low

TABLE C

HYDRAULIC CONDUCTIVITIES ESTIMATED FROM GRAIN-SIZE DESCRIPTIONS
(In Feet Per Day)

Grain-Size Class or Range From Sample Description	Degree of Sorting			Silt Content		
	Poor	Moderate	Well	Slight	Moderate	High
Fine-Grained Materials						
Clay			Less than .001			
Silt, clayey			1 - 4			
Silt, slightly sandy			5			
Silt, moderately sandy			7 - 8			
Silt, very sandy			9 - 11			
Sandy silt			11			
Silty sand			13			
Sands and gravels[(1)]						
Very fine sand	13	20	27	23	19	13
Very fine to fine sand	27	27	-	24	20	13
Very fine to medium sand	36	41-47	-	32	27	21
Very fine to coarse sand	48	-	-	40	31	24
Very fine to very coarse sand	59	-	-	51	40	29
Very fine sand to fine gravel	76	-	-	67	52	38
Very fine sand to medium gravel	99	-	-	80	66	49
Very fine sand to coarse gravel	128	-	-	107	86	64
Fine sand	27	40	53	33	27	20
Fine to medium sand	53	67		48	39	30
Fine to coarse sand	57	65-72	-	53	43	32
Fine to very coarse sand	70	-	-	60	47	35
Fine sand to fine gravel	88	-	-	74	59	44
Fine sand to medium gravel	114	-	-	94	75	57
Fine sand to coarse gravel	145	-	-	107	87	72
Medium sand	67	80	94	64	51	40
Medium to coarse sand	74	94	-	72	57	42
Medium to very coarse sand	84	98-111	-	71	61	49
Medium sand to fine gravel	103	-	-	84	68	52
Medium sand to medium gravel	131	-	-	114	82	66
Medium sand to coarse gravel	164	-	-	134	108	82
Coarse sand	80	107	134	94	74	53
Coarse to very coarse sand	94	134	-	94	75	57
Coarse sand to fine gravel	116	136-156	-	107	88	68
Coarse sand to medium gravel	147	-	-	114	94	74
Coarse sand to coarse gravel	184	-	-	134	100	92

(1) Reduce by 10 percent if grains are subangular.
Source: Lappala (1978).

(continued)

TABLE C (Continued)

Grain-Size Class or Range From Sample Description	Degree of Sorting Poor	Moderate	Well	Silt Content Slight	Moderate	High
<u>Sands and Gravels</u>[1]						
Very coarse sand	107	147	187	114	94	74
Very coarse sand to fine gravel	134	214	-	120	104	87
Very coarse sand to medium gravel	1270	199-227	-	147	123	99
Very coarse sand to coarse gravel	207	-	-	160	132	104
Fine gravel	160	214	267	227	140	107
Fine to medium gravel	201	334	-	201	167	134
Fine to coarse gravel	245	289-334	-	234	189	144
Medium gravel	241	231	401	241	201	160
Medium to coarse gravel	294	468	-	294	243	191
Coarse gravel	334	468	602	334	284	234

(1) Reduce by 10 percent if grains are subangular.
Source: Lappala (1978).

2.11 Leachate conductivity using laboratory methods: Many primary and secondary leachates found at disposal sites may be nonaqueous liquids or aqueous fluids of high ionic strength. These fluids may significantly alter the intrinsic permeability of the porous medium. For example, Anderson and Brown (1981) have demonstrated increases in hydraulic conductivity of compacted clays of as much as two orders of magnitude after the passage of a few pore volumes of a wide range of organic liquids. Consequently, the effects of leachate on these materials should be evaluated by laboratory testing. The preceding laboratory methods can all be used to determine leachate conductivity by using the following guidelines.

2.11.1 **Applicability:** The determination of leachate conductivity may be required for both fine-grained and coarse-grained materials. Leachates may either increase or decrease the hydraulic conductivity. Increases are of concern for compacted clay liners, and decreases are of concern for drain materials. The applicability sections of the preceding methods should be used for selecting an appropriate test for leachate conductivity. The use of the modified compaction method (Section 2.7) for determining leachate conductivity is discussed extensively in EPA Publication SW870 (EPA 1980).

2.11.2 **Leachate used:** A supply of leachate must be obtained that is as close in chemical and physical properties to the anticipated leachate at the disposal site as possible. Methods for obtaining such leachate are beyond the scope of this report. However, recent publications by EPA (1979) and Conway and Malloy (1981) give methodologies for simulating the leaching environment to obtain such leachate. Procedures for deairing the leachate supply are given in Section 2.4. The importance of preventing bacterial growth in leachate tests will depend on the expected conditions at the disposal site. The chemical and physical properties that may result in corrosion, dissolution, or encrustation of laboratory hydraulic conductivity apparatus should be determined prior to conducting a leachate conductivity test. Properties of particular importance are the pH and the vapor pressure of the leachate. Both extremely acidic and basic leachates may corrode materials. In general, apparatus for leachate conductivity tests should be constructed of inert materials, such as acrylic plastic, nylon, or Teflon. Metal parts that might come in contact with the leachate should be avoided. Leachates with high vapor pressures may require special treatment. Closed systems for fluid supply and pressure measurement, such as those in the modified triaxial-cell methods, should be used.

2.11.3 **Safety:** Tests involving the use of leachates should be conducted under a vented hood, and persons conducting the tests should wear appropriate protective clothing and eye protection. Standard laboratory safety procedures such as those as given by Manufacturing Chemists Association (1971) should be followed.

2.11.4 **Procedures:** The determination of leachate conductivity should be conducted immediately following the determination of hydraulic conductivity (Anderson and Brown, 1981). This procedure maintains fluid saturation of the sample, and allows a comparison of the leachate and hydraulic conductivities under the same test conditions. This procedure requires modifications of test operations as described below.

2.11.5 **Apparatus:** In addition to a supply reservoir for water as shown in Figures 3 through 6, a supply reservoir for leachate is required. Changing the inflow to the test cell from water to leachate can be accomplished by providing a three-way valve in the inflow line that is connected to each of the reservoirs.

2.11.6 **Measurements:** Measurements of fluid potential and outflow rates are the same for leachate conductivity and hydraulic conductivity. If the leachate does not alter the intrinsic permeability of the sample, the criteria for the time required to take measurements is the same for leachate conductivity tests as for hydraulic conductivity tests. However, if significant changes occur in the sample by the passage of leachate, measurements should be taken until either the shape of a curve of conductivity versus pore volume can be defined, or until the leachate conductivity exceeds the applicable design value for hydraulic conductivity.

2.11.7 **Calculations:** If the leachate conductivity approaches a constant value, Equations (8) and (9) can be used. If the conductivity changes continuously because of the action of the leachate, the following modifications should be made. For constant-head tests, the conductivity should be determined by continuing a plot of outflow volume versus time for the constant rate part of the test conducted with water. For falling-head tests, the slope of the logarithm of head versus time should be continued.

2.11.7.1 If the slope of either curve continues to change after the flow of leachate begins, the leachate is altering the intrinsic permeability of the sample. The leachate conductivity in this case is not a constant. In this case, values of the slope of the outflow curve to use in Equation (8) or (9) must be taken as the tangent to the appropriate outflow curve at the times of measurement.

3.0 FIELD METHODS

This section discusses methods available for the determination of fluid conductivity under field conditions. As most of these tests will use water as the testing fluid, either natural formation water or water added to a borehole or piezometer, the term hydraulic conductivity will be used for the remainder of this section. However, if field tests are run with leachate or other fluids, the methods are equally applicable.

The location of wells, selection of screened intervals, and the appropriate tests that are to be conducted depend upon the specific site under investigation. The person responsible for such selections should be a qualified hydrogeologist or geotechnical engineer who is experienced in the application of established principles of contaminant hydrogeology and ground water hydraulics. The following are given as general guidelines.

1. The bottom of the screened interval should be below the lowest expected water level.

2. Wells should be screened in the lithologic units that have the highest probability of either receiving contaminants or conveying them down gradient.

3. Wells up gradient and down gradient of sites should be screened in the same lithologic unit.

Standard reference texts on ground water hydraulics and contaminant hydrogeology that should be consulted include: Bear (1972), Bouwer (1978), Freeze and Cherry (1979), Stallman (1971), and Walton (1970).

The success of field methods in determining hydraulic conductivity is often determined by the design, construction, and development of the well or borehole used for the tests. Details of these methods are beyond the scope of this report; however, important considerations are given in Sections 3.1 and 3.2. Detailed discussions of well installation, construction, and development methods are given by Bouwer, pp. 160-180 (1978), Acker (1974), and Johnson (1972).

