LEACHATE CLOGGING ASSESSMENT OF GEOTEXTILE AND SOIL LANDFILL FILTERS

by

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FOREWORD

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E. Timothy Oppelt, Director
National Risk Management Research Laboratory

ABSTRACT

The liquids management strategy for any municipal or hazardous waste landfill requires a knowledgeable design strategy for the leachate collection system located at the base of the waste mass. Such leachate collection systems generally consist of sumps, perforated pipes, drainage materials (gravel soils or geonets) and filter materials (sand soils or geotextiles). The solid waste mass lies above the filter, although sometimes a protective soil acts as a intermediate layer. As leachate migrates through the waste mass it must be intercepted, collected and transported by the leachate collection system. Since leachate is often high in suspended solids and microorganism content, concerns over excessive clogging of the leachate collection system are often expressed. More specifically, the filter is the target material due to its small opening spaces with respect to the other materials that are involved. Thus, this project is completely oriented toward the filter material of the leachate collection system. Both sand soil and geotextile filters are investigated, although more emphasis is on geotextiles due to their greater current usage in this application.

A multifaceted approach leading to a design methodology was the focus of this study. The project consisted of exhuming four sites-of-opportunity which essentially established ground truth. Three of the sites had excessively clogged geotextiles and the fourth was marginally adequate. Parallel to the field work was the laboratory evaluation of 12 commonly used filters (10 geotextiles and 2 soils) using four different permeating liquids (water and 3 different leachates) under three different accelerated leachate flow rates. This required 144 ASTM D1987 flow columns to be constructed. Each were used for time periods of up to one year in order to establish equilibrium permeability values. From this data, master curves of the twelve filters were generated. Parallel to the field and laboratory efforts just noted was a computer modeling effort (using the HELP model) to investigate flow rate sensitivity to different leachate collection materials and to obtain the required, i.e., site specific, flow rates for the four exhumed sites.

These three activities (field exhuming, laboratory testing and computer analysis) were brought together in a design model which can be used to calculate the factor-of-safety for a leachate collection system filter in any site specific and material specific design. The formula is as follows.

$$FS = \frac{k_{allow}}{k_{reqd} \times DCF}$$

where

FS = factor-of-safety against excessive filter clogging
k_{allow} = allowable permeability for the specific filter material being considered
k_{reqd} = required permeability for the site specific hydraulic and solid waste situation
DCF = drainage correction factor for the particular design geometry being considered

The DCF is a unique addition to the conventional type of factor-of-safety equations in the literature and is necessary because some designs significantly limit the available flow area downstream of

the filter. Indeed, the incorporation of the DCF completely substantiated the field findings of the exhumed sites. Example calculations of DCF for common design geometries are presented in the report.

Some ancillary studies were also included, such as the "no filter" strategy and the use of biocides within the various filter materials. Such design strategies are indeed possible but are felt to be unnecessary in light of utilization of the design method. Clearly, landfill filters can be designed so as not to result in excessive clogging of leachate collection systems. Design guidance is presented in this report. For the case of mild leachates, recommendations are offered as to typical geotextile filters which should result in conservative factors-of-safety against excessive filter clogging. For more aggressive leachates, the procedures, test methods and computer generated design flows detailed in this report must be utilized accordingly.

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1. Introduction and Scope of Project

The concept of the containment system at a landfill is to envelope the solid waste and isolate it from the surrounding environment. Insofar as total containment is concerned, landfills consist of three major parts: A liner system beneath the waste, the liquids management system within the waste, and cover system above the waste. Each system is quite complex having a number of geosynthetic and natural soil components involved. While the cross section of Figure 1 emphasizes the use of geosynthetics in double lined facilities, it also clearly identifies the three essential parts of a landfill containment system. .

This report, and the study upon which it was developed, does not address the liner or cover system, per se. Rather it addresses the liquids management system and, in particular, the leachate collection system at the bottom of the solid waste mass, see Figure 2. Irrespective of the method of ultimately handling the leachate, i.e., withdrawal on demand or leachate recycling, it must flow within a drainage system consisting of granular soil or a geonet composite to a perforated pipe network. Cross sections of several types of drainage and pipe systems are shown in Figure 3. Once the leachate drains into the pipe, the flow rate increases rapidly and it continues to flow gravitationally to a low area, i.e., a sump, where it accumulates. From the sump, a manhole or pipe riser allows for pumping and eventual removal of the leachate as per the site-specific plan and permit.

Leachate collection systems must remain functioning for the service life and postclosure care period of a landfill. Time frames of 30 years are generally required, although in reality, the postclosure period could be longer depending upon the site-specific liquids management scheme, e.g., those landfills using leachate recycling schemes.

A typical leachate collection system consists of the following components shown in Figure 2 (from the bottom of the solid waste downward):

- Filter layer—either sand or geotextile.
- Drainage material—either coarse sand, gravel, geonet or geocomposite.
- Perforated pipe—required for sand or gravel drainage materials.
- Sump—low area in the facility from which extends a vertical manhole or sidewall riser in order to remove the leachate.

Note that a "soil protection layer" above the layer has not been specifically identified. When used, this protection layer is usually native soil, and is often very low in its hydraulic conductivity or permeability. If the permeability of the protection layer is less than that of the filter, it becomes a de-facto liner and dominates the flow regime of the leachate collection system. The undesirable result of "perched leachate" in the body of the waste would result, with possible cover seeps and/or high hydraulic pressure exerted against the base and side slopes of the facility. Thus if a soil protection layer is used above the filter, its permeability should be equal to that of the filter, or higher, so as not to inhibit the flow of leachate. Note that "permeability", rather than hydraulic conductivity will be used in this report since it is the common term used in geosynthetics which is the majority of this study.

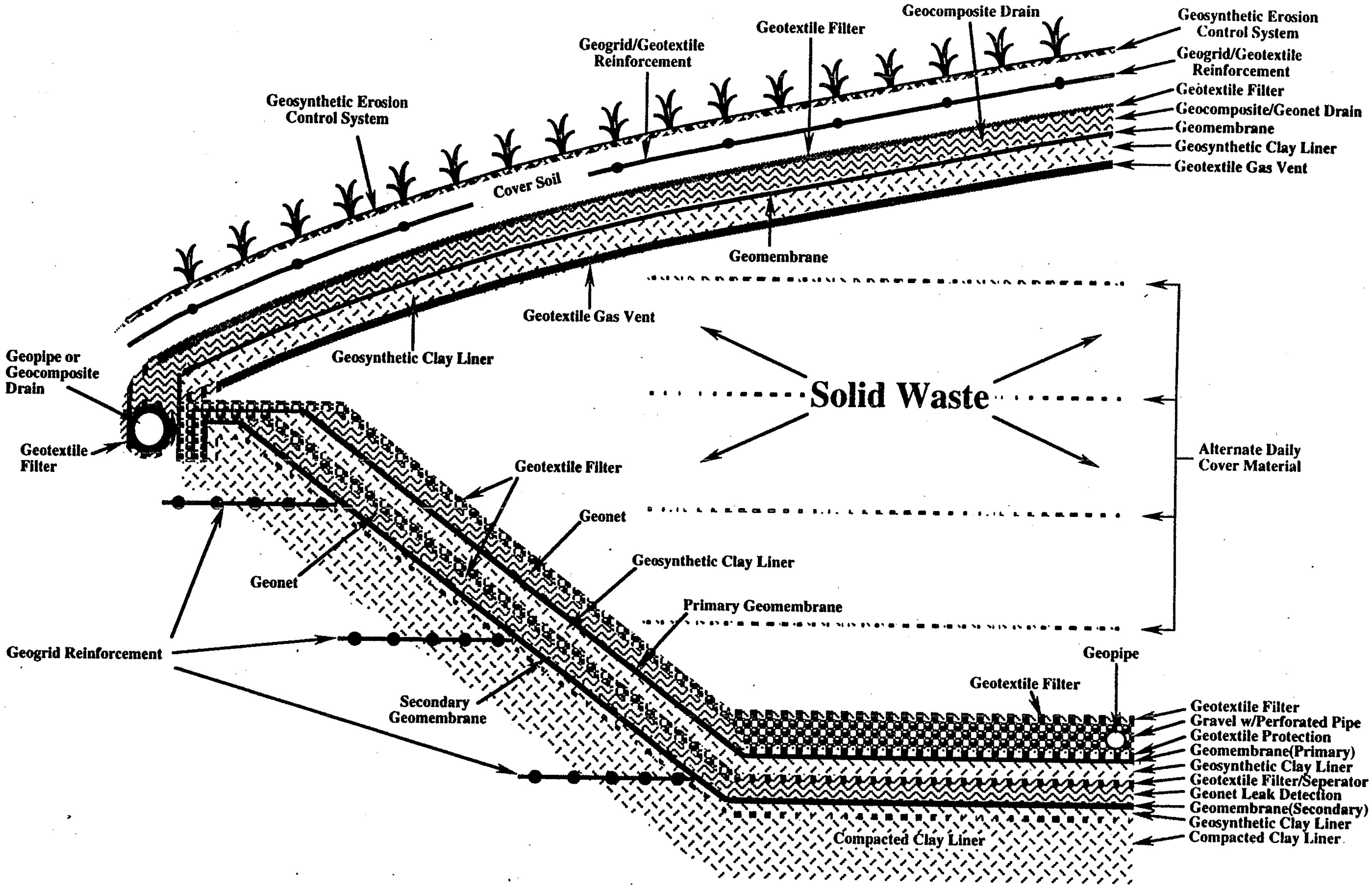


Figure 1 - Generalized Cross Section of Double Lined Landfill Liner and Cover System

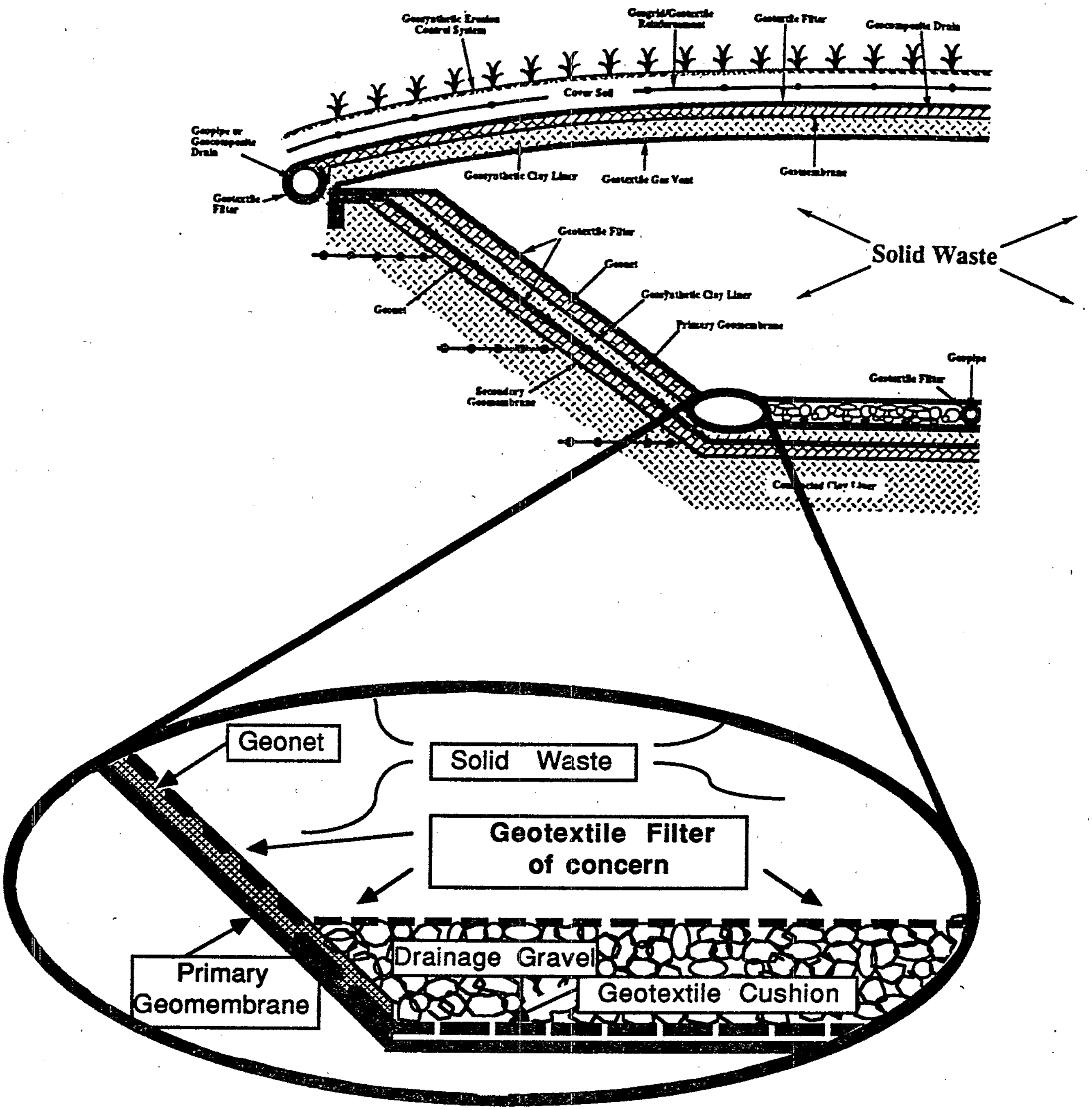
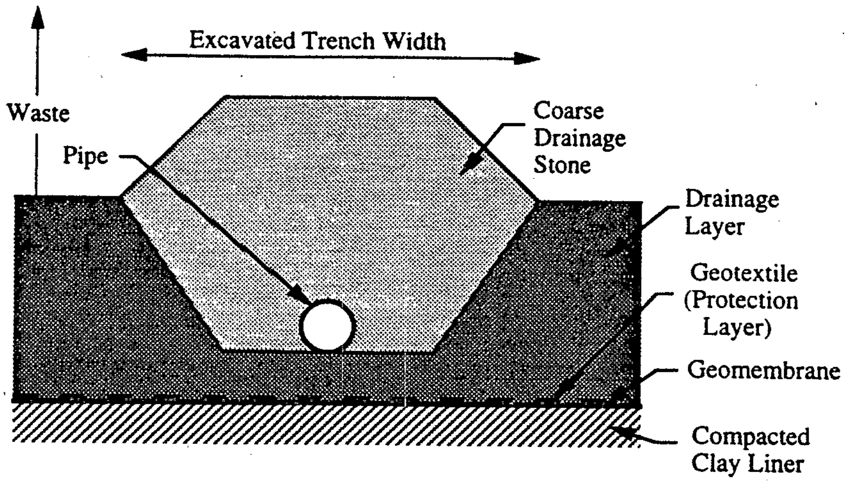
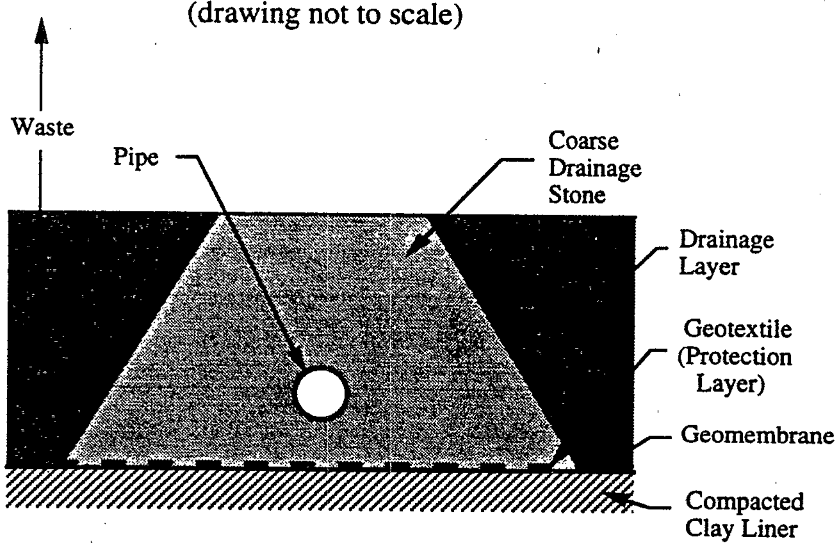


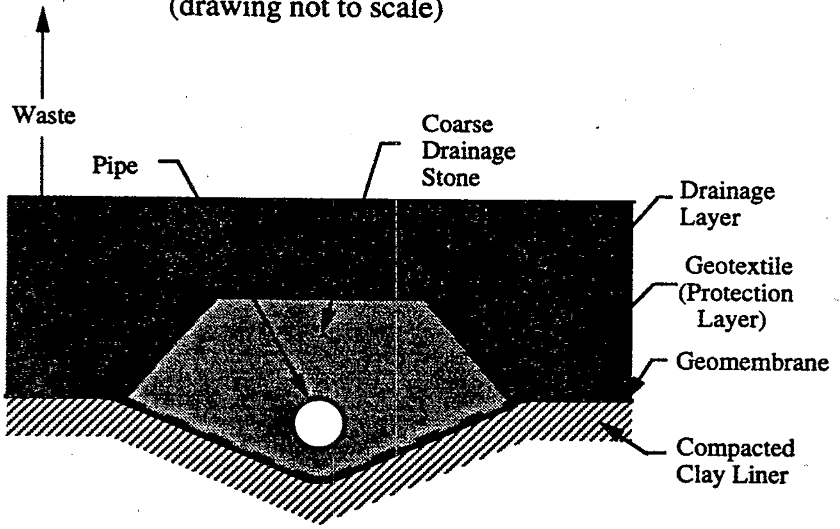
Figure 2 - Common Cross Section of Leachate Collection System



(a) Trench Type of Installation
(drawing not to scale)



(b) Embankment Type of Installation
(drawing not to scale)



(c) Embankment with V-Trench Type of Installation
(drawing not to scale)

Figure 3 - Location of Removal Pipes Within a Drainage Layer

Considering the components of a leachate collection and removal system as just described, it is felt that the filter layer will be the first component to excessively clog, if the phenomenon does indeed occur. This is because the filter is the first layer of the drainage system that the leachate encounters (thus the leachate has its highest sediment and microorganism content) and the voids of the filter are the smallest of any component of the drainage system.

While such a filter could be either granular soil or geotextile, the current trend is toward the use of geotextiles for a number of reasons:

- The savings in thickness provides additional landfill volume for placement of waste.
- A wide variety of geotextiles are readily available with various opening sizes via quality controlled-manufacturing processes.
- Geotextiles are easily placed even in tight or poorly accessible locations.
- Geotextiles must be used to cover geosynthetic drainage systems, e.g., for geonets or geocomposites, due to the large open spaces of these materials.

Regarding the design elements of any type of filter, including those used for leachate collection systems, there are three primary considerations. The first and foremost filtration criterion is to allow adequate flow of leachate into the underlying drainage layer so that a hydraulic head does not build up in the solid waste mass. The second criterion is to adequately retain the particles above the filter layer (solid waste or protection layer soil) so that the downstream drain itself does not become excessively clogged with fine-grained particles. The third criterion is to design the filter against long-term excessive clogging.

Hence, the proper design of a filter must establish a balance between flow capacity and upstream particle retention as well as guard against the potential problem of long-term excessive clogging.

It must be recognized that some degree of flow reduction, i.e., clogging, of the filter is to be expected. Such clogging can occur without adversely affecting the drainage system, at least until the clogged filter begins to "starve" the downstream drain. At that point, leachate will begin to build up into the solid waste as with the low permeability soil protection layers described earlier. The implication of such buildup, called "perched leachate," is unknown but probably is not desirable. In the extreme case, leachate may exit the cover soil of the facility in the form of leachate seeps and exert hydraulic pressure on the base and side slopes which are the location of sumps and pipe penetrations. Furthermore, if leachate recycling is the liquids management scheme, the situation is aggravated due to the constant reintroduction of leachate back into the waste mass. Leachate recycling will simply not work if the leachate collection filter becomes excessively clogged. Therefore, "excessive clogging" is defined as the point in time when the permeability of the filter renders the downstream leachate collection system ineffective for the site specific application.

There exists many analytic attempts at achieving the above three criteria of adequate permeability, proper retention of upstream soil, and long-term flow equilibrium. Christopher and Fisher [1] summarize the existing criteria in each of the three above mentioned areas. Illustrative of many of these criteria is that recommended by the Federal Highway Administration as proposed by Christopher and Holtz [2]. This set of criteria has recently been substantiated in the exhuming of a series of different highway drainage systems (Koerner, et al. [3]).

It is felt, however, that the state-of-the-practice for filters in highway applications should be challenged for the design of filters used in landfill leachate collection systems in regards to the excessively clogging situation. This is primarily due to the fundamental differences between leachate and water and the critical nature of the essentially inaccessible filter of a landfill leachate collection system. Specifically, it is felt that the permeability and clogging criteria should be linked together and that an overall global factor-of-safety against excessive clogging be introduced. For the general landfill situation, soil retention is probably not as major of an issue as is permeability and excessive clogging.

This report presents the findings of a multi-phased study in light of the previous discussion. Field behavior, laboratory testing, computer modeling and design method development are presented. In all cases focus is on the filter component of the leachate collection system. Both geotextile and soil filters are addressed, but (as mentioned previously) the emphasis is on geotextiles since they are used much more widely than sand filters at this time. Additionally, the work has direct applicability to the drainage soil, sand or gravel, if no filter is used at all. This "no filter" strategy is also investigated and is presented in an Appendix.

2. Sources of Leachate Collection Filter Clogging

As stated in the introduction, the filter zone above the leachate collection system is an ideal location for excessive clogging from several sources within the leachate. These sources include particulate clogging, biological clogging and precipitate clogging. The range of leachate characteristics in Table 1 is such that particulates, microorganisms and precipitates are all common to municipal solid waste (MSW) leachates. It is of interest to note that hazardous waste leachates may be significantly less troublesome to leachate collection filters than MSW leachates, unless such waste is co-disposed waste or has other extenuating circumstances.

Table 1 - Range of Leachate Characteristics, after Chain and de Walle [4]

Potential Clogging Mechanism	Property of Concern	Range of Values (mg/l)
Particulate	pH	3.7 - 8.5*
	Total solids (TS)	0 - 59,200
	Total dissolved solids (TDS)	584 - 44,900
	Total suspended solids (TSS)	10 - 700
Biological	Chemical oxygen demand (COD)	40 - 89,520
	Biochemical oxygen demand (BOD)	81 - 33,360
	Total organic carbon (TOC)	256 - 28,000
Precipitate	Specific conductance	2,810 - 16,800
	Alkalinity ($CaCO_3$)	0 - 20,800
	Hardness ($CaCO_3$)	0 - 22,800
	Total phosphorus	0 - 130
	Ammonia	0 - 1,106
	Nitrate	0.2 - 10.29
	Calcium	60 - 7,200
	Chlorine	4.7 - 2.467
	Sodium	0 - 7,700
	Sulfate	1 - 1,558
	Manganese	0.09 - 125
	Magnesium	17 - 15,600

***pH is in pH units, all others are in units of mg/l**

Particulate clogging is rather self explanatory. It is merely the settling out of suspended particles from the leachate. Particulate clogging generally occurs at the upper surface of a filter. This is sometimes referred to as formation of a surface cake. In contrast to this surface phenomenon, depth filtration within the filter may also occur. This is the situation where successively smaller particles in the leachate are removed from suspension within the thickness of the filter. The mechanism of depth filtration incorporates embedment of suspended particles from the leachate in the pores of the filter. In so doing the sediment that collects within the filter acquires a gradation from large to small particles. It is possible that depth filtration is more conductive to fluid flow than surface or cake filtration [5].

