D R A F T

R C R A G U I D A N C E D O C U M E N T

S U R F A C E I M P O U N D M E N T S

L I N E R S Y S T E M S, F I N A L C O V E R, A N D

F R E E B O A R D C O N T R O L

[TO BE USED WITH RCRA REGULATIONS
SECTIONS 264.221(a) and (c), 264.222(a), and 264.228(a)]

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A. Purpose and Use

This guidance document presents surface impoundment design specifications which the Agency believes comply with the Design and Operating Requirements of §§264.221(a) and (c), and 264.222(a), and the Closure Requirements of §264.228(a) of the RCRA surface impoundment regulations (40 CFR -------). These regulations have been formulated with the goal of elimiating the escape of hazardous waste and hazardous constituents from surface impoundments for all time to the extent practical. Given that few things work perfectly, or work for all time, absolute prevention of escape for all time is probably not realistic. However, the regulations require surface impoundments to come as close to the containment ideal as possible. In actual practice, the Agency believes containment is practical during the operating life of the impoundment in the absence of damage to the containment system. In the long run, however, in the distant years after closure of disposal impoundments, minimization of leachate formation and escape is the best that current technology can practically achieve.

For most impoundments, the regulations provide that liner systems be installed that are capable of preventing release of hazardous wastes and their derivatives during operating life. At closure, the wastes and contaminated liners, equipment, and subsoils must either be removed or a cap must be installed that is designed to minimize release of pollutants into the distant future. To protect surface waters, a surface impoundment must be designed, constructed, and operated so that it does not overflow. The closure rules essentially eliminate the potential for significant overflow to surface waters after closure.

To provide flexibility, the design and operating characteristics required are expressed in terms of performance standards for system components as a whole. Optimally, these standards would contain numeric limits to the performance required of each system component, and most, but not all of the currently promulgated standards are expressed in numeric terms. For others, minimum specifications are not included. For example, though a final cap must be incorporated, unless the waste is removed at closure, the required capabilities of that system are currently expressed in general terms, specifically "Provide long-term minimization of the migration of liquids". The Agency intends to augment the general statements currently in the regulations by applying numeric limits to the performance required of each component. But, there will still be substantial flexibility in designing facilities to meet the standard and substantial uncertainty involved in judging whether a given design will, in fact, achieve the performance level prescribed.

This document is designed to provide specific guidance on designs the Agency believes accomplish the performance statements in the regulations. As a result, permit applicants designing their facilities in accordance with the specifications contained herein, will be considered in compliance with §§264.221(a) and (c), 264.222(a)(3), and 264.228(a). This provides certainty to the permitting process because, if these specifications and procedures are followed, a draft permit will be issued. (Final permits cannot be issued until input from the public participation process is evaluated.)

The Agency wishes to emphasize that the specifications contained herein are guidance, not regulations. The Agency is not requiring, and does not intend, that all facilities be built in this way. On the contrary, the Agency believes there are many designs which can be acceptably used, depending on waste characteristics and location. Owners or operators wishing to use a different design, but one that contains the basic design components of §§264.221(a) and (c), 264.222(a), and 264.228(a), i.e., liners, overtopping controls, and caps (if disposal units),may demonstrate compliance with the performance requirements for the specific components, directly to the permitting official. An easy way to demonstrate compliance with the performance requirements would be to show that the specific design incorporated at a particular unit provides the same level of performance as would the design incorporated in this guidance under similar circumstances (waste characteristics, location, rainfall, etc.). For example, the specifications for the final cover in this guidance call for a final slope of between three and five percent to promote drainage without causing erosion. To demonstrate the acceptability of a greater slope, the applicant could attempt to show that because of the materials used, perhaps in combination with other design features, a greater slope will result in no more erosion than would be the case utilizing the slope specifications contained herein. The Agency will accept convincing demonstrations of equivalency of performance to the specifications in this guidance as adequate demonstration of compliance with the performance standards of §§264.221(a) and (c) and 264.228(a).

This document contains only initial guidance--it will be significantly expanded over time to include additional designs and specifications which the Agency believes acceptably comply with the performance requirements of the regulations. Since the Agency can only recommend the liner specifications contained in this document for installation above the water table, it is diligently working on similar specifications for location in saturated soils. EPA hopes to issue this additional guidance in the near future. This document will also be expanded by incorporating the experience gained in implementing the performance standards.

The document is arranged according to the section of the regulation to which it corresponds. Those wishing to send technical information or suggestions concerning this document should address them to: Rod Jenkins, Chief, Land Disposal Branch, Office of Solid Waste (WH-564), U.S. Environmental Protection Agency, 401 M Street, S.W., Washington, D.C. 20460. EPA is particularly interested in information and suggestions concerning the usefulness of the document, expansion of it, and the effectiveness of the guidance contained in ensuring compliance with the performance requirements in the regulations.

EPA is also producing a series of Technical Resource Documents (TRDs), two of which cover caps and liners, and a third that covers surface impoundment closure. The TRDs are designed to comprehensively but concisely present the sum total of the body of information and experience gained by the Agency over the years on a given topic. As such, they contain factual summaries concerning the experiences and effectiveness of design alternatives, covering what has been found not to work as well as what has been found to be effective. They contain no policy-related direction. The TRDs can be considered as technical background or development documents supporting these guidance documents and the regulations. Based partially on the information contained in the TRD's, the Agency made the policy decisions which resulted in the regulations and these guidance documents. TRD's corresponding to the guidance in this document are:

(1) Evaluating Cover Systems for Solid and Hazardous Waste (SW-867) NTIS Publication No. PB-81-166-340.

(2) Lining of Waste Impoundment and Disposal Facilities (SW-870) NTIS Publication No. PB-81-166-365.

(3) Closure of Hazardous Waste Surface Impoundments (SW-873) NTIS Publication No. PB-81-166-894.

These documents can be obtained from the National Technical Information Service, U.S. Department of Commerce, Springfield, Virginia 22161. The Agency plans to publish amended versions of these documents in the fall of 1982.

B. Liner System Function, Components, and Life

1. The Regulations

The regulations require the system to function through scheduled closure and to consist of at least one liner designed and constructed to prevent transmission of liquids through it. In the case of disposal impoundments (i.e., where the waste will be left in place at closure), the liner must be essentially impermeable to liquids, allowing no more than de minimum infiltration of liquids into the liner itself.

2. Guidance

(a) Liner systems should be constructed wholly above the seasonal high water table, i.e., in the unsaturated soil.

(b) Liner systems for storage or treatment impoundments where the waste will be removed at closure should consist of a single soil (clay) or synthetic liner, as a minimum.

(c) Liner systems for disposal impoundments where the waste will remain at closure should consist of a single synthetic liner, as a minimum.

(d) Where a synthetic liner is used in any surface impoundment which will not complete closure for 30 or more years after first placement of wastes, the underliner system should consist of the following as a minimum:

(i) A primary synthetic liner; and

(ii) A secondary soil liner (e.g., clay).

3. Discussion

In developing the designs contained in this guidance, the Agency has attempted to come as close as possible to complete containment for as long as the impoundment is in operation. However, the Agency also used, as an overriding criterion, that the designs developed be based on conventional technology, utilizing readily available equipment and materials, at practical

Essentially complete but temporal containment is practical using the synthetic liners developed over recent years. However, experience with synthetic liners in contact with chemicals is relatively recent. As a result, EPA has no experience base from which to predict the life of these materials. However, predictions based on chemistry, and limited recent real world experience are sufficient to convince the Agency that with proper selection for resistance to chemicals in the waste and proper design and installation (tight seams and no tears), containment for at least 30 years is practical, utilizing a single synthetic liner (some believe that such liners will last in excess of 100 years). Therefore, a single synthetic liner system is described for impoundments which will be closed in less than 30 years. Some surface impoundments, however, are designed to function almost perpetually. While the regulations require a prediction that the synthetic liner will last as long as the projected life of the impoundment, regardless of how long that is, there is no historic proof that such liners will, in fact, function into the distant future. Therefore, if final closure is not scheduled for 30 years or more, a double liner system incorporating a top synthetic liner and a secondary clay liner is recommended. The secondary soil liner functions as a backup to the synthetic primary liner, taking over the task of minimizing liquid transmission when and if the primary liner deteriorates.

Some soil materials, typically those classified as clays, can be deposited and compacted to produce a liner system of very low permeability. Movement of liquids through well constructed clay liners is very slow as long as the structure of the liner is not affected by the waste materials or is not otherwise damaged. As a result, when functioning properly, the release of liquid-containing hazardous constituents to the ground water and surrounding soils, is effectively minimized (but not prevented) by the secondary clay liner. Thus, absolute prevention of escape of liquid wastes and pollutants through the liner system during the life of very long lived surface impoundments may not be fully achievable, in those cases where the primary synthetic liner deteriorates, though the use of the secondary clay liner should keep the rate of escape to very low levels.

Storage impoundments are a special case, however. Storage impoundments must be closed by removing the wastes, contaminated liners, and equipment at closure. Should there be any leakage through the liner during operation of a storage impoundment, the contaminated soil must either be removed or decontaminated as well. Since removing and decontaminating soil can be an expensive process, it is important to owners and operators of storage impoundments that liner systems not leak during the life of the impoundment. The Agency is convinced that synthetic liners will readily achieve this goal, assuming the liner is not attacked chemically or damaged physically, and assuming that site life is 30 years or less. The Agency also believes that adequate protection can be achieved by use of a single clay liner of sufficient thickness and impermeability to ensure that no waste travels through the liner before closure when both the waste and the saturated liner are removed.

Thus, in the Agency's view, for storage impoundments, clay and synthetic liners can be considered equally protective under most circumstances.

For disposal impoundments, where the waste and contaminated liners will remain after closure, the Agency believes that synthetic liners are preferable because they constitute a more efficient barrier to escape of liquid wastes. As long as they are intact, they are essentially 100 percent effective as a barrier. If the synthetic liner remains intact through closure, when the cap functions to greatly reduce leachate generation, then little, if any, pollutants should have exfiltrated to the surrounding soil. (The Agency realizes that very small amounts of liquids may enter the structure of synthetic membranes causing them to swell, but the amount is truly negligible and leads to no future ground water contamination.) Clay liners, on the other hand, are somewhat permeable. Some of the liquid waste will infiltrate the pore structure of the liner and will be released over time. In the Agency's view, therefore, a single clay liner in a disposal impoundment does not fully prevent release of hazardous waste constituents reaching its surface during operating life. It simply reduces the amount that is released, retards release, and minimizes the rate of release. EPA concludes, therefore, that clay liners are not nearly so effective as synthetics in the near term (at least 30 years). The Agency realizes, however, that there is disagreement as to the relative effectiveness of various types of liners. The Agency is, therefore, seeking data and information concerning:

(1) The relative efficiency of synthetic, soil, and other liners in preventing and minimizing the transmission of liquids and the release of pollutants;

(2) The effective life of various liner designs;

(3) The causes of physical and chemical damage to various liners and how damage can be avoided;

(4) The potential for saturated soil liners to attenuate pollutants;

(5) The potential for soil liners to release pollutants after proper closure has removed liquids from the impoundment and reduced the amount of leachate formed; and

(6) The benefits to be realized and the risks posed by various liner designs.

The designs suggested are specifically recommended for installation in the unsaturated zone. The Agency views location in the ground water as fraught with additional risks and design difficulties. This does not mean, however, that the Agency frowns on such locations; EPA is convinced that given certain hydrogeological circumstances and accommodating designs, location in saturated soils can be environmentally acceptable. In addition to the potential for leak detection systems to fill with ground water, the problems associated with location in the saturated zone are primarily associated with the external pressure applied by the saturated earth against the liner system. This can result in damage to the integrity of the liner system and to difficulties in removing wastes at closure. The former is of real concern to the Agency; the latter would be primarily a nuisance to the operator at closure. Depending on the active earth pressure and the way the surface impoundment is designed and operated, the force exerted by the liquid contents of the impoundment may offset the external force exerted by the saturated earth. If so, no damage to the liner system would be expected. Accordingly, the Agency intends to expand the designs in this guidance to include designs which will effectively minimize the rate of pollutant release in saturated locations. In the interim, those wishing to locate in the saturated zone and whose units have the basic components specified in the regulations (i.e., at least one liner) may be able to readily show that the location, design, and operating characteristics of the unit prevent the migration of pollutants from the unit and that the surface impoundment will function with an equivalent degree of certainty (e.g., longevity, damage potential, etc.). The closer the actual design is to those in this guidance document, the easier that demonstration may be to make. In any event, for now, a case-by-case demonstration of containment will be necessary for impoundments located wholly or partially in the saturated zone.

C. Leak Detection Systems

1. The Regulations

The regulations do not require a leak detection system.

However, under the requirements of §264.222 (Double-lined surface impoundments), owners or operators choosing to install a double liner system with a leak detection system may be exempted from the monitoring and other requirements of Subpart F unless and until contaminated liquid is identified in the leak detection system.

2. Guidance

Leak detection systems should have:

(a) At least a 30 centimeter (12-inch) drainage layer with a hydraulic conductivity not less than 1×10^{-3} cm/sec and a minimum slope of 2 percent;

(b) A drainage tile system of appropriate size and spacing and a sump pump or other means to efficiently conduct liquids.

3. Discussion

The leak detection system is the means by which one can determine if the primary liner has failed or is leaking. Under the operating requirements of §264.222(b), the owner or operator must then either repair the primary liner or institute ground-water monitoring, if he is not already doing so.

There are, of course, many possible designs for detecting leaks including the use of advanced instrumentation. The system described here is essentially a gravity collection system, very similar to the leachate collection system for landfills and waste piles.