The methods for field determination of hydraulic conductivity are restricted to well or piezometer type tests applicable below existing water tables. Determinations of travel times of leachate and dissolved solutes above the water table usually require the application of unsaturated flow theory and methods which are beyond the scope of this report.

3.1 Well-construction considerations: The purpose of using properly constructed wells for hydraulic conductivity testing is to assure that test results reflect conditions in the materials being tested, rather than conditions caused by well construction. In all cases, diagrams showing all details of the actual well or borehole constructed for the test should be made. Chapter 3 of the U.S. EPA, RCRA Ground Water Monitoring Technical Enforcement Guidance Document (TEGD) should be consulted.

3.1.1 **Well installation methods:** Well installation methods are listed below in order of preference for ground water testing and monitoring. The order was determined by the need to minimize side-wall plugging by drilling fluids and to maximize the accurate detection of saturated zones. This order should be used as a guide, combined with the judgment of an experienced hydrogeologist in selecting a drilling method. The combined uses of wells for hydraulic conductivity testing, water-level monitoring, and water-quality sampling for organic contaminants were considered in arriving at the ranking.

1. Hollow-stem auger;
2. Cable tool;
3. Air rotary;
4. Rotary drilling with non-organic drilling fluids;
5. Air foam rotary; and
6. Rotary with organic-based drilling fluids.

Although the hollow stem-auger method is usually preferred for the installation of most shallow wells (less than 100 feet), care must be taken if the tested zone is very fine. Smearing of the borehole walls by drilling action can effectively seal off the borehole from the adjacent formation. Scarification can be used to remedy this.

3.1.2 **Wells requiring well screens:** Well screens placed opposite the interval to be tested should be constructed of materials that are compatible with the fluids to be encountered. Generally an inert plastic such as PVC is preferred for ground water contamination studies. The screen slot size should be determined to minimize the inflow of fine-grained material to the well during development and testing. Bouwer (1978) and Johnson (1972) give a summary of guidelines for sizing well screens.

3.1.2.1 The annulus between the well screen and the borehole should be filled with an artificial gravel pack or sand filter. Guidelines for sizing these materials are given by Johnson (1972). For very coarse materials, it may be acceptable to allow the materials from the tested zone to collapse around the screen forming a natural gravel pack.

3.1.2.2 The screened interval should be isolated from overlying and underlying zones by materials of low hydraulic conductivity. Generally, a short bentonite plug is placed on top of the material surrounding the screen, and cement grout is placed in the borehole to the next higher screened interval (in the case of multiple screen wells), or to the land surface for single screen wells.

3.1.2.3 Although considerations for sampling may dictate minimum casing and screen diameters, the recommended guideline is that wells to be tested by pumping, bailing, or injection in coarse-grained materials should be at least 4-inches inside diameter. Wells to be used for testing materials of low hydraulic conductivity by sudden removal or injection of a known volume of fluid should be constructed with as small a casing diameter as possible to maximize measurement resolution of fluid level changes. Casing sizes of 1.25 to 1.50 inches usually allow this resolution while enabling the efficient sudden withdrawal of water for these tests.

3.1.3 **Wells not requiring well screens:** If the zone to be tested is sufficiently indurated that a well screen and casing are not required to prevent caving in, it is preferable to use a borehole open to the zone to be tested. These materials generally are those having low to extremely low hydraulic conductivities. Consolidated rocks having high conductivity because of the presence of fractures and solution openings may also be completed without the use of a screen and gravel pack. Uncased wells may penetrate several zones for which hydraulic conductivity tests are to be run. In these cases, the zones of interest can be isolated by the use of inflatable packers.

3.2 Well development: For wells that are constructed with well screens and gravel packs, and for all wells in which drilling fluids have been used that may have penetrated the materials to be tested, adequate development of the well is required to remove these fluids and to remove the fine-grained materials from the zone around the well screen. Development is carried out by methods such as intermittent pumping, jetting with water, surging, and bailing. Adequate development is required to assure maximum communication between fluids in the borehole and the zone to be tested. Results from tests run in wells that are inadequately developed will include an error caused by loss of fluid potential across the undeveloped zone, and computed hydraulic conductivities will be lower than the actual value. Bouwer (1978) and Johnson (1975) give further details on well development including methods to determine when adequate development has occurred. The U.S. EPA TEGD should also be consulted.

3.3 Data interpretation and test selection considerations: Hydraulic conductivity may be determined in wells that are either cased or uncased as described in Section 3.1. The tests all involve disturbing the existing fluid potential in the tested zone by withdrawal from or injection of fluid into a well, either as a slug over an extremely short period of time, or by continuous withdrawal or injection of fluid. The hydraulic conductivity is determined by measuring the response of the water level or pressure in the well as a function of time since the start of the test. Many excellent references are available that give the derivation and use of the methods that are outlined below, including Bouwer (1978), Walton (1969), and Lohman (1972).

3.3.1 The selection of a particular test method and data analysis technique requires the consideration of the purposes of the test, and the geologic framework in which the test is to be run. Knowledge of the stratigraphic relationships of the zone to be tested and both overlying and underlying materials should always be used to select appropriate test design and data interpretation methods.

3.3.2 The equations given for all computational methods given here and in the above references are based on idealized models comprising layers of materials of different hydraulic conductivities. The water-level response caused by disturbing the system by the addition or removal of water can be similar for quite different systems. For example, the response of a water-table aquifer and a leaky, confined aquifer to pumping can be very similar. Consequently, it is not considered acceptable practice to obtain data from a hydraulic conductivity test and interpret the type of hydraulic system present without supporting geologic evidence.

3.3.3 The primary use of hydraulic conductivity data from tests described subsequently will usually be to aid in siting monitoring wells for facility design as well as for compliance with Subpart F of Part 264. As such, the methods are abbreviated to provide guidance in determining hydraulic conductivity only. Additional analyses that may be possible with some methods to define the storage properties of the aquifer are not included. The U.S. EPA TEGD has an expanded discussion on the relationship between K tests and siting design (Chapter 1) and should be consulted.

3.3.4 The well test methods are discussed under the following two categories: 1) methods applicable to coarse-grained materials and tight to extremely tight materials under confined conditions; and 2) methods applicable to unconfined materials of moderate permeability. The single well tests integrate the effects of heterogeneity and anisotropy. The effects of boundaries such as streams or less permeable materials usually are not detectable with these methods because of the small portion of the geologic unit that is tested.

3.4 Single well tests: The tests for determining hydraulic conductivity with a single well are discussed below based on methods for confined and unconfined conditions. The methods are usually called slug tests because the test involves removing a slug of water instantaneously from a well and measuring the recovery of water in the well. The method was first developed by Hvorslev (1951), whose analysis did not consider the effect of fluid stored in the well. Cooper and others (1967) developed a method that considers well bore storage. However, their method only applied to wells that are open to the entire zone to be tested and that tap confined aquifers. Because of the rapid water-level response in coarse materials, the tests are generally limited to zones with a transmissivity of less than about 70 cm^2/sec (Lohman, 1972). The method has been extended to allow testing of extremely tight formations by Bredehoeft and Papadopulos (1980). Bouwer and Rice (1976) developed a method for analyzing slug tests for unconfined aquifers.

3.4.1 **Method for moderately permeable formations under confined conditions:**

3.4.1.1 Applicability: This method is applicable for testing zones to which the entire zone is open to the well screen or open borehole. The method usually is used in materials of moderate hydraulic conductivity which allow measurement of water-level response over a period of a hour to a few days. More permeable zones can be tested with rapid response water-level recording equipment. The method assumes that the tested zone is uniform in all radial directions from the test well. Figure 7 illustrates the test geometry for this method.