The clogging associated with biological growth is quite complex. It occurs when microorganisms metabolize on and within the filter material. Biological growth depends on the presence of microorganisms, appropriate nutrients and environmental conditions which sustain growth. Factors which influence biological clogging include carbon-to-nitrogen ratio of the leachate, rate of nutrient supply, concentration of micro and macro nutriets, moisture conditions of the waste mass and temperature.

Anaerobic rather than aerobic conditions prevail in most locations in a landfill. Hence, methanogenic bacteria are most prevalent. The quantity and quality of nutrients available to the methanogenic bacteria is significant in this metabolism. Nutrients required by methanogenic bacteria include; carbon, hydrogen, oxygen, nitrogen and phosphorous. In addition, they require limited concentrations of trace metals such as sodium, potassium, calcium, and magnesium. Biofilm growth starts on the surface of the filter media and eventually incrusts it. Upon incrusting the filter surface, it then moves into the pore spaces and further reduces flow through the filter.

Current practice at many landfills is to co-dispose sewage treatment plant sludge and municipal solid waste. This practice ensures the presence of a population of methanogenic bacteria as well as the needed nutrients needed for them to metabolize. In addition, leachate recirculation is being practiced at a number of landfills and is currently permitted by Subtitle D regulations for municipal solid waste landfills. Leachate recirculation is intended to make a "bioreactor" of the landfill and to result in enhanced degradation of the waste mass by continuously recasting leachate onto or into the waste. This practice further ensures the presence of a population of viable microorganisms to degrade the waste mass and potentially cause excessive clogging of the filter of the leachate collection system.

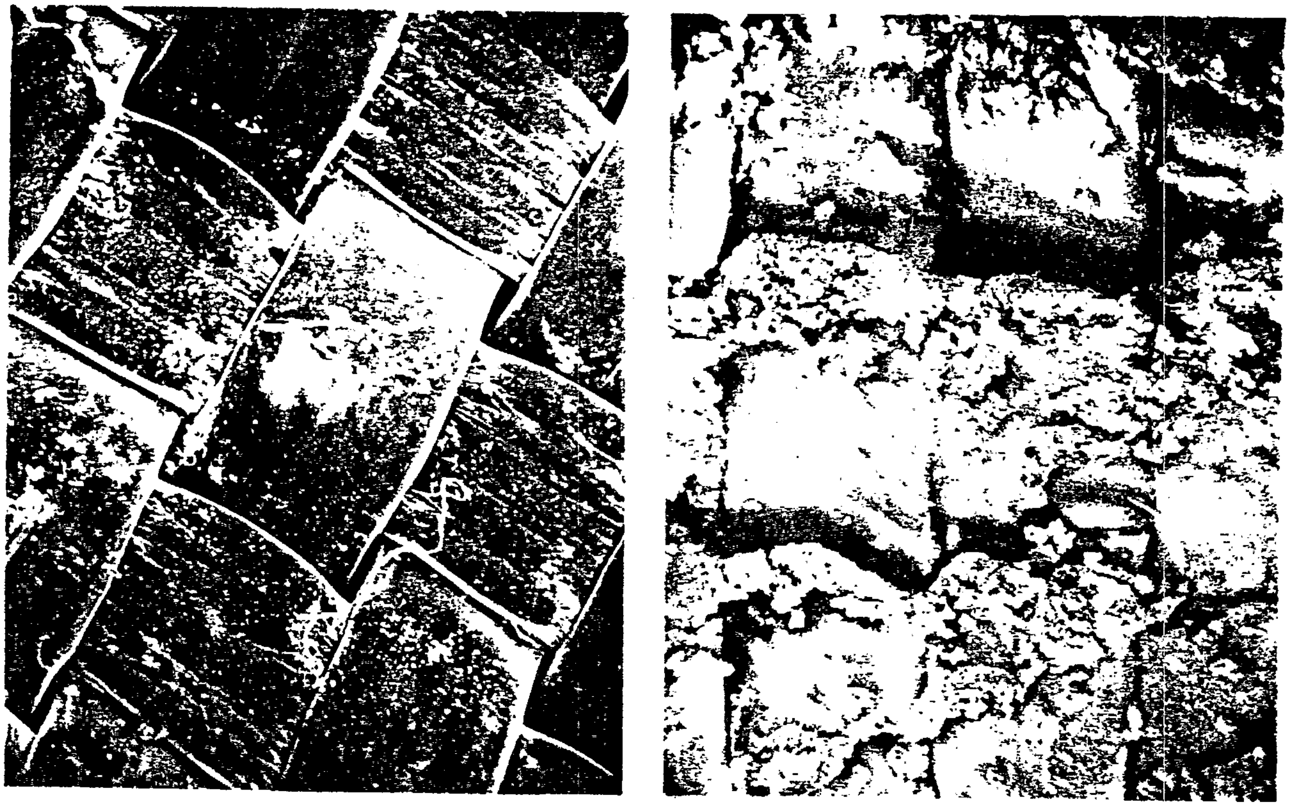
The final type of potential filter clogging is associated with chemical precipitation. It occurs as a result of chemical processes which include the precipitation of calcium carbonate, manganese carbonate and other insoluble forms such as sulfides, chlorides and silicates. Inorganic chemical precipitates can form when the pH exceeds 7. Hardness and total alkalinity of the leachate are also important. Precipitation can be caused by the presence of oxygen, changes in pH, changes in the partial pressure of carbon dioxide, or evaporation of residual liquid. Biochemical precipitation can also exist. The biochemical mechanisms usually involve the complexation of iron or manganese. The most frequently complexed metal is iron which results in the formation of "ochre" deposits.

Both inorganic precipitate and biochemical precipitate clogging are iterative and sometimes synergistic. When conditions are optimum, precipitate clogging can quickly decrease the permeability of a filter. A modified list of precipitate clogging indicator parameters is given in Table 2.

Table 2 - Indicators of Precipitate Clogging Parameters, after Driscoll [6]

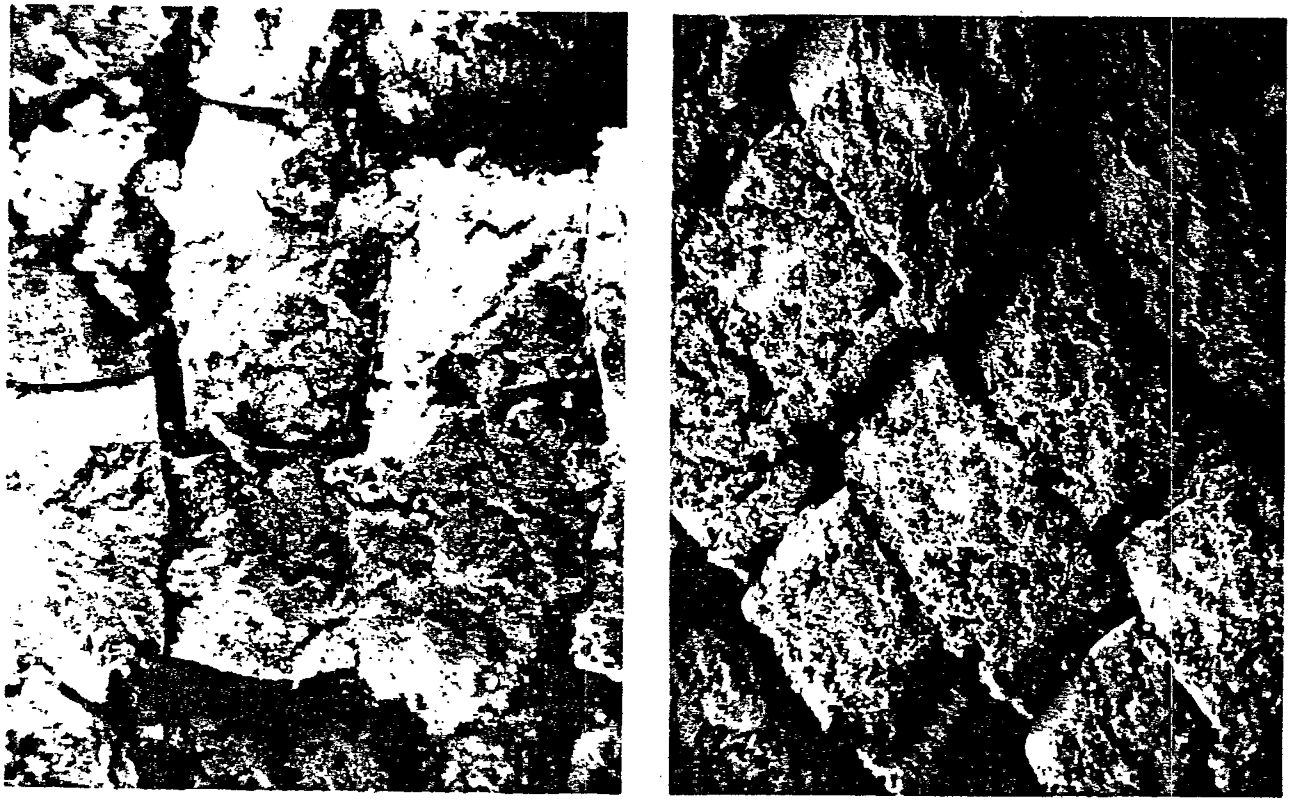
Corrosive Condition	Incrusting Condition
At pH less than 7	At pH greater than 7
Dissolved oxygen in excess of 2 mg/l	Total iron (Fe) in excess of 2 mg/l
Hydrogen sulfide (H_2S) in excess of 1 mg/l	Total manganese (Mn) in excess of 1 mg/l in conjunction with high pH and the presence of oxygen
Total dissolved solids in excess of 1,000 mg/l, indicating an ability to conduct electric current great enough to cause electrolytic corrosion	Total carbonate hardness in excess of 300 mg/l
Sulfate in excess of 300 mg/l	
Carbon dioxide in excess of 50 mg/l	
Chloride in excess of 500 mg/l	

Indeed, the combination of particulate, biological and precipitate potential for filter clogging is of concern. For example, the scanning electron micrographs of Figures 4 (for woven geotextiles) and 5 (for nonwoven geotextiles) illustrate the progressive reduction in void space in the respective filter materials. However, it remains to be seen how these reductions effect (and to what extent) the flow of leachate in landfill filters and if proper design can negate the phenomenon to the point where acceptable performance of the filter can be achieved.



(a) After 1 Month of Flow

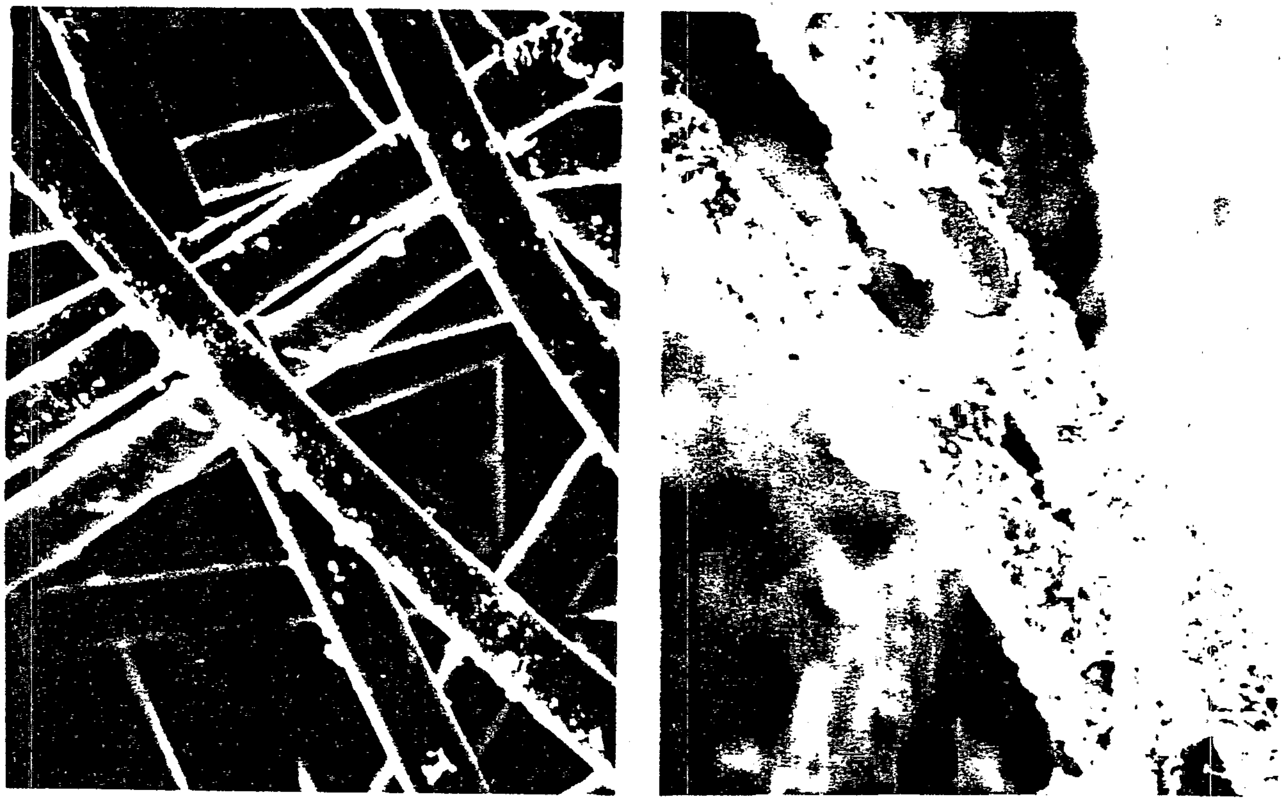
(b) After 3 Months of Flow



(c) After 6 Months of Flow

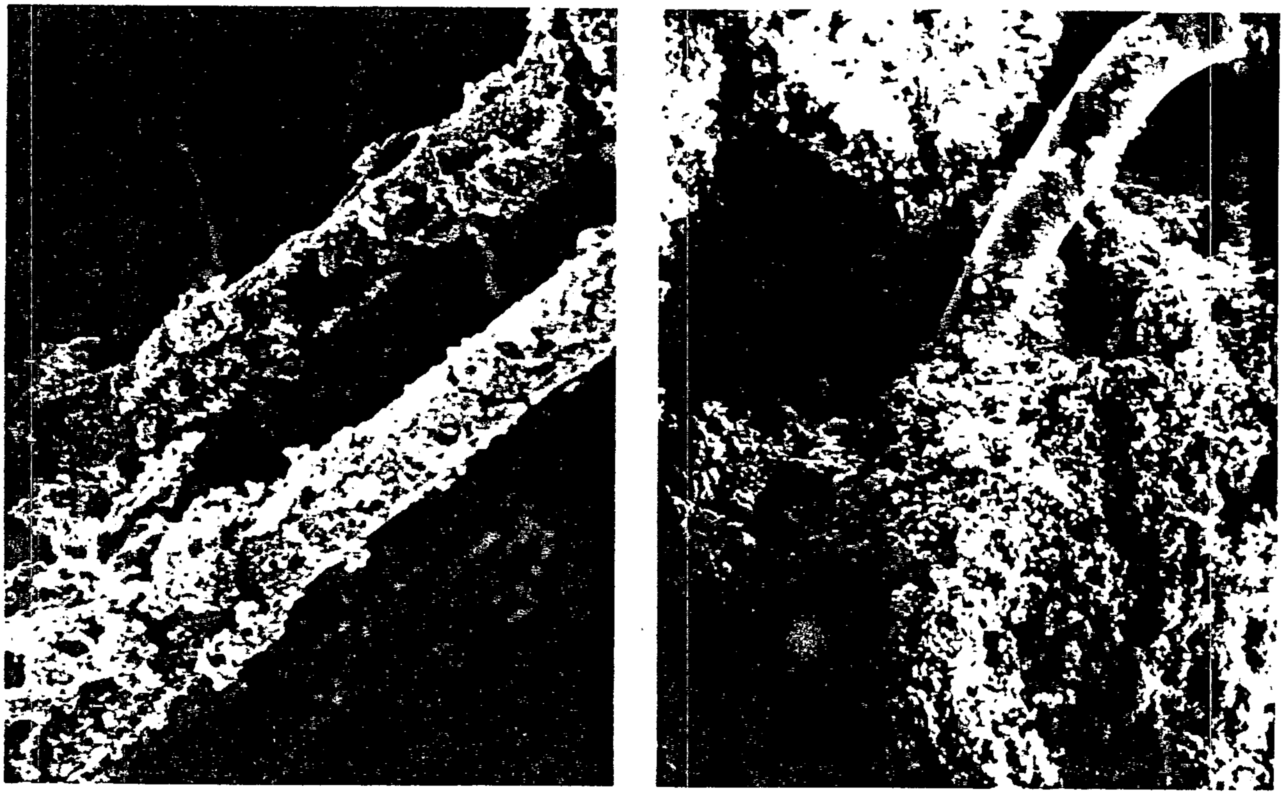
(d) After 12 Months of Flow

Figure 4 - Mosaic of Woven Geotextile Filter Under Very High Flow Rate Conditions for 1, 3, 6 and 12 Months Time Periods at 30X Magnification



(a) After 1 Month of Flow

(b) After 3 Months of Flow



(c) After 6 Months of Flow

(d) After 12 Months of Flow

Figure 5 - Mosaic of Nonwoven Geotextile Filter Under Very High Flow Rate Conditions for 1, 3, 6 and 12 Months Time Periods at 400X Magnification

3. Results of Previous Investigation

This project represents a continuation and expansion of a previous U.S. EPA sponsored project [Assistance ID No. CR-814965] which was performed for the Agency by this same research team between September 1, 1987 and August 30, 1990. So as to set the stage for the work to follow the results from the earlier project will be briefly described. For complete details see EPA/600/2-91/025 [7].

To investigate the behavior of several geotextile filters and a sand soil filter, six landfill leachates were used under different experimental conditions. The characteristics of the leachates are shown in Table 3. It was determined from particle size analyses that all of the sediment and microorganisms contained in the six leachates fell into a relatively tight particle size distribution within the silt-size classification, i.e., they ranged from 0.074 mm to 0.002 mm.

Table 3 - Details of Municipal Solid Waste Landfill Leachates Evaluated and Average Leachate Characteristics

Site Designation	Average Leachate Characteristics			
	pH	COD* (mg/l)	TS* (mg/l)	BOD_5* (mg/l)
PA-1	8.0	15,000	8,000	2,000
NY-2	5.5	20,000	9,000	5,000
DE-3	5.8	40,000	17,000	24,000
NJ-4	7.4	45,000	16,000	25,000
MD-5	6.8	1,000	100	150
PA-6	6.5	10,000	5,000	2,500

*COD = chemical oxygen demand; TS = total solids content; and BOD_5 = biochemical oxygen demand at five days.

3.1 Phase I - Initial Flow Rate Evaluations

The first phase, which lasted for 12 months between September, 1987 and September, 1988, used flow boxes for aerobic evaluation of the filters and large containers for anaerobic incubation of the various filters.

In the aerobic tests, 300 x 300 mm (12 x 12 in.) wooden flow boxes, 600 mm (24 in.) high were used. The boxes were constructed using a base plate, a geonet drain, a geotextile filter, and 150 mm (6 in.) of free draining sand. The remaining 450 mm (18 in.) of the boxes were empty so that falling head permeability tests could be conducted. Leachate passed through the sand cover and geotextile filter and then flowed within the geonet, which was open at one end only. The time for given quantities of leachate to pass through the system was measured. Each of the six sites had at least four boxes, the only difference being the type of geotextile filter. Both woven and nonwoven geotextiles were evaluated. They consisted of various polymer types and manufacturing styles.

The following findings were based on the flow rate behavior over the 12 month evaluation period at each site.

(a) The flow rate measurements from the original values all decreased but varied considerably.

(b) The relatively tightly woven geotextile filter, with a 4% open area, performed the poorest. For each of the four different sites in which it was used, it clogged beyond the detection limit. The time periods were from 4.5 to 12 months.

(c) Opening up the void space of the same type of woven geotextile to a 10% open area helped considerably. Flow rates still decreased but were more equivalent to the needle punched nonwoven geotextile types.

(d) The needle punched, nonwoven geotextiles performed equivalently. They were similarly constructed but were of different polymer types. The results indicate that polypropylene, polyester, and polyethylene fibers do not appear to give significantly different values in their flow rate response behavior.

(e) A heat bonded, nonwoven geotextile was used at two sites. Its response was somewhat poorer than that of the needle-punched nonwovens but better than that of the 4% open area woven geotextile.

(f) The Phase I study indicated that use of open woven geotextiles and each of the needle punched, nonwoven geotextiles resulted in equilibrium flow conditions being established between 6 and 12 months. The flow rate was reduced from as little as 20% of the original values (at four sites) to as much as 80% (at two sites). These reductions appeared to be related to the strength of the leachate insofar as their total solids (TS) and microorganism content (BOD) were concerned. In the worst cases, flow rates were usually greater than 0.68 $l/sec\text{-}m^2$ (1.0 $gal/min\text{-}ft^2$). This is the equivalent to 58×10^7 l/ha-day (6.2×10^7 gal/acre-day), which far exceeds most design requirements for leachate collection system filters.

(g) The cause of the flow reductions created somewhat of a dilemma. By cutting a cross section of the boxes at the end of the 12 month period it was evident that the 150 mm (6 in.) of cover sand placed over the geotextile filter was a major source of the flow reduction. The experiments showed that soil clogging can also be imporrtant. The soil used was an open graded, rounded sand (Ottawa sand) having a permeability coefficient of approximately 0.02 cm/sec (0.04 ft/min). Thus, it actually meets, and even exceeds, most regulatory criteria for a drainage soil, let alone for a filter soil.

(h) Microscopic examination of the cross sectioned soil/geotextile systems showed heavy particulate clogging within the upper portion of the soil layer. Thereafter, the clogging was either fibrous or consisted of very small clusters. Although not conclusively proven, it was felt that the upper portion of the soil column filtered the suspended solids out of the leachate and thereafter biological activity spread throughout the remaining portion of the soil column and into the underlying geotextile. This biological activity took numerous forms including the deposition of precipitates in the soil and in geotextile voids. Thus, different geotextiles (all other things being equal) responded differently to a site specific leachate.

(i) The relative amounts of flow rate reduction between leachate sediment, biological precipitates, and biological growth could not be distinguished in these particular tests.

The anaerobic tests were performed under completely submerged conditions in 210 litre (55 gallon) drums. Twelve samples of each type of geotextile were suspended on stainless steel racks and placed in leachate from the various sites. One sample of each type was removed for testing each month. Four geotextile types were evaluated for each of the six landfill leachates. After the samples were removed, they were brought to the laboratory and were tested for the retained flow capability and possible strength reduction. The general findings follow:

(a) Relatively minor flow reduction occurred in all types of geotextiles evaluated. The reduction values varied from 10% to 20%. Note that these amounts were distinctly less than those that occurred in most of the previously described aerobic tests. It is felt that sediment clogging did not form since flow was not occurring during the incubation periods. Furthermore the absence of a soil column had a dramatic (but quantitatively unknown) effect on improving the flow rates.

(b) All of the exhumed geotextiles had heavy biological growth that could be easily seen and felt.

(c) Informative scanning electron micrographs taken at various times of incubation were compared with the as-received geotextiles. After 3 months of incubation, complete growth around the individual fibers or growth in clusters could generally be seen, recall Figures 4 and 5. Although difficult to quantify, the amount of growth was clearly related to the time of immersion.