The minimum thickness (30 centimeters or 12 inches) of the drainage layer is designed to allow sufficient head to promote drainage. The two percent minimum slope is also designed to promote drainage. The hydraulic conductivity of not less than 1 X 10^{-3} cm/sec was chosen because materials used widely as drainage media are typically at least that coarse.

Drainage tile diameter and spacing are important because they affect the removal of liquids. The closer the tiles are together, the more quickly a leak is likely to be detected. Unlike leachate collection systems for landfills and piles, however, the primary purpose here is detection of leaks, not removal. Thus, tile size and spacing need only be sufficient for rapid detection of the initial leak and need not be designed to remove some predetermined volume rate of flow. The Agency is therefore not specifying minimum tile spacing or size in this guidance. Nevertheless, a reasonably sized drainage system coupled with an efficient means (such as a sump pump) for removing collected liquids, will result in capacity to remove leaking fluids except in the case of severe breaches of the primary liner. By so doing, the liquid head on the bottom liner will remain low providing an extra measure of protection. EPA believes that a design incorporating 4 in. diameter tiles on 50 to 200 foot (15 to 60 meter) centers will provide efficient leak detection and capacity to remove leakage from minor breaches of the primary liner.

D. Liner Specifications

1. The Regulations

As discussed in Section B of this guidance, the liner system must be designed and built to achieve containment of fluids during the life of the site thus preventing the escape of hazardous constituents to surrounding soils and ultimately to the ground water. At least one liner must be installed and the material used must be resistant to the chemicals it will encounter in the wastes and in the leachate, and be of sufficient strength to withstand the forces it will encounter during installation and operation. In this regard, a base is required which must provide sufficient support to the liner to prevent failure. The liner system must, of course, cover all areas likely to be exposed to waste and to leachate.

2. <u>Guidance</u>

In Section B of this guidance, the Agency identified the conditions under which it believes single and double liner systems are appropriate and the nature of the materials recommended. Following are liner specifications which the Agency believes will produce stable construction and which will prevent the release of hazardous constituents.

(a) Synthetic liners should:

(1) Consist of at least a 30 mil membrane that is chemically resistant to the waste managed at the unit. In judging chemical compatibility of wastes and membranes, the Agency will consider appropriate historical data, demonstrations involving theoretical chemistry, and actual test data. Testing of chemical resistance of liners should be performed using either the EPA test method attached or an equivalent test method.

(2) Be protected from damage from below the membrane, by at least 15 centimeters (6 inches) of bedding material no coarser than Unified Soil Classification System (USCS) sand (SP) and which is free of rock, fractured stone, debris, cobbles, rubbish, roots, and sudden changes in grade. Leak detection systems, soil (clay) liners, or natural, in situ soils may serve as bedding materials when in direct contact with synthetic liners if they meet the requirements specified herein.

(3) Be protected from damage when mechanical equipment is used to remove sludge or for other reasons, by means of a minimum of 45 centimeters (18 inches) of protective soil (or the equivalent) above the top liner.

(4) Be protected from damage due to sunlight or wind, where exposed to the elements, by means of a minimum of 15 centimeters (6 inches) of protective soil (or the equivalent) over exposed surfaces, unless it is known that the liner material used is not physically or chemically impaired by exposure.

(b) Soil liners should:

(1) Consist of at least 60 centimeters (24 inches) of natural or recompacted emplaced soil (e.g., clay) with a saturated hydraulic conductivity not more than 1 X 10^{-7} cm/sec. Saturated hydraulic conductivity testing should be conducted using either the EPA test method attached or an equivalent.

(2) Have a saturated hydraulic conductivity which is not increased beyond 1×10^{-7} cm/sec as a result of contact with the waste or leachate generated at the facility. Testing of the effect of waste or leachate on soil liner hydraulic conductivity should be performed using either the EPA test method attached or an equivalent test method.

(3) Have no lenses, cracks, channels, root holes, or other structural nonuniformities that can increase the nominal hydraulic conductivity of the liner above 1×10^{-7} cm/sec; and

(4) Where recompacted emplaced soil liners are used, be placed in lifts not exceeding 15 centimeters (6 inches) before compaction to maximize the effectiveness of compaction.

(c) Single soil liners used in storage or treatment impoundments from which wastes will be removed at closure should:

(1) Have a sufficient thickness of natural or recompacted emplaced soil to provide containment of the waste in the liner system (i.e., no fluid flow moves beyond the liner) during the operating life of the unit; the necessary thickness of soil should be determined by use of the following:

$$d = 0.5 \left[\frac{tk}{\theta} + \sqrt{\frac{(tk)^2}{\theta} + 4 \frac{(tkh)}{\theta}} \right]$$

where:

d = necessary thickness of soil (feet)

θ = total porosity

k = hydraulic conductivity (ft/yr)

h = maximum fluid head on the liner (feet)

t = facility life from startup through closure (years);

(2) Have a hydraulic conductivity which is not increased beyond that used in the calculations as a result of contact with the waste managed at the facility. Testing of the effect of leachate on soil liner hydraulic conductivity should be performed using either the EPA test method attached or an equivalent test method.

(3) Have no lenses, cracks, channels, root holes or other structural nonuniformities which can act to increase the nominal hydraulic conductivity above that used in the calculations; and

(4) Where recompacted emplaced soil liners are used, be placed in lifts not exceeding 15 centimeters (6 inches) before compaction to maximize the effectiveness of compaction.

3. Discussion

EPA believes synthetic liners should be at least 30 mils thick. Thinner synthetic membrane liners are known to be readily damaged. One of the primary reasons for failure of synthetic liners, is damage (i.e., punctures, rips, and tears). Damage occurs during installation or during operation. With surface impoundments, punctures occur as a result of the pressure applied by liquid wastes forcing the membrane against sharp objects below (rocks, sticks, debris, etc.). If the impoundment is a double lined unit with a leak detection system, this is not usually a problem because the leak detection system normally provides protection. However, even leak detection systems are sometimes constructed of coarse rock to promote drainage, but which can damage the liner. To protect against this, EPA recommends that a minimum six inch bedding layer be placed under the liner. The bedding layer should consist of materials which are no coarser than sand (SP) as defined by the Uniform Soil Classfication System (USCS). Use of a sand layer is common practice for protection of membranes and other delicate materials from damage due to contact with grading equipment and materials, sharp materials in soil, etc. The bedding material need not be a separate layer as natural soils or the leak detection, collection, and removal systems materials will often meet the necessary criteria.

Bedding material is not usually necessary above the liner, since direct contact is normally with the liquid waste contents or material settling out of it. However, the liner can also be damaged by sludge removal or other mechanical equipment used in the impoundment. Where such equipment is used, EPA recommends a minimum of 45 centimeters (18 inches) of protective soil, or the equivalent covering the top liner. EPA believes this will be sufficiently protective in most cases since sludge removal equipment is usually carefully controlled. Additionally, some liner materials are known to be degraded substantially by sunlight. In some circumstances, wind can get under the edge of exposed liners, causing flapping and whipping, which can lead to tears. These problems have occurred most commonly above the liquid level near the edge of the liner. As a result, it has become common practice to cover exposed liner areas with six inches or so of earth materials to hold the liner down and prevent degradation. Of course, if the design is such that wind creates no difficulties, and if it is known that the liner is not subject to solar degradation, then these precautions are not necessary.

Chemical testing is prudent because synthetic liners are degraded by certain species which may be present in the waste. Because wastes and liner chemical characteristics are almost infinitely variable, it is difficult to generalize concerning incompatibility. The Agency therefore prefers test data as the preferable way to demonstrate the compatibility of waste and liner materials, but recognizes that historic data (results elsewhere with similar wastes) or theoretical chemistry may provide sufficient information in some cases. Data currently available to EPA indicate the following combinations of waste types and liner materials are often incompatible:

(a) Chlorinated solvents tend to dissolve polyvinyl chloride (PVC)

(b) Chlorosulfonated polyethylene can be dissolved by aromatic hydrocarbons.

(c) Clays may exhibit high permeability when exposed to concentrated organics, especially organics of high and low pH

(d) Asphaltic materials may dissolve in oily wastes

(e) Concrete and lime based materials are dissolved by acids

The Agency is currently developing a more comprehensive summary of waste/liner compatibility information which will be included in a later edition of this document.

An acceptable test method for evaluating waste/liner compatibility is included in the Appendix to this document. The test method exposes a liner sample to the waste or leachate encountered at the unit. After exposure, the liner sample is tested for important characteristics--saturated hydraulic conductivity in the case of soil liners; strength (tensile, tear, and puncture) in the case of synthetics. The Agency considers any significant deterioration in any of the measured properties to be evidence of incompatibility unless a convincing demonstration can be made that the deterioration exhibited will not impair the integrity of the liner over the life of the unit. Even though the tests may show the amount of deterioration to be relatively small, the Agency is concerned about the accumulative effects of exposure over very much longer periods than those actually tested.

The Agency had intended to incorporate the National Sanitation Foundation's (NSF) standard specifications for flexible membrane liners as part of this guidance. This would have provided minimum recommendations with regard to physical properties, construction practices, seaming tests, etc. An NSF committee has been studying the subject for some time, and EPA believes that the specifications which are being developed are reasonable and well thought out. However, at this point, the NSF has not formally adopted the draft standards. Therefore, given the possibility that they might still be changed, EPA believes it prudent to wait for formal adoption by the NSF before incorporating them into this guidance document. The NSF specifications are not intended to comprehensively cover the myriad of hazardous waste applications, however. Therefore, the Agency believes that some augmentation of the NSF specifications is appropriate. A few additions have been made (e.g., the 30 mil thickness requirement) and more may be incorporated if experience warrants.

Once the NSF strength secifications have been published, EPA plans to incorporate them in conjunction with the compatibility test method for synthetic liners attached, in an improved method of evaluating liner compatibility. While the Agency does not want to prejudge how this may be done, one possible approach might be to extrapolate the strength curves developed by the test method to the expected life of the unit. The expected strength at that time could then be compared against minimally acceptable strength levels, e.g., the NSF strength specifications or some acceptable fraction of them. The Agency is interested in comments, suggestions, and data on the subject of evaluating strength loss on exposure to chemical leachates.

Soil liners will normally be of clay soils. They should have a saturated hydraulic conductivity of not more than 1×10^{-7} cm/sec and be at least 60 centimeters (2 feet) thick. To minimize the transmission of waste or leachate fluids, the soil liner should be as tight as practical. Many clays can readily be recompacted to meet the specified level. It is not clear, however, that recompacting to meet a tighter specification can be routinely accomplished. In concert with the philosophy of these designs, soil liners are usually incorporated as backup systems and are depended upon to minimize the rate of liquid flow through them (except in the case of storage impoundments to be discussed later). Thickness of the soil is not as important as other factors in minimizing flow but a minimum thickness is necessary to retain structural stability (reduce cracking potential, etc.). Two feet is generally accepted to be a minimum stable thickness for recompacted clay.

When discussing the relative tightness of soils, the term permeability is most often used. This is a generic term, referring to the property in general. In this guidance, EPA uses the more specific term--"hydraulic conductivity". An acceptable method for soil hydraulic conductivity has been included in the Appendix to this document.

In situ soils can be considered acceptable as soil liner material provided the specifications in this guidance are met. Natural soil liners should be free of conduits and channels which would convey liquids through the liner. This includes root holes, sand lenses, cracks, fractures, etc.

In addition to meeting the other specifications specified for clay liners, those wishing to use a single clay liner for storage impoundments should be convinced that the liner system is capable of retarding liquid flow through it to the extent that the liquid is wholly contained within the liner through the life of the unit. Ideally, one would calculate containment time on an unsaturated basis, but the unsaturated flow equations are difficult, complex, and controversial. As a surrogate which approximates the ideal and can be practically applied, the Agency has chosen a transit time formula assuming saturated conditions. The formula is based on the life of the facility through closure, the porosity of the soil liner, the saturated hydraulic conductivity, and the maximum fluid head in the surface impoundment.

The determination of the effective porosity of the soil is difficult to do in practice. Because of the controversial nature of the determination of effective porosity, the Agency believes total porosity should be used in the calculation even though in doing so the requirement is somewhat less protective. The Agency is evaluating methods for determining effective porosity and may change this guidance should confidence in a specific method be established.

Use of the saturated flow equation is environmentally conservative, and will result in a thicker soil liner than would absolutely be necessary to assure containment in the liner system. This is partially offset however by the use of total porosity instead of effective porosity which causes the equation to be somewhat less protective. Additionally, the saturated flow equation does not account for the effects of capillary tension which will also cause the regulation to be somewhat less protective. On balance, the use of total porosity and the lack of consideration of capillary tension will somewhat offset the error introduced by use of a saturated flow equation. While EPA realizes that this formula is not perfect, it is the only performance based approach which is practically implementable. It is somewhat environmentally conservative; erring on the side of added protection. This is prudent, in the Agency's opinion, given the uncertainties associated with prediction of pollutant movement through soils.

Owners and operators choosing to incorporate a leak detection system between a double liner design as a means of qualifying for exemption from the ground-water protection and associated monitoring requirements of Subpart F in accordance with §264.222, must incorporate a secondary liner, under the leak detection system, which meets the containment requirement for liner systems under §264.221(a). For most surface impoundments, this means that a synthetic secondary liner must be used as a minimum. If the impoundment is being designed to operate for more than 30 years, then the synthetic liner should be backed up by a tertiary soil liner meeting the specifications discussed herein. The regulations are written to require prevention of release by the secondary liner in surface impoundments exempted from Subpart F because the owner or operator may choose, upon detection of a leak, to convert his unit to the same status as other surface impoundments installing ground-water monitoring facilities and becoming subject to the ground-water protection standards of Subpart F. Most other surface impoundments must have a liner system capable of preventing migration of liquids.

E. Cap (Final Cover) Design

1. The Regulation

The cap or final cover must be designed to minimize infiltration of precipitation into the surface impoundment after closure. It must be no more permeable than the liner system.