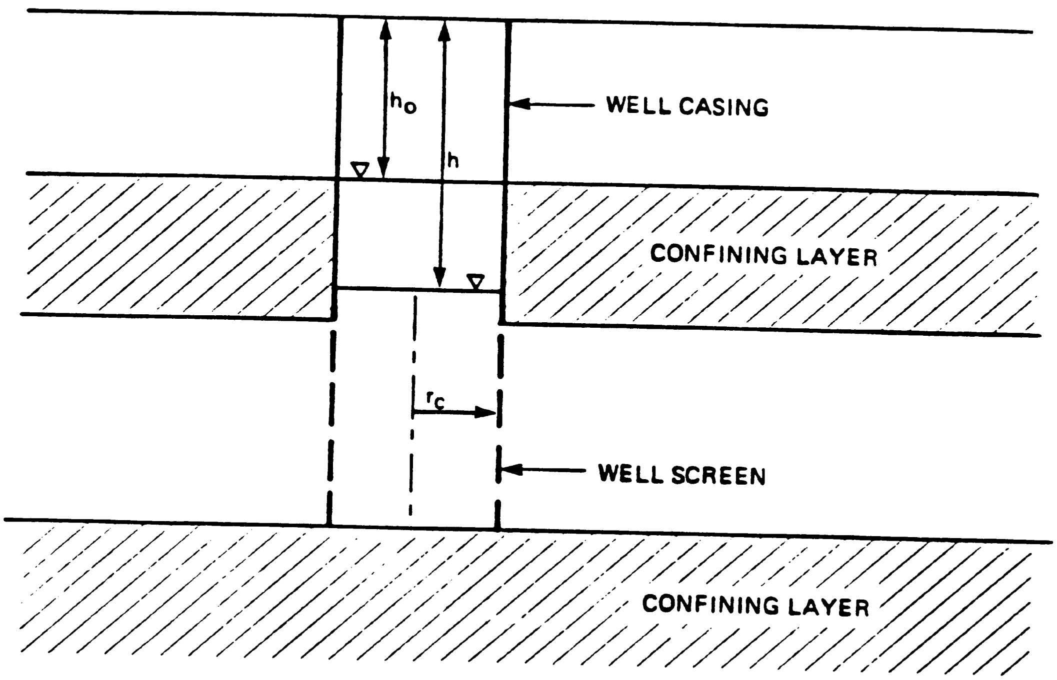

Figure 7.--Geometry and variable definition for slug tests in confined aquifers.

3.4.1.2 Procedures: The slug test is run by utilizing some method of removing or adding a known volume of water from the well bore in a very short time period and measuring the recovery of the water level in the well. The procedures are the same for both unconfined and confined aquifers. Water is most effectively removed by using a bailer that has been allowed to fill and stand in the well for a sufficiently long period of time so that any water-level disturbance caused by the insertion of the bailer will have reached equilibrium. In permeable materials, this recovery time may be as little as a few minutes. An alternate method of effecting a sudden change in water level is the withdrawal of a weighted float. The volume of water displaced can be computed using the known submersed volume of the float and Archimedes' principle (Lohman, 1972).

Water-level changes are recorded using either a pressure transducer and a strip chart recorder, a weighted steel tape, or an electric water-level probe. For testing permeable materials that approach or exceed 70 cm^2/sec, a rapid-response transducer/recorder system is usually used because essentially full recovery may occur in a few minutes. Because the rate of water-level response decays with time, water-level or pressure changes should be taken at increments that are approximately equally spaced in the logarithm of the time since fluid withdrawal. The test should be continued until the water level in the well has recovered to at least 85 percent of the initial pre-test value.

3.4.1.3 Calculations: Calculations for determining hydraulic conductivity for moderately permeable formations under confined conditions can be made using the following procedure:

1. Determine the transmissivity of the tested zone by plotting the ratio h/h_o on an arithmetic scale against time since removal of water (t) on a logarithmic scale. The observed fluid potential in the well during the test as measured by water level or pressure is h, and the fluid potential before the instant of fluid withdrawal is h_o. The data plot is superimposed on type curves, such as those given by Lohman (1972), Plate 2, or plotted from Appendix A, with the h/h_o and time axes coincident. The data plot is moved horizontally until the data fits one of the type curves. A value of time on the data plot corresponding to a dimensionless time (β) on the type curve plot is chosen, and the transmissivity is computed from the following:

$$T = \frac{\beta r_c^2}{t} \tag{10}$$

where:

r_C is the radius of the casing (Lohman, p. 29 (1972)).

The type curves plotted using data in Appendix A are not to be confused with those commonly referred to as "Theis Curves" which are used for pumping tests in confined aquifers (Lohman, 1972). The type curve method is a general technique of determining aquifer parameters when the solution to the descriptive flow equation involves more than one unknown parameter. Although both the storage coefficient and transmissivity of the tested interval can be determined with the type curve method for slug tests, determination of storage coefficients is beyond the scope of this report. See Section 3.4.1.4 for further discussion of the storage coefficient.

If the data in Appendix A are used, a type curve for each value of α is prepared by plotting $F(\alpha,\beta)$ on the arithmetic scale and dimensionless time (β) on the logarithmic scale of semi-log paper.

2. Determine the hydraulic conductivity by dividing the transmissivity (T) calculated above by the thickness of the tested zone.

3.4.1.4 Sources of error: The errors that can arise in conducting slug tests can be of three types: those resulting from the well or borehole construction; measurement errors; and data analysis error.

Well construction and development errors: This method assumes that the entire thickness of the zone of interest is open to the well screen or boreholes and that flow is principally radial. If this is not the case, the computed hydraulic conductivity may be too high. If the well is not properly developed, the computed conductivity will be too low.

Measurement errors: Determining or recording the fluid level in the borehole and the time of measurement incorrectly can cause measurement errors. Water levels should be measured to an accuracy of at least 1 percent of the initial water-level change. For moderately permeable materials, time should be measured with an accuracy of fractions of minutes, and, for more permeable materials, the time should be measured in terms of seconds or fractions of seconds. The latter may require the use of a rapid-response pressure transducer and recorder system.

Data analysis errors: The type curve procedure requires matching the data to one of a family of type curves, described by the parameter , which is a measure of the storage in the well bore and aquifer. Papadopulos and others (1973) show that an error of two orders of magnitude in the selection of would result in an error of less than 30 percent in the value of transmissivity determined. Assuming no error in determining the thickness of the zone tested, this is equivalent to a 30 percent error in the hydraulic conductivity.

3.4.2 **Methods for extremely tight formations under confined conditions:**

3.4.2.1 Applicability: This test is applicable to materials that have low to extremely low permeability such as silts, clays, shales, and indurated lithologic units. The test has been used to determine hydraulic conductivities of shales of as low as 10^{-10} cm/sec.

3.4.2.2 Procedures: The test described by Bredehoeft and Papadopulos (1980) and modified by Neuzil (1982) is conducted by suddenly pressurizing a packed-off zone in a portion of a borehole or well. The test is conducted using a system such as shown in Figure 8. The system is filled with water to a level assumed to be equal to the prevailing water level. (This step is required if sufficiently large times have not elapsed since the drilling of the well to allow full recovery of water levels.) A pressure transducer and recorder are used to monitor pressure changes in the system for a period prior to the test to obtain pressure trends preceding the test. The system is pressurized by addition of a known volume of water with a high-pressure pump. The valve is shut and the pressure decay is monitored. Neuzil's modification uses two packers with a pressure transducer below the bottom packer to measure the pressure change in the cavity and one between the two packers to monitor any pressure change caused by leakage around the bottom packer.

3.4.2.3 Calculations: The modified slug test as developed by Bredehoeft and Papadopulos (1980) considered compressive storage of water in the borehole. These authors considered that the volume of the packed-off borehole did not change during the test and that all compressive storage resulted in compression of water under the pressure pulse. Neuzil (1980) demonstrated that under some test conditions this is not a valid assumption. The computational from either Lohman, Plate 2 (1972) or plotted from data given in Appendix A as described in Section 3.4.1.3. The values of time (t) and dimensionless time (β) are determined in the same manner as for the conventional tests. If compression of water only is considered, transmissivity is computed by replacing r_c by the quantity ($V_W C_W \rho_g/\pi$) in Equation 10:

$$T = \frac{\beta (V_W \; C_W \; \rho_g/\pi)^2}{t} \qquad (10)$$

where:

V_W is the volume of water in the packed-off cavity, L^3;

C_W is the compressibility of water, LT^2M^{-1}

ρ is the density of water, ML^{-3}; and

g is the acceleration of gravity, LT^{-2}.