(d) The micrographs also revealed that the biological growth was easily removed from the fiber's surface. There appeared to be no fixity or attachment of the biofilm clusters to the fibers.

(e) The above observation was corroborated by various strength tests performed on the geotextiles after immersion. Within the statistical limits of testing there was no strength reduction over the 12 month period. This suggests that for these leachates, *biological degradation* of geotextiles is not a problem. As a result, Phase II studies did not include the polymer degradation concern.

3.2 Phase II(a) - Improved Flow Rate Columns and Remediation Attempts

This phase of the project, which lasted 24 months, between September 1988 and August 1990 used improved flow rate measuring systems. This type of improved system has been developed into a test method procedure and subsequently adopted by the American Society for Testing and Materials as a Standard Test Method (ASTM D1987-91, "Test Method for Biological Clogging of Geotextile or Soil/Geotextile Filters"). The flow rate measuring column is a 100 mm

(4.0 in.) diameter fixed wall permeameter which can function in either a variable or falling head test mode. Ninety-six (96) of these test devices were used under the following set of conditions;

- four different geotextile filters,
- without sand and with sand above geotextile filters,
- aerobic and anaerobic conditions, and
- six landfill leachates.

Continuous flow rate testing on the 96 permeameters was monitored for 6 months. Once trends were established, a series of different remediation procedures were attempted which lasted for approximately 14 months.

The following comments apply to the first 6 months of flow testing, i.e., before the first remediation was attempted.

(a) The columns with sand above the geotextiles clogged considerably more than those with the geotextile alone, i.e., 23% of the flow was retained for sand/geotextile columns versus 34% flow retained for geotextile columns. Note that if the heat bonded nonwoven fabrics were eliminated from the geotextile group, the flow rate retained by the geotextile group would be 45%. This suggests that geotextiles can clog less than natural sand filters.

(b) Of the four geotextiles evaluated, the highest retained flow rate was achieved with the lightweight needle punched nonwoven (38%), with the heavyweight needle punched nonwoven (34%) and woven monofilament (32%) slightly behind. The nonwoven heat bonded geotextile had the lowest retained flow of only 10% after 6 months of evaluation.

(c) Of the various landfill leachate types, the lowest retained flow rate resulted from use of the NJ-4 (14%) and DE-3 (17%) leachates. Recall from Table 3 that these are the leachates with the highest TS and BOD concentrations. The other four landfill leachates and their percentages of flow retained after 6 months of testing were PA-6 (26%), MD-5 (29%), PA-1 (38%), and NY-2 (41%).

After the initial 6 months of flow rate testing confirmed the results of the Phase I study, several remediation procedures of the flow columns were attempted. The first remediation, a leachate backflush, improved flow rate but to varying amounts between the different columns. After 4 months of resumed flow testing, the flow rates decreased and allowed for a second remediation. This remediation used a water backflush. Upon resumption of flow the flow rates increased, but over the next 5 months they gradually decreased.

The third remediation utilized a nitrogen gas backflush. It improved flow rates, but 3 months later they were once again reduced. The fourth, and last, remediation was a forward vacuum extraction, which only nominally improved flow rates when it was performed. Thereafter, the flow rate again decreased. The overall average behavior of the 96 columns is shown in Figure 6. It visually describes the decreasing flow rate trends between remediation attempts and the rapid increase in flow rates immediately following remediation. Individual filter

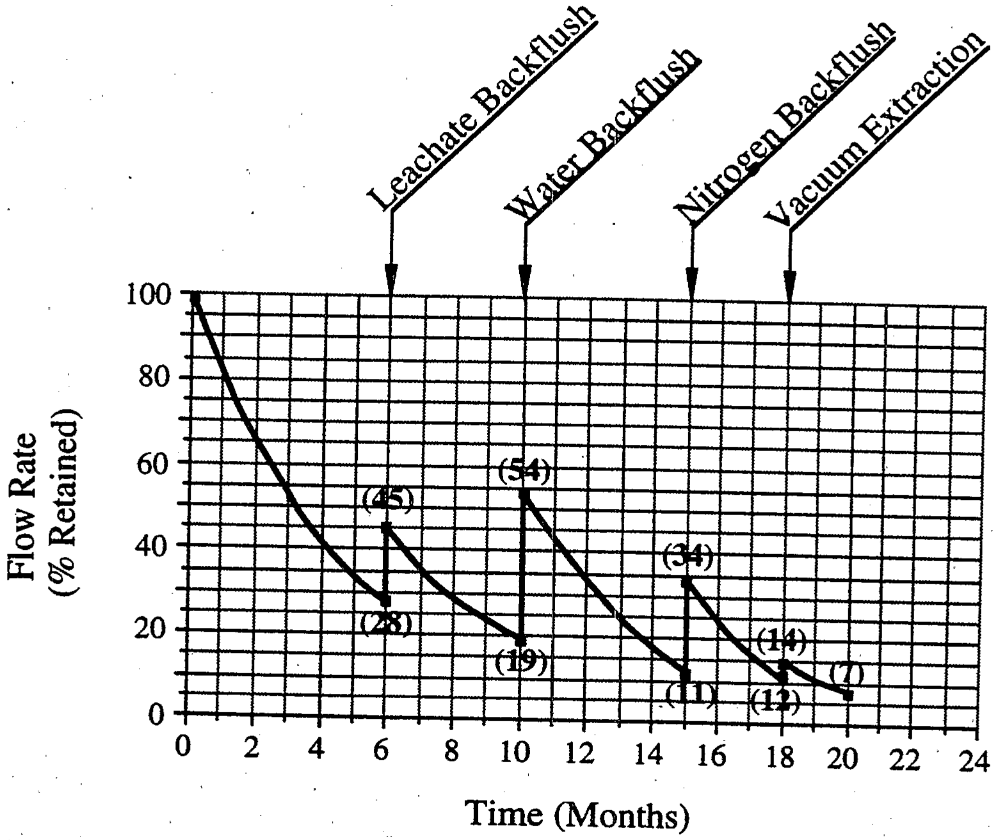


Figure 6 - Average Response of 96 Flow Rate Columns from Phase II(a) Activites

types and their respective behavior for all 96 combinations are found in Reference 7.

To quantitatively assess the overall performance of the remediation attempts and their relative performances in contrast to one another, the data were analyzed with respect to their percent of flow rate improvement. Within each combination, however, there were decided differences. For example:

(a) Backflushing of geotextiles by themselves was more efficient than backflushing of geotextile/sand systems. The average recovery efficiencies were 29% and 13%, respectively.

(b) With sand overlying a geotextile there was no measurable difference from one type of geotextile to another.

(c) With only a geotextile, remediation was most effective with the woven monofilament geotextiles (38% recovery efficiency), slightly less effective with the nonwoven needle punched lightweight (31%) and the heavyweight (30%) geotextiles, and relatively ineffective with the nonwoven heat bonded geotextiles (16%).

(d) When sand was placed over the geotextile, there was no difference between anaerobic and aerobic remediation schemes.

(e) With only a geotextile, remediation was slightly better under anaerobic conditions than with aerobic conditions.

(f) When sand was placed over the geotextile, the remediation recovery efficiency rankings were: water > nitrogen > leachate > vacuum

(g) With only a geotextile, the remediation recovery efficiency rankings were: water > leachate > nitrogen > vacuum

3.3 Phase II(b) - Biocide Treated Geosynthetics

Because of the relatively large flow rate decreases observed during the course of this study, an investigation into the use of biocides in the flow system was undertaken. This was done under the assumption that the biocide would kill the microorganisms that come into contact with it and that the nonviable (i.e., dead) matter would pass through the system in much the same way that fine particles or sediment moves through any other filtration/drainage system. Because it was believed that the biocide should be introduced on a long-term basis rather than as one bulk dose, the biocide was added to the polymer compound during manufacture of the selected geonets or geotextiles. The reasoning was that the biocide would gradually release via molecular diffusion through the polymer structure and migrate to the surface of the ribs or fibers over a long period of time.

From the results of these biocide treated geosynthetics, the location of the biocide vis-a-vis the initial formation of a biofilm layer was felt to be critical. This was confirmed at the end of the tests after solidifying the test columns with epoxy and cutting them apart. Clearly, the biofilm layer was occurring at the top of the sand column some 50 mm (2 in.) above the biocide-treated geosynthetics. Although there may have been some flow rate improvement due to high concentrations of biocide, it was very subtle (at best) and was masked by the inherent scatter in the test data. There was essentially no difference between flow rates in anaerobic versus aerobic conditions.

These findings led to additional tests without sand above the biocide-treated geosynthetics that forced the leachate to interface directly with the biocide. Rather than use a single type of geotextile, three different types were utilized. The opening sizes varied from 0.15 mm (nonwoven needle punched style), to 0.21 mm (a woven monofilament) to 0.42 mm (another woven monofilament). Quite clearly, the flow rates through the largest opening size geotextiles, i.e., the 0.42 mm, were the highest. This suggests that microorganisms (dead or alive) must be able to pass through the system. Whenever these microorganisms reside on or within the small pores of the filter, partial, or even excessive, clogging is possible.

3.4 Summary of the Previous Investigation

A simulated field-oriented project concerning biological clogging of landfill drainage systems was focused on geotextile filter clogging. Six different MSW landfill leachates were used. The filter was singled out (versus the geonet drain, drainage stone or perforated pipe) since it had the smallest openings and was likely to become clogged before other components. Geotextiles were emphasized because they are generally used for this particular application.

Phase I results reoriented the initial goals since the granular soils covering the filters were clogging before the underlying geotextiles. Furthermore, sediment and/or particulates were a major factor in flow rate reductions, which appeared to be synergistic with the biological clogging. Clearly, partial filter clogging was occurring with a gradual reduction of flow rate over time. These trends were common to all six landfill leachates used. All of the landfills were domestic municipal solid waste facilities; but their waste stream, volume of waste deposited, and liquid management schemes differed. Recognized early in this Phase I activity was that remediation attempts would be a necessary part of the overall study, but the Phase I experimental setup could not accommodate such activities. New and different test devices would be needed if such attempts were to be made. Some conclusions, however, were drawn from Phase I activities.

- Filter clogging (as indicated by flow rate reductions) over the 12 month test period varied widely, the range being between 10% and the limit of the test devices.
- A geotextile filter must be relatively open in its pore structure if it is to limit the amount of clogging, i.e., the geotextile must be capable of passing the sediment or particulates along with the associated microorganisms into the down-gradient drainage system.
- The polymer type (polypropylene, polyester or polyethylene) comprising the geotextile fibers appears to be a nonissue.
- Both anaerobic and aerobic conditions promote clogging; the relative amounts, however, were not capable of being identified because of differing setups.
- The mechanical strength of the geotextiles was not adversely affected by the 12 month exposure to the various leachates. This finding, coupled with numerous micrographs which showed no chemical attachment of bacteria clusters to the fibers, led to the conclusion that biological degradation of polymeric based geotextiles does not occur.

Phase II(a) of the study saw the development of a new and vastly improved test device for flow rate evaluation. The 100 mm (4.0 in.) diameter flow columns developed during the project have the following capabilities.

- All types of cross sections can be evaluated: geotextiles by themselves, soil/geotextile systems, soil/geotextile/geonet systems, or soil/geotextile/gravel systems.
- Anaerobic or aerobic conditions can be maintained.
- Flow rates can be evaluated using falling head or constant head measurements.
- The flow columns are small and portable. Therefore, they can be stored indoors and taken to a site for evaluation, or stored at the site, or even stored within the leachate storage tank or sump.
- Various methods for remediation of clogged systems can be evaluated.
- The test columns and their measurement protocol have been adopted as an ASTM Test Method under the designation of D1987-91.
- The test columns and their contained materials can be solidified by epoxy and cut in half to visually observe the conditions existing within the cross section.
- Since all parts of the columns consist of PVC plumbing and swimming pool accessories, they are readily available, easily sealed by chemical wipes, and inexpensive.

The following conclusions were reached from this Phase II(a) study.

- Flow rate reductions were similar to the results of Phase I, and the conclusions drawn earlier were substantiated.
- If geotextile and/or soil filters are to be used in leachate collection systems, they should have sufficiently open voids to pass the sediment or particulates along with the microorganisms contained in the leachate into the downstream drainage system.
- The limiting or equilibrium flow rate retained must exceed the site specific design requirement. If flow rates over time are not adequate, remediation is necessary. It was found that the water backflush technique gave the best results (35% improvement), nitrogen gas backflush (23%), and leachate backflush (17%) methods were next. The vacuum extraction was the least effective; it provided only nominal improvement (2%).
- The periodicity of backflushing to open up a clogged or semi-clogged filter system appears to be approximately 6 months.

Incorporating biocides into the geotextile (or geonet) polymer structure to keep the flow system open was Phase II(b) of the study. The concept was to add various amounts of a time-released biocide into the polymer compound as the product was manufactured—biocide that would essentially diffuse to the surface of the fibers during its service life. On contact, the biocides would kill the viable microorganisms in the leachate. In the tests that were conducted on 16 separately built flow rate measuring devices, some experimental evidence indicated that 2% and 4% biocide was partially effective. The remains of of the dead bacteria must, however, be permitted to pass through the system, and this apparently could not happen for these particular test setups. Thus, the idea of a very open filter system was further substantiated. While not successful within the context of this setup, the suggested use of biocides continues. Thus Appendix "A" of this report presents the results of the biocide experiments of this precursor study.

3.5 Recommendations of the Previous Investigation

Based on the major findings of this project, namely,

- under continuous flow of landfill leachate, a gradually decreasing flow rate occurred for all types of filters (soil or geotextile) and eventually reached an equilibrium value,
- the equilibrium value of flow rate varied according to the type of filter, the type of leachate, and the hydraulic gradient, and
- the equilibrium flow rate for any given filter system must be compared with the design required flow rate to ultimately assess the adequacy of the filter's design,

it was felt that the following recommendations be considered regarding geotextile and soil filters placed over different types of leachate collection drains.

(a) Leachate collection systems at landfills that are decommissioned or exhumed for other reasons should be investigated in light of the results of this study.

(b) Design criteria should be developed that consider the amount and type of microorganisms and sediment present in the leachate along with conventional issues such as hydraulic gradient and type of filter.

(c) This particular project should be followed by another effort aimed at a larger variety of geotextile filters along with design guidance to predict the field performance of those existing sites which are exhumed.

4. Overview of This Project

Based upon the recommendations of the project just described, a second U.S. EPA funded study entitled "Leachate Clogging Assessment of Geotextile and Soil Landfill Filters," was conducted. Its designation was CR-819371 and the project period extended from March 1, 1992 through February 28, 1995. The remainder of this report focuses on this second project. The project consisted of field, laboratory, computer modeling and design oriented tasks. Figure 7 is a flow chart of the project and the interrelated tasks. The major tasks of the project were the following:

- Field exhuming of sites-of-opportunity which came about in the 18-month time frame between the two projects and extended into the beginning of this second project.
- Long-term laboratory tests of a wide range of geotextile filters and two sand filters using ASTM D1987 permeameters of the type developed in the earlier study.
- Utilization of the U.S. EPA sponsored HELP computer model for design input parameters for the field exhumed sites.
- Formulation of a new methodology for the design of filters used with landfill leachate collection systems.
- Posing a challenge to the newly developed design formulation with the results of the field exhumed sites-of-opportunity where "ground truth" had been established.
- Forming final summaries and conclusions for both of the projects described in this report, i.e, CR-814965 and CR-819371.

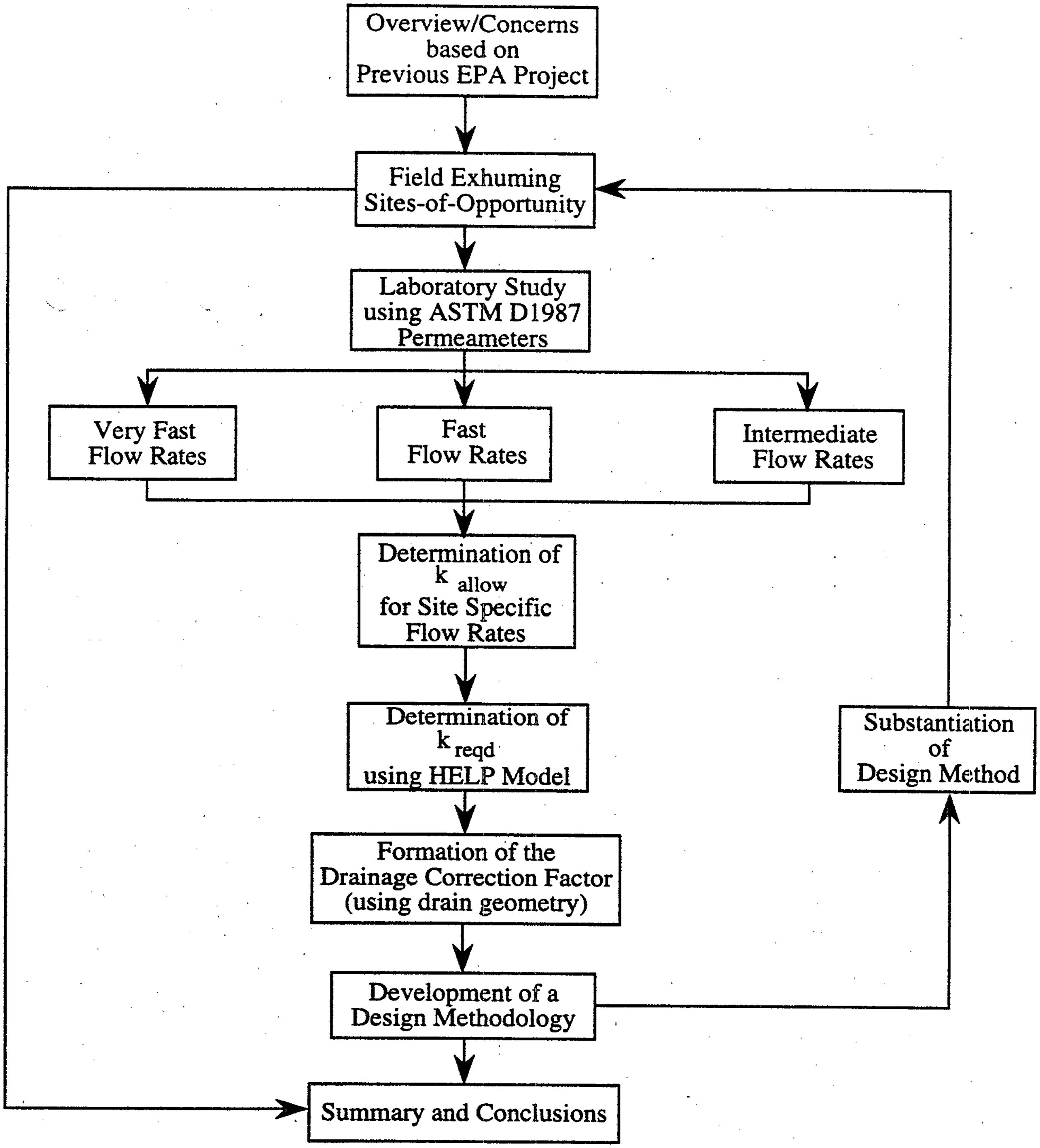


Figure 7 - Flow Chart for This Project

5. Field Exhuming of Leachate Collection Systems

This section describes the field exhuming of actual solid waste landfill leachate collection systems with focus on the geotextile filter and its performance [8]. All sites were obtained as "sites-of-opportunity," whereby permission was granted by the owner/operator of the facility. By request of the owner/operators, the description of the sites is not identified as to the location and other details unimportant to this study.

5.1 Field Exhuming Details

Four (4) landfill leachate collection systems were exhumed during the course of this study. The leachate collection systems had been in service for up to 11 years at the time of their exhuming. The design of each system, the reason for exhuming, the findings and performance levels were quite different for each site. Table 4 counterpoints these differences. Each site is detailed in regard to specific conditions in the following sections. It is relevant to note that all four sites were in the northeast region of the USA, thus temperature, precipitation, and general climatic conditions were reasonably similar to one another.

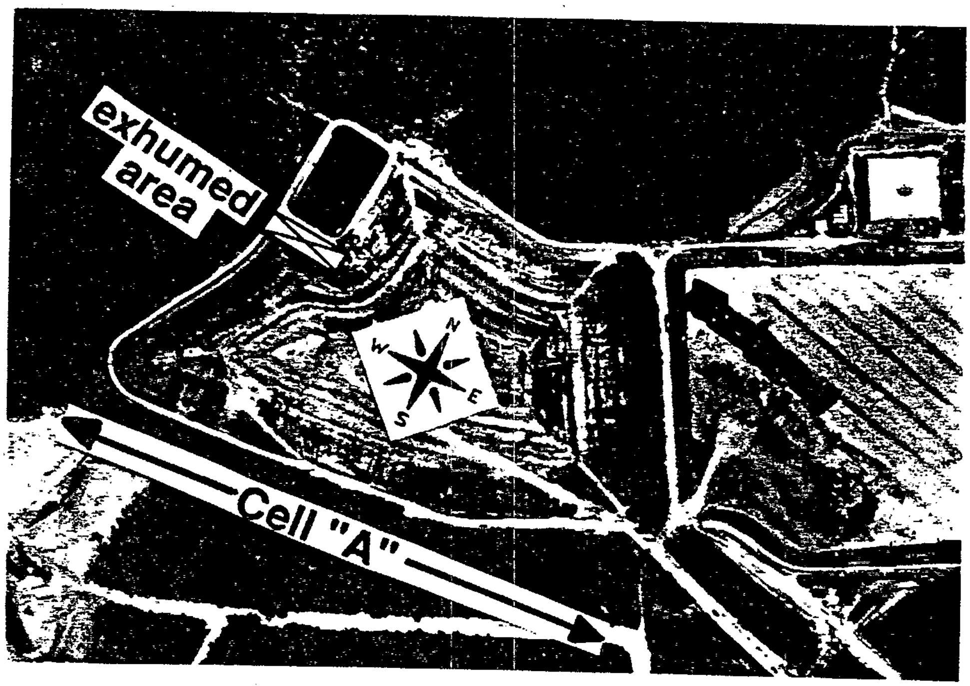
Table 4 - Overview of the Leachate Collection Sites Exhumed

Site	Waste Type	Construction	Liquid Management Scheme	Reason for Exhuming	Critical Element in Drainage System
1	domestic and light industrial	1981	leachate recycling	no flow in the collection system	geotextile filter
2	domestic and light industrial	1985	leachate recycling	leachate seeps through the landfill cover	drain location
3	industrial solids and sludge	1990	leachate withdrawal	no flow in the collection system	geotextile filter
4	domestic and rural	1976	leachate recycling	no flow of methane into extraction wells	geotextile filter

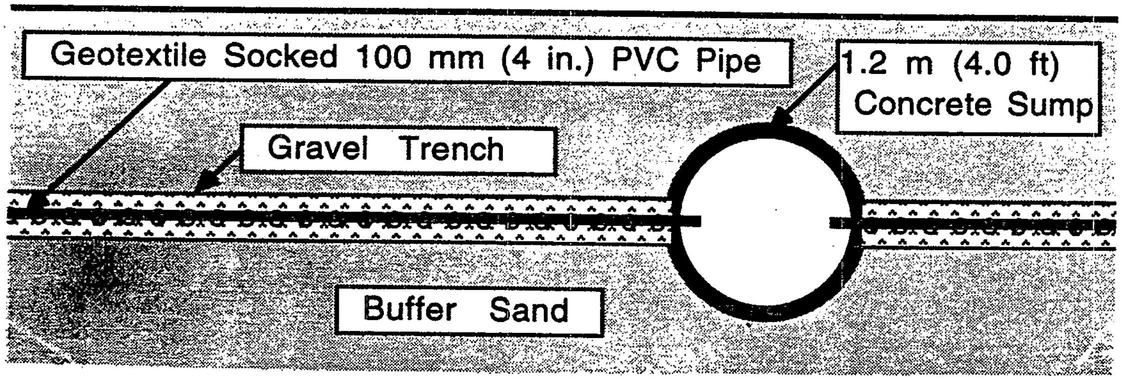
5.2 Field Exhumed Site #1

The solid waste in this landfill was a mixture of domestic and light industrial wastes. The facility was constructed in 1981. The plan and cross-sectional views are shown in Figures 8 and 9, respectively. Within one year after the cell was complete, leachate flow rates began to significantly decrease. Since the liquid management scheme was a form of leachate recycling, the situation was of concern with respect to the lack of leachate for reinjection and the high leachate heads being built up and imposed on the liner. Furthermore, leachate seeps began appearing through the landfill cover (which was a compacted clay liner) which resulted in leachate running down the side slopes of the landfill.