It must operate with minimum maintenance and promote drainage from its surface while minimizing erosion. It must also be designed so that settling and subsidence are accommodated to minimize the potential for disruption of continuity and function of the final cover.

2. Guidance

(a) The cap or final cover should be placed over the surface impoundment after the waste has been solidified and compacted enough to support it. Solidification and fixation processes often take days or weeks to completely stabilize. Final cover should not be applied before stabilization is nearly complete.

(b) The cap (final cover) should consist of the following as a minimum:

(1) A vegetated top cover, as described in paragraph (c) of this section;

(2) A middle drainage layer as described in paragraph (d) of this section; and

(3) A low permeability bottom layer as described in paragraph (e) of this section.

(c) The vegetated top cover should:

(1) Be at least 60 centimeters (24 inches) thick;

(2) Support vegetation that will effectively minimize erosion without need for continuing application of fertilizers, irrigation, or other man-applied materials to ensure viability and persistence (Fertilizers, water, and other materials may be applied during the closure or post-closure period if necessary to establish vegetation or to repair damage.);

(3) Be planted with persistent species that will effectively minimize erosion, and that do not have a root system that will penetrate beyond the vegetative and drainage layer;

(4) Have a final top slope, after allowance for settling and subsidence, of between three and five percent, unless the owner or operator knows that an alternate slope will effectively promote drainage and not subject the closed facility to erosion. For slopes exceeding five percent, the maximum erosion rate should not exceed 2.0 tons/acre using the USDA Universal Soil Loss Equation (USLE); and

(5) Have a surface drainage system capable of conducting run-off across the cap without forming erosion rills and gullies.

(d) The drainage layer should:

(1) Be at least 30 centimeters (12 inches) thick with a saturated hydraulic conductivity not less than 1×10^{-3} cm/sec;

(2) Have a final bottom slope of at least two percent, after allowance for settling and subsidence;

(3) To prevent clogging, be overlain by a graded granular or synthetic fabric filter. Where a granular filter is used, the grain size ratio should meet the following criteria:

$$\frac{D15 \text{ (filter soil)}}{D85 \text{ (drainage layer)}} \leq 5$$

$$\frac{D50 \text{ (filter soil)}}{D50 \text{ (drainage layer)}} \leq 25$$

and

$$\frac{D15 \text{ (filter soil)}}{D15 \text{ (drainage layer)}} = 5\text{-}20$$

where:

D15 = grain size, in millimeters, at which 15% of the filter soil used, by weight, is finer;

D85 = grain size, in millimeters, at which 85% of the drainage layer media, by weight, is finer;

D50 = grain size, in millimeters, at which 50% of the filter soil or drainage media, by weight, is finer; and

(4) Be designed so that discharge flows freely in the lateral direction to minimize head on and flow through the low permeability layer.

(e) The low permeability layer should have two components:

(1) The upper component should:

(A) Consist of at least a 20 mil synthetic membrane;

(B) Be protected from damage below and above the membrane by at least 15 centimeters (6 inches) of bedding material no coarser than Unified Soil Classification System (USCS) sand (SP) and which is free of rock, fractured stone, debris, cobbles, rubbish, roots, and sudden changes in grade (slope). The drainage layer and lower soil (clay) component may serve as bedding materials when in direct contact with synthetic caps if they meet the specifications contained herein;

(C) Have a final upper slope (in contact with the bedding material) of at least two percent after allowance for settling; and

(D) Be located wholly below the average depth of frost penetration in the area;

(2) The lower component should:

(A) Include at least 60 centimeters (24 inches) of soil recompacted to the maximum practical extent, but capable, if placed on a firm base, of being recompacted to a saturated hydraulic conductivity of not more than 1 X 10^{-7} cm/sec;

(B) Have the soil emplaced in lifts not exceeding 15 centimeters (6 inches) before compaction to maximize the effectiveness of compaction;

(f) In designing the final cover, owners and operators should estimate and accommodate the amount of settling and subsidence expected as a result of degradation and long-term consolidation of waste.

3. Discussion

The guidance calls for placing the final cover once the waste remaining in the surface impoundment has been solidified through sorption, fixation, or some other means and after it has stabilized and been compacted sufficiently to support the final cover. Some of the fixation processes take days or weeks to stabilize and placement of final cover should not commence until the process is nearly complete.

The Agency believes that a three layer final cover (cap) will adequately minimize infiltration of precipitation, which is the primary purpose of the final cover. The final cover acts to minimize infiltration by causing precipitation to run off through use of slopes, drainage layers, and impermeable and slightly permeable barriers. By minimizing infiltration, the generation of leachate will also be minimized, thereby reducing long-term discharge of pollutants to the ground water to a bare minimum. To prevent the "bathtub effect," i.e., to prevent the landfill from filling with leachate after closure, the final cover must be no more permeable than the most impermeable component of the liner system (or of the underlying soils). In this way, no more precipitation is allowed to infiltrate into the closed impoundment than can escape through the bottom liner. Prevention of the "bathtub effect" is important to eliminate the possibility of surface overflow or migration through porous surface strata. Other functions of the final cover include prevention of contamination of surface run-off, prevention of wind dispersal of hazardous wastes, and prevention of direct contact with hazardous wastes by people and animals straying onto the site.

The top layer should have at least two feet of soil capable of sustaining plant species which will effectively minimize erosion. Two feet was chosen because it will accommodate the root systems of most nonwoody cover plantings and is typical practice within the waste management industry today. Species planted should not require continuing man-made applications of water or fertilizers to sustain growth since such applications cannot be guaranteed in the long term. Application of water and fertilizer is, of course, acceptable during the early stages of the post-closure care period as the plant growth is being established. The plant species chosen should also not have root systems which can be expected to penetrate beyond the vegetated and drainage layers. If they penetrate deeper, they can damage the integrity of the low permeability layer.

After allowance for settling and subsidence, the final slope should be at least three percent to prevent pooling due to irregularities of the surface and vegetation but less than five percent to prevent excessive erosion. Owners and operators using different final slopes should determine that an alternate slope will not be beset with erosion problems and that it will promote efficient drainage.

The U.S. Department of Agriculture Universal Soil Loss Equation (USLE) is recommended as a tool for use in evaluating erosion potential. The USLE predicts average annual soil loss as the product of six quantifiable factors. The equation is:

$$A = RKLSCP$$

where R = rainfall and run-off erosivity index

K = soil erodibility factor, tons/acre

L = slope-length factor

S = slope-steepness factor

C = cover/management factor

P = practice factor

The data necessary as input to this equation are described in Evaluating Cover Systems for Solid and Hazardous Waste (SW-867), September 1980, U.S. EPA. The maximum rate of erosion for any part of the cover should not exceed 2.0 tons/acre in order to minimize the potential for gully development and future maintenance. The agricultural data base indicates that rates as low as 1/3 ton per acre are achievable for a silt-loam soil, sloped four percent with a blue grass vegetative cover. The Agency believes that two tons per acre is more readily achieved and does not significantly increase cover maintenance. The top layer should also have some means of conducting runoff (e.g., swales or conduits) to safely pass run-off velocities and volumes without eroding the cover.

The second layer or drainage layer is analogous in function to the leachate collection system over the liner of a landfill. It should be at least 12 inches thick to provide capacity to handle water from major sustained storm events, and should be constructed of porous materials (at least 1 X 10^{-3} cm/sec hydraulic conductivity). Drainage tiles or other collection devices are not necessary. The Agency believes that the combination of very porous media, a final minimum two percent slope after settling, and the impermeable nature of the layer beneath will effectively conduct precipitation infiltrating the vegetative layer, off of the unit. As with the leak collection system, the drainage layer should be overlain with a graduated granular or synthetic fabric filter to prevent plugging of the porous media with fine earth particles carried down from the vegetated layer. To prevent fluid from backing up into the drainage layer, the discharge at the side should flow freely.

The function of the low permeability layer is to reject fluid transmission, thereby causing infiltrating precipitation to exit through the drainage layer. It should consist of at least two components. The upper component should be at least a 20 mil thick synthetic membrane. While the regulations do not specify that the cap <u>prevent</u> infiltration, the requirement that it be no more permeable than the bottom liner, as a practical matter, necessitates the use of a synthetic membrane. This is so because the regulatory requirement for the liner system does specify that liquids be contained, and this will be translated, in most cases, into a very nearly impermeable synthetic membrane liner.

The minimum thickness specified for the synthetic component of the final cover (20 mil) is less than that recommended for the liner (30 mil) because (1) the final cover is not expected to come in contact with chemical wastes which will tend to hasten failure, and (2) once placed, the potential for damage is small as compared to the potential for underliner damage where waste is in contact with the liner throughout the operating life of the cell. While intact (30 + years in the absence of damage), the synthetic component will essentially prevent transfer of precipitation through it and leachate production should be very nearly zero. As with underliners, synthetic caps should be protected from from punture and tears by at least six inches of bedding materials with the consistency of sand or finer. In most cases, the drainage layer media above the synthetic cap, together with the soil (clay) liner under it, can effectively function as the bedding material.

Even with protection from damage, the synthetic cap will not last forever. At some point, perhaps in the far distant future, the synthetic membrane will degrade. At that time, the function of minimizing infiltration will fall to the second component, a 2-foot minimum clay soil cap with a maximum hydraulic conductivity of 1×10^{-7} cm/sec. Although some small amount of precipitation will seep through this secondary cap, the amount of leachate generated will be quite small and escape to the ground water should be mimimal. Unless damaged or affected by differential settling, the secondary soil liner should remain intact and effective for all time. One source of damage is frost heaving which can disrupt the continuity of the impermeable layers. For this reason, the impermeable layer should be wholly below the average depth of frost penetration in the area. This may necessitate a thicker cover than would otherwise be necessary.

One of the more difficult problems associated with designing final cover is how to allow for settling and subsidence. Settling occurs as a result of natural compaction and consolidation and biological degradation of organics. It tends to be relatively uniformly distributed and usually occurs shortly after closure. Subsidence is a more difficult problem since it tends to be unevenly distributed, resulting in differential sinking which in turn can cause disruption in continuity of the final cover. It most often occurs as the result of final release of liquids from, and collapse of drums and is, therefore, not normally a major problem with closed surface impoundments. Settling on the other hand, may pose a significant problem if remaining wastes are high in organic content or if organic sorbants (e.g., sawdust or paper) are used to solidify the wastes. Chemical fixation processes are usually inorganic in nature and are not prone to significant settling once stabilization is complete.

EPA intends to develop specific design requirements which will ensure adequate allowance for settling and subsidence. As of this writing, however, the Agency lacks sufficient information to judge the effectiveness of various design options. Therefore, this guidance suggests simply that owners and operators estimate the amount of subsidence and allow for it in the final cover design as best they can. The final result should be a minimum three percent final slope after settling and subsidence. During the postclosure period, the regulations require that the damaging effects of settling and subsidence (e.g., disruption of the continuity and slope of the cap) be repaired. It thus behooves the owner or operator to adequately allow for subsidence and settling. As the Agency evaluates alternative methods of designing final cover to effectively allow for settling and subsidence, it will issue further guidance or perhaps even additional regulations covering the subject.

One suggestion which owners and operators may consider as a means of at least partially accomodating settling and subsidence, is to stage final closure and the placement of the final cover. Unsubstantiated information from the field leads EPA to believe that the most severe subsidence and settling problems occur rather soon after closure. It may be preferable therefore, from both an environmental and cost standpoint, to delay placement of the relatively expensive final cover for six months or more in those cases where substantial subsidence or settling are expected. By so doing, expensive repairs to the final cover may be avoided. This would require an extension in the 180 day limit to the closure period imposed in Subpart G. In deciding whether to grant such an extension, in accordance with the rules of Subpart G, the permitting official will normally require installation of an expendable interim cover, capable of minimizing precipitation migration into the unit. The Agency solicits information on the effectiveness of this and other approaches to dealing with the settling/subsidence problem.

F. Freeboard Control

1. The Regulations

The regulations require simply that the owner or operator prevent overtopping of his surface impoundment from virtually any eventuality, including normal or abnormal operations, overfilling, wind and wave action, rainfall, equipment malfunctions, and human error.

To implement this requirement, the owner or operator must demonstrate in his permit application that design features and operating procedures at his operation will prevent overtopping. If acceptable to the permitting official, these features and procedures will be incorporated in the permit.

2. Guidance

(a) Where possible, surface impoundments should be designed with outfall mechanisms such as weirs or spillways which are relatively insensitive to inflow.

(b) Where outfalls are sensitive to inflow, i.e., where adjustments must be made to maintain impoundment freeboard as inflow increases, outfall devices (or inflow controls) should be automaticaly controlled by signals from level-sensing instruments.

(c) Except when equipped with inflow insensitive outfalls as described in (a) above, surface impoundments should be equipped with a high-level alarm based on a different level sensor than that used for automatic control.

(d) Surface impoundments should be designed to maintain at least 60 centimeters (two feet) of freeboard unless it is known that normal fluctuations in level and maximum wave action will not cause overtopping.

(e) A surface impoundment should be designed so that any flow of waste into the impoundment can be immediately shut off in the event of overtopping or liner failure.

(f) A surface impoundment should have a run-on control system designed to prevent flow into the impoundment during the peak discharge from a 100-year storm unless the impoundment is designed to accommodate the extra flow without detrimental effects to the impoundment or appreciable loss of freeboard.

3. Discussion

Preventing overtopping of surface impoundments is not normally a difficult engineering problem. Many impoundments are operated on a flow through basis. Typically, these are impoundments used for treatment of wastes; biological oxidation lagoons are a common example, though not one normally associated with hazardous wastes. Frequently, these impoundments are designed with simple spillway or weir-type discharge structures which maintain a constant freeboard level in the impoundment. With this type of arrangement, the sensitivity of freeboard level is dependent directly on the relative width of the spillway or weir.