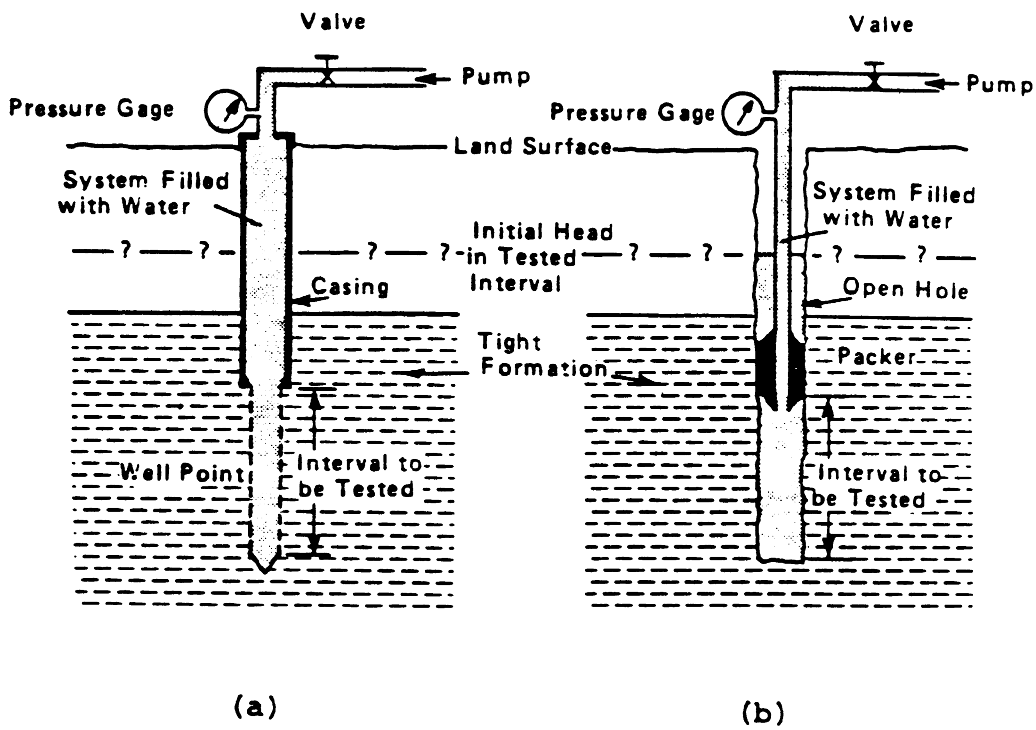

Figure 8.--Schematic diagram for pressurized slug test method in unconsolidated (a) and consolidated (b) materials. Source: Papadopulos and Bredehoeft, 1980.

If the compressive storage is altered by changing the volume of the packed-off cavity (V), then the combined compressibility of the water and the expansion of the cavity (C_o) is used. C_o is computed by measuring the volume of water injected during pressurization (ΔV) and the pressure change (ΔP) for the pressurization:

$$C_o = \frac{\Delta V}{V \Delta P} \qquad (11)$$

(Neuzil, p. 440 (1982)). Use of C_O requires an accurate method of metering the volume of water injected and the volume of the cavity.

3.4.2.4 Sources of error: The types of errors in this method are the same as those for the conventional slug test. Errors may also arise by inaccurate determination of the cavity volume and volume of water injected. An additional assumption that is required for this method is that the hydraulic properties of the interval tested remain constant throughout the test. This assumption can best be satisfied by limiting the initial pressure change to a value only sufficiently large enough to be measured (Bredehoeft and Papadopulos, 1980).

3.4.3 Methods for moderately permeable materials under unconfined conditions:

3.4.3.1 Applicability: This method is applicable to wells that fully or partially penetrate the interval of interest (Figure 9). The hydraulic conductivity determined will be principally the value in the horizontal direction (Bouwer and Rice, 1976).

3.4.3.2 Procedures: A general method for testing cased wells that partly or fully penetrate aquifers that have a water table as the upper boundary of the zone to be tested was developed by Bouwer and Rice (1976). The geometry and dimensions that are required to be known for the method are shown in Figure 9. The test is accomplished by effecting a sudden change in fluid potential in the well by withdrawal of either a bailer or submerged float as discussed in Section 3.4.1.2. Water-level changes can be monitored with either a pressure transducer and recorder, a wetted steel tape, or an electric water-level sounder. For highly permeable formations, a rapid-response transducer and recorder system is required. The resolution of the transducer should be about 0.01 m.

3.4.3.3 Calculations: The hydraulic conductivity is calculated using the following equation from Bouwer and Rice (1976), in the notation of this report:

$$K = \frac{r_c^2 \ln \bar{R}/r_w}{2 L_e t} \ln \frac{Y_o}{Y} \qquad (12)$$

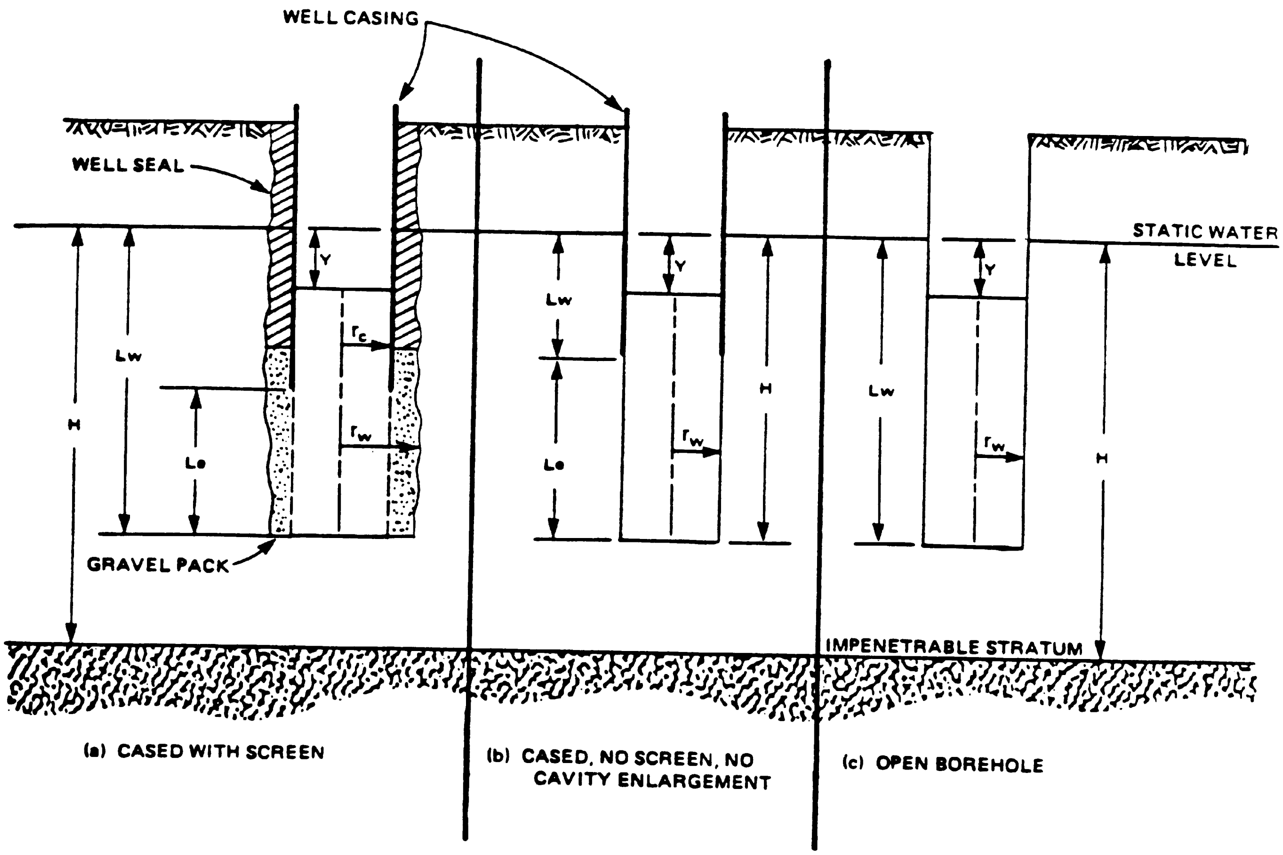

Figure 9.--Variable definitions for slug tests in unconfined materials. Cased wells are open at the bottom.

where r_c, r_w, L_e, t, Y, and K have been previously defined or are defined in Figure 8a. Y_o is the value of Y immediately after withdrawal of the slug of water. The term $\bar{R}$ is an effective radius for wells that do not fully penetrate the aquifer that is computed using the following equation given by Bouwer and Rice (1976):

$$\ln \frac{\bar{R}}{r_w} = \left\{ \frac{1.1}{\ln (L_w/r_w)} + \frac{A + B \ln[(H_o - L_w)/r_w]}{(L_e/r_w)} \right\}^{-1} \qquad (13)$$

If the quantity $(H_o\text{-}L_w)/r_w)$ is larger than 6, a value of 6 should be used.