The original design called for a drain along the low elevation side of the facility intended to intercept leachate flowing down gradient on the geomembrane as seen in Figure 9. The facility was constructed accordingly. Once in the drainage system, the leachate flowed to a sump for

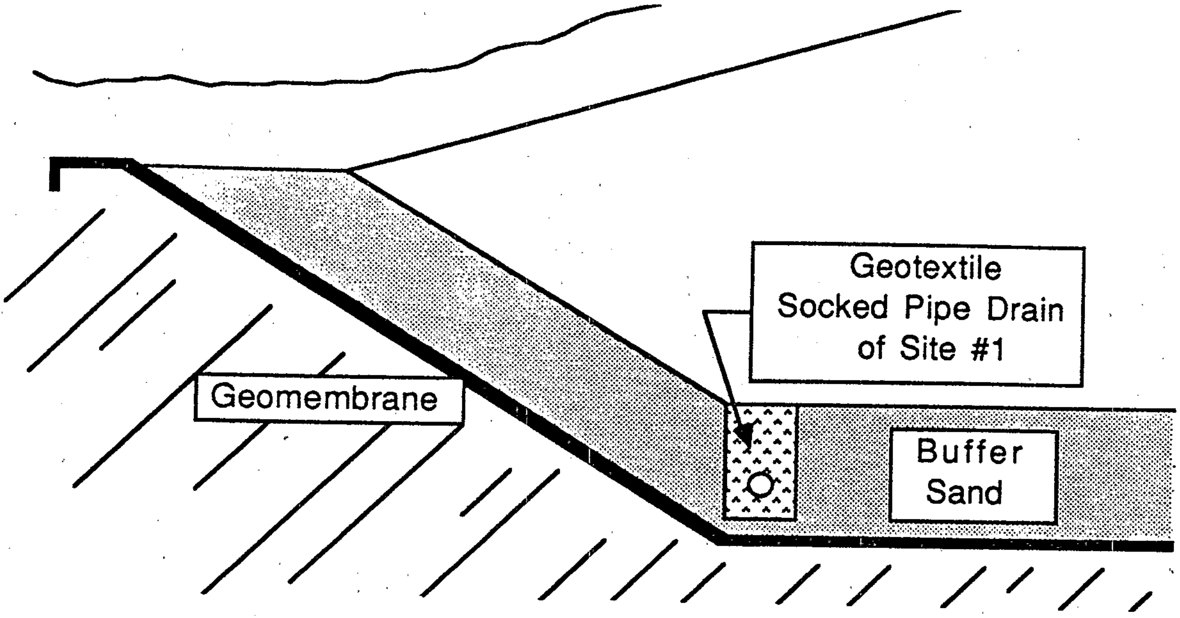


(a) Aerial Photographs of Site #1 Showing Exhumed Area of Cell "A". Cell "A" is 11 ha (27 acres) in Area and the Exhumed Area was Rectangular in Shape Measuring 30 m × 91 m (100 ft × 300 ft).

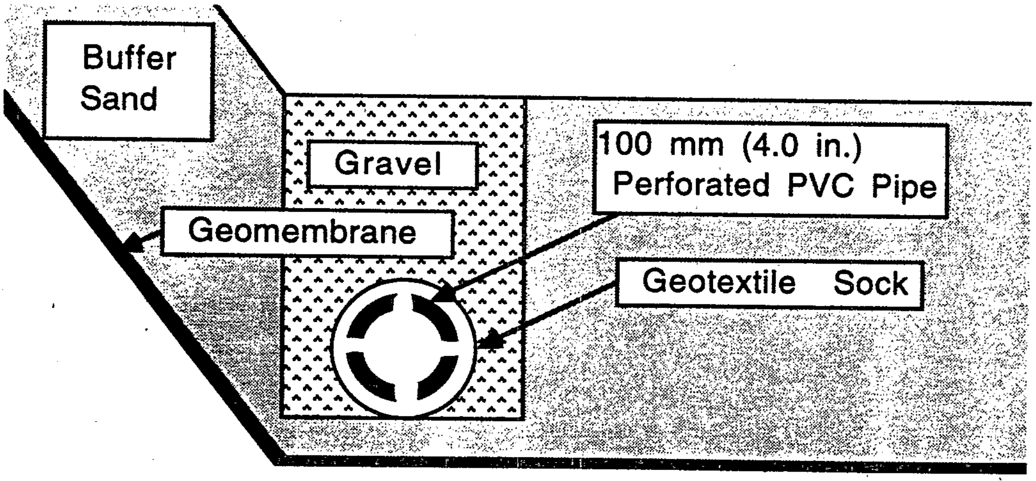


(b) Plan View of Site #1 Leachate Collection System. The Leachate Collection System was Embedded in a Sand Layer and Consisted of a Geotextile Socked 100 mm (4.0 in.) Perforated PVC Pipe Within a Gravel Trench.

Figure 8 - Plan Views of Site #1



(a) Cross-Sectional View of Cell "A" of Site #1 Showing the Location of the Leachate Collection System with Respect to the Other Components of the Cell.



(b) Cross-Section Detail of the Leachate Collection System of Site #1 Showing the Arrangement of the Geotextile Filter Socking the Perforated Pipe.

Figure 9 - Cross-Sectional Views of Site #1

collection and removal. The drainage system included a 100 mm (4 in.) diameter SDR 41 perforated PVC drainage pipe wrapped (or "socked") with a nonwoven heat bonded geotextile. This socked pipe was embedded in 600 mm (24 in.) of crushed stone. The drainage system fed into a sump within a concrete standpipe which was located at the lowest elevation of the landfill cell.

Due to the nature and magnitude of the clogging of this drainage system, a large section of the landfill was exhumed. A total area of 19,000 m^3 (25,000 yd^3) of solid waste was removed to uncover the leachate collection system. Upon exhuming the collection system, numerous problems were found to be contributing to the malfunctioning of the leachate collection system. The most obvious shortcoming of the system was the geotextile encapsulating the perforated pipe. The geotextile was excessively clogged with particulates as well as biomass. The fouled geotextile was no longer allowing leachate to flow into the perforated pipe. Permittivity tests on the geotextile confirmed these findings as the original value of 1.1 sec^{-1} decreased to 8.2×10^{-4} sec^{-1}. (Permittivity of a geotextile is defined as the permeability divided by its thickness and is a commonly used geotextile value used in permeability and flow rate calculations).

A second problem was the 100 mm (4 in.) PVC drainage pipe used in this installation. It was crushed in several locations and noticeably damaged in others. This SDR 41 pipe was significantly underdesigned on the basis of strength. The damage most probably occurred at the time of installation of the pipe due to stresses imposed by construction equipment. It was noted that the pipe, although crushed, was still able to transmit leachate. This was apparent from leachate removal records and the fact that leachate flowed into the perforated pipe when the excessively clogged geotextile was stripped away from its outer surface.

The shale aggregate which surrounded the socked pipe was a third limiting factor. This collection stone was classified as a poorly graded gravel (GP) as defined by the Unified Soil Classification system. Its gradation corresponded to that of an AASHTO #57 stone which was from 6 to 30 mm (0.25 to 1.25 in.) particle size. Upon exhuming, this gravel was filled with fines and biomass to the point that it had agglomerated together forming "biorock" as shown in Figure 10(b). In its current state the gravel permeability had been reduced from 25 cm/sec to 1.2×10^{-2} cm/sec. The presence of fines and biomass were the result of years of filtering leachate.

The fourth component of this system which deserved attention was the protection layer of soil in which the leachate collection drain was embedded. The primary purpose of the protection soil was to insure that the waste did not come into direct contact with the geomembrane. The secondary purpose of the protection soil was to provide a granular media for the leachate to be transmitted down gradient to the drainage trench. The protection soil was a well graded sand (SW) as classified by the Unified Soil Classification system. Its gradation was that of AASHTO #10. Upon exhuming this 450 mm (18 in.) layer of protection soil layer, a heavily contaminated lense of fines and biomass was observed. In addition there were several regions of the protection soil that were heavily marbled with black and orange ochre staining. This staining was due to

(a) Photograph of the Leachate Collection System of Site #1. Note that Even Though the Pipe was Crushed it Still could Convey Leachate.

(b) Photograph of Several Components of the Drain of Site #1. Note that the Gravel had Agglomerated Into "Biorock" and That the Geotextile was Heavily Strained and Clogged with Sediment and Biomass.

(c) Photograph of the Geotextile Socking and Perforated Pipe of Site #1. Note that the Geotextile was Bound Around the Pipe by String Ties Placed at 1 m (3 ft.) Intervals. This Technique Allowed for Fines to Infiltrate Through the Flap Edge. In Addition Note that the Intensity of "Biofouling" was Directly Above the Perforations in the Pipe.

Figure 10 - Photographs of Site #1

metals (particularly iron) and organics in the leachate. It was apparent from observing the standing sidewall of solid waste that the majority of the leachate was draining on top of the protection soil layer and not within it. This indicated that the protection sand was no longer allowing leachate to enter and be drained. It should be pointed out that the protection sand may never have exhibited the required transmissivity for this application and is the reason why most current facilities require drainage gravel in this application rather than sand, or no protection layer whatsoever. The drainage soil must be capable of limiting the head on the liner to a maximum of 300 mm (12 in.) under current federal regulations and some state regulations and private facilities require 0.1 to 1.0 cm/sec as the minimum permeability.

A sample of the leachate was taken from this site and analyzed with the following results:

- pH = 6.9
- COD = 31,000 mg/l
- TS = 28,000 mg/l
- BOD_5 = 27,000 mg/l

In comparison to the leachates described in Table 3, this was a relatively harsh MSW leachate with high organic content and relatively high solids content. The nature of the leachate was due in part to the practice of leachate recirculation. Recirculation of leachate was intended to induce degradation of the waste mass. It must be recognized, that the practice of recirculating leachate can contribute to the clogging of geotextile and natural soil filter layers at this, and any other site where the technique is utilized.

Upon making observations in the field, all of the materials were sampled and brought to the laboratory for testing and evaluation, see Table 5. Sample S1 was of the sand layer underlying the waste and around the drainage gravel of the leachate collection system. These protection soil samples were evaluated for their permeability in constant head permeability tests, per ASTM D2434. The sand was evaluated as close as possible to its in-situ conditions, which was labeled the S1(a) value. The soil was then thoroughly washed to obtain the original, as-placed value. This material was called S1(b). As seen in Table 5(a), the resulting differences were remarkable. During the 10 year period of operation the permeability of the sand decreased from 4.3×10^{-2} cm/sec to 1.6×10^{-5} cm/sec, i.e., a reduction of over three orders of magnitude.

Table 5 - Results of Laboratory Tests on Materials of Sites #1

(a) Permeability Results of Natural Soil Materials

Number	Description	Permeability, k (cm/sec)
S1(a)	Field Sample of Biofouled Sand	1.6×10^{-5}
S1(b)	Cleaned Sand	4.3×10^{-2}
S2(a)	Field Sample of Biofouled Angular Gravel	1.2×10^{-2}
S2(b)	Cleaned Angular Gravel	25

Table 5 - (continued)

(b) Permittivity Results of Geotextiles
(The thickness of the heat bonded nonwoven geotextile was 0.038 cm)

Number	Description	Permittivity (sec^{-1})	Permeability (cm/sec)
GT1(a)	Nonwoven Heat Bonded Geotextile	8.2×10^{-5}	1.8×10^{-4}
GT1(b)	Cleaned Nonwoven Heat Bonded Geotextile	1.1	4.2×10^{-2}

Sample S2 consisted of the angular gravel taken from the leachate collection system. This gravel was located between the protection sand and the geotextile around the perforated collection pipe. The sample was also evaluated for its permeability per ASTM D2434. As with the previous sample the soil was thoroughly washed to obtain the original (as placed) value. Seen in Table 5(a) is that the average permeability coefficient decreased from 25 cm/sec to 1.2×10^{-2}, which is again over three orders of magnitude decrease.

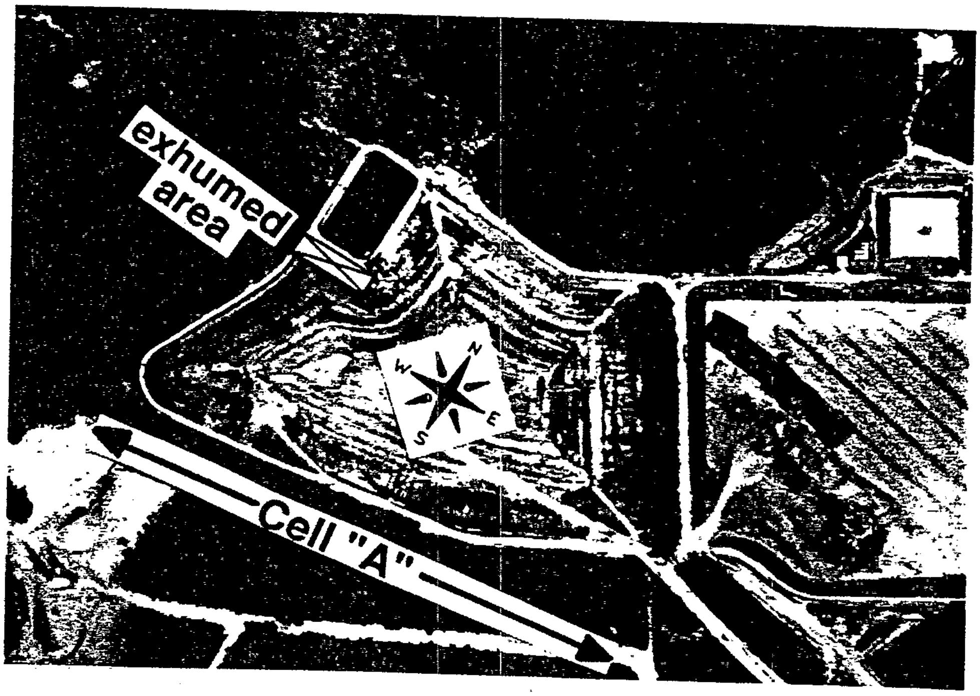
Gradation curves were generated for all soils in the test program. There was very little difference between the washed samples and the exhumed field samples. This indicated that the mass of the biomass was insignificant with respect to the overall sample's mass; however, its impact on permeability was quite another matter.

In addition to testing the sand and gravel, the geotextile of the leachate collection system was tested for its cross plane flow characteristics, see Table 5(b). Sample GT1 was taken from the geotextile which was socked around the perforated pipe in the leachate collection system. The test utilized was the geotextile permittivity test which, as noted previously, is defined as the permeability divided by the geotextile's thickness, in units of sec^{-1}. The ASTM designation for the permittivity test is D4491. This test was performed on the exhumed geotextile, labeled GT1(a), and then on the washed sampled, labeled GT1(b). Table 5(b) indicates that there was a three order of magnitude decrease in permittivity between the exhumed and cleaned samples.

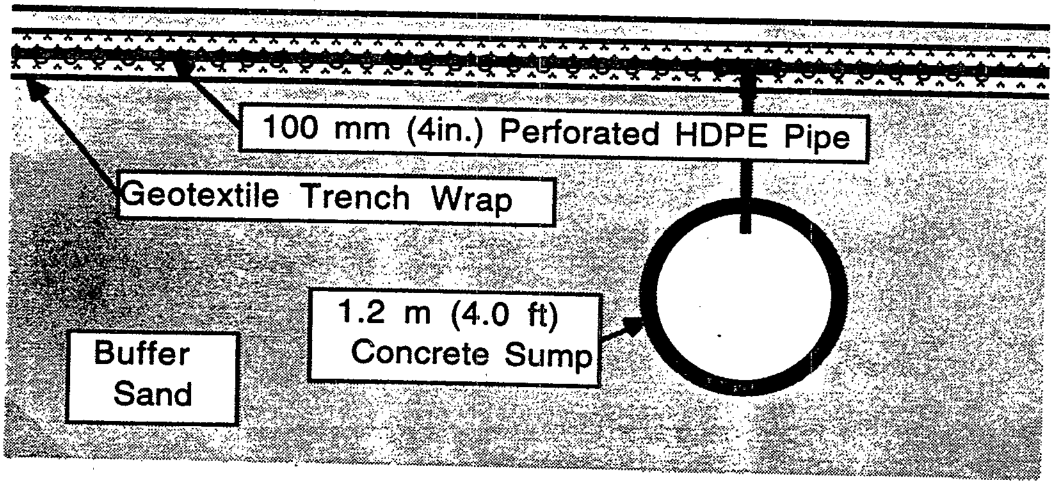
5.3 Field Exhumed Site #2

Site #1 and Site #2 were at the same landfill and were exhumed at the same time. They are differentiated because they were installed five years apart, they were constructed of different materials, they were configured very differently, they were in different locations and ultimately they were installed for different purposes.

As a relief system to the excessively clogged drain in Site #1 just described, an auxiliary underdrain was installed as shown in plan and cross sectional views in Figures 11 and 12, respectively. Note that the drain was placed as an auxiliary retrofit near the top of the geomembrane lined slope where it intersects the cover soil. The drain was installed to intercept leachate seeps on the sidewall of the landfill. Apparently the drain remediated the problem of side wall seeps but left a considerable depth of leachate in the facility, approximately 4.6 m (15 ft.).

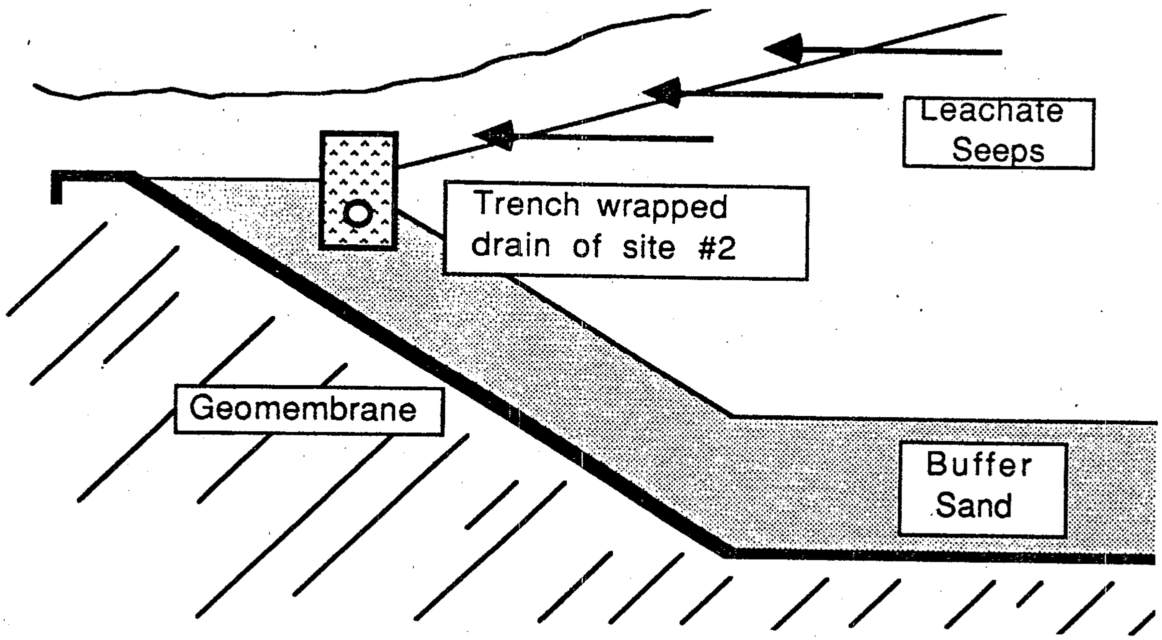


(a) Aerial Photograph of Site #2 Showing Exhumed Area of Cell “A”. Note that There were Two Drains at This One Location. The Drain of Site #2 was Designed and Constructed to Intersect Side Wall Seeps and Divert the Leachate to the Sump for Collection and Removal.

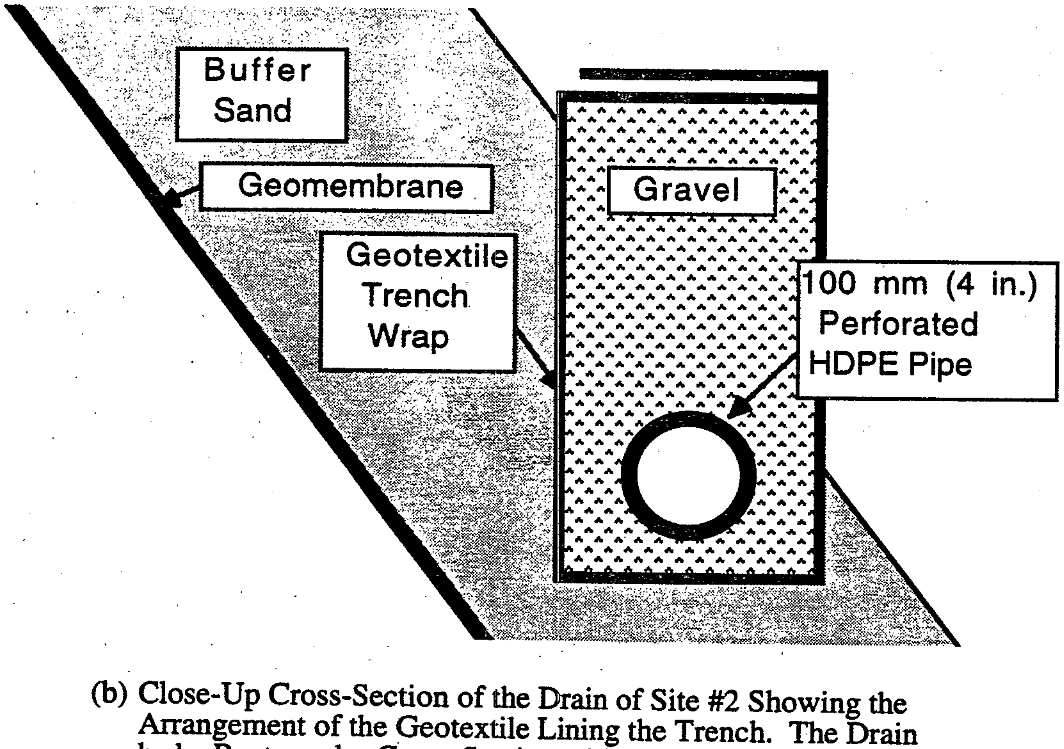


(b) Plan View of Site #3. The Drain Consisted of a Geotextile Wrapped Trench with a Perforated Pipe Filled with Rounded Gravel. The Drain was Located Near the Edge of the Landfill at the Intersection of the Cap and the Liner.