Normally, but not always, these structures are sufficiently wide so that freeboard level is insensitive to normal changes in flow. In these cases, all that is necessary is the provision of a modest freeboard level sufficient to deal with wave action and with the minor fluctuations which will occur as flow changes. The Agency believes two feet will be sufficient in virtually all cases where discharge structures of this type are used.

Some impoundments are designed with other discharge arrangements. Some, such as circular weirs, operate on the same principle as those discussed above, but because the effective discharge width of the device is often narrow relative to potential flow fluctuations, they may be overwhelmed by abnormal rainfall events or malfunctions in the production equipment which feeds wastes to the impoundment. Other arrangements require adjustment to the discharge structures to maintain freeboard level. A common design of this type incorporates underflow pipes through a dike. Freeboard level in these designs is usually controlled by a valve on the pipe. Others operate off of a sump arrangement with the freeboard level controlled by turning a pump on and off. Storage impoundments are often constructed this way. Some other impoundments, usually those operated as seepage or evaporation impoundments, have no discharge arrangment. Obviously, with no discharge, the freeboard level is very sensitive to substantial changes in inflow to the impoundment.

Where freeboard level is sensitive to flow changes, there are a number of design and operating alternatives which can be adopted to provide adequate protection against overtopping:

(1) Emergency spillways or overflow piping arrangements can sometimes be provided which can be connected to holding ponds or tanks to contain the overflow. If these latter units are to be used only in the unlikely event of an emergency, they need not be permitted. However, if they are to be used on a statistically predictable recurring basis for surge capacity, permitting is necessary. This will require exercise of some judgment on the part of the permitting official.

(2) Another alternative is to control the discharge device automatically based on freeboard level. The underflow valve can be opened or closed automatically to maintain a set freeboard level through signals from level-sensing instruments. Reliable level sensing devices have been available for many years and are usable in most situations. Discharge pumps can be similarly controlled (turned on and off to maintain level within a range). Where it can be accommodated from the point of view of production or operation of the rest of the facility, it may be possible to automatically control the amount of waste flowing into the impoundment through valves or pumps based on freeboard level.

(3) Outfall devices can also be controlled manually. In these cases, the owner or operator must be prepared to demonstrate that he either maintains sufficient freeboard to accommodate any reasonably possible increase in influent flow or that the combination of the rate of possible flow increases coupled with the frequency of operator inspections to control discharge

will eliminate potential overtopping due to sudden unnoticed increases in level. In these cases, the frequency of control inspections must be specified in the inspection schedule and thus in the permit.

(4) Some impoundments have no discharge devices. In these cases, owners or operators must be able to demonstrate either that the rate of seepage or evaporation is such that overflow is not possible even under maximum possible inflow rates or, more probably, that operating procedures are in effect which will effectively control inflow so that overtopping conditions will not occur. A convincing demonstration should be required in any case.

Of the acceptable options discussed above, the Agency clearly favors automatic controls and/ or provision of foolproof arrangements such as weir discharges or emergency collection devices (tanks or ponds). These are not as sensitive to human error as are those arrangements requiring human inspection and manual operation. Automatic control devices based on level sensors are, however, subject to malfunctions. Because of this, the Agency recommends that any freeboard control arrangement that is sensitive to inflow variations (i.e., all except those using weirs, spillways, or similar devices, or where possible inflow variations cannot cause overflow) be equipped with an alarm to warn of loss of freeboard so emergency action can be taken to prevent overtopping. Typically, these will also be based on a level sensor device, and the Agency recommends that a different sensing unit and different type of device be used than that used for automatic control. This will help ensure that both do not fail at the same time.

EPA believes that two feet of freeboard will normally provide sufficient protection against overtopping due to inflow fluctuations or wave action. However, where manual operation is involved, freeboard levels substantially greater may be necessary to assure adequate protection. Smaller freeboard levels may be justified if the owner or operator can demonstrate that level variations due to possible flow changes are very small or that the automatic level control response is such that level variations will be very small.

In the event overtopping does occur, in spite of the safeguards built into the system, or in the event of some other catastrophic failure (e.g., dike failure), there must be some way to quickly shut off inflow to the impoundment. The Agency does not care how this is accomplished so long as it can be done quickly and without causing significant environmental or human health problems elsewhere. Possible options might include provision of redundant units or immediate shutdown of production operations feeding the impoundment.

Many ponds and lagoons are designed, either purposefully or circumstantially, to collect run-off from adjacent plant areas or even sometimes from whole watersheds. Depending on the magnitude of rainfall events and the size of the drainage area relative to the impoundment capacity, storm run-off can provide an overwhelming inflow to the impoundment, causing overtopping. Unless it can be shown at the time of permitting that the impoundment is designed to handle storm water without appreciable loss of freeboard or other detrimental effects on the impoundment, a run-on control system should be installed. The Agency recommends that the capacity of the run-on control system be designed to divert the maximum flow from a 100-year storm event. This is the most severe storm event for which historical meteorologic data is normally available. Choice of this rather stringent storm event indicates the Agency's level of concern over potential overtopping of surface impoundments containing hazardous wastes. This level of concern stems not only from the inherent threat posed by the uncontrolled escape of hazardous wastes into the environment but also from the potential for overtopping to threaten the very stability of the dike itself; leading to a possible complete washout with accompanying catastrophic results. Nevertheless, there are some events, including storm events with greater than a 100 year severity, that separately or in combination can result in overtopping, which must be ignored as a practical matter. Many of these border on the absurd such as the remote possibility of an airplance crash in the impoundment. Others are improbable combinations of events such as the possibility that all liquid containing storage tanks in a manufacturing operation will break at once, releasing their contents to the sewer system feeding the impoundment, causing it to overflow. EPA does intend such events to be protected against. Judgement must be exercised during the permit process in dealing with the more remote possibilities.

APPENDIX A

METHODS FOR DETERMINING SATURATED HYDRAULIC CONDUCTIVITY, SATURATED LEACHATE CONDUCTIVITY, AND INTRINSIC PERMEABILITY

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METHOD SW-846

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 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 parts of the RCRA Guidance Documents and associated Part 264 Standards and these test methods is provided by Table A.

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.

In this report, the laboratory and field methods that are considered the most appropriate to meeting the requirements of

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 system	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.

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.

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.

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 reference to standard texts and scientific journals.

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.

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.

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) is 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:

$$h = h_p + h_z \tag{1}$$

where

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

h_p is the pressure potential caused by the weight of fluid above the point in question, L.

h_p 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},

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 is the fluid potential when water is the fluid.

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

1.2.5 Fluid conductivity is 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:

$$K = \frac{k\rho g}{\mu}; \tag{2}$$

where

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

k is the intrinsic permeability, a property of the porous medium alone, L^{-2};

μ 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:

$$q = \frac{Q}{A} = KI, \tag{3}$$

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 discussed in this report. The range of validity of Darcy's law is discussed in Section 1.4 (Lohman, 1972).

1.2.6 Leachate conductivity is the fluid conductivity when leachate is the fluid.

1.2.7 Aquifer is 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 is 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}], is 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:

$$K_f = \frac{K_t \mu_t \rho_f}{\mu_f \rho_t} , \tag{4}$$

where

the subscript f refers to field conditions, and
t refers to test conditions.

Most temperature corrections are necessary because of the viscosity dependence on temperature. Fluid density variations caused by temperature changes are usually very small for most liquids. The temperature correction for water can be significant. A temperature decrease from 75°C to 10°C results in a 68 percent reduction in viscosity and hence hydraulic conductivity. 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 exists that shows that this alteration occurs 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 disposal site. Special considerations for using non-aqueous fluids are given in Section 3.3 of this report.

1.4 INTRINSIC PERMEABILITY

Rearrangement of Equation 2 results in a definition of intrinsic permeability.

$$k = \frac{K\mu}{\rho g} \quad . \tag{5}$$

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 low gradient 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:

$$Re = \frac{\rho q d}{\mu}, \qquad (6)$$

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:

$$I \leq \frac{10\mu}{\rho K D_{50}}, \qquad (7)$$

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 would be to measure 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 to determine fluid conductivity into two divisions: laboratory and field methods. Ideally, compliance with the Part 264 disposal facility requirements should be evaluated by using field methods that test the materials under in-situ conditions whenever possible. In general, field methods can usually provide more representative values than laboratory methods because they test a larger volume of material, thus integrating the effects of macro-structure 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, to be fairly thick, or to 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 result in the requirement to determine the fluid conductivity of liner and cover materials 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.

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.

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 METHODS

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 judgment 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:

- 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.

  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.

- For purposes 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 investigations, sampling, and material classification.

2.2 CONSTANT-HEAD METHODS

The constant-head method is the simplest method to determine 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

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:

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

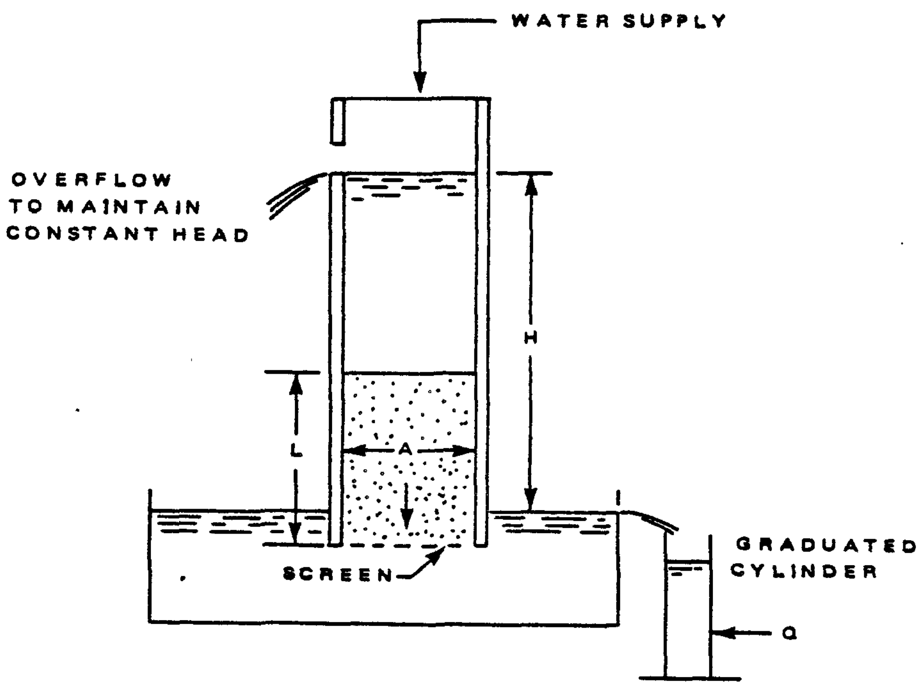

Figure 1.--Principle of the constant head method

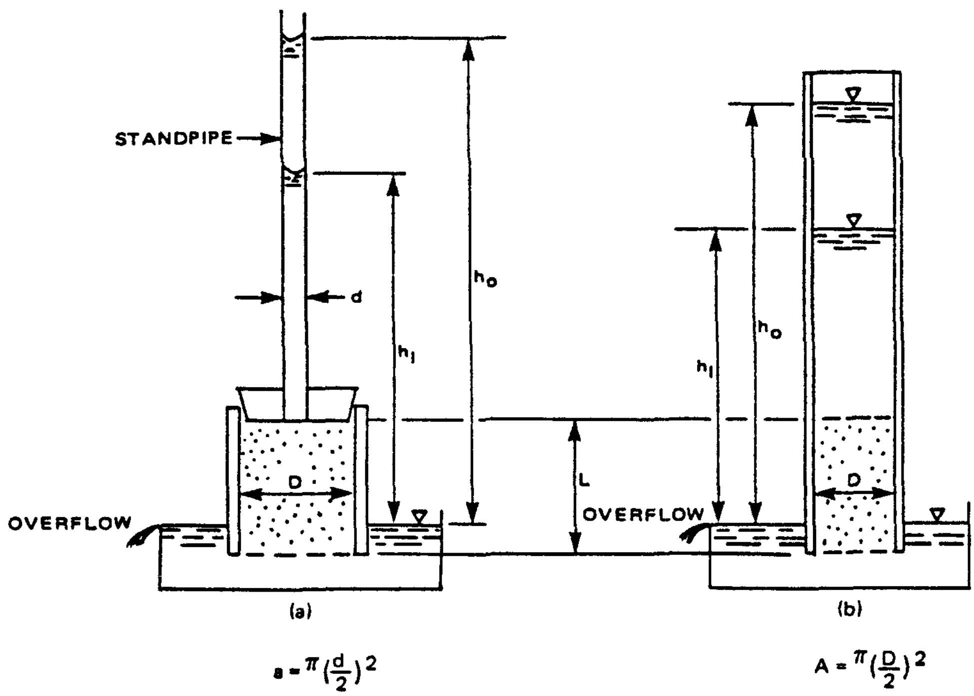

Figure 2.--Principle of the falling head method using a small (a) and large (b) standpipe.

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

t = elapsed time from t_1 to t_2, T.

For the setup shown 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.

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.

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:

a. A permeameter cylinder having a diameter at least 8 times the diameter of the largest particle of the material to be tested,

b. Constant-head filter tank,

c. Perforated metal disks and circular wire to support the sample,

d. Filter materials such as Ottawa sand, coarse sand, and gravel of various gradations,

e. Manometers connected to the top and bottom of the sample,

f. Graduated cylinder, 100 ml capacity,

g. Thermometer,

h. Stop watch,

i. Deaired water,

j. Balance sensitive to 0.1 gram, and

k. Drying oven.

2.5.3 Sample Preparation

1. Oven-dry the specimen. Allow it to cool, and weigh to the nearest 0.1 g. Record the oven-dry weight of material on a data sheet as W_s. 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 openings 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.