For wells that completely penetrate the aquifer, the following equation is used:

$$\ln \frac{\bar{R}}{r_w} = \left\{ \frac{1.1}{\ln (L_w/r_w)} + \frac{C}{L_e/r_w} \right\}^{-1} \qquad (14)$$

(Bouwer, 1976). The values of the constants A, B, and C are given by Figure 10 (Bouwer and Rice, 1976).

For both cases, straight-line portions of plots of the logarithm of Y or Y_o/Y against time should be used to determine the slope, $(\ln Y_o/Y)/t$.

Additional methods for tests under unconfined conditions are summarized by Bower (1976) on pages 117-122. These methods are modifications of the cased-well method described above that apply either to an uncased borehole or to a well or piezometer in which the diameter of the casing and the borehole are the same (Figures 9b and 9c.)

3.4.3.4 Sources of error: The method assumes that flow of water from above is negligible. If this assumption cannot be met, the conductivities may be in error. Sufficient flow from the unsaturated zone by drainage would result in a high conductivity value. Errors caused by measuring water levels and recording time are similar to those discussed in Sections 3.4.1.4 and 3.4.2.4.

3.5 Multiple well tests: Hydraulic conductivity can also be determined by conventional pumping tests in which water is continuously withdrawn or injected using one well, and the water-level response is measured over time in or near more observation wells. The observation wells must be screened in the same strata as the injection or pumping well. These methods generally test larger portions of aquifers than the single well tests discussed in Section 3.4. For some circumstances these tests may be appropriate in obtaining data to use in satisfying requirements of Part 264 Subpart F. However, the large possibility for non-uniqueness in interpretation, problems involved in pumping contaminated fluids, and the expense of conducting such tests generally

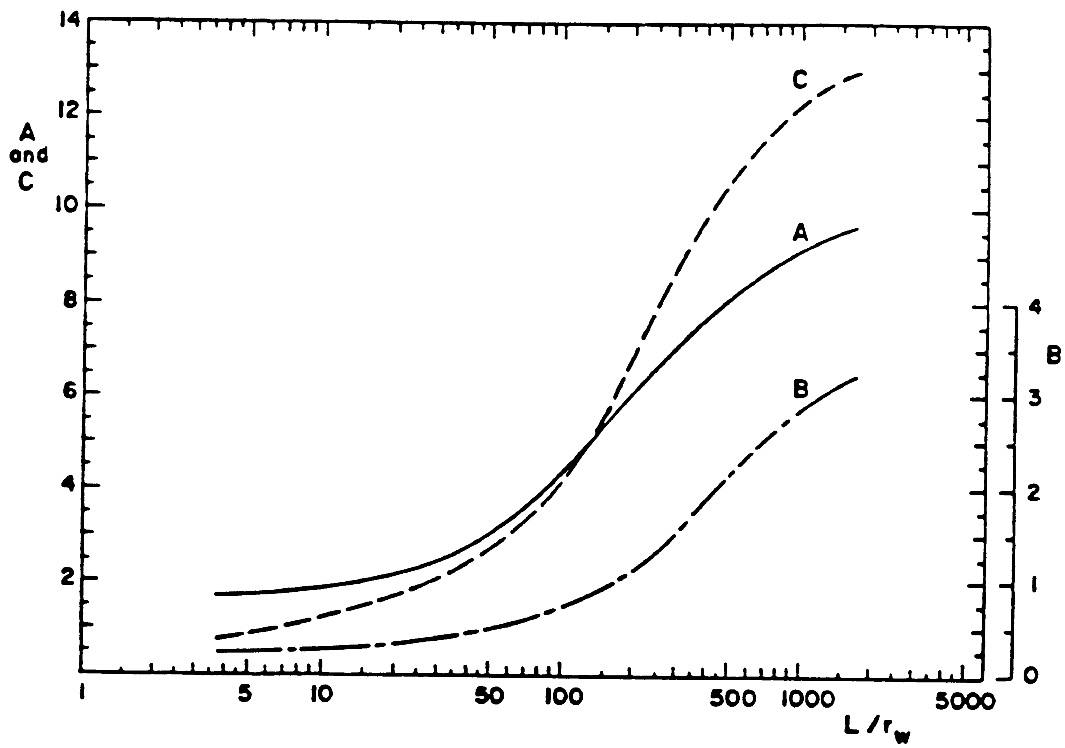

Figure 10. --Curves defining coefficients A, B, and C in equations 13 and 14 as a function of the ratio L/rw. Source: Bower and Rice, 1976.

preclude their use in problems of contaminant hydrogeology. The following references give excellent discussions of the design and interpretation of these tests: Lohman (1972), Stallman (1971), and Walton (1970).

3.6 Estimates of hydraulic conductivity for coarse-grained materials: The characterization of ground water flow systems to satisfy the intent of Part 264 Subpart F is preferably done with flow nets based on borehole measurements rather than relying on interpolation from grain-size analyses.

An empirical approach that has been used by the U.S. Geological Survey (Lappala, 1978) in several studies relates conductivity determined by aquifer testing to grain-size, degree of sorting and silt content. Table C provides the estimates of hydraulic conductivity.

Although estimates of K from analysis of grain-size and degree of sorting do provide a rough check on test values of K, repeated slug tests provide a better check on the accuracy of results.

3.7 Consolidation tests: As originally defined by Terzagi (Terzaghi and Peck, 1967) the coefficient of consolidation (C_v) of a saturated, compressible, porous medium is related to the hydraulic conductivity by:

$$C_v = \frac{K}{\rho g \alpha} \qquad (15)$$

where:

K is the hydraulic conductivity, LT^{-};

ρ is the fluid density, ML^{-3};

g is the gravitational constant, LT^{-2}; and

α is the soil's compressibility, $LM^{-1}T^{2}$.

The compressibility can be determined in the laboratory with several types of consolidometers, and is a function of the applied stress and the previous loading history. Lambe (1951) describes the testing procedure.

3.7.1 The **transfer value** of results from this testing procedure is influenced by the extent to which the laboratory loading simulates field conditions and by the consolidation rate. The laboratory loadings will probably be less than the stress that remolded clay liner will experience; therefore, the use of an already remolded sample in the consolidometer will probably produce no measurable results. This suggests that the test is of little utility in determining the hydraulic conductivity of remolded or compacted, fine-grained soils. Second, the consolidation rate determines the length of the testing period. For granular soils, this rate is fairly rapid. For fine-grained soils, the rate may be sufficiently slow that the previously described methods, which give faster results, will be preferable. Cohesive soils (clays) must be trimmed from undisturbed samples to fit the mold, while cohesionless sands can be tested using disturbed, repacked samples (Freeze and Cherry, 1979).

3.7.2 In general, EPA believes that consolidation tests can provide useful information for some situations, but prefers the previously described methods because they are direct measurements of hydraulic conductivity. Hydraulic conductivity values determined using consolidation tests are **not to be used** in permit applications.

3.8 Fractured media: Determining the hydraulic properties of fractured media is always a difficult process. Unlike the case with porous media, Darcy's Law is not strictly applicable to flow through fractures, although it often can be applied empirically to large bodies of fractured rock that incorporate many fractures. Describing local flow conditions in fractured rock often poses considerable difficulty. Sowers (1981) discusses determinations of hydraulic conductivity of rock. This reference should be consulted for guidance in analyzing flow through fractured media.

3.8.1 Fine-grained sediments, such as glacial tills, are commonly fractured in both saturated and unsaturated settings. These fractures may be sufficiently interconnected to have a significant influence on ground water flow, or they may be of very limited connection and be of little practical significance.

3.8.2 Frequently, a laboratory test of a small sample of clay will determine hydraulic conductivity to be on the order of 10^{-8} cm/sec. A piezometer test of the same geologic unit over an interval containing fractures may determine a hydraulic conductivity on the order of perhaps 10-5 or 10-6 cm/sec. To assess the extent of fracture interconnection, and hence the overall hydraulic conductivity of the unit, several procedures can be used. Closely spaced piezometers can be installed; one can be used as an observation well while water is added to or withdrawn from the other. Alternately, a tracer might be added to one piezometer, and the second could be monitored. These and other techniques are discussed by Sowers (1981).

3.8.3 For situations that may involve flow through fractured media, it is important to note in permit applications that an apparent hydraulic conductivity determined by tests on wells that intersect a small number of fractures may be several orders others of magnitude lower or higher than the value required to describe flow through parts of the ground water system that involve different fractures and different stress conditions from those used during the test.