Figure 11 - Aerial and Plan of Site #2



(a) Cross-Section View of Site #2 Showing the Location of the Drain Near the Anchor Trench. This Drain was a Retrofit to the Original Design and Intended to Remediate the Problem of Side Wall Seeps.



(b) Close-Up Cross-Section of the Drain of Site #2 Showing the Arrangement of the Geotextile Lining the Trench. The Drain had a Rectangular Cross-Section with Dimensions of 600 × 1200 mm (24 × 48 in.).

Figure 12 - Aerial and Plan of Site #2

This head was directly imposed on the liner at the base of the landfill.

Upon exhuming Site #2, see Figure 13, a rectangular drainage trench measuring approximately 600 mm (24 in.) wide and 1200 mm (48 in.) high was located. The drainage trench included a 100 mm (4 in.) diameter SDR 30 perforated HDPE drainage pipe. The geotextile wrapped trench drain was filled with rounded quartz gravel of particle sizes ranging from 6 to 18 mm (0.25 to 0.74 in.). The geotextile used at this site was a 7 percent open area woven monofilament geotextile. The drain fed into a sump where the leachate was periodically removed. The drain was seen to be functioning reasonably well. As can be seen in Table 6(a), the permeability (per ASTM D2434) of the in-situ rounded gravel is 28 cm/sec which was slightly less than its cleaned (assumed to be original) value. In addition, the woven monofilament geotextile experienced only a nominal decrease in permittivity. As can be seen in Table 6(b) the geotextile decreased from its as-received permittivity (as per ASTM D4491) of 0.9 sec^{-1} to an in situ permittivity of 0.33 sec^{-1}, which was considered to be only a nominal decrease.

A sample of the leachate from the site was taken and analyzed with the following results:

- pH = 7.5
- COD = 10,000 mg/l
- TS = 3,000 mg/l
- BOD_5 = 7,500 mg/l

This is an average municipal solid waste landfill leachate (recall Table 3) with organic content in the high thousands and solids content in the low thousands.

The shortcoming of the drain was its location with respect to the elevation of the liner of the landfill. The invert of the perforated pipe in this drain was 4.6 m (15 ft.) above the liner. This means that the drain only intercepted leachate from the upper two thirds of the landfill and could not alleviate the problem of leachate head on the liner. However, the drain did eliminate the side wall seeps.

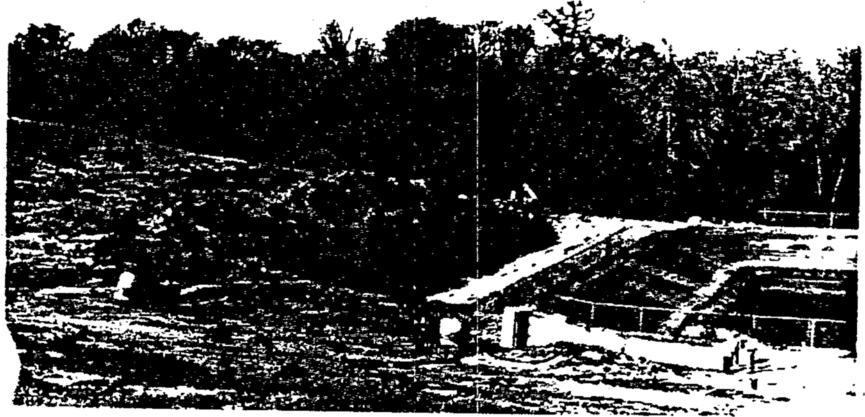
Table 6 - Results of Laboratory Tests on Materials of Sites #2

(a) Permeability Results of Natural Materials

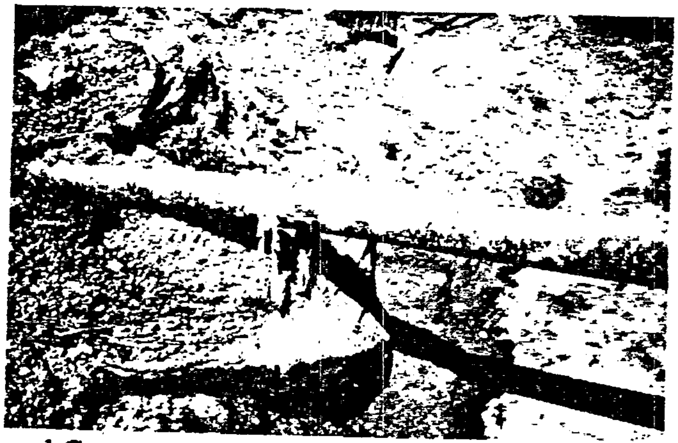
Number	Description	Permeability, k (cm/sec)
S3(a)	Field Sample of Biofouled Rounded Gravel	28
S3(b)	Cleaned Rounded Gravel	53

(b) - Permittivity Results of Geotextiles
(The thickness of the woven monofilament geotextile was 0.041 cm)

Number	Description	Permittivity (sec^{-1})	Permeability (cm/sec)
GT2(a)	Woven Monofilament Geotextile	0.33	1.35×10^{-2}
GT2(b)	Cleaned Woven Monofilament Geotextile	0.9	3.69×10^{-2}



(a) Photograph of the Exhumed Area. The Large Majority of the Waste was Exhumed Via a Backhoe and Disposed of in an Adjacent Active Facility. The Entire Volume of the Waste Exhumed Amounted to Approximately 19,000 m^3 (25,000 yd^3)



(b) Photograph of Several Components of Site #2 Drain. The Drain was Difficult to exhume due to its Location, the Nature of the Rounded Gravel and the Saturated Condition of the Waste Mass Below the Drain.

(c) Photograph of the Geotextile Trench Wrap of Site#2. Note that the Geotextile was Stained but not Clogged. This Woven Geotextile was still Functioning after 3 years of service.

Figure 13 - Photographs of Site #2

5.4 Field Exhumed Site #3

Site #3 was a relatively small industrial landfill 240 by 120 m (800 by 400 ft.) in size with 9.1 m (30 ft.) of waste depth. A plan view of the site is shown in Figure 14(a) and an isometric diagram in Figure 14(b). This facility was a double lined landfill with a leak detection system between the primary and secondary liners. Only the primary geomembrane and its overlying leachate collection system is shown in Figure 14(b) since it was the focus of our investigation.

This industrial waste facility co-disposed industrial plant refuse, slurried fines and lime stabilized waste. It will be shown that the slurried fines were the problem materials in this situation. The slurried fines were seventy percent less than a #100 sieve (0.15 mm) and forty-five percent less than the #200 sieve (0.074 mm). Both of these values were less than the apparent opening size of the geotextile used at this site which was equal to #80 sieve (0.19 mm).

From visual observations, it was determined that the leachate collection system was not functioning as intended, for example:

- The leachate in the drainage gravel was under pressure as measured by a piezometer in the area of the sump. This indicated that the leachate entered the drainage gravel at high elevations but was unable to enter the pipe network at lower elevations.
- There were several areas of ponded rain water on the surface of the solid waste. With time, the ponds grew and required surface pumping.
- The amount of leachate collected in the sump was much less than that anticipated by the operators on the basis of other similar cells at the facility.

Of the three observations listed above, the first one focused attention on the problem element of the design. By observing the leachate stained pea gravel of the leachate collection system it was assumed that the leachate entered the stone drainage system through the upper geotextile. The geotextile wrapped around the 100 mm (4.0 in.) perforated pipe however, was excessively clogged. This put the leachate in the drainage stone under hydraulic pressure causing the liquid to build pressure in the area of the sump due to the elevation head at this point. The photographs of Figure 15 show some of the conditions that existed at the site and the condition of the geotextile that socked the perforated pipe. As with Site #1 described previously, the primary cause of the malfunctioning leachate collection system was the geotextile filter wrapped directly around the perforated pipe. This type of "socked pipe" design for leachate collection systems appears to be a flawed design particularly when the pipe is of the smooth (i.e., non-profiled) type.

A series of three permittivity tests per ASTM D4491 were conducted on the geotextiles at this site. As seen in Table 7 both the upper geotextile and the geotextile socking the perforated pipe were evaluated. A sample of geotextile was also washed clean of particulates and then retested. This procedure gave a basis for comparison by approximating the as-received permittivity of the geotextile. Due to the nonuniformity of the clogging and the small size of the sample taken in the field, it was decided to condition a sample of geotextile in the laboratory to simulate field conditions before testing. This geotextile was conditioned with site specific slurried fine waste for 6 months prior to testing for permittivity.

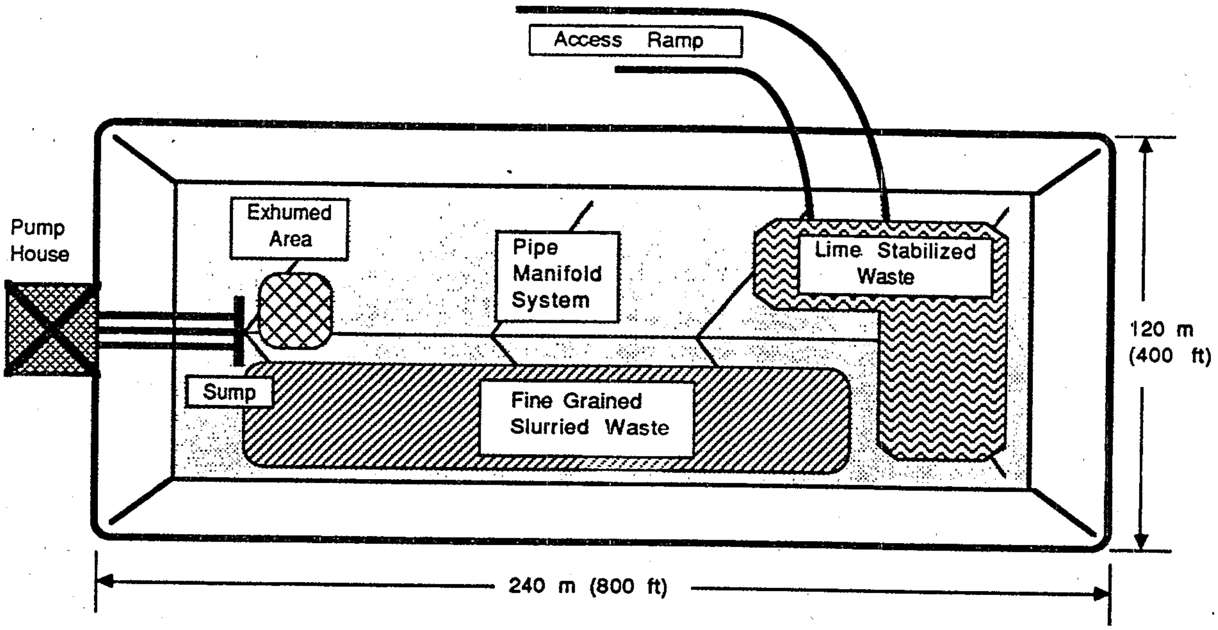


Figure 14(a) - Plan View of Site #3 Showing the Dimensions of the Cell as Well as the Exhumed Area with Respect to the Different Waste Disposal Locations.

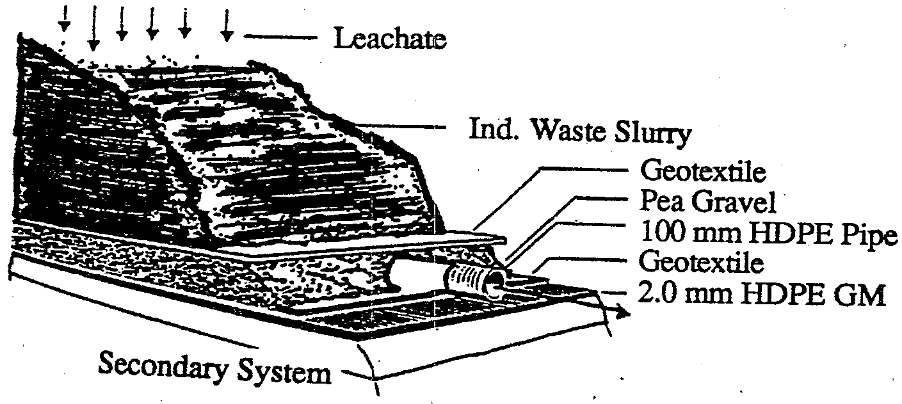
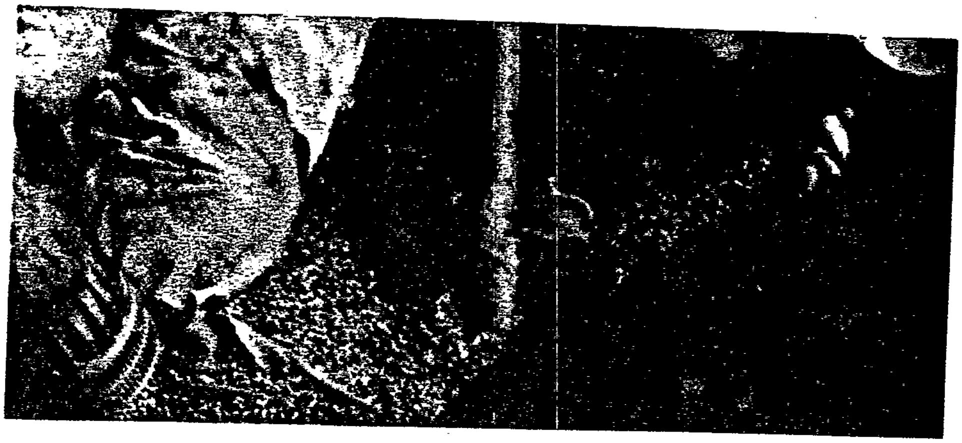


Figure 14(b) - Isometric View of Site #3 Showing the Geotextile Filter Separating the Slurried Waste from the Pea Gravel and a Second Geotextile Filter Beneath the Pea Gravel Which also Socked the Perforated Pipe.

(a) Photograph of the Landfill. A Large Majority of the Waste Consisted of Slurried Fines from an Industrial Process.

(b) Photograph of Exhuming Process of Site #3. A 6 m by 6 m (20 ft by 20 ft) Pit was Exhumed Near the Area of the Sump.



(c) Photograph of the Geotextile Torn Away from the Perforated Pipe. It was observed that Leachate in the Pea Gravel Drained into the Sump After the Geotextile Sock was Removed.

Figure 15 - Photographs of Site #3

Table 7 - Permittivity Test Results for Site 3 (ASTM D-4491)
(The thickness of the needled punched nonwoven geotextile was 0.274 cm)

Number	Description	Permittivity (sec^{-1})	Permeability (cm/sec)
1	Cleaned geotextile	1.82	0.5
2	Upper geotextile	3.14×10^{-2}	8.6×10^{-3}
3	Geotextile sock*	1.57×10^{-5}	4.3×10^{-6}

*The geotextile used for this test was the site specific geotextile conditioned with the fine slurried waste for six months and then tested.

The results of Table 7 show a two order of magnitude decrease in permeability of the upper geotextile over the as-received geotextile. Also, there was a five order of magnitude decrease in permeability of the geotextile used in socking the perforated pipe over the as-received geotextile. As will be discussed later, these test results show that the upper geotextile is much less clogged than the geotextile socking the perforated pipe.

In addition, grain size analyses were conducted on the waste and granular drainage media of the site. These results are shown in Figure 16.

Finally, an apparent opening size test was conducted on the 330 g/m^2 (10 oz/yd^2) needle punched nonwoven geotextile. A value of #80 sieve size (0.19 mm) was obtained.

In addition to characterizing the waste as far as its particle size, a sample of the landfill leachate taken from Site #3 was analyzed and resulted in the following values:

- pH = 9.9
- COD = 3,000 mg/l
- TS = 12,000 mg/l
- BOD_5 = 1,000 mg/l

As expected, this industrial waste did not have much organic matter which was indicated by relatively low readings of COD and BOD_5, recall Table 3. In contrast, the value of 12,000 mg/l of total solids (TS) was quite high. The clogging at this site was felt to be due primarily to the particulate clogging by the sludge rather than biofouling from microorganisms.

As can be seen from the site photograph of Figure 15(a), the slurried turbid waste was at a very high moisture content (greater than 40%). Figure 15(b) shows the actual exhuming process. Approximately 9.1 m (30 ft) from the sump, a pit was dug through the waste, protection soil layer, upper geotextile, pea gravel and finally reached the geotextile socking the drainage pipe. Upon going through the different layers, the geotextile socking the pipe was identified as the layer inhibiting flow of leachate into the drainage pipe. Once the geotextile sock was removed from around the drainage pipe the system flowed freely.

The geotextile above the pea gravel and below the protection sand was the same type of geotextile that was socking the drainage pipe. It was important to observe that the upper geotextile

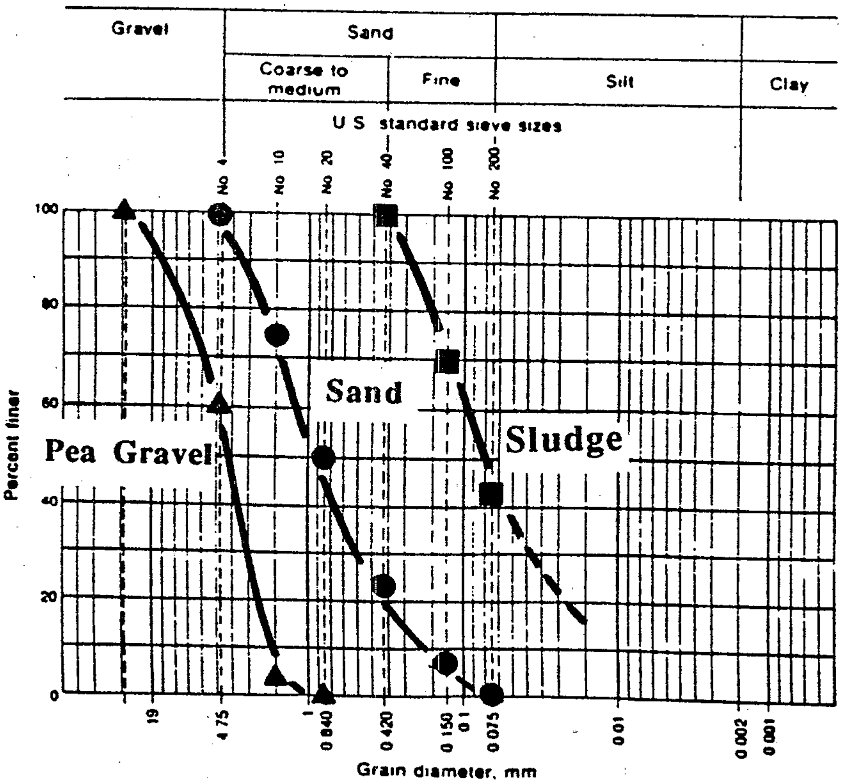


Figure 16 - Grain Size Distribution of Soils and Sludge at Site #3

appeared to be functioning while the geotextile socking the drainage pipe was excessively clogged. This was known because the pea gravel was relatively clean and full of leachate. The reason was felt to be due to the different demands on the two filters. The upper geotextile was still functioning while the geotextile surrounding the drainage pipe was not functioning because all of the landfill leachate had to flow through the perforations in the drainage pipe. At each of the 12 mm (0.5 in.) diameter perforations in the pipe, the 500 mm^2 (0.78 in.^{2}) of geotextile which filtered the leachate had clogged with particulates and the leachate was subsequently backing up in the cell. Conversely, the upper geotextile covered the entire base, or footprint, of the cell. Its available area for flow was enormous compared to the geotextile socking the holes in the drainage pipe. Hence, the available area for flow through the upper geotextile was far in excess over that of the geotextile socking the perforated holes in the drainage pipe.

5.5 Field Exhumed Site #4

The solid waste in this landfill was a mixture of domestic, light industrial, and sewage treatment plant sludge. The facility began to receive waste in 1976. The landfill utilized the concept of leachate recirculation via injection wells. In so doing leachate was reintroduced into the waste mass through a series of wells. This particular liquid management scheme was intended to decompose the solid waste into harmless constituent end products over the operating life of the facility.

This field site was different from those previously exhumed in that a leachate collection system at the base of a landfill was not being exhumed. In this case the investigation was on the geotextile filter around well casings from a methane gas extraction system. The methane produced by this solid waste landfill was being collected for use as fuel for electric power generation. The methane was extracted by 23 m (75 ft) deep wells at the top of the landfill which were on a 30 m (100 ft) rectangular spacing. The wells were made from 100 mm (4.0 in.) perforated schedule 40 PVC pipe with a needle punched nonwoven geotextile surrounding them acting as filters. The geotextiles were attached to the well casing via duct tape spaced every 900 mm (36 in.). A plan view of the landfill is shown in Figure 17(a) and a typical extraction well is shown in isometric form in Figure 17(b). The gas from these wells was collected, cleaned, condensed and it was then used as fuel to run a turbine which powered a generator and produced electricity.

The gas extracted from the landfill at a vacuum of 7 to 20 kPa (1 to 3 lb/sq. in.) of mercury was essentially saturated. This is one of the drawbacks of performing both leachate recirculation and gas extraction in close proximity to one another. The gas wells at such sites are inevitably going to attract leachate. The leachate entering the wells was very strong (compared to the sites listed in to Table 3) and had the following characteristics:

- pH = 6.1
- COD = 24,000 mg/l
- TS = 9,000 mg/l
- BOD_5 = 11,000 mg/l

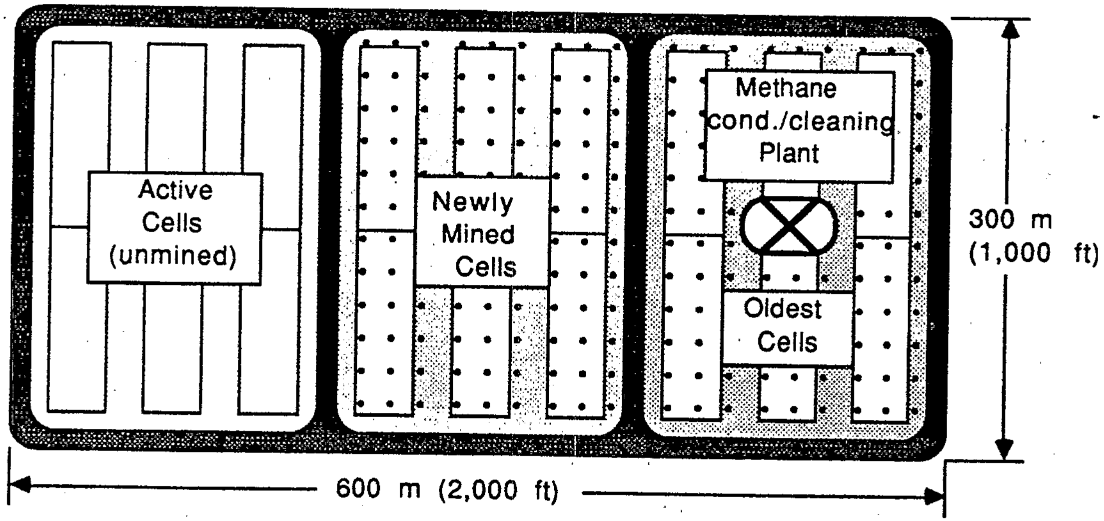


Figure 17(a) - Plan View of Site #4 Showing the Configuration and Dimensions of the Cells. The Exhumed Wells Were in the Area of the Oldest Cells and Very Close to the Methane Condensing and Cleaning Plant.