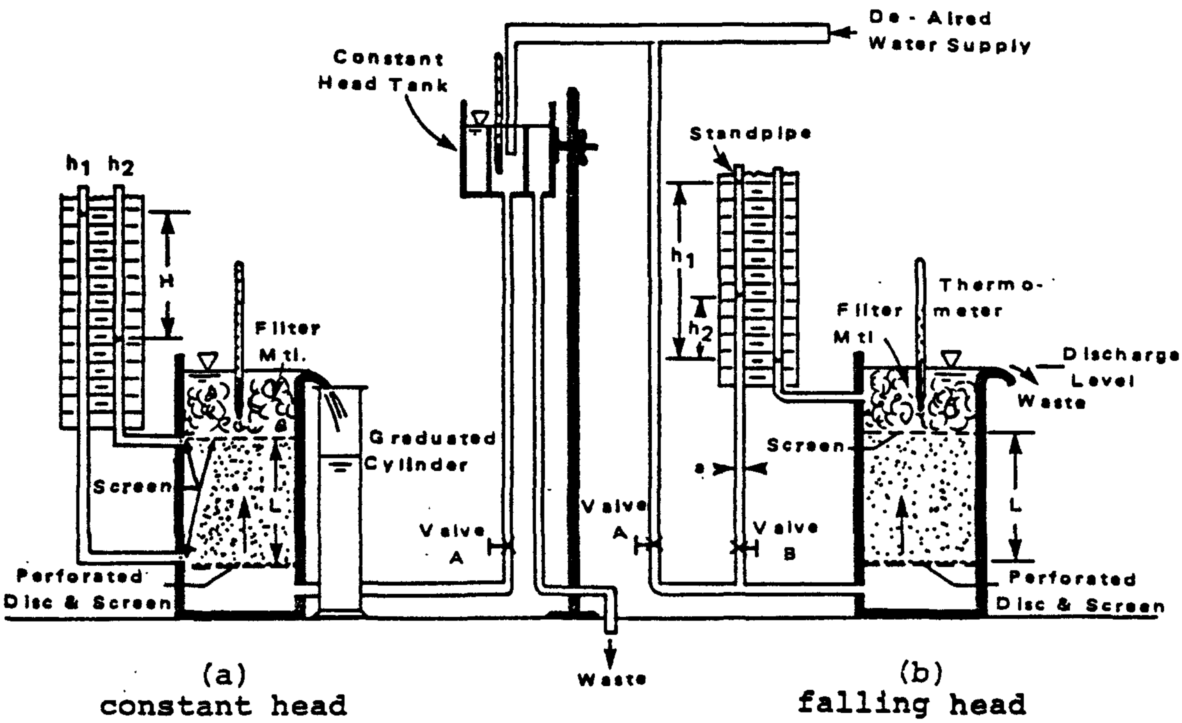

Figure 3.-- Apparatus setup for the constant head (a) and falling head (b) methods.

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 may be formed at the surface of the compacted layer which might affect the permeability results. 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 <u>Test Procedure</u>

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. The 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 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

(7) Thermometer.

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.

5. Observe and record the temperature of the water in the permeameter.

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).

2.7.2 Apparatus

The apparatus is shown in Figure 4 and consists of the equipment and accessories as follows:

a. 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,

b. Fluid Chamber - A compaction mold sleeve having the same diameter as the soil chamber,

c. 2 kg hammer,

d. Rubber rings used for sealing purposes,

e. A coarse porous stone having higher permeability than the tested sample,

f. Regulated source of compressed air, and

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

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, 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). Air pressure must be applied gradually 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 portion of the curve is defined.

2.7.5 Calculations

The hydraulic conductivity can be calculated using Equation 8.

2.8 TRIAXIAL-CELL METHOD WITH BACK PRESSURE

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.

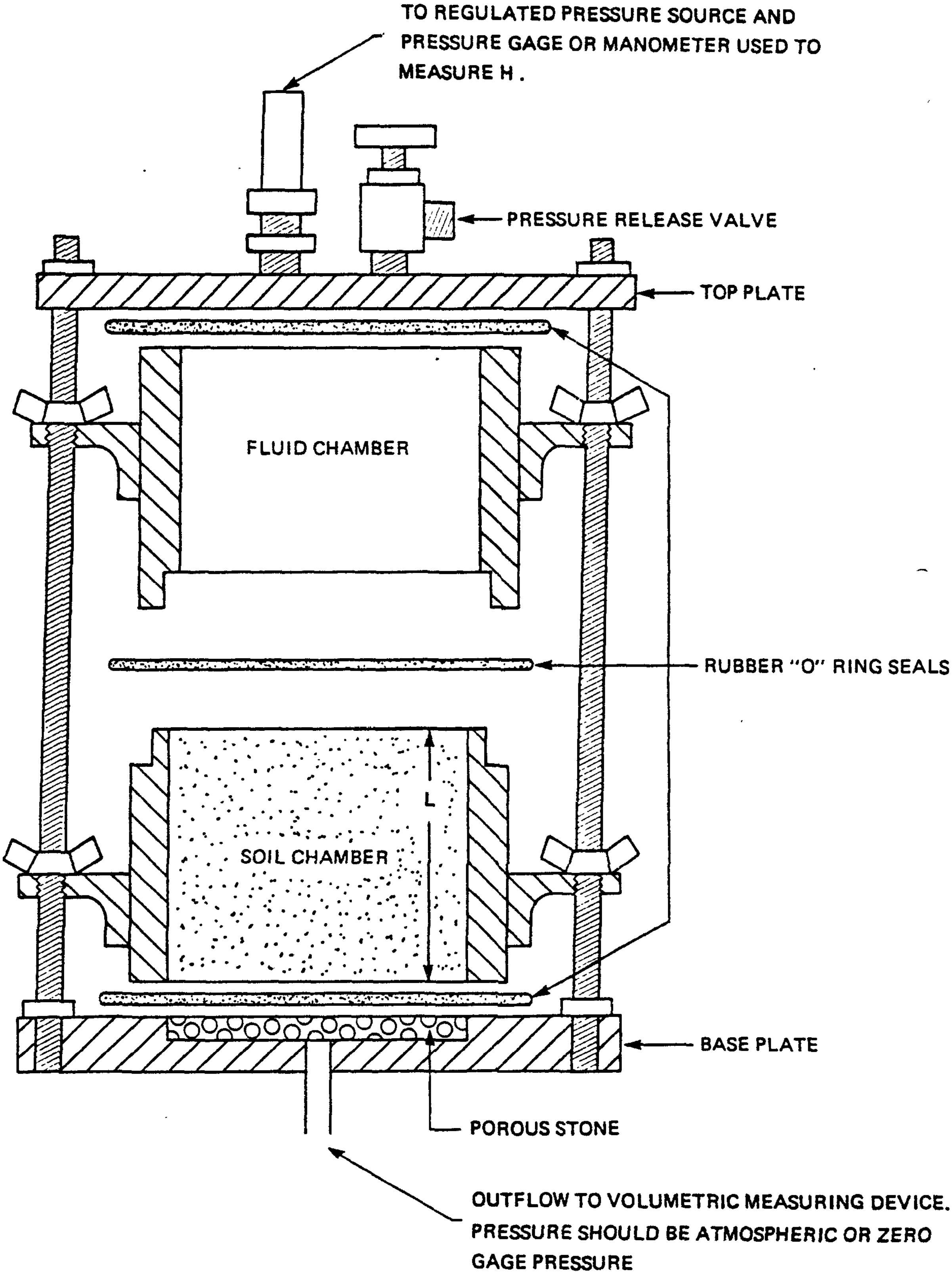

Figure 4.--Modified compaction permeameter. Note: h in Equation 8 is the difference between the regulated inflow pressure and the outflow pressure. Source: Anderson and Brown, 1981.

2.8.2 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 as shown in Figure 9. Place the specimen cap on the top of 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.

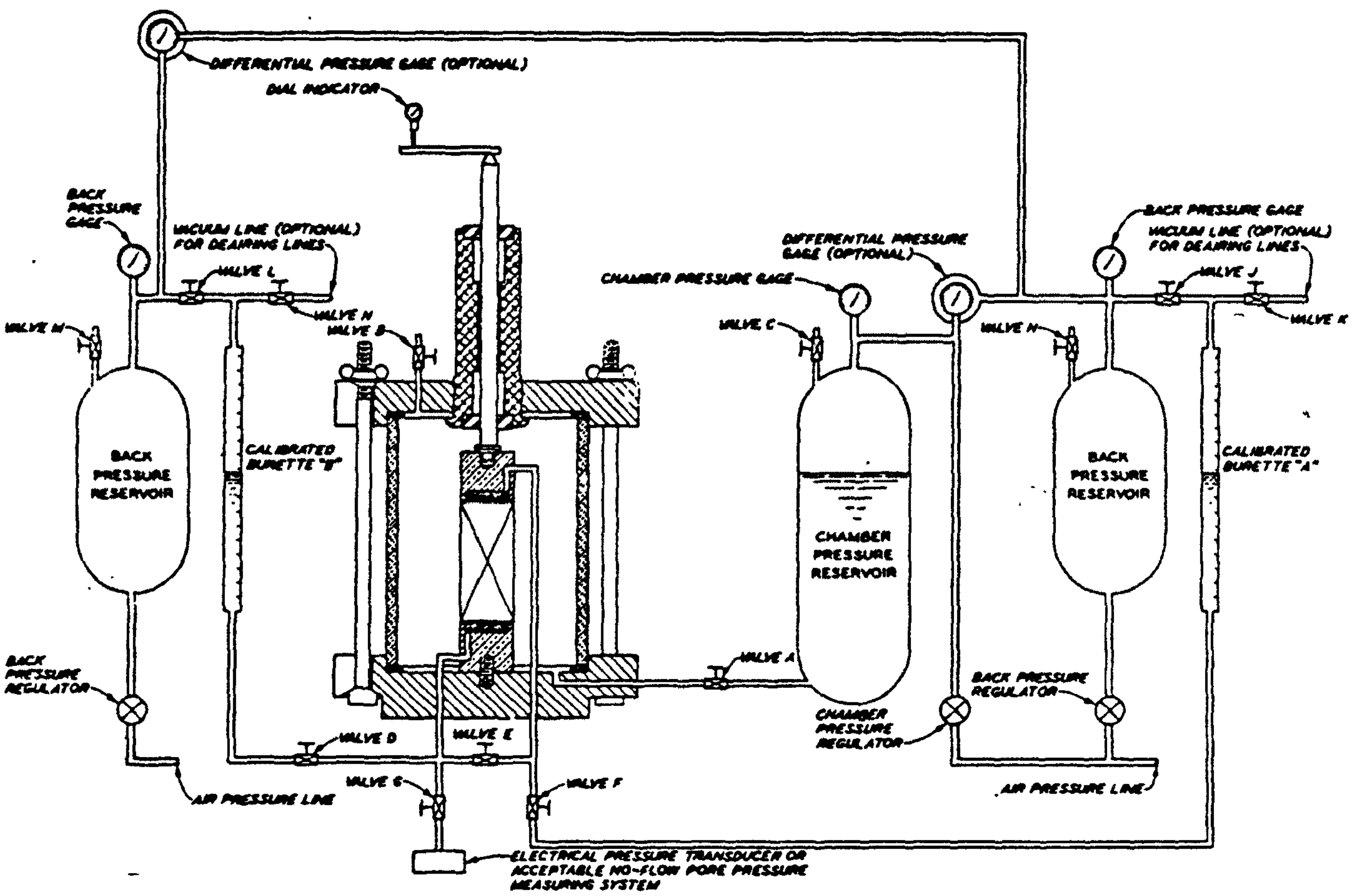

Figure 5.--Schematic diagram of typical triaxial compression apparatus for hydraulic conductivity tests with back pressure.
Source: U.S. Army Corps of Engineers, 1970

(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.

(6). Specimens should be completely saturated before any appreciable consolidation is permitted, for ease and uniformity of saturation, as well as to allow volume changes during consolidation to be measured with the burette; 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, as shown in Figure 6, consists of

a. pressure chamber,
b. standpipe,
c. specimen cap and base, and
d. coarse porous plates.

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.