4.0 CONCLUSION

4.1 By following laboratory and field methods discussed or referenced in this report, the user should be able to determine the fluid conductivity of materials used for liners, caps, and drains at waste-disposal facilities, as well as materials composing the local ground water flow system. If fluid-conductivity tests are conducted and interpreted properly, the results obtained should provide the level of information necessary to satisfy applicable requirements under Part 264.

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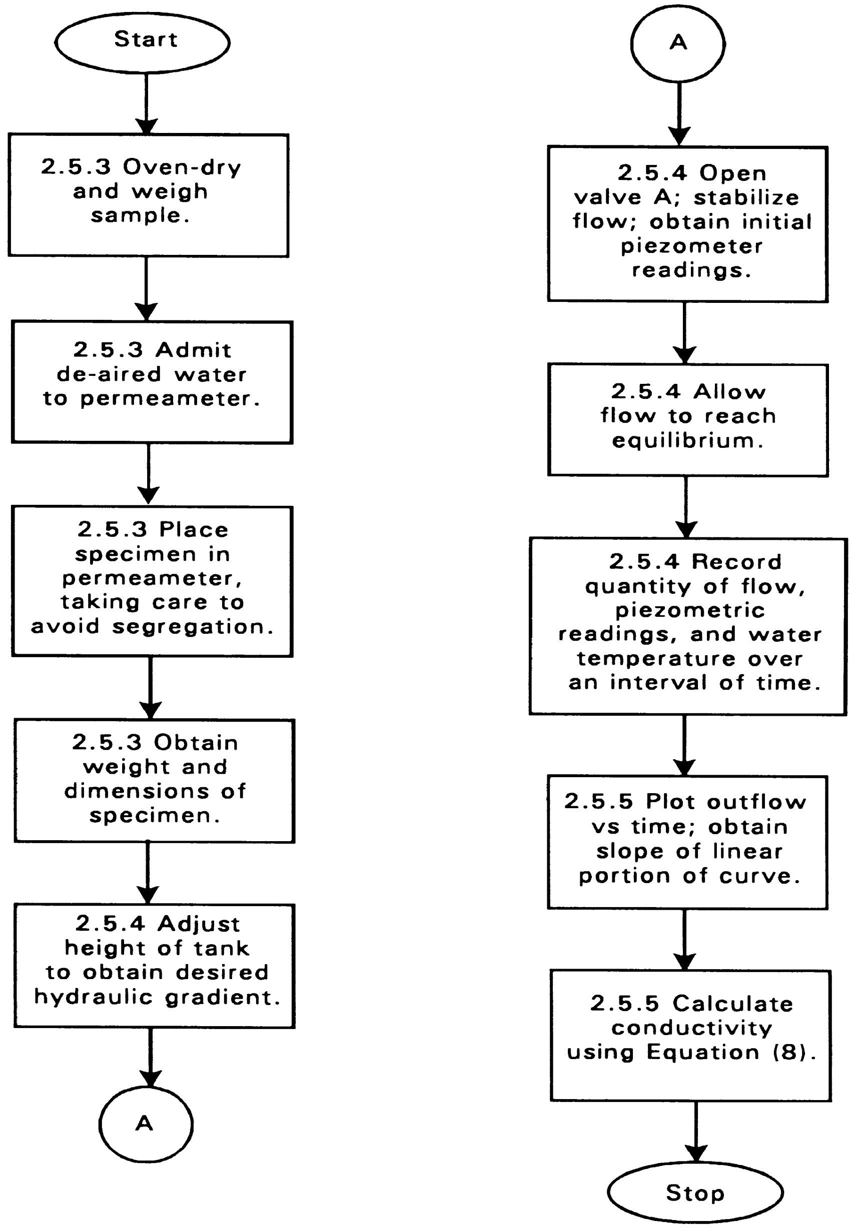
METHOD 9100
HYDRAULIC CONDUCTIVITY OF SOIL SAMPLES:
CONSTANT-HEAD TEST WITH CONVENTIONAL PERMEAMETER