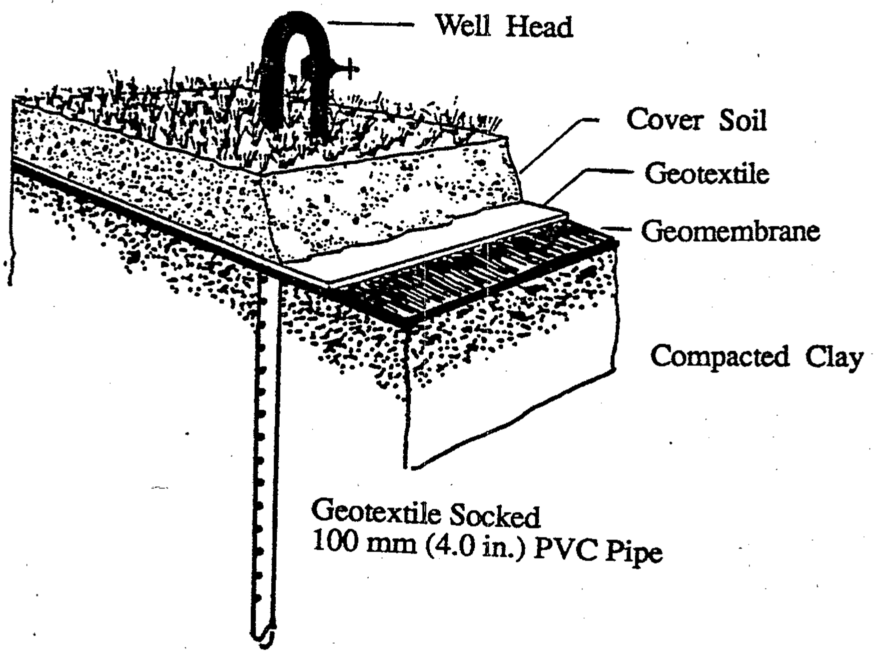


Figure 17(b) - Isometric View of Site #4 Showing the Geotextile Filter Socking the Perforated Pipe Which was Used as Extraction Wells.

Over the last year of service, approximately 30% of the extraction wells stopped producing recoverable amounts of methane. The operator tried to increase the vacuum on the wells; however, this resulted in drawing air into the well head. The result was a decrease in the methane content of the extracted gas which made it useless as fuel.

The malfunctioning wells were extracted by a small crane as shown in Figure 18. All-thread was drilled through the pipe and slings were attached to it. The well casing was pulled by extending the boom and allowing the hoist to compensate for the inclination of the crane.

Field samples of the geotextile filter were retrieved and brought back to the laboratory for testing. The samples of geotextile were retrieved from 3.0, 7.5 and 15 m (10, 25 and 50 ft) depths into the well. The samples at the two lower elevations were black with organic material and caked with fine sediment. There were particularly large deposits in those areas where the geotextile covered the pipe perforations. The sample retrieved from the upper elevation was fouled like the lower two samples, although not to such a great extent. Table 8 shows the results of permittivity tests conducted on the three exhumed geotextile samples as well as a geotextile sample which was washed clean. The cleaned geotextile was tested to give an estimate of the as-received permittivity of the needle punched nonwoven geotextile. The average thickness of the four geotextiles tested was 0.22 cm. This value was used in the conversion of permittivity to permeability.

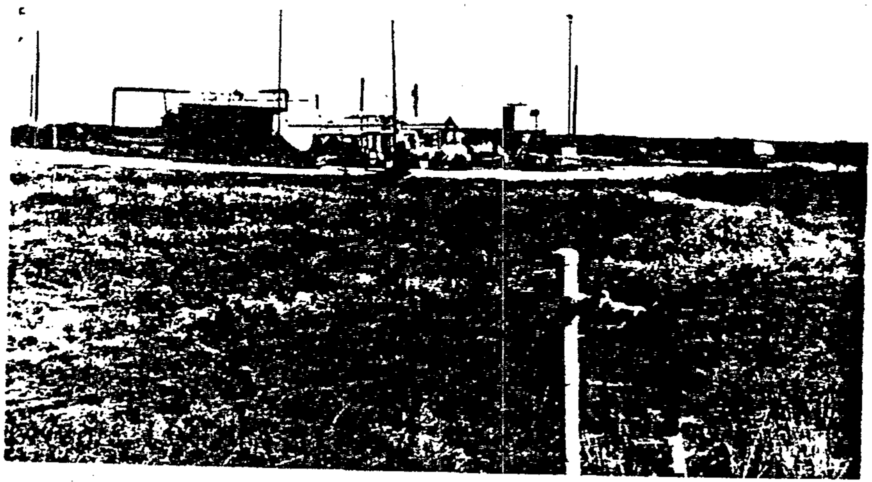
Table 8 - Permittivity Test Results of the Geotextiles of Site #4

(The thickness of the nonwoven needle punched geotextile was 0.22 cm)

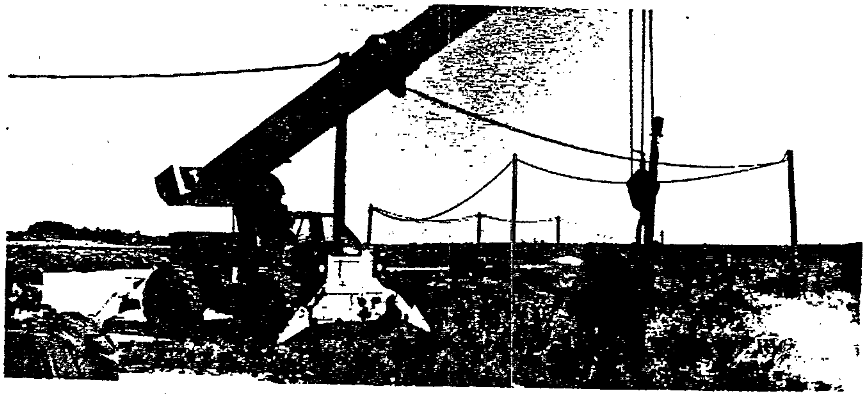
Description	Permittivity, (sec^{-1})	Permeability, (cm/sec)
Washed Geotextile	1.1	2.3×10^{-1}
Geotextile from 3.0 m (10 ft) depth	1.7×10^{-2}	3.8×10^{-3}
Geotextile from 7.5 m (25 ft) depth	7.3×10^{-5}	1.6×10^{-5}
Geotextile from 15 m (50 ft) depth	3.4×10^{-4}	7.5×10^{-5}

Note that there was a decrease in permeability by as much as four orders of magnitude. This decrease in permeability substantiated the lack of methane recoverability from the geotextile socked gas extraction wells. As proof that the landfill was still producing recoverable amounts of methane, new wells of a different design were placed in the same location as the old ones and significant amounts of methane emerged. It also should be mentioned that the permittivity of the geotextile at the 3.0 m (10 ft) depth was not as low as the geotextile permittivities at the 7.5 m (25 ft) and 15 m (50 ft) depths because the geotextile did not encounter as much leachate at the upper elevations in the landfill.

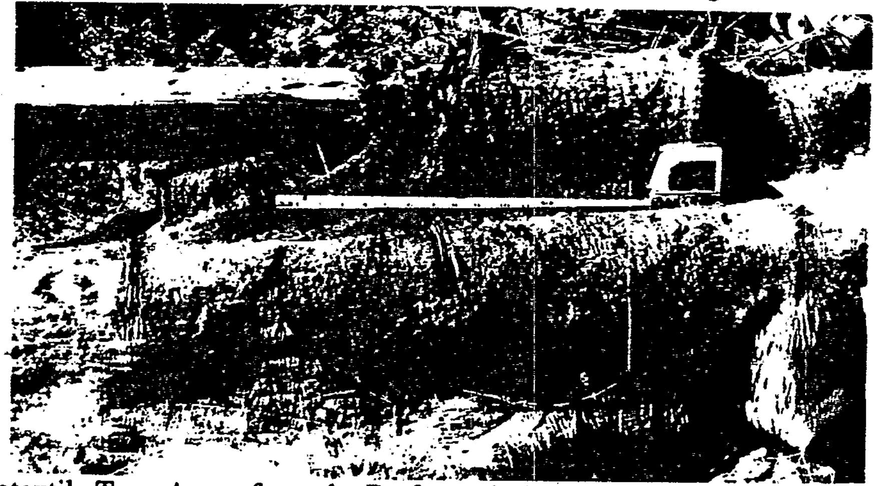
Due to the poor performance of the extraction wells the owner of the facility changed the well design over time. The casing was changed from drilled to slotted well screens which increased the surface area of the pipe penetrations. The geotextile sock was eliminated from the



(a) Photograph of the Top of the Oldest Landfill Cell. The Photograph shows the Head of a Typical Extraction Well and the Methane Condensing and Cleaning Plant.



(b) The Exhuming Process of Site #4. A Crane was used to remove the Wells so that Additional Wells Could be Set in the Same Locations Utilizing the Existing Manifold System.



(c) The Geotextile Torn Away from the Perforated Pipe. Note the Heavy Layer of Fines and Biomass on the Geotextile Filter which was Limiting Flow of Methane into the Pipe.

Figure 18 - Photographs of Site #4

design completely. The elimination of the filter was a result of excessive clogging of the geotextile. Last, but not least, the diameter of the augured hole was increased from 200 mm (8 in.) to 300 mm (12 in.). This increase in diameter allowed for an annulus of gravel to surround the perforated pipe and also insured that the gravel extended for the entire depth of the well.

5.6 Summary of Field Exhuming Study

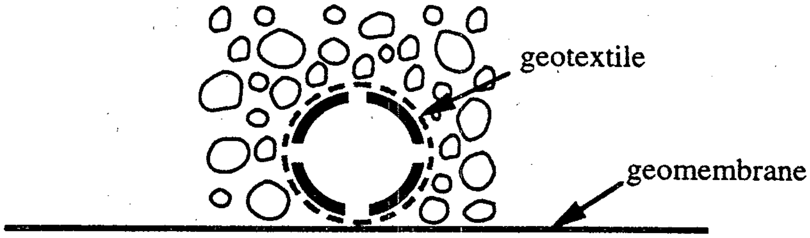
The exhuming of leachate collection system filters was critically important to the overall thrust of the project. While sparse in number, the four sites provide ground truth and a target for the design method development to follow. At each of the four sites we acted as technical observers and performed the following tasks:

- Observed and recorded (via reports, photographs and video) the exhuming processes.
- Sampled the various components (leachate, geotextiles and soils).
- Repaired geotextiles and soil components of the leachate collection system.
- Conducted required tests on the various samples (natural soils and geotextiles) taken from the field.
- Cleaned and tested the various materials as close as possible to their as-received condition and compared the results to the in-situ values.

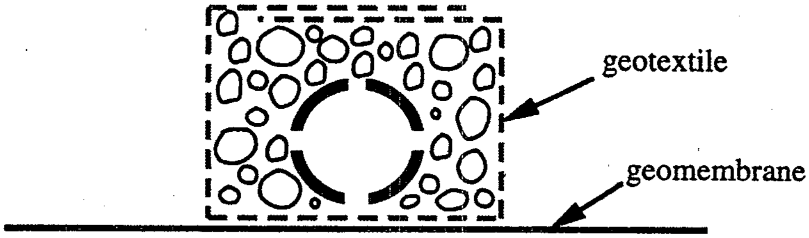
In addition to these tasks we hand excavated the final thickness of solid waste and buffer soil overlying the leachate collection filter so as not to cause removal damage. Bulk excavation of the solid waste with construction equipment was performed by the facility operator or by an outside contractor.

The experience of these four sites allows us to conclude that geotextile socking of perforated pipe for the application of leachate collection filters of solid waste landfills simply does not work. As seen in the different location sketches of Figure 19, the positioning of a geotextile as a pipe sock, trench wrap or aerial filter is significant with respect to its long term performance. The ratio of the leachate to be filtered to the available open area of the downstream drain is a critical design detail. It should not be a surprise that an aerial filter will function in an application where the same type of filter material placed around a perforated pipe will fail. All of the excessively clogged sites that were exhumed were socked pipe filters. It should also be noted that all of the pipes exhumed were of the solid wall type. It is possible that profiled or corrugated pipe would have performed better, but this is not known to be the case.

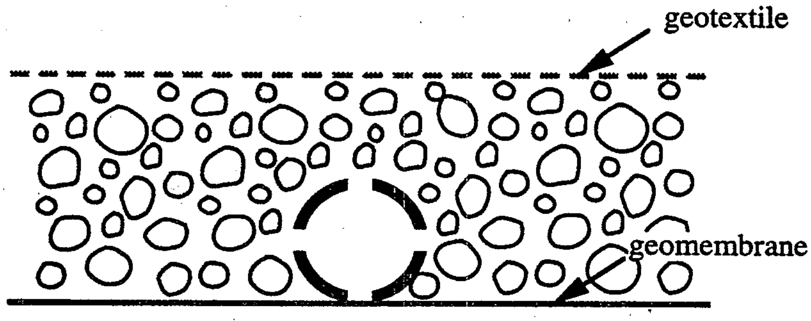
In addition to the configuration of the filter with respect to the drain, a major consideration in the performance of the drain is the type of leachate being filtered. If the leachate had high total suspended solids and high organic content, a filter criteria which is based on water as the permeant will probably not be suitable. See Table 9 for a comparison of leachates from these four exhumed case histories. In cases where the leachate contained high amounts of organics and solids, the available filtration area of the filter and probably the opening size of the geotextile filter must be increased to encourage transmission of the suspended matter through the filter rather than causing excessive clogging. Recognize that in so doing, the suspended materials will move into the drainage pipe network. Thus, the drainage pipe network may have to be set at a high gradient



(a) Socked Pipe



(b) Trench Wrap



(c) Aerial Filter

Figure 19 - Various Leachate Collection Filter Configurations

and/or be available for periodically flushing and cleaning to insure its effectiveness.

Table 9 - Summary of the Leachate Characteristics of the Exhumed Field Sites

Site No.	Landfill Type	pH	COD (mg/l)	TS. (mg/l)	BOD_5 (mg/l)
1	Municipal	6.9	31,000	28,000	27,000
2	Municipal	7.5	10,000	3,000	7,500
3	Industrial	9.9	3,000	12,000	1,000
4	Municipal	6.1	24,000	9,000	11,000

Additional summarizing details for the four exhumed sites are given in Table 10. It is clear from these particular sites that geotextile filters of leachate collection systems can become excessively clogged with fine particles and/or microorganisms. Three of the four sites had excessively clogged geotextile filters. All three of the excessively clogged geotextile filters were designed as filters socking perforated solid wall pipe. It is obvious from these findings that the practice of socking perforated pipe for the application of leachate collection filters in solid waste landfills is not good landfill practice and is fraught with problems.

The task of designing a filter at the base of a landfill facility for long-term transmission of leachate is difficult. Leachate, while varying tremendously in its characteristics, is invariably laden with suspended solids, dissolved solids and microorganisms as was seen in Table 9. The filter, being the component with the smallest opening sizes is the first to encounter this leachate. The choices are to either pass the sediment and microorganisms or collect them entirely or partially. The first choice risks downstream drain clogging, the second excessive filter clogging. For geotextile filters this decision must be made relatively quick in comparison to a soil filters due to their thinness. As described earlier, the possible preferred mechanism for geotextile filtration is by depth filtration [5], but many geotextiles used are quite thin. One way of assessing the potential behavior of such filters is the ASTM D1987-91 test method. This test is the subject of the following laboratory portion of this project.

Table 10 - Overview of Exhumed Leachate Collection Systems

Site No.	Waste Type	Age Upon Exhuming	Liquid Management Scheme	Performance Upon Exhuming	Critical Element in Drainage System
1	domestic and light industrial	10	leachate recycling	excessively clogged	geotextile filter
2	domestic and light industrial	6	leachate recycling	marginally clogged	drain location
3	industrial solids and sludge	0.5	leachate withdrawal	excessively clogged	geotextile filter
4	domestic and rural	6	leachate recycling	excessively clogged	geotextile filter

6. Laboratory Investigations for Allowable Flow Rate

This section describes the laboratory test setup, materials used and experimental results of flow rate (permeability) tests which constituted the laboratory testing portion of the project [9]. The experimental design consisted of 48 separate flow columns with four replicate sets of 12 different columns (10 with geotextiles and 2 with sands). The four replicate sets were each permeated with the following liquids:

- water (as the control liquid)
- leachate "D"
- leachate "P"
- leachate "L"

Furthermore, the permeation rate through each of the columns was varied between very high, high and intermediate; thus 144 long term flow tests were conducted. All tests were continued until flow equilibrium or excessive clogging was reached. By determining the equilibrium permeability of each filter at three higher-than-typical field flow rates, the resulting curve can be back extrapolated to obtain the allowable permeability of the filter in question at a site specific flow rate. Thus these tests represented accelerated flow rate tests to determine site specific allowable flow rates of the various filters under investigation.

6.1 Test Setup

The laboratory test set-up for this project was aimed at determining the relative potential for particulate and microorganism clogging of geotextile and soil filters as they were permeated with leachate from various municipal solid waste landfills. The test method involved measuring the permeability of the filter over an extended period of time to determine the decrease in permeability caused by the accumulation of particulates and microorganisms. The permeabilities were measured under constant head conditions initially and as flow rates decreased, the permeability measurements were changed to falling head conditions.

Before discussing the specific test setup, details of the test method will be described. A geotextile filter specimen or soil/geotextile composite specimen was positioned in a flow column (permeameter) so that leachate could permeate perpendicular to its plane. The setup provided great liberty in selecting the test conditions for which the column was exposed for the duration of the test. Test specimens could be exposed to either partial wetting or full saturation. Partial wetting resulted in aerobic conditions. Full saturation resulted in anaerobic conditions. It should be noted that all of the tests in this study were conducted under fully saturated conditions. This decision was made on the basis that the base of a municipal solid waste landfill is generally considered to be anaerobic.

The main element of the test setup was the flow column or permeameter. The terms flow column and permeameter will be used interchangeably in this report. The flow column conformed to the specification of the ASTM D1987-91 Test Method. Each device held a specific geotextile specimen on a cylindrical mount of 100 mm (4.0 in.) inside diameter. The geotextile was temporarily attached to the mount by a model glue. The geotextile was then permanently bonded to the ring with a soldering iron or hot scissors. Finally, the specimen mount was seated against a containment ring which was a 100 mm (4.0 in.) coupling with PVC solvent cement. For these tests, the bond had to last for up to one year in the presence of leachate and in some cases with soil overburden.

Upper and lower tube sections are then joined to the containment ring. A support gravel of 37 mm (1.5 in.) size was placed under the geotextile. To do so, the upper tube, coupling and lower tube assemblage was inverted and the geotextile was supported on a cylindrical stump which fit into the lower assembly. Rounded quartz gravel filling the lower tube was then placed on the geotextile. The lower cap was then bonded to the lower tube thus encapsulating the gravel in the lower half of the flow column. The column was inverted to its proper orientation and was ready to be threaded onto a 32 mm (1.25 in.) male adapter of the recirculation system. Some of the gravel used was small enough in size to fit through the 1.25 in. threaded opening at the base of the flow column. To ensure that this would not happen a stainless steel nail was drilled into the male adapter connecting the flow column to the recirculation system. This effectively halved the opening and held the gravel in the flow column while not restricting flow. It should be noted that the upper end cap was not yet been bonded to the upper 100 mm (4.0 in.) tube. This afforded a last chance to inspect the geotextile containment ring seal as well as the ability to carefully place an optional soil layer above the geotextile. The controlled densification of the soil would not be possible with the upper end cap in place. The final step in the assemblage of the flow column was to bond the upper end cap to the upper tube.

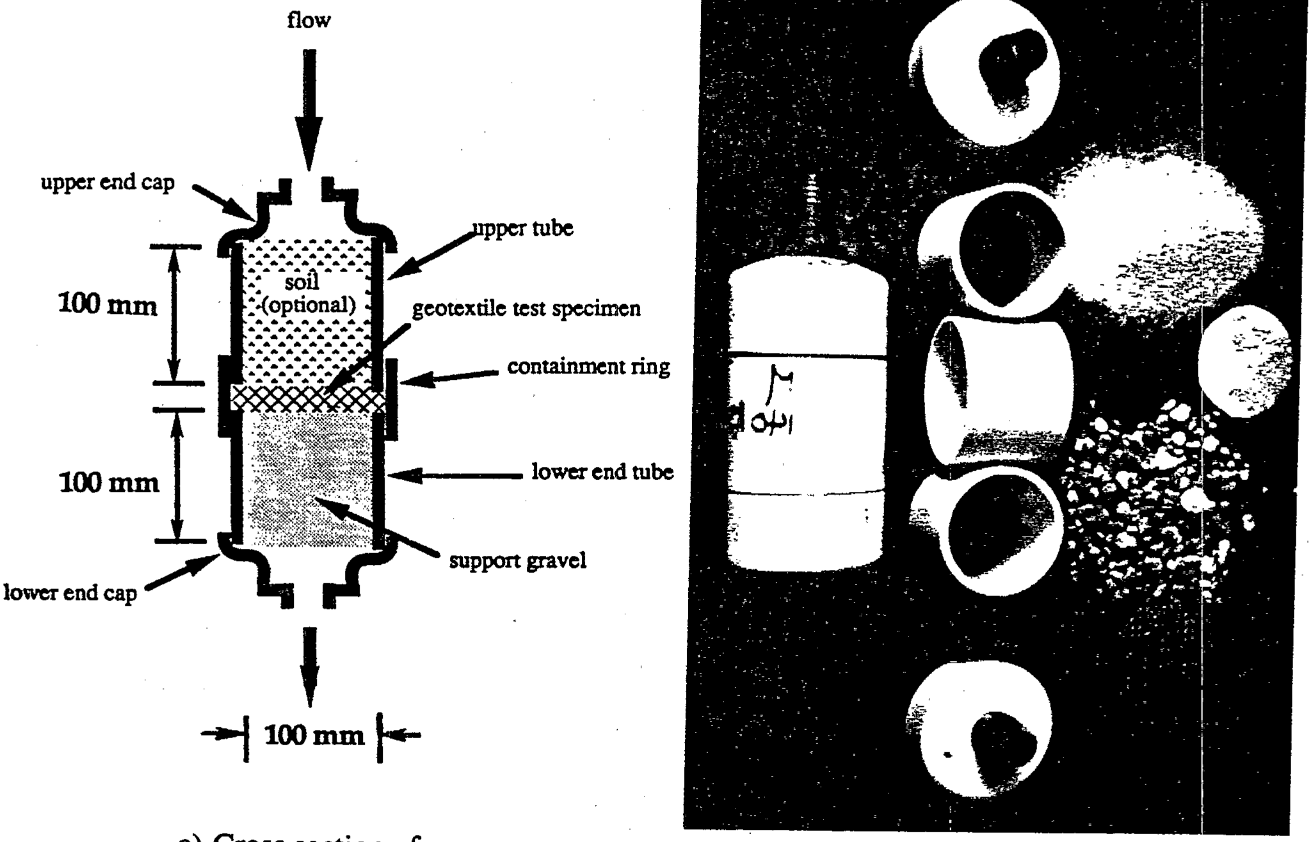
Figure 20(a) shows a schematic diagram of the system, while a photograph of the flow column is shown in both component and assembled form in Figure 20(b).