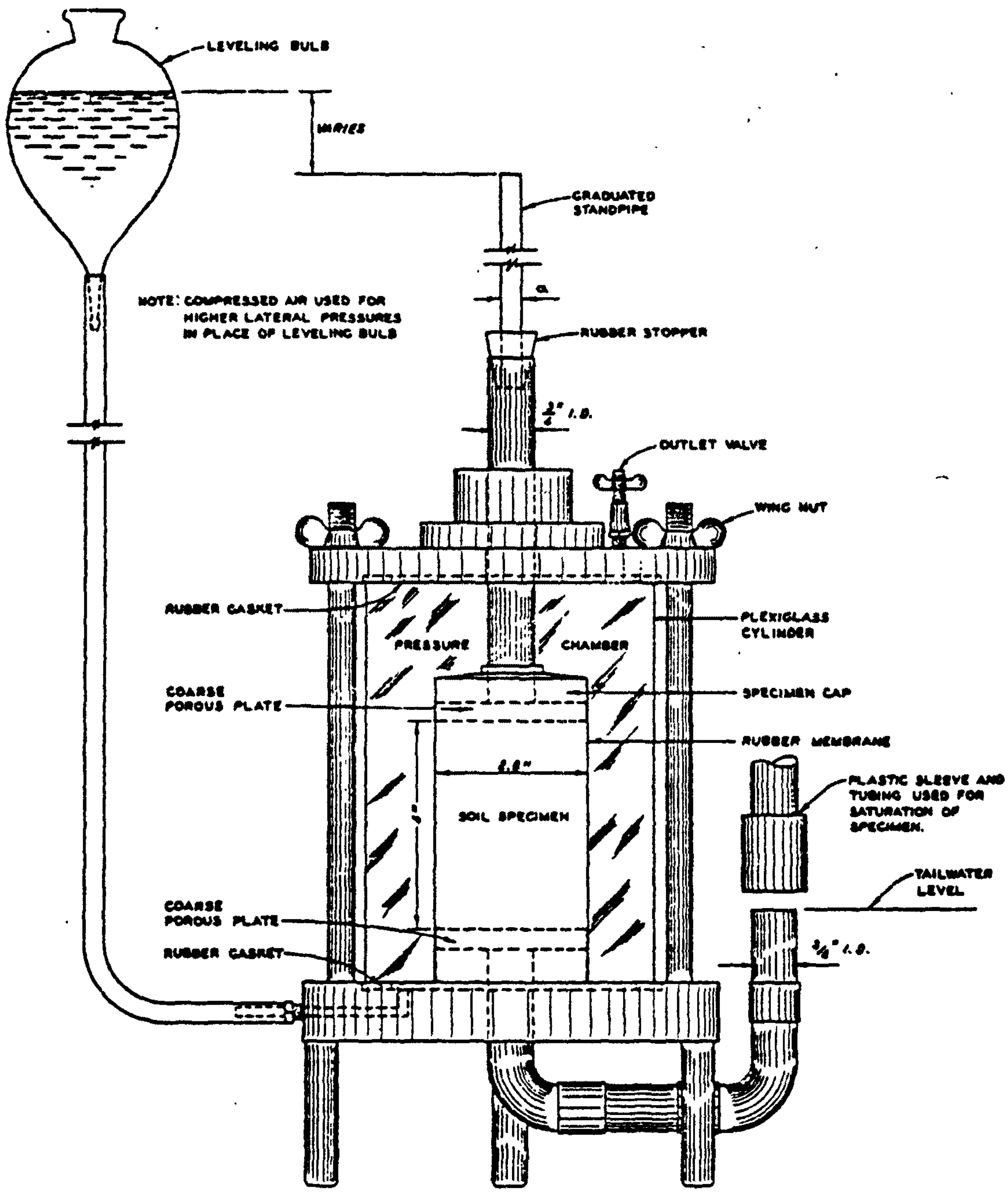

Figure 6.--Pressure chamber for hydraulic conductivity.
Source: U.S. Army Corps of Engineers, 1980.

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. Table B summarizes some potential errors that can occur. Olson 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 and References	Measured K Too Low or Too High?
1.	Voids Formed in Sample Preparation (Olson and Daniel, 1981).	High
2.	Smear Zone Formed During Trimming (Olson 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. (No confining pressure used).	High
9.	Performing Laboratory Rather than In-Situ Tests (Olson 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

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 (1980, SW-870; 1982, SW-846) and Conway and Malloy (1981) discuss 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 conductivity versus pore volume curve 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.

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 (1978), pages 160-180, 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. Determination 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 the 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.

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.

a. Hollow-stem auger,
b. Cable tool,
c. Air rotary,
d. Rotary drilling with non-organic drilling fluids,
e. Air foam rotary, and
f. 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.

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.

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.

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 is not required to prevent caving then, 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.

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).

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.

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.

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 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 applies 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 an 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.

3.4.1.2 Procedures. The slug test is run by utilizing some method of removing 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

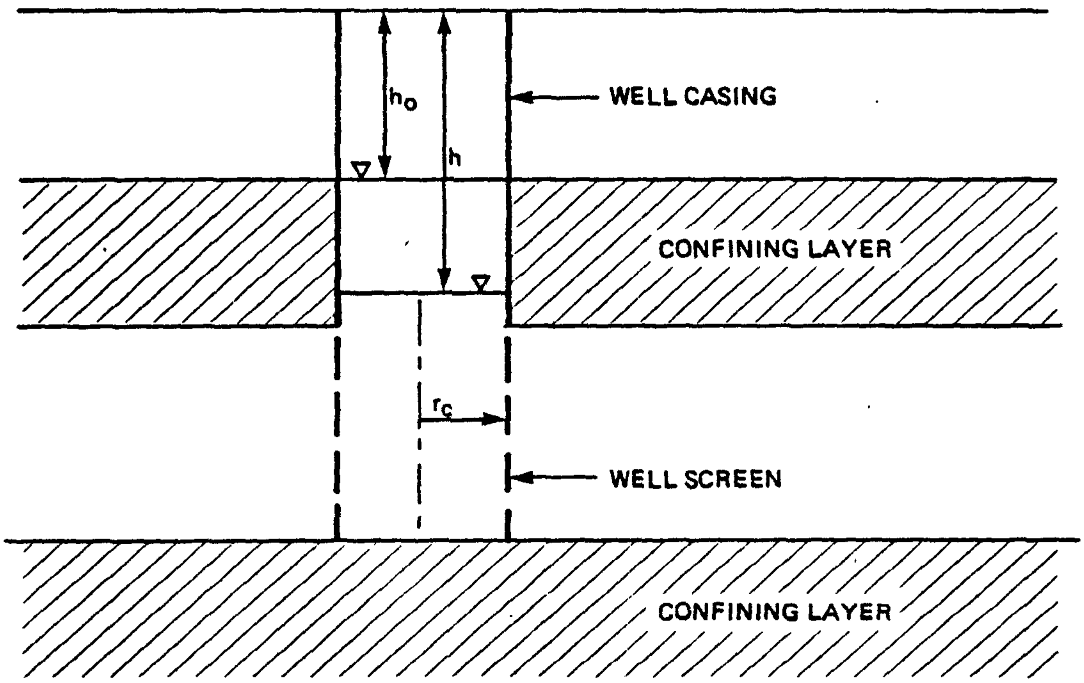

Figure 7.--Geometry and variable definition for slug tests in confined aquifers.

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.

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

a. 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 h_o is the fluid potential before the instant of fluid withdrawal. 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, 1972, p. 29).

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 detemining 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.

b. Determine the hydraulic conductivity by dividing the transmissivity 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.

<u>Well construction and development errors</u>. 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 can result from determining or recording the fluid level in the borehole and the time of measurement incorrectly. 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 α, 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 α 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 pressurizing suddenly 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

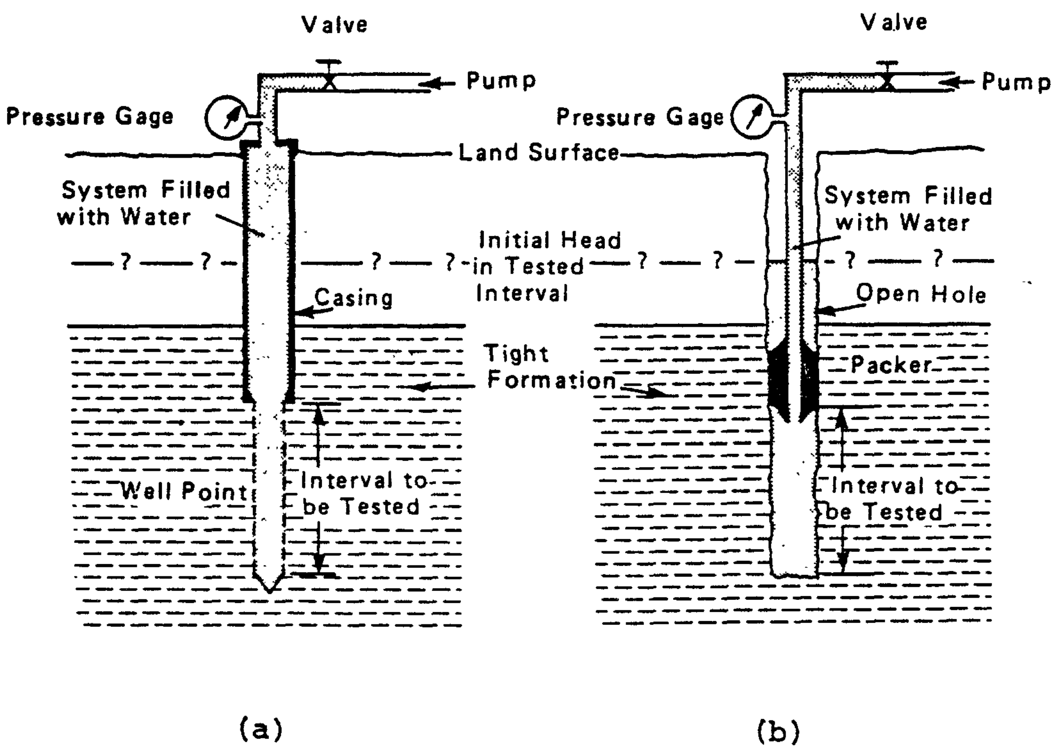

Figure 8.--Schematic diagram for pressurized slug test method in unconsolidated (a) and consolidated (b) materials. Source: Papadopulos and Bredehoeft, 1980.

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 procedure is the same in either case. Data is plotted in the same manner as for the conventional slug test and type curves are used from either Lohman (1972, Plate 2) or plotted from data given in Appendix A as described in Section 3.1.1.3. The value 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^2 by the quantity (V_w $C_w \rho$ g/π) 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 accleration of gravity, LT^{-2}.

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:

$$C_o = \frac{\Delta V}{V \Delta P} \tag{11}$$

(Neuzil, 1982, page 440). 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 <u>Methods for Moderately Permeable Materials Under Unconfined Conditions</u>

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

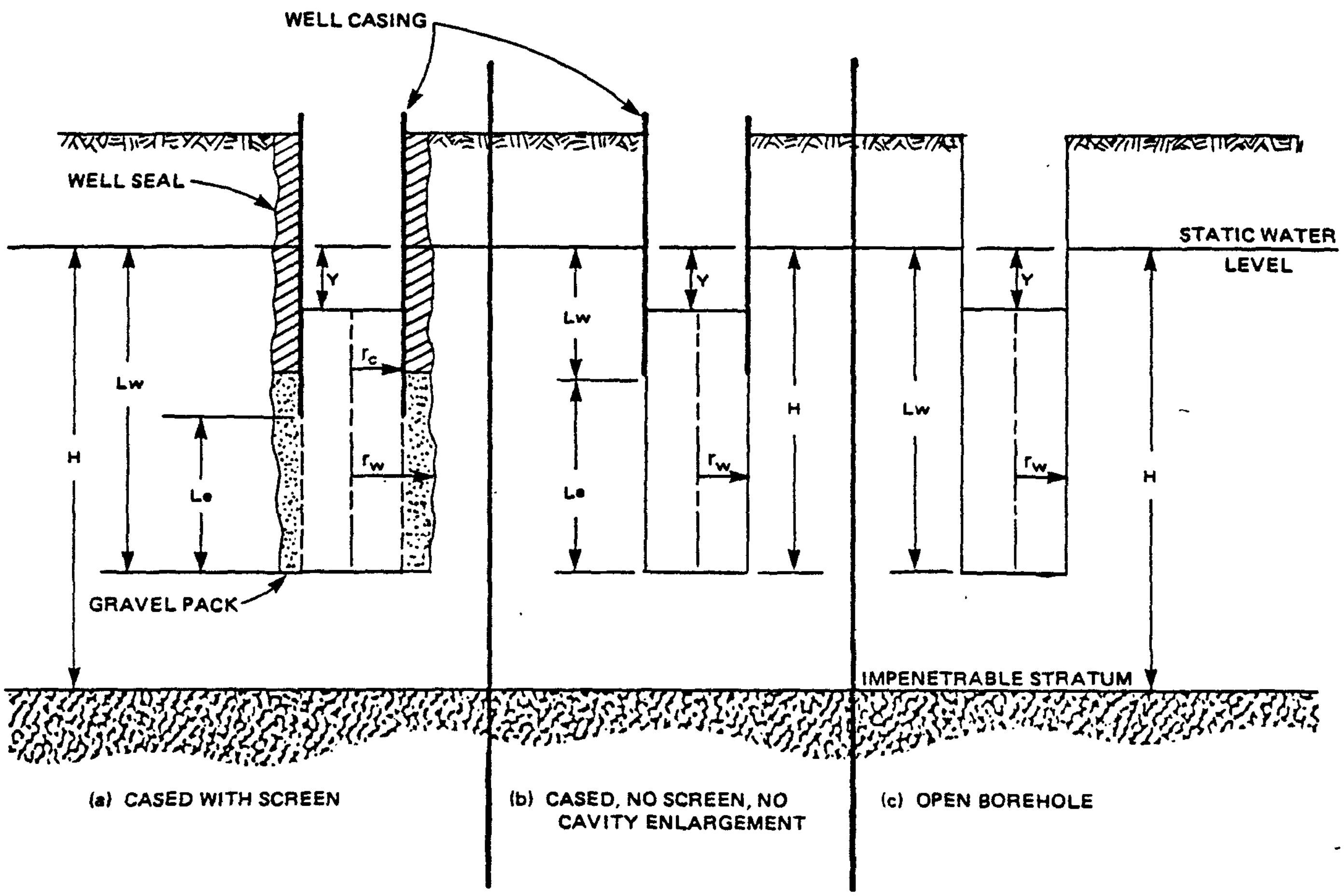

Figure 9.--Variable definitions for slug tests in unconfined materials. Cased wells are open at the bottom.

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 submersed float as discussed in Section 4.4.1.2. 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 in the notation of this report, taken from Bouwer and Rice (1976)

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

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 that is computed using the following equation given by Bouwer and Rice (1976).

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

for wells that do not fully penetrate the aquifer. If the quantity $(H_o-L_w)/r_w)$ is larger than 6, a value of 6 should be used.

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

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

(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, $\frac{\ln Yo/Y}{t}$.

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.