METHOD 9100
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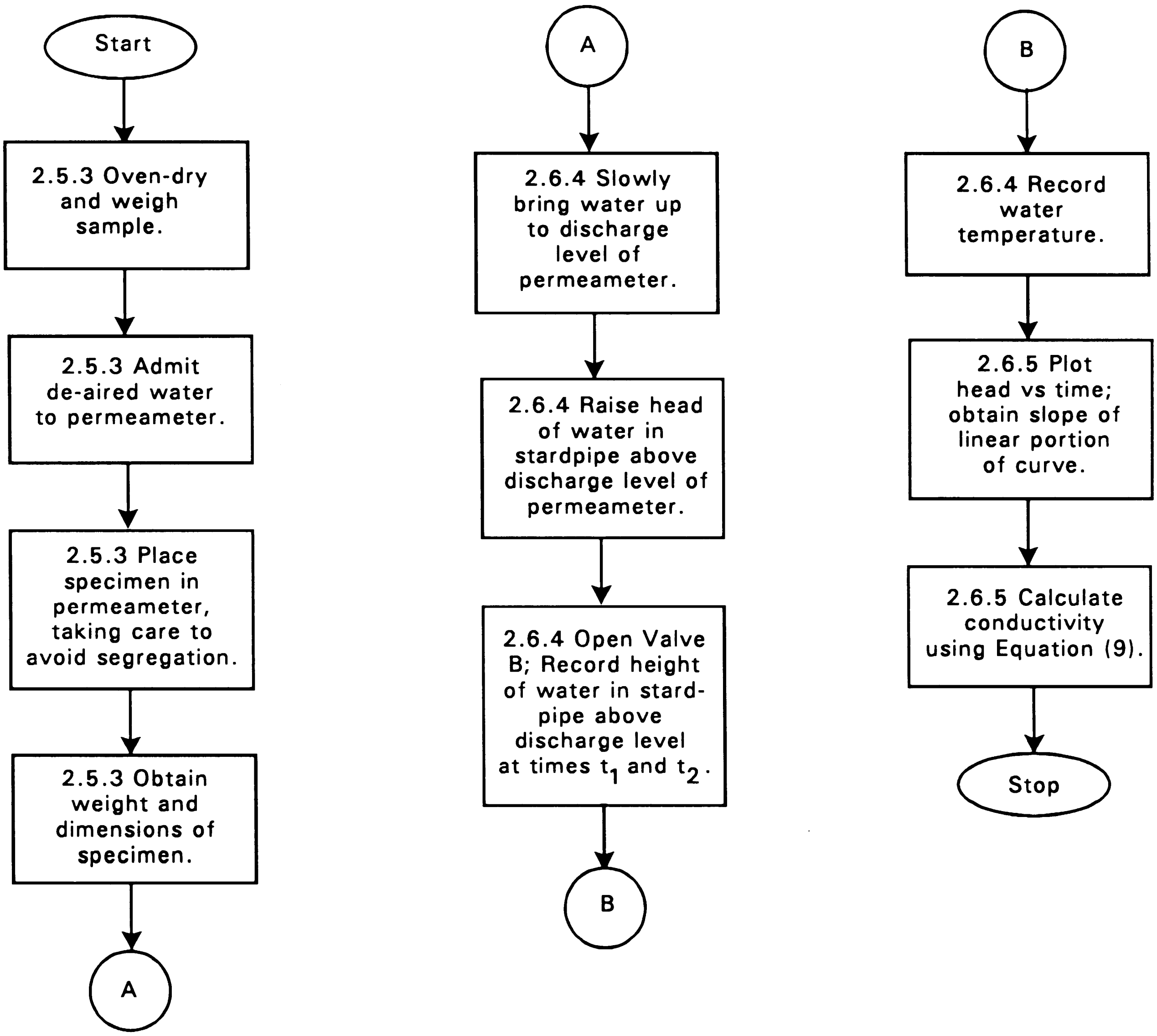
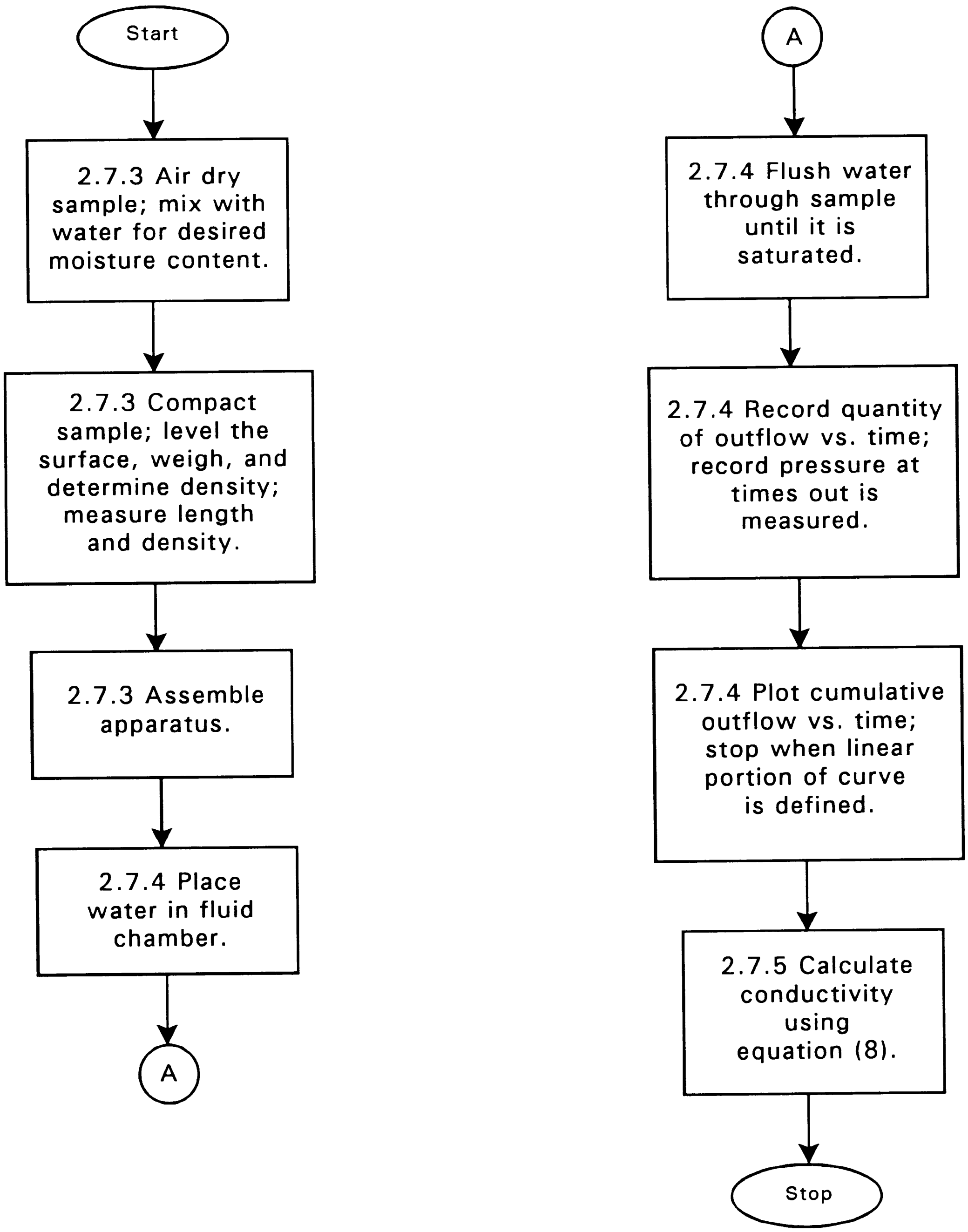
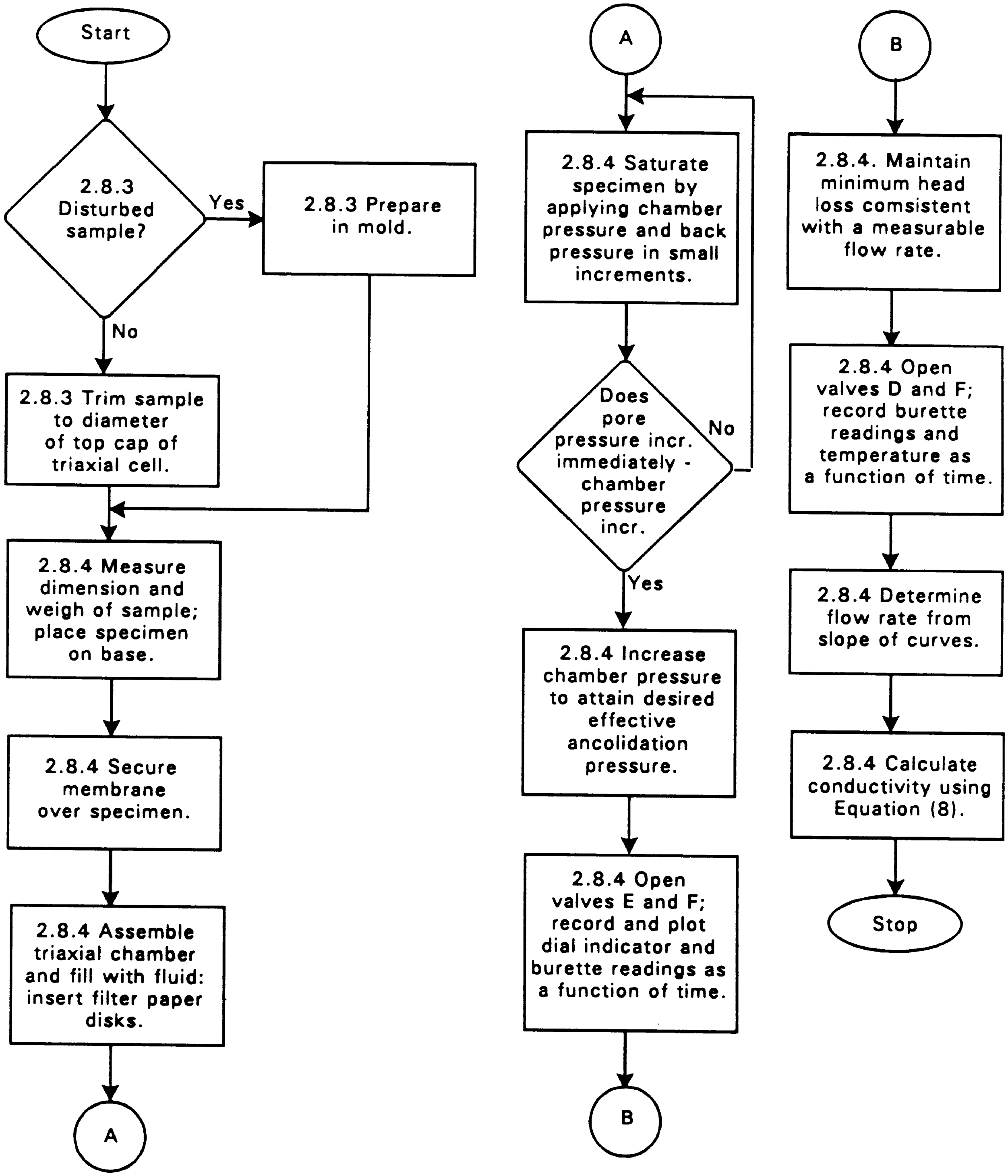
FALLING-HEAD TEST WITH CONVENTIONAL PERMEAMETER