Once the flow column was assembled it was connected to a leachate recirculation system. The recirculation system was designed to run continuously and uniformly for at least one year. It was constructed entirely of plastic materials thus ensuring that buildup of rust would not occur.

The recirculation system consisted of piping network connecting twelve flow columns in series to a reservoir recirculation system. The system was capable of supplying approximately 95 l/day (25 gal/day) of leachate to each column. As can be seen from the cross section of Figure 21(a) and the photograph of Figure 21(b) the network consisted of two valves, a threaded "T", a union, an inlet manifold and an outlet manifold for each flow column.

The valves in the system regulated the flow through the columns as well as provided a means of isolating any one column from the rest of the set of twelve during the flow reading periods. The threaded "T" provided access to the flow columns so that a telescoping "L" tube assembly could be attached to the columns for permeability testing. As can be seen in Figure 22(b), the "L" tube assembly could be made from different diameter tubing and could be utilized in both the constant and falling head measurement modes. The union was only included in the piping network to connect and disconnect the columns from the recirculation system at the conclusion of the test series. Without the unions the recirculation system could not be reused.

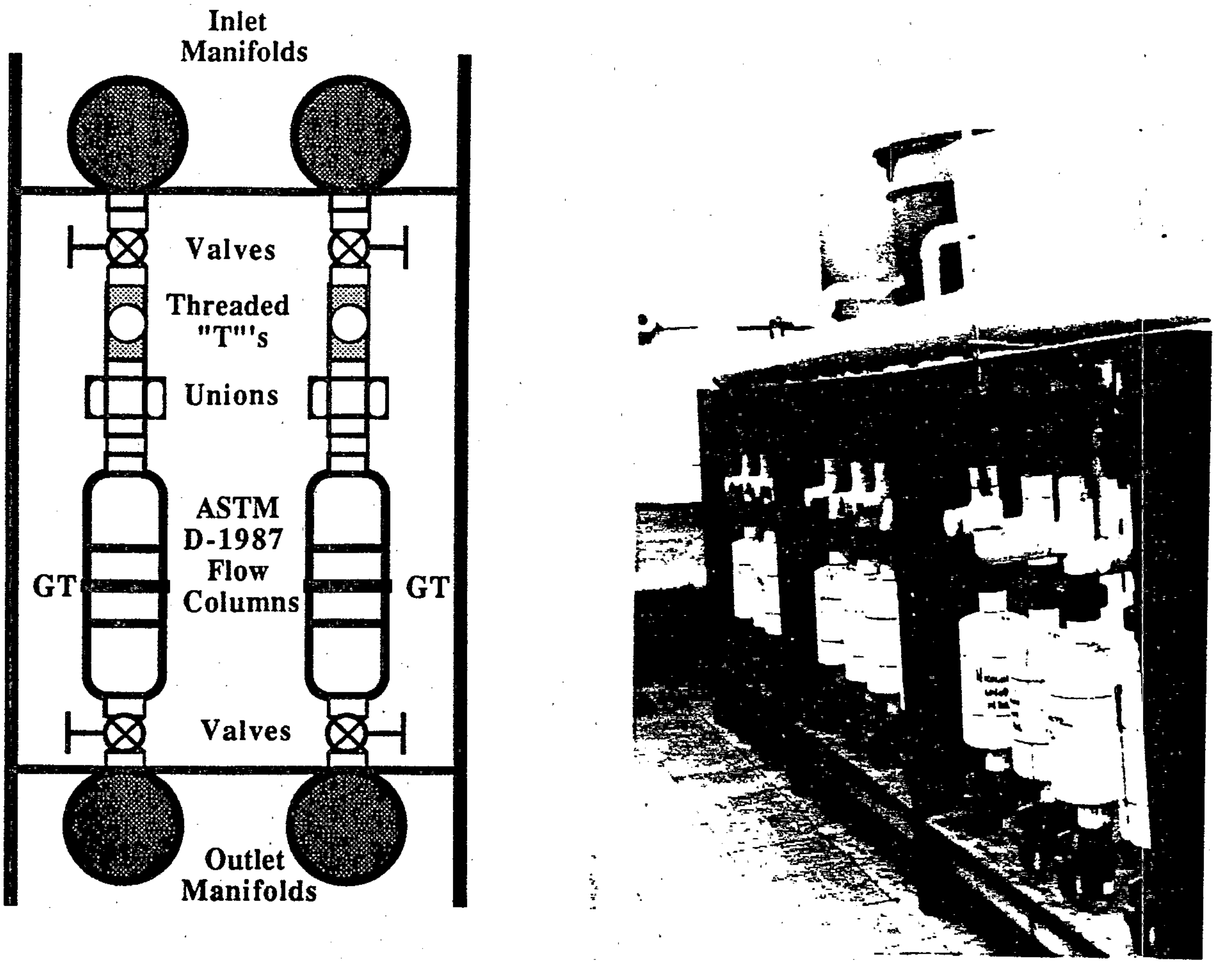
The inlet and outlet manifold systems consisted of a 3.0 m (10 ft) piece of PVC pipe tapped



a) Cross section of permeameter

b) Photograph of Components and Total Assembly

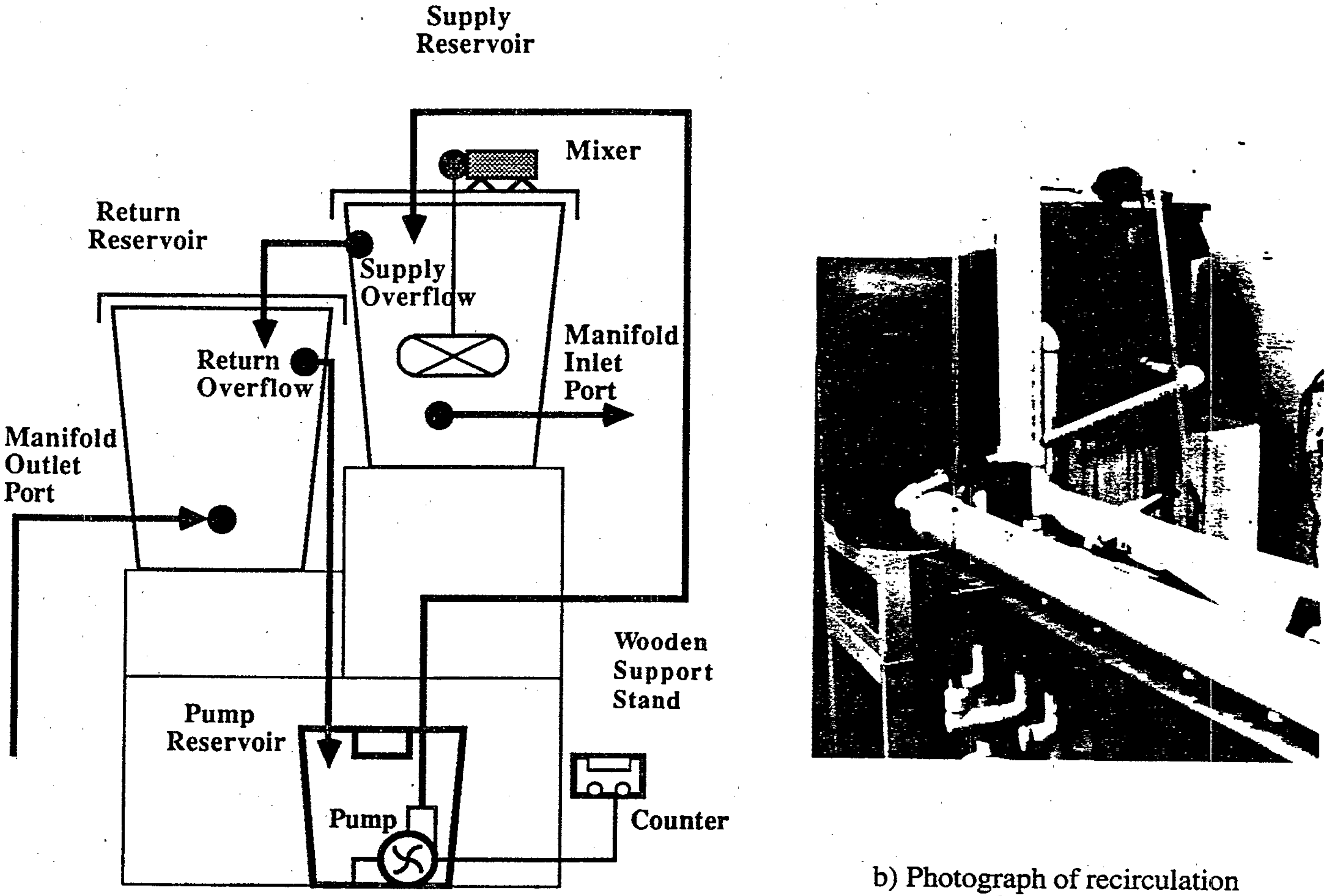
Figure 20 - Flow Columns Developed During This Study and Adopted by ASTM as D-1987 Test Method



a) Cross section of flow column racks showing two typical permeameters

b) Photograph of flow column rack showing twelve permeameters

Figure 21 - Flow Columns and Support System with Associated Pipe Network



a) Cross section of recirculation reservoir system

b) Photograph of recirculation reservoir system

Figure 22 - Reservoir Recirculation System

and fitted to the twelve separate test cylinders to accommodate the leachate flow. One end of the manifold was attached to the reservoir and the other end was fitted with a 100 mm (4.0 in.) diameter elbow and standpipe to act as an air vent for the system. The final component of the test setup was the reservoir system for the recirculation of leachate to the flow columns. As seen in Figure 22(a) and (b), the system consisted of a series of three reservoirs. The supply and return reservoirs each held 190 litre (50 gal) of leachate whereas the pump reservoir held only 57 litre (15 gal) of leachate. The supply reservoir was connected to the inlet manifold while the outlet manifold was connected to the return reservoir. An elevation head of 300 mm (12 in.) was imparted on the two reservoirs by separating the supply and the return overflows by this exact amount. This differential in elevation head was the driving force of the entire system. The supply reservoir was filled by way of the 57 litre (15 gal) pump reservoir contained within a wooden stand and upon which the other two reservoirs were located. The pump was a centrifugal pump equipped with a automatic float switch which triggered a charge of 57 litre (5 gal) of leachate through a pipeline up into the supply reservoir. The submersible centrifugal pump was also equipped with a solid state counting module which accumulated the number of times the pump was triggered over the course of a day. Knowing this number afforded a means to calculate the average amount of leachate that passed through each column.

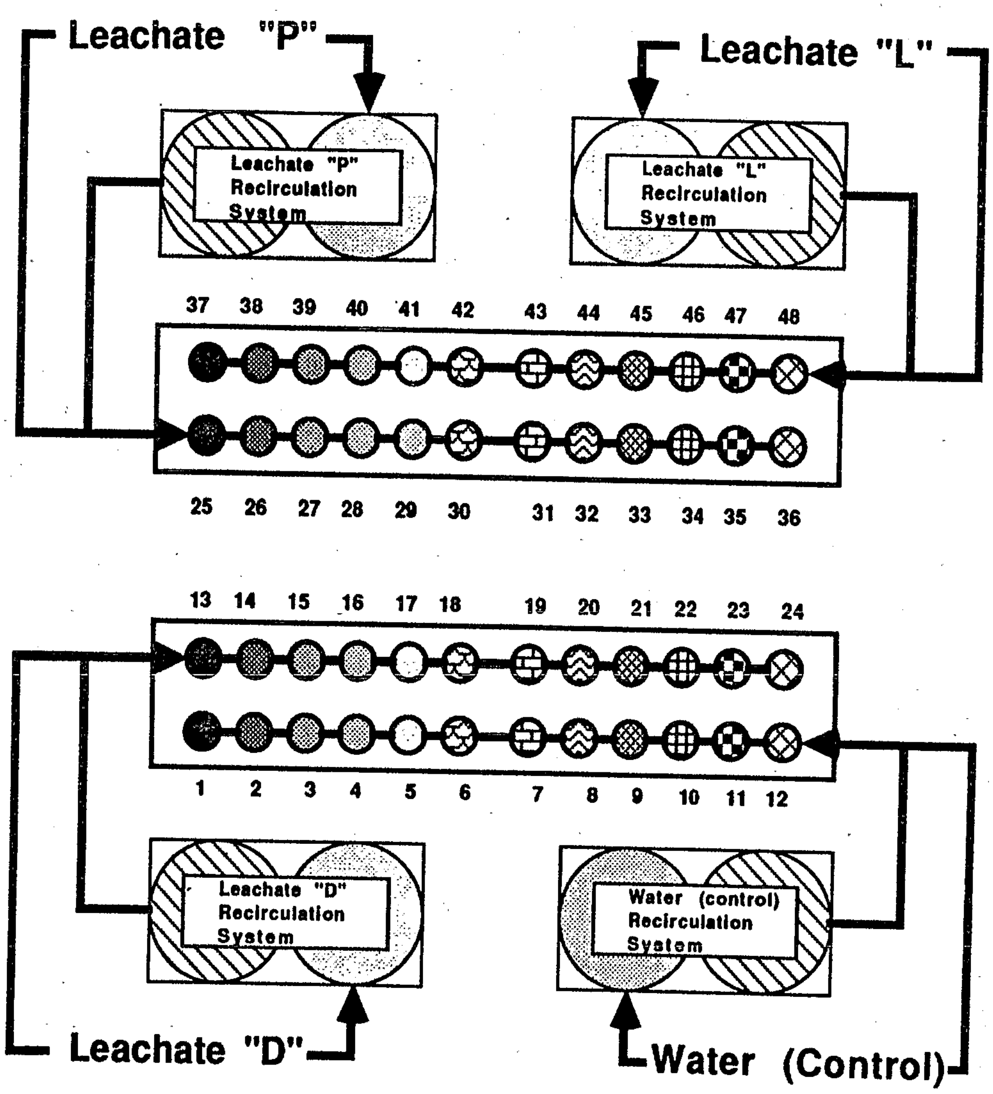
The entire system of four 12-column units, with associated leachate reservoirs is shown in plan view in Figure 23. The set-up was totally contained within an isolated room in a laboratory separated from other activities. The room was well ventilated to ensure that methane, carbon monoxide and sulfur dioxide did not accumulate. In addition, standard temperature and humidity were maintained in the room for the duration of the test program.

6.2 Test Procedure

Due to the wide range of flow rates that were to be encountered in using the flow columns just described, three different testing procedures were used to obtain permeability values. The constant head test was used to determine permeabilities in the range of 1 cm/sec to 0.01 cm/sec. Falling head tests, utilizing a 34 mm (1.33 in.) diameter tube, were used for determining permeabilities in the range of 0.01 cm/sec to 10^{-4} cm/sec. Permeability values less than 10^{-4} cm/sec were also obtained via the falling head procedure; however, in this case a 9.7 mm (0.38 in.) diameter tube was used for the measurements due to better sensitivity and control.

Since the procedure for running the different leachate flow streams are identical, the general procedure shared by all the flow columns will be described and then the differences involved with running the constant head and falling head tests will be addressed. Additional details are found in the thesis of G. R. Koerner [10].

The flow columns were periodically tested for permeability at various intervals of leachate throughput. Initially, tests were conducted every day. This frequent interval of testing was due to the fact that during the first stage of the test the various materials being permeated were undergoing considerable change. After a month of testing, the testing frequency decreased to once a week. In



LEGEND

Column Number	Filter Material
1, 13, 25 & 37	Ottawa Sand
2, 14, 26 & 38	Concrete Sand
3, 15, 27 & 39	N 32 W
4, 16, 28 & 40	N 14 W
5, 17, 29 & 41	N 7 W
6, 18, 30 & 42	H 4 NPNW
7, 19, 31 & 43	H 8 NPNW
8, 20, 32 & 44	H 16 NPNW
9, 21, 33 & 45	T 4 HBNW
10, 22, 34 & 46	A 10 W
11, 23, 35 & 47	P 6 NPNW
12, 24, 36 & 48	N 22 NW/W

Figure 23 - Plan View of the 48-Permeameter Set-Up

testing the 48 flow columns, the three sets of leachates were utilized first and then the water (control) was evaluated. This practice allowed the room to be washed clean of leachate at the end of the day with water.

The following sequence was the procedure for running a series of 48 permeability tests.

1. The valves leading from the supply and return tanks were closed.
2. Leachate was released to the elevation of the threaded "T" connection by opening the main drainage port of the system. Note that at no time during the permeability measurement process was the geotextile or geotextile/soil specimen unsaturated. This was important to verify because of the premise that the system was functioning under anaerobic conditions.
3. The valves upstream and downstream of the twelve flow columns in a series were closed.
4. Outlets were placed on the discharge end of the system.
5. The individual column permeabilities were then measured. Typically, the test run was started at columns 12, 24, 36 and 48 and work progressed down the rack. See Figure 23 for the ordering of the columns along the rack. This insured that the two soil filled columns were tested last. Back pressure trapped in the system would cause the sandy soils to become unstable and this obviously would disturb the soil matrix. For this reason, 5 minutes was allowed to pass before testing any column after it was disconnected from the recirculation system.
6. The upstream and downstream valves were opened for a single flow column to be tested.
7. The plug was removed from the threaded "T".
8. The "L" tube assembly was attached to the threaded "T".
9. A constant head of the permeant was regulated into the "L" tube assembly maintaining a constant hydraulic head of 50 mm (2.0 in.). This value was selected because it was the prescribed head for measuring geotextile permeability according to ASTM D4491.
10. Upon maintaining a constant head for 1 minute, 1000 cc of liquid was collected from the outlet, the time for which was recorded.
11. After the time was confirmed from a second run, and was in agreement with the first reading, the test was complete. Caution was taken so as not to have excessive hydraulic head over the geotextile specimen at this stage in the testing sequence. The danger here was of flushing the geotextile free of particulates and biomass leading to erroneous results.
12. It was felt to be good practice to check the outlet works for zero datum after the test was complete. This is done by waiting a few minutes after the test was over and checking if the residual water level was zero after the system reached equilibrium. If the test did not return to the zero datum mark, the test was rerun.
13. Upon confirming the result, the test was over and the next flow column in the series

was evaluated. To do so, the upstream and downstream valves of the flow column just tested were closed. It was important to isolate one column at a time during this phase of the testing process.

14. As described previously, the flow columns decreased in permeability over time. As the columns fell below 0.01 cm/sec, a falling head test was conducted in the same "L" tube assembly as with the constant head test. The only difference in the procedure was that the flow rate was so slow that a higher head and longer time was required to expedite data collection. Therefore the "L" tube assembly was filled to a elevation head of 900 mm (36 in.) above the geotextile and allowed to fall to 450 mm (18 in.) above the geotextile. Time was recorded during which the 450 mm (18 in.) charge of leachate traveled through the flow column. This procedure did not require a known quantity of leachate to be collected from the outlet works. Only the time for the 450 m (18 in.) charge of leachate to pass through the geotextile was needed for the required permeability calculation.
15. During the progress of permeability reduction, the flow columns were again in need to shorten the time interval for testing. When the permeability of the columns was less than 10^{-4} cm/sec, falling head tests lasted over 30 min. The easiest way to shorten this testing time was to decrease the volume of permeant passing through the column. Hence the diameter of the standpipe in the "L" tube assembly was reduced from 34 mm (1.33 in.) to 9.7 mm (0.38 in.). The measurements were conducted at 300 mm (12 in.) of head independent of the diameter of the standpipe.
16. After the twelve flow columns in a series had been completed, the columns to the recirculation system were reconnected. The supply and return reservoirs were again connected to the inlet and outlet manifolds and all valves were opened slightly except for the valve leading to the supply reservoir. Note that the valves were opened slightly and not opened completely. This allowed the system to back saturate slowly, purging the air out of the supply manifold stack. It was important not to back saturate the columns quickly. The process took at least 30 minutes to perform. If one was too quick, the risk of backflushing the entire twelve flow columns was taken. This would lead to irregular and false results.
17. After the entire system was saturated again, the valves were set to regulate 95 l/day (25 gal/day), or other targeted value, through each column. The system was then left to recirculate until the next flow measurements were made.
18. Upon completing a series of 12 measurements, the next set of 12 were evaluated with a second leachate, then the third set of 12 with the third leachate and finally the last set of 12 with water as the permeant.

6.3 Leachate, Filters and Flow Rates Used in Tests

As illustrated in Figure 23 there were four identical flow rate systems in the study. Water (as the control) was used in one series and three different municipal solid waste leachates were used in the others. The leachates were retrieved fresh from their respective landfills every two weeks. Over the entire test period (lasting from 120 to 365 days), a significant change in the leachate characteristics was observed. What was originally intended to be three very distinct and different leachates, ended up being three quite similar leachates with only slightly different characteristics. The average values over the test period are listed in Table 11. Here it is seen that the BOD_5 and TSS values (the two preferred indicator values for filter clogging assessment) were quite close to one another.

The various filters used were both natural soils (sands) and geotextiles. The two soils were Ottawa sand and concrete sand, each in the medium sand size range, the difference being the particle shape and gradation. Ottawa sand was rounded, while concrete sand (being quarried) was angular. Furthermore, the Ottawa sand was poorly graded while the concrete sand was well graded. The Ottawa sand was approximately an order of magnitude higher in its permeability and was classified as an "SP" soil by the Unified Classification System. The concrete sand was classified as "SW".

Table 11 - Description of Average Leachate Characteristics Evaluated in this Study

Permeanent	Leachate Management	Landfill Age	pH	COD (mg/l)	BOD_5 (mg/l)	Chlorides (mg/l)	TSS (mg/l)	TDS (mg/l)
Water (Used as Control)	n/a	n/a	7.0	20	7	5	0	0
Leachate "D"	Recycled through the landfill	10	6.4	3500	2000	1000	300	4600
Leachate "P"	Continuously removed and pumped to a treatment facility	5	7.1	4000	2500	1500	500	4000
Leachate "L"	Continuously removed and pumped to a treatment facility	2	6.7	3000	2100	3500	600	5900

n/a = not applicable

The ten geotextiles used in this study had properties shown in Table 12. It was felt that this selection resulted in a relatively complete assortment of geotextile filters used as leachate collection filters at solid waste landfills. All ten of these geotextiles are commercially available and most (if not all) are currently being used as primary leachate collection filters in landfill applications.