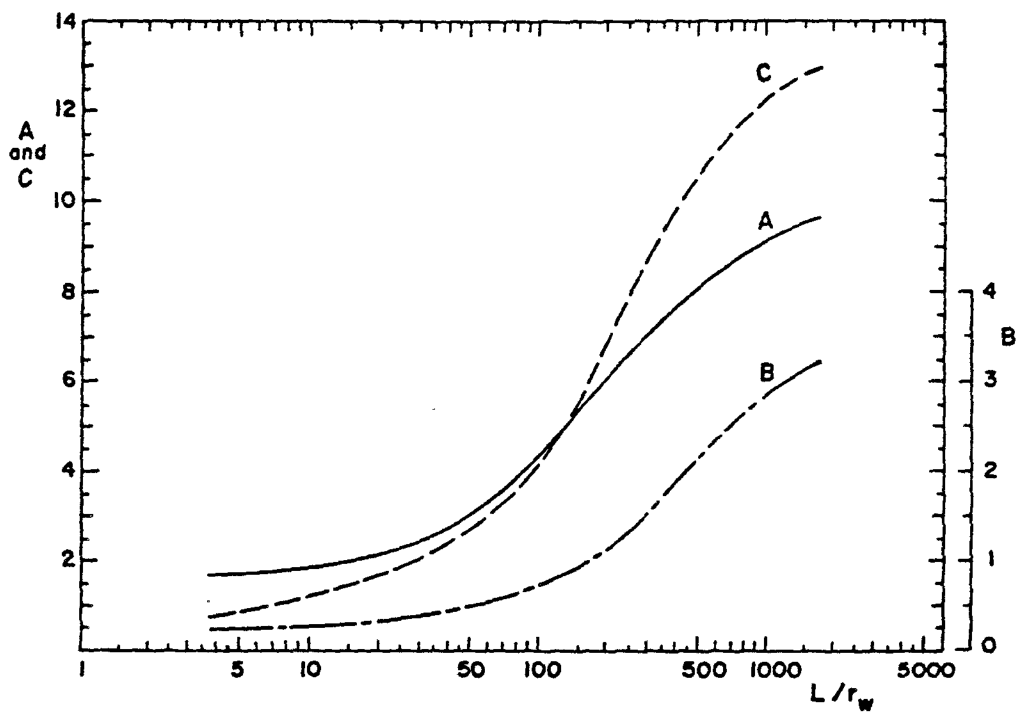

Figure 10. --Curves defining coefficients A, B, and C in equations 13 and 14 as a function of the ratio L/rw. Source: Bower and Rice, 1976.

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. These methods generally test larger portions of aquifers than the single well tests discussed in Section 4.4. 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 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

To characterize the ground-water flow system to satisfy the intent of Part 264 Subpart F, estimates of the hydraulic conductivity based on grain-size analyses or visual grain-size classification may be appropriate. However, hydraulic conductivities determined by these methods are not to be used in permit applications. Several theoretical models are available to provide these estimates, with one of most widely used being

Kozeny-Carmen equation which defines the intrinsic permeability, as adapted from Bear (1972):

$$k = \frac{n^3}{(1-n)^2} \frac{D_{50}{}^2}{180} , \qquad (15)$$

where n is the effective porosity, and all other terms are as previously defined.

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.

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
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

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:

$$C_v = \frac{K}{\rho g \alpha} , \tag{16}$$

where

K is the hydraulic conductivity, LT^{-1},

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.

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, so that the previously described methods will provide faster results. 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).

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 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 in rock. This reference should be consulted for guidance in analyzing flow through fractured media.

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.

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).

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

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.

5.0 REFERENCES

Acker, W. L., III, Basic procedures for soil sampling and core drilling, Acker Drill Co., 246 p., 1974.

Allison, L. E., Soil Science, vol. 63, pp. 439-450, 1947.

American Society for Testing and Materials (ASTM), Annual Book of ASTM Standards, part 19, 1978.

Anderson, D., and K. W. Brown, Organic leachate effects on the permeability of clay liners, in Proceedings of Solid Waste Symposium, U.S. EPA, p. 119-130, 1981.

Bear, J., Dynamics of fluids in porous media, American Elsevier, 764 p., 1972.

Bouwer, H., Groundwater hydrology, McGraw Hill, 480 p., 1978.

Bouwer, H., and R. C. Rice, A slug test for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells, Water Resources Research, 12, p. 423-428, 1976.

Bredehoeft, J. D., and S. S. Papadopulos, A method for determining the hydraulic properties of tight formations, Water Resources Research, 16, p. 233-238, 1980.

Conway, R. A., and B. C. Malloy (eds.), Hazardous Solid Waste Testing: first conference, ASTM Special Technical Publication 760, 1981.

Cooper, H. H., J. D. Bredehoeft, and I. S. Papadopulos, Response of a finite diameter well to an instantaneous charge of water, Water Resources Research, 3, p. 263-269, 1967.

Fireman, M., Soil Science, vol. 58, pp. 337-355, 1944.

Freeze, R. A., and J. A. Cherry, Groundwater, Prentice Hall, 604 p., 1979.

Gordon, B. B., and M. Forrest, Permeability of soil using contaminated permeant, in Permeability and Groundwater Contaminant Transport, ed. T. F. Zimmie and C. O. Riggs, ASTM Special Technical Publication 746, p. 101-120, 1981.

Hillel, D., Soil and Water, Academic Press, 288 p., 1971.

Hvorslev, M. J., Time lag and soil permeability in groundwater observations, U.S. Army Corps of Engineers Waterways Experiment Station Bull. 36, 1951.

Johnson, A. I., Symposium on soil permeability, ASTM STP 163, American Society of Testing and Materials, Philadelphia, pp. 98-114, 1954.

Johnson, E. E., Inc., Groundwater and wells, Johnson Division, UOP, 440 p., 1975.

Lappala, E. G., Quantitative hydrogeology of the Upper Republican Natural Resources District, Southwest Nebraska, U.S. Geological Survey Water Resources Investigations 78-38.

Lambe, T. W., Soil testing for engineers, John Wiley, N.Y., 1951.

Lohman, S. W., Groundwater hydraulics, U.S. Geological Survey Professional Paper 708, 70 p., 1972.

Lohman, S. W., and others, Definitions of selected ground water terms - revisions and conceptual refinements, U.S. Geological Survey Water Supply Paper 1988, 1972.

Manufacturing Chemists Association, Guide for safety in the chemical laboratory, Van Nostrand, Reinhold Co., N.Y., 1971.

Mitchell, A. K., and J. S. Younger, Permeability and capillarity of soils, ASTM STP 417, American Society for Testing and Materials, Philadelphia, pp. 106-139, 1967.

Neuzil, C. E., On conducting the modified 'slug' test in tight formations, Water Resources Research, vol. 18, no. 2, pp. 439-441, 1982.

Olsen, R. E., and D. E. Daniel, Measurement of the hydraulic conductivity of fine-grained soils, in Permeability and Groundwater Transport, ed. T. F. Zimmie and C. O. Riggs, ASTM Special Publication 746, p. 18-64, 1981.

Papadopulos, S. S., J. D. Bredehoeft, and H. H. Cooper, Jr., On the analysis of "slug test' data, Water Resources Research, 9, p. 1087-1089, 1973.

Schwartzendruber, D., Soil Science Society of America Proceedings, vol. 32, no. 1, pp. 11-18, 1968.

Sowers, G. F., Rock permeability or hydraulic conductivity - an overview, in Permeability and Groundwater Transport, ed. T. F. Zimmie and Co. O. Riggs, ASTM Special Technical Publication 746, 1981.

Stallman, R. W., Aquifer-test design, observation and data analysis, TWRI, Chap. B1, Book 3, U.S. Geological Survey, U.S. Govt. Printing Office, WAshington, D.C., 1971.

Terzaghi, K., and R. B. Peck, Soil Mechanics in Engineering Practice, 2nd Ed., John Wiley & Sons, N.Y., 729 p., 1967.

Walton, W. C., Groundwater resource evaluation, McGraw Hill, 664 p., 1970.

Wilkinson, W. B., In situ investigation in soils and rocks, British and Geotechnical Society, Institution of Civil Engineers, London, pp. 311-313, 1969.

U.S. Army Corps of Engineers, Laboratory Soil Testing, Waterways Experiment Station Publication EM1110-2-1906, 1970.

U.S. Environmental Protection Agency, Lining of waste impoundment and disposal facilities, Office of Solid Waste Report SW-870, 1980.

U.S. Environmental Protection Agency, Hazardous waste guidelines and regulations (proposed), Federal Register, Part IV, Dec. 18, 1978.

U.S. Environmental Protection Agency, Test Methods for Evaluating Solid Waste, 2nd Edition, SW-846, Superintendent of Documents, Government Printing Office, Washington, D.C., 1982.

6.0 Appendix A

Values of the function F(α,β) for use in the conventional and pressurized slug tests. Source: Papadopulos et.al. (1973)

Tt/r_c^2	$\alpha = 10^{-6}$	$\alpha = 10^{-7}$	$\alpha = 10^{-8}$	$\alpha = 10^{-9}$	$\alpha = 10^{-10}$
0.001	0.9994	0.9996	0.9996	0.9997	0.9997
0.002	0.9989	0.9992	0.9993	0.9994	0.9995
0.004	0.9980	0.9985	0.9987	0.9989	0.9991
0.006	0.9972	0.9978	0.9982	0.9984	0.9986
0.008	0.9964	0.9971	0.9976	0.9980	0.9982
0.01	0.9956	0.9965	0.9971	0.9975	0.9978
0.02	0.9919	0.9934	0.9944	0.9952	0.9958
0.04	0.9848	0.9875	0.9894	0.9908	0.9919
0.06	0.9782	0.9819	0.9846	0.9866	0.9881
0.08	0.9718	0.9765	0.9799	0.9824	0.9844
0.1	0.9655	0.9712	0.9753	0.9784	0.9807
0.2	0.9361	0.9459	0.9532	0.9587	0.9631
0.4	0.8828	0.8995	0.9122	0.9220	0.9298
0.6	0.8345	0.8569	0.8741	0.8875	0.8984
0.8	0.7901	0.8173	0.8383	0.8550	0.8686
1.0	0.7489	0.7801	0.8045	0.8240	0.8401
2.0	0.5800	0.6235	0.6591	0.6889	0.7139
3.0	0.4554	0.5033	0.5442	0.5792	0.6096
4.0	0.3613	0.4093	0.4517	0.4891	0.5222
5.0	0.2893	0.3351	0.3768	0.4146	0.4487
6.0	0.2337	0.2759	0.3157	0.3525	0.3865
7.0	0.1903	0.2285	0.2655	0.3007	0.3337
8.0	0.1562	0.1903	0.2243	0.2573	0.2888
9.0	0.1292	0.1594	0.1902	0.2208	0.2505
10.0	0.1078	0.1343	0.1620	0.1900	0.2178
20.0	0.02720	0.03343	0.04129	0.05071	0.06149
30.0	0.01286	0.01448	0.01667	0.01956	0.02320
40.0	0.008337	0.008898	0.009637	0.01062	0.01190
50.0	0.006209	0.006470	0.006789	0.007192	0.007709
60.0	0.004961	0.005111	0.005283	0.005487	0.005735
80.0	0.003547	0.003617	0.003691	0.003773	0.003863
100.0	0.002763	0.002803	0.002845	0.002890	0.002938
200.0	0.001313	0.001322	0.001330	0.001339	0.001348

6.0 Appendix A (continued)

Extended values of F(α,β) for use in slug tests.
Source: Bredehoeft and Papadopulos (1980)