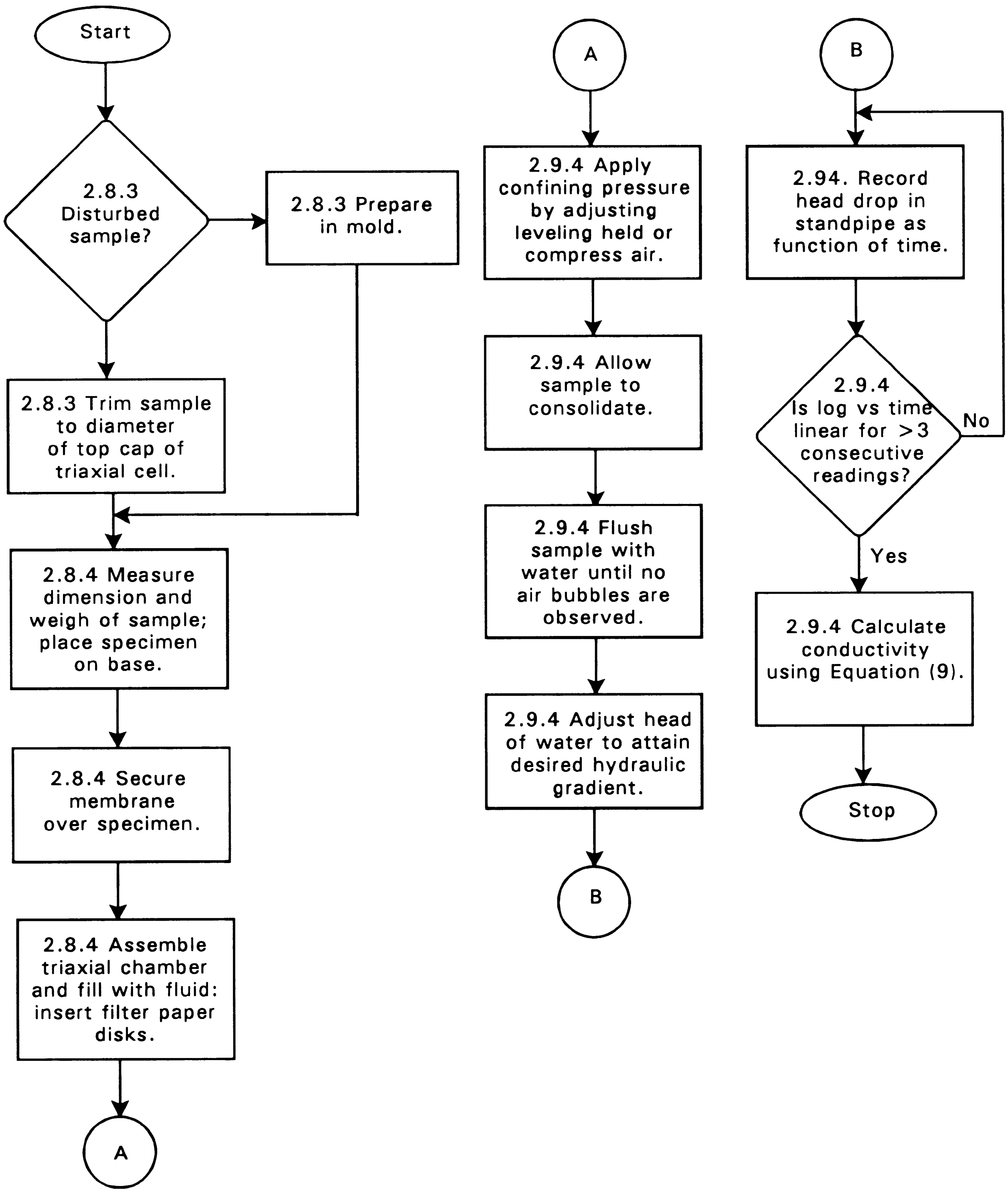
METHOD 9100
HYDRAULIC CONDUCTIVITY OF SOIL SAMPLES:
MODIFIED COMPACTION PARAMETER METHOD

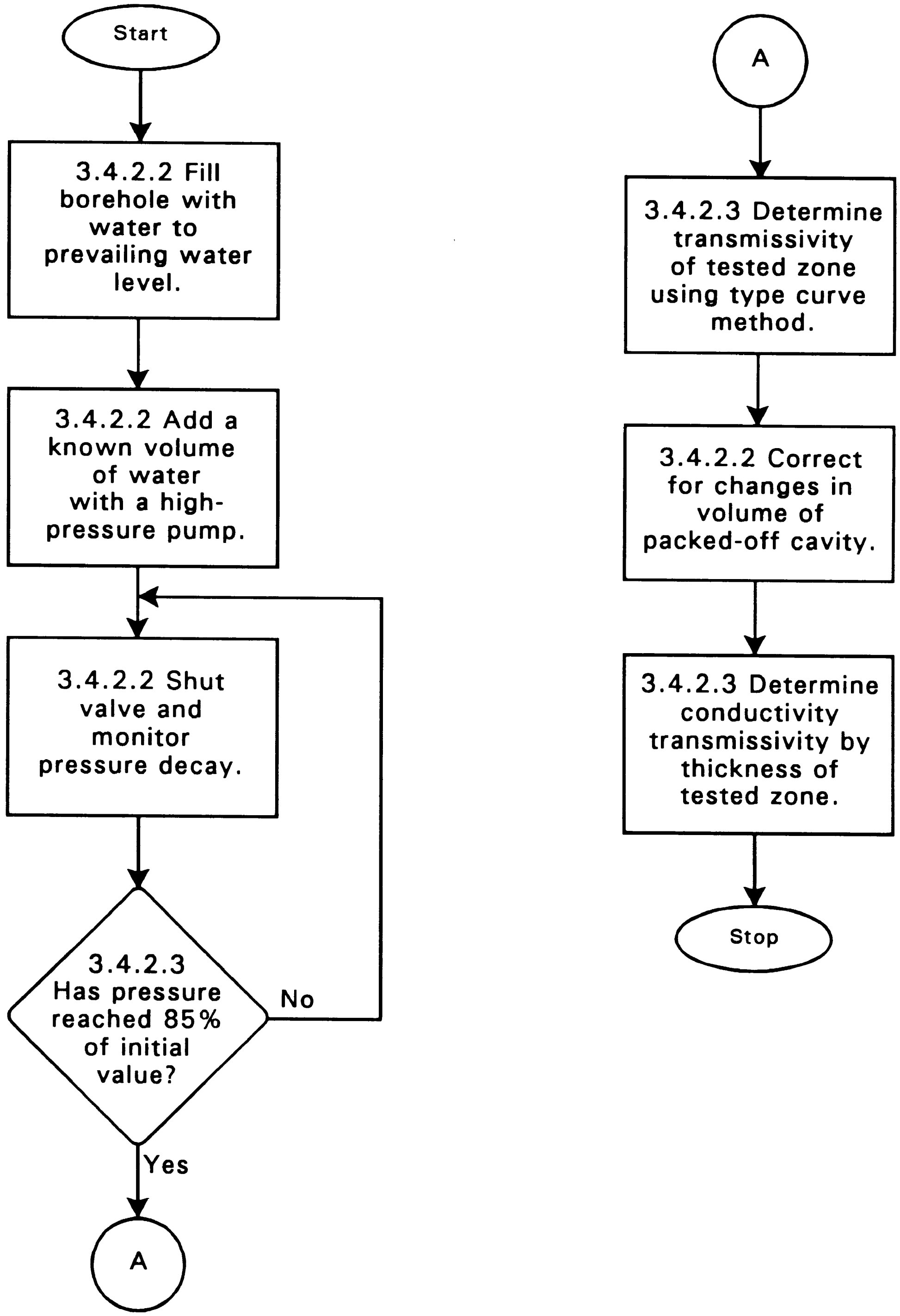


METHOD 9100
HYDRAULIC CONDUCTIVITY OF SOIL SAMPLES: TRIAXIAL CELL METHOD WITH BACK PRESSURE

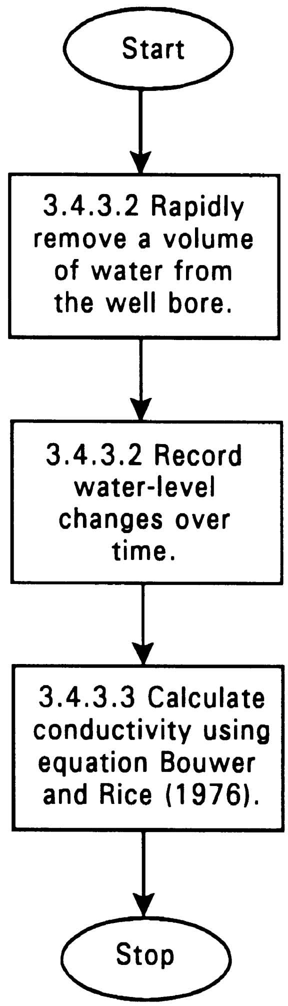


METHOD 9100
HYDRAULIC CONDUCTIVITY OF SOIL SAMPLES:
PRESSURE-CHAMBER PARAMETER METHOD




Start
3.4.2.2 Fill borehole with water to prevailing water level.
3.4.2.2 Add a known volume of water with a high-pressure pump.
3.4.2.2 Shut valve and monitor pressure decay.
3.4.2.3 Has pressure reached 85% of initial value?
No
Yes
A
A
3.4.2.3 Determine transmissivity of tested zone using type curve method.
3.4.2.2 Correct for changes in volume of packed-off cavity.
3.4.2.3 Determine conductivity transmissivity by thickness of tested zone.
Stop

METHOD 9100
HYDRAULIC CONDUCTIVITY OF SOIL SAMPLES:
FIELD METHOD FOR MODERATELY PERMEABLE MATERIALS UNDER UNCONFINED CONDITIONS



METHOD 9100
HYDRAULIC CONDUCTIVITY OF SOIL SAMPLES:
FIELD METHOD FOR MODERATELY PERMEABLE FORMATIONS UNDER CONFINED CONDITIONS