β	α=0.1	α=0.2	α=0.5	α=1	α=2	α=5	α=10
0.000001	0.9993	0.9990	0.9984	0.9977	0.9968	0.9948	0.9923
0.000002	0.9990	0.9986	0.9977	0.9968	0.9955	0.9927	0.9894
0.000004	0.9986	0.9980	0.9968	0.9955	0.9936	0.9898	0.9853
0.000006	0.9982	0.9975	0.9961	0.9945	0.9922	0.9876	0.9822
0.000008	0.9980	0.9971	0.9955	0.9936	0.9910	0.9857	0.9796
0.00001	0.9977	0.9968	0.9949	0.9929	0.9900	0.9841	0.9773
0.00002	0.9968	0.9955	0.9929	0.9900	0.9858	0.9776	0.9683
0.00004	0.9955	0.9936	0.9899	0.9858	0.9801	0.9687	0.9558
0.00006	0.9944	0.9922	0.9877	0.9827	0.9757	0.9619	0.9464
0.00008	0.9936	0.9909	0.9858	0.9800	0.9720	0.9562	0.9387
0.0001	0.9929	0.9899	0.9841	0.9777	0.9688	0.9512	0.9318
0.0002	0.9898	0.9857	0.9776	0.9687	0.9562	0.9321	0.9059
0.0004	0.9855	0.9797	0.9685	0.9560	0.9389	0.9061	0.8711
0.0006	0.9822	0.9752	0.9615	0.9465	0.9258	0.8869	0.8458
0.0008	0.9794	0.9713	0.9557	0.9385	0.9151	0.8711	0.8253
0.001	0.9769	0.9679	0.9505	0.9315	0.9057	0.8576	0.8079
0.002	0.9670	0.9546	0.9307	0.9048	0.8702	0.8075	0.7450
0.004	0.9528	0.9357	0.9031	0.8686	0.8232	0.7439	0.6684
0.006	0.9417	0.9211	0.8825	0.8419	0.7896	0.7001	0.6178
0.008	0.9322	0.9089	0.8654	0.8202	0.7626	0.6662	0.5797
0.01	0.9238	0.8982	0.8505	0.8017	0.7400	0.6384	0.5492
0.02	0.8904	0.8562	0.7947	0.7336	0.6595	0.5450	0.4517
0.04	0.8421	0.7980	0.7214	0.6489	0.5654	0.4454	0.3556
0.06	0.8048	0.7546	0.6697	0.5919	0.5055	0.3872	0.3030
0.08	0.7734	0.7190	0.6289	0.5486	0.4618	0.3469	0.2682
0.1	0.7459	0.6885	0.5951	0.5137	0.4276	0.3168	0.2428
0.2	0.6418	0.5774	0.4799	0.4010	0.3234	0.2313	0.1740
0.4	0.5095	0.4458	0.3566	0.2902	0.2292	0.1612	0.1207
0.6	0.4227	0.3642	0.2864	0.2311	0.1817	0.1280	0.09616
0.8	0.3598	0.3072	0.2397	0.1931	0.1521	0.1077	0.08134
1.	0.3117	0.2648	0.2061	0.1663	0.1315	0.09375	0.07120
2.	0.1786	0.1519	0.1202	0.09912	0.08044	0.05940	0.04620
4.	0.08761	0.07698	0.06420	0.05521	0.04668	0.03621	0.02908
6.	0.05527	0.04595	0.04331	0.03830	0.03326	0.02663	0.02185
8.	0.03963	0.03658	0.03254	0.02933	0.02594	0.02125	0.01771
10.	0.03065	0.02870	0.02600	0.02376	0.02130	0.01776	0.01499
20.	0.01408	0.01361	0.01288	0.01219	0.01133	0.009943	0.008716
40.	0.006680	0.006568	0.006374	0.006171	0.005897	0.005395	0.004898
60.	0.004367	0.004318	0.004229	0.004132	0.003994	0.003726	0.003445
80.	0.003242	0.003214	0.003163	0.003105	0.003072	0.002853	0.002668
100.	0.002577	0.002559	0.002526	0.002487	0.002431	0.002313	0.002181
200.	0.001271	0.001266	0.001258	0.001247	0.001230	0.001194	0.001149
400.	0.0006307	0.0006295	0.0006272	0.0006242	0.0006195	0.0006085	0.0005944
600.	0.0004193	0.0004188	0.0004177	0.0004163	0.0004141	0.0004087	0.0004016
800.	0.0003140	0.0003137	0.0003131	0.0003123	0.0003110	0.0003078	0.0003035
1000.	0.0002510	0.0002508	0.0002504	0.0002495	0.0002490	0.0002469	0.0002440

APPENDIX B

IMMERSION TEST OF MEMBRANE LINER MATERIALS FOR COMPATIBILITY WITH WASTES

DRAFT

Method 9082

IMMERSION TEST OF MEMBRANE LINER MATERIALS FOR COMPATIBILITY WITH WASTES

1.0 Scope and Application

1.1 This test method is for use in determining the effects upon the physical properties of flexible membrane liner materials intended to contain chemicals in a pit, pond, lagoon or landfill-type installation, of the chemical environment expected to be encountered. Data from these tests will assist in deciding whether a given liner material is acceptable for the intended application.

1.2 This method is based on material resulting from work by the National Sanitation Foundation, Dr. Henry E. Haxo, Dr. Robert Landreth, Matrecon, Inc. and EPA's Municipal Environmental Research Laboratory in Cincinnati, Oh.

2.0 Summary of Method

2.1 In order to estimate long term compatability, the liner material is exposed to the expected chemical environment for a period of 120 days at an elevated temperature. A comparison of the membrane's physical properties before and after this contact period is used to estimate the properties of the liner at the time of site closure.

3.0 Interferences

4.0 Apparatus and Materials

4.1 Exposure tank - A size sufficient to contain the samples with provisions for [illegible] they do not touch bottom or sides of the tank, or each other; for stirring the liquid in the tank; and for holding the specimens in such a manner that the liner material contacts the test solution only on the surface that would face the waste in an actual disposal site. The tank shall be equipped with a means of maintaining the solution at a temperature of 50±2°C and for preventing evaporation of the solution (e.g., cover equipped with a reflux condenser). The Agency understands that one such device is manufactured by Associated Design and Manufacturing Company, 814 North Henry Street, Alexandria, VA 22314, (703)549-5999.

4.2 Stress-strain machine suitable for measuring elongation, tensile strength, tear and puncture resistance.

4.3 Jig for testing puncture resistance.

4.4 Labels and holders for specimens, of materials known to be resistant to the specific wastes. Holders of stainless steel, and tags made of 50 mil polypropylene, embossed with machinist's numbering dies and fastened with stainless steel wire, are resistant to most wastes.

5.0 Reagents

6.0 Sample Collection Preservation and Handling

6.1 For information on what constitutes a representative sample of the waste fluid to employ, refer to the appropriate guidance document listed below:

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

July 1982 and used with 40 CFR 254.221(a) and (c), and 264.228(a).

2. RCRA Guidance Document: Waste Pile Design, Liner Systems. Issued July 1982 and used with 40 CFR 264.251(a), 264.252, and 264.253.

3. RCRA Guidance Document: Landfill Design, Liner Systems and Final Cover. Issued July 1982 and used with 40 CFR 264.301(a) and 264.310(a).

7.0 <u>Procedure</u>

7.1 Obtain a representative sample of the waste fluid.

7.2 Perform the following tests on <u>unexposed</u> samples of the polymeric membrane liner material:

7.2.2 Puncture resistance, three specimens

7.2.3 Tensile strength in two directions, three specimens in each direction

7.2.4 Elongation at Break, (This test is only to be performed on membrane material which does not have a fabric or other non-elastomeric support on its reverse [away from waste] face.)

Tests are to be performed according to the protocols referenced in Table 9082-1. See Figure 9082-1 for a suggested cutting pattern.

7.3 Cut pieces of the lining material, of a size to fit sample holder, and of a sufficient number to permit at least three samples for each test at each test period.

7.4 Label the test specimens with a plastic identification tag and suspend in sample of the waste fluid.

7.5 Expose the sample to the stirred waste fluid held at $30 \pm 2^{\circ}C$

7.6 At the end of 30, 60, 90 and 120 days of exposure, remove sufficient specimen from the waste fluid to determine the membrane's physical properties (see 7.2). Place wet specimen in a labelled container of fresh waste fluid at room temperature for at least one hour to effect cooling prior to testing.

7.7 The sample should be tested within 24 hours of removal from the bath. To test the immersed sample, wipe off any waste fluid, rinse with deionized water, blot specimen dry, and measure the physical properties listed in 7.2.

7.8 <u>Results and Reporting</u>

7.8.1 Plot the curve for each property over the time period 0 to 120 days.

7.8.2 Report all raw, tabulated, and plotted data. Evaluation of the results is described in the RCRA guidance documents listed under 6.1.

8.0 <u>Quality Control</u>

8.1 Determine the mechanical properties of identical non-immersed and immersed specimens in accordance with the standard methods for the specific physical property test. Conduct mechanical property tests on nonimmersed and immersed specimens prepared from the same sample or lot of material in the same manner and run under identical conditions. Test immersed specimens immediately after they are removed from the room temperature test solution.

PVC(... etic thermoplastic polymer made ...

CPE (... Family of polymers produced by c... on polyethylene. The resulting t... ain 25 to 45% chlorine by weight a...

Butyl ... ased on isobutylene and a small a... ites for vulcanization.

CR (P... ne for a synthetic rubber based pr...

HDPE (... A polymer prepared by the low pr... ylene as the sole monomer.

EPDM (... mer): A synthetic elastomer ba... nd a small amount of nonconju- gat... vulcanization.

CO, ECC ... Synthetic rubber including two ... ers which are saturated, high mol... hers with chloromethyl side cha... nopolymer(CO) and a copolymer of ... ethylene oxide(ECO).

CM (Crosslinked chlorinated polyethylene): See chlorinated polyethylene.

PE-EP-A (Polyethylene ethylene/propylene alloy): A blend of polyethylene and poly(ethylene/propylene).

HDPE-A (High density polyethylene / rubber alloy): A blend of high density polyethylene and rubber.

CSPE (Chlorosulfonated polyethylene): Family of polymers that are produced by polyethylene reacting with chlorine and sulfur dioxide and usually containing 25 to 43% chlorine and 1.0 to 1.4% sulfur.

TN-PVC (Thermoplastic nitrile-polyvinyl chloride): An alloy of thermoplastic unvulcanized nitrile rubber and polyvinyl chloride.

T-EPDM (Thermoplastic EPDM): An ethylene-propylene diene monomer blend resulting in a thermoplastic elastomer.

EIA (Ethylene interpolymer alloy): A blend of polyethylene and polyvinyl chloride resulting in a thermoplastic elastomer.

PVC-CPE (Polyvinyl chloride - chlorosulfonated polyethylene alloy): A blend of polyvinyl chloride and chlorosulfonated polyethylene

TABLE 9082-1

PHYSICAL PROPERTY TESTING PROCEDURES

[Appropriate ASTM or FTMS(*) Testing Method]

Polymer	Tensile Strength & Elongation at Break	Tear Resistance	Puncture Resistance
PVC	D882	D1004 Die C	*2065
CPE	D882	D1004 Die C	*2065
Butyl rubber	D412	D624 Die C	*2065
CR	D412	D624 Die C	*2065
HDPE	D638	D1004 Die C	*2065
EPDM	D412	D624 Die C	*2065
CO, ECO	D412	D624 Die C	*2065
CM	D412	D624 Die C	*2065
PE-EP-A	D412	D1004 Die C	*2065
HDPE-A	D638 Type IV Dumbell at 2 inches/second	D1004 Die C	*2065
CSPE	D751 Method A	D751 as modified in Appendix A	*2065
TN-PVC	D751 Method A	D751 as modified in Appendix A	*2065
T-EPDM	D751 Method A	D751 as modified in Appendix A	*2065
EIA	D751 Method A	D751 as modified in Appendix A	*2065
PVC-CPE	D882 Method A	D1104 Die C	*2065

Abbreviations:

ASTM: American Society for Testing and Materials

FTMS: [illegible]

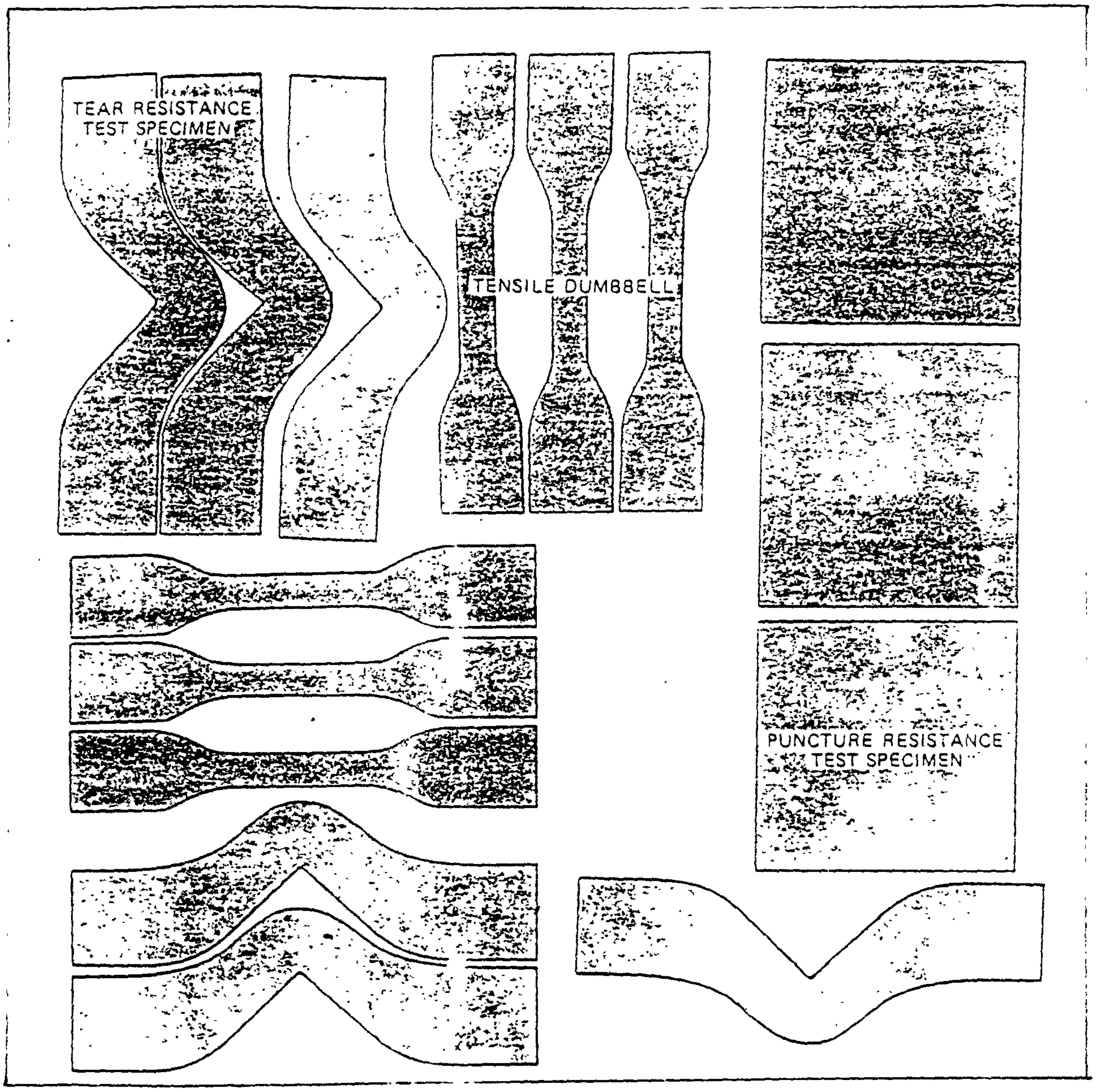

Figure 9082-1. Suggested pattern for cutting test specimens.