Table 12 - Description of Geotextile Characteristics Evaluated in This Study

Description	Polymer Type*	Structure	Mass per Unit Area		POA (%)	Apparent Opening Size			Permittivity (sec^{-1})	Permeability (cm/sec)
			g/m^2	oz/yd^2		(mm)	(in.)	(sieve #)		
N 32 W	PP	woven	200	6	32	0.71	0.028	n/a	4.8	0.34
N 14 W	PP	woven	270	8	14	0.81	0.032	n/a	2.4	0.19
N 7 W	PP	woven	200	6	7	0.41	0.016	n/a	0.4	0.016
A 10 W	PP	woven	250	7.5	10	0.64	0.025	n/a	1.0	0.064
N 22 NW/W	PE	nonwoven and woven	740	22	n/a	6.32	0.249	50	2.4	1.50
H 4 NPNW	PET	needled nonwoven	130	4	n/a	1.12	0.044	70	2.1	0.24
H 8 NPNW	PET	needled nonwoven	270	8	n/a	2.44	0.096	80	1.5	0.37
H 16 NPNW	PET	needled nonwoven	540	16	n/a	4.65	0.183	100	0.5	0.23
T 4 HBNW	PP	heat bonded nonwoven	120	3.5	n/a	0.38	0.015	90	0.6	0.023
P 6 NPNW	PP	needled nonwoven	220	6.5	n/a	1.98	0.078	70	1.6	0.32

*PP = polypropylene, PE = polyethylene, PET = polyester

Regarding the flow quantity passing through the various filters considerable thought and deliberation went into the selection process. Knowing that flow rates of approximately 18,700 l/ha-day (2000 gal/acre-day) are typical for New York State landfills [11], it would take years to begin to see measurable decreases in permeability in the experimental setup just described. For example, the above stated flow rate for a 100 mm (4.0 in.) diameter permeameter is a minisule 0.015 l/day (0.004 gal/day) which was too low for measurement tests of this type. Thus it was decided to accelerate the testing by greatly increasing the flow rate. Three accelerated flow rate values were selected. By performing all of the 48 tests at each flow rate, the data could be plotted and back extrapolated to the lower field anticipated value. The entire setup was changed with new geotextile and sand filters after each flow rate cycle was completed. The flow rates selected were those given in Table 13.

Table 13 - Flow Rate Values Used for Accelerated Testing

Classification	Flow Rate (Actual)		Flow Rate (Equivalent)	
	l/week	gal/week	l/ha-day	gal/acre-day
very high	95	25	17,000,000	1,800,000
high	3.8	1.0	660,000	71,000
intermediate	0.95	0.25	170,000	18,000

Thus the number of long term permeability tests that were performed was increased by a factor of three. The experimental design included four permeants, twelve filters and three flow rates for a total of 144 long term permeability tests.

6.4 Test Results from Very High Flow Rates

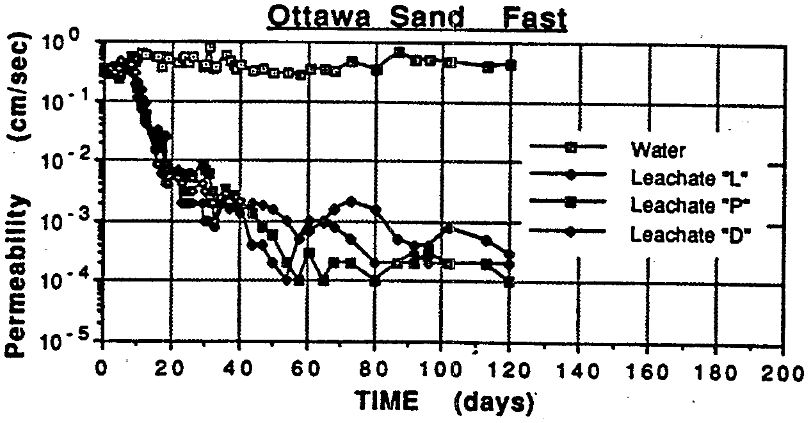
The set of figures presented in this section are the permeability results of the 48 flow columns at the "very high" flow rate. Note that each set of 12 repeat columns were permeated with four leachates; water (as the control) and leachates "L", "P" and "D" as described previously in Table 11 at the very high flow rate of 95 l/week (25 gal/week).

Figure 24 presents the results of the two sand soil filters which were evaluated. In both cases, the water control was relatively constant, with the permeability of the Ottawa sand being an order of magnitude higher than the concrete sand. This was understandable since the Ottawa sand has round and uniform particles versus the angular and more well graded particles of the concrete sand. The variations in shape and size distribution of the concrete sand allowed for greater packing and densification. Therefore, a tighter matrix and a more tortuous path for the liquid to follow was created.

The two sands also behaved differently to the permeation of leachate. When contrasting the two sets of results, the concrete sand was seen to exhibit an immediate decrease in permeability while the Ottawa sand exhibited a few days of near "water like" flow before gradually decreasing in permeability. This can be attributed to the difference in void space of the two soils. Apparently the concrete sand trapped suspended particles from the onset of the test while the Ottawa sand did not trap suspended particulates until after an initial flow period.

In comparing the shape of the two soil curves, a similar gradual decrease in permeability was noticed. When compared to the geotextile response curves it was seen that the soils take much longer to come to equilibrium than do geotextiles. This is due that the soils being 100 mm (4.0 in.) thick while the geotextiles range in thickness from 0.38 to 6.3 mm (0.015 to 0.249 in.). This is an illustration of depth versus cake filtration as described in Section 2.

In comparing the shape of the two soil leachate response curves to one another it was seen that the Ottawa sand reached equilibrium faster than the concrete sand. This was due to the fact that the Ottawa sand consisted of uniform particles with uniform pores. Ottawa sand came to equilibrium in 70 days while concrete sand took 100 days to reach its residual value. Hence the



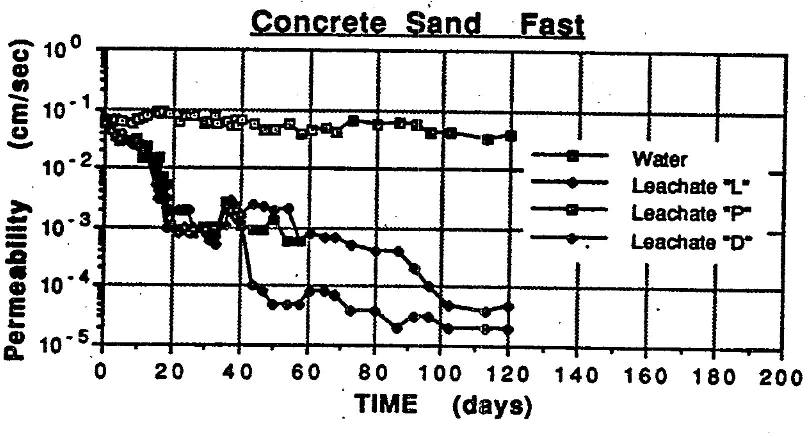


Figure 24 - Sand Response Curves After 120 Days of Leachate Permeation

Ottawa sand was uniformly decreasing in porosity while the concrete sand was very sporadic in its behavior.

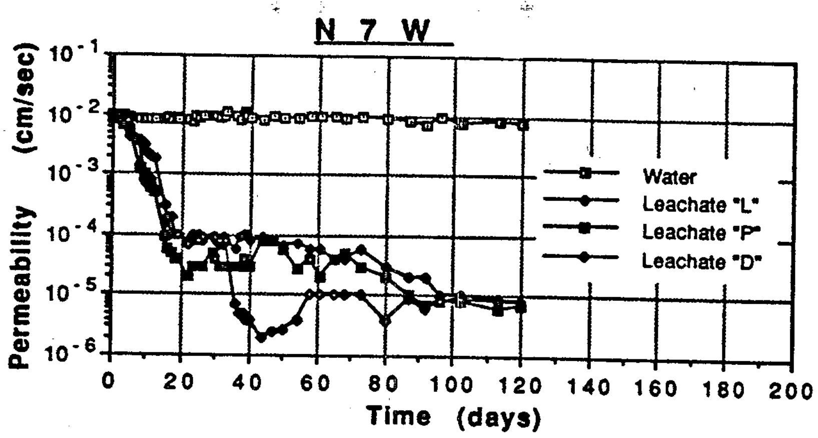
Figure 25 presents the data from three woven, monofilament, polypropylene geotextile filters having different percent open areas, e.g., 7%, 14% and 32%. The geotextiles were different with regard to yarn construction, not weave. A plain weave was utilized in all cases for these woven geotextiles. The N 7 W geotextile was manufactured with calendered fibers while the N 14 W and the N 32 W geotextiles were made with noncalendered fibers having a rounded cross section instead of a flattened tape. The variation between the N 14 W and the N 32 W was that the N 14 W geotextile utilized a multifilament yarn. In addition to the obvious difference in percent open area (POA), the shape of the opening also changed with the selection of the different yarns. The N 14 W geotextile exhibited a three dimensional quality where the other woven geotextiles were very flat. Although there was not much thickness added to the geotextile it did allow flow in the bias of the material. This aided in the performance of the filter as well as formed an irregular surface for filter cake formation. Hence, filter performance was improved.

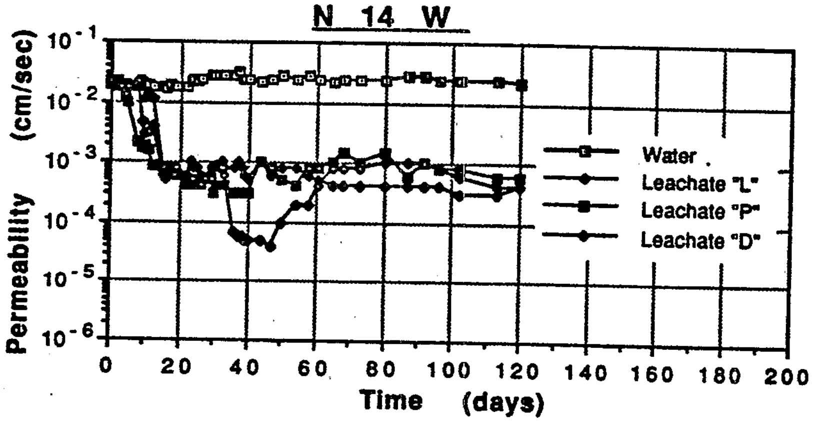
It should be noted that in all cases the woven geotextiles took 4 to 8 days for measurable permeability decrease to occur. This time corresponded to 380 to 770 litre (100 to 200 gal) of leachate passing through the geotextile filter. This suggested that the openings in the woven geotextiles were too large to immediately catch or trap particulates. Note that during this time the downstream drainage media was accommodating the particulates that were carried into the drainage system.

As far as the overall performance of the group of three woven geotextiles was concerned, the N7W geotextile performed poorly with respect to the N14W and N32W geotextiles. This indicated that the POA of a woven geotextile was the limiting characteristic which distinguishes between different performances. It was felt that the minimum POA for acceptable woven geotextiles was approximately 10%.

There was no discernible difference between the various leachates for any of the woven geotextiles. This was not surprising for at this time all three leachates had quite similar characteristics. As can be seen in Table 11, the leachates were all of different ages and individual properties; however their combined effect yielded similar responses for this method of testing. In retrospect, the intent was to expose the geotextiles to four very different permeants; however, the experiment materialized into testing the filters with water and three quite similar leachates.

Figure 26 presents the data from three nonwoven, needlepunched, polyester, geotextile filters. They were 130, 270 and 540 gm/m^2 (4, 8, and 16 oz/yd^2) mass per unit area (often called "weight"). The water control tests remained quite constant throughout the 120 day testing duration. In contrast, decreases in permeability from leachate testing were observed immediately. This was indicative of particles being trapped in the relatively thick geotextiles from the inception of the test. All of the geotextiles in this group were constructed of 10 denier polyester fibers which are fine in comparison to the other geotextile fibers used in this study. The porosity of





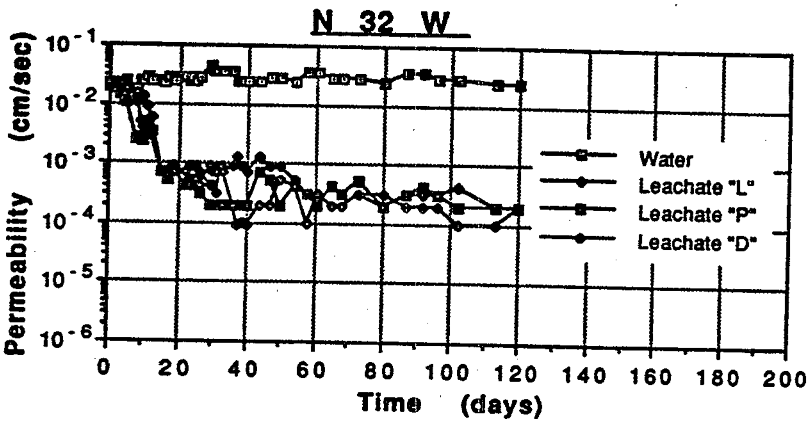
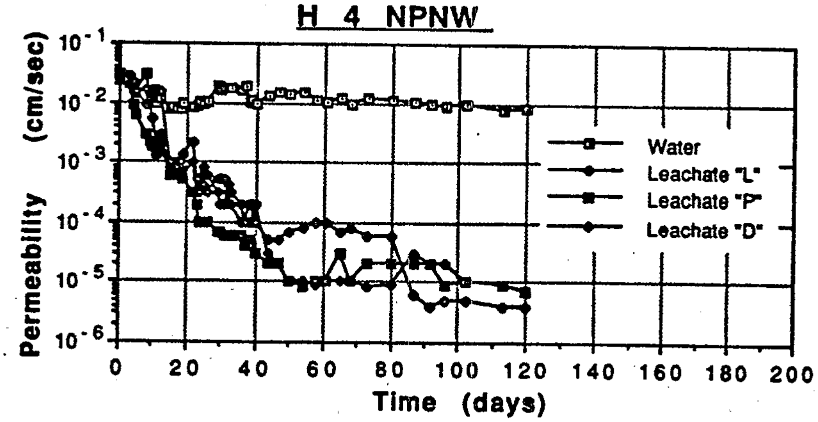
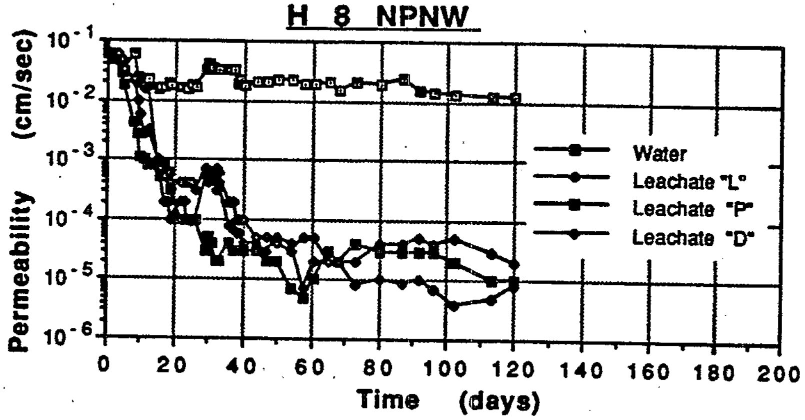


Figure 25 - Woven Response Curves After 120 Days of Leachate Permeation





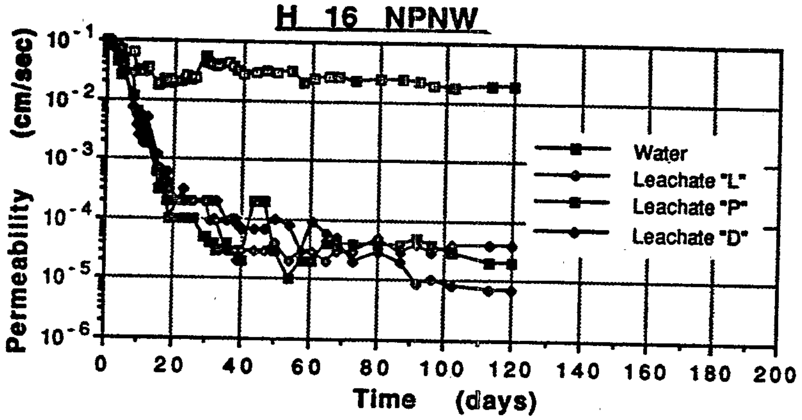


Figure 26 - Nonwoven Needlepunched Curves After 120 Days of Leachate Permeation

these geotextiles varied due to the nature of the spunbonding process and mass per unit area by which they were made.

From the curves on Figure 26 it is apparent that the higher weight geotextiles outperform the lighter weight geotextiles. This is a result of the thickness of the geotextile. From Darcy's equation we note that ;

$$Q = k\,i\,A \qquad (1)$$

$$V/t = k\,(\Delta h/L)\,A \qquad (2)$$

where

Q = flow rate
k = permeability
i = hydraulic gradient
A = cross sectional area
V = volume of fluid passed
t = time
Δh = change in hydraulic head
L = thickness of the specimen

Solving for k we find that;

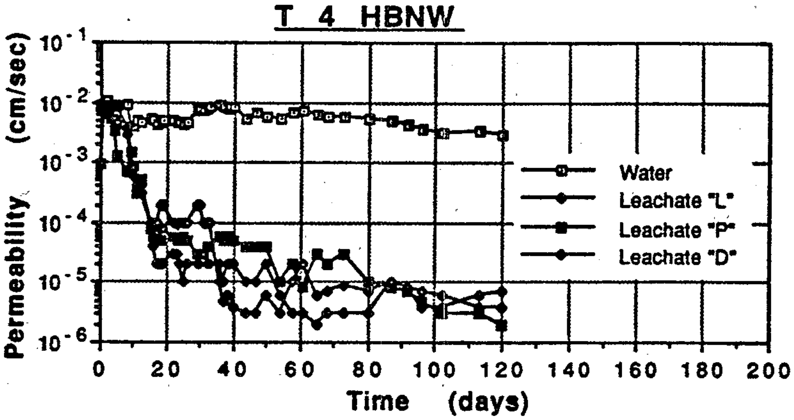
$$k = (VL)/[(A)(\Delta h)(t)] \qquad (3)$$

Knowing that V, A, and Δh are constants, the permeability (k) is seen to be directly proportional to the thickness (L) of the specimen. As the thickness increases it influences the gradient "$\Delta h/L$" in a similar manner and the result is a higher permeability. This confirms that small differences in thickness will greatly influence the calculated permeability value. It is for the reason that geotextile permittivity is often used in lieu of geotextile permeability.

All the geotextiles in this group had equilibrium values of approximately 1×10^{-5} cm/sec with no significant differences on the basis of mass per unit area. When compared to the other geotextiles in the study these materials where at the low end of the performance scale.

Figure 27 (upper curve) presents the results of T 4 HBNW, a nonwoven, heat bonded, polypropylene geotextile. The water control was consistent, but the leachate curves throughout the test continued to decrease. At 120 days (the conclusion of the tests) they were less than 7×10^{-6} cm/sec which was low in comparison to other filters. It is conceivable that this value could have decreased further and resulted in an excessively clogged filter.

Figure 27 (lower curve) presents results for a nonwoven, needlepunched, polypropylene geotextile filter of 200 g/m^2 (6 oz/yd^2) mass per unit area. The water control is constant, and the trend in permeability decreased gradually to a limiting value around 1×10^{-5} cm/sec. Once again, the type of leachate did not make a discernible difference. The values are seen to be similar to the set of curves shown in Figure 26 for polyester fabrics. Thus fabric structure, rather than polymer



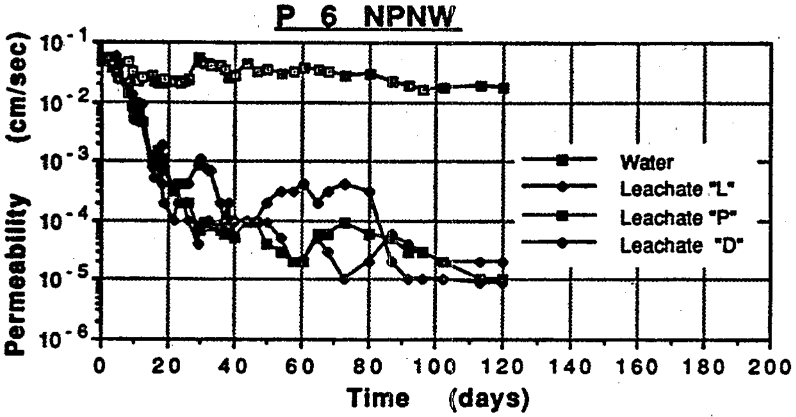


Figure 27 - Nonwoven Curves After 120 Days of Leachate Permeation

type, appeared to be the more relevant issue in permeability testing. This geotextile had much greater variability in its structure due to a more random laydown process. Therefore this product had a range of opening sizes instead of a near constant opening size as with the series of polyester geotextiles.

Figure 28 presents two "special" geotextiles which are uniquely targeted to the filtration market by their respective manufacturers. Figure 28 (upper curve) gives the permeability results of A 10 W which was a woven, monofilament, polypropylene, geotextile. The warp and weft fibers were constructed of greatly dissimilar yarns. The warp fibers were rounded monofilaments while the weft fibers were crimped staple yarns. The concept behind the construction of this geotextile was to weave these two dissimilar materials into an open mesh. Since the staple yarn was bulky it also provided the geotextile with openings on the bias and some thickness.

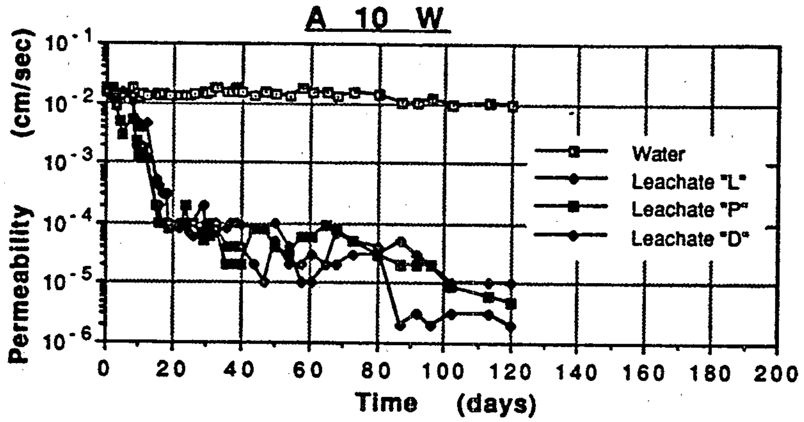
Like the other tests in the study the water control was well behaved for this geotextile. The curves for the leachate tests quickly decreased to 1×10^{-5} cm/sec (within 45 days) and then gradually decreased further until the terminal permeability was somewhat less than 1×10^{-5} cm/sec. There was no discernible difference between the different types of leachate.

Figure 28 (lower curve) is of the N 22 NW/W geotextile. This was the most uniquely different geotextile in the entire study. The geotextile was comprised of many staple polyethylene fibers supported on an open woven polyethylene mesh. The designation of NW/W was assigned to this geotextile because it was configured from both nonwoven and woven yarns. The staple fibers in the matrix consisted of a 500 denier fiber which gave the geotextile a high initial permeability as well as a low apparent opening size (AOS). This geotextile behaved best of the nonwoven geotextiles and exhibited an average equilibrium permeability of 3.5×10^{-4} cm/sec, irrespective of the type of leachate. However, the N22 W/NW geotextile did take 10 days to show any decrease in flow. This behavior was indicative of a filter with relatively large openings. The response of the N22 NW/W geotextile was encouraging for it brings some additional possibilities of the utilization of geotextiles into this application other than simply opening up the pore structure.

6.5 Test Results from High and Intermediate Flow Rates

A second set of 48 flow columns were constructed using a completely new set the same 12 filters as given in Table 12. They were permeated by the same (as near as possible) four permeants as given in **Table** 11. For these tests, however, the flow rate was reduced from 95 l/week (25 gal/week) to 3.8 l/week (1.0 gal/week). It was designated as a "high" flow rate, recall Table 13. While the behavior was generally similar to that shown in Figures 24 to 28, there were two important differences. First, the time to equilibrium was slightly longer, and second the equilibrium permeability was considerably higher. The complete set of curves are given in Appendix "B" and accompanying text is found in Reference 10.

A third, and final, set of 48 flow columns was constructed in the same manner (again, a completely new set of filters was used) with the flow rate being further reduced from 3.8 l/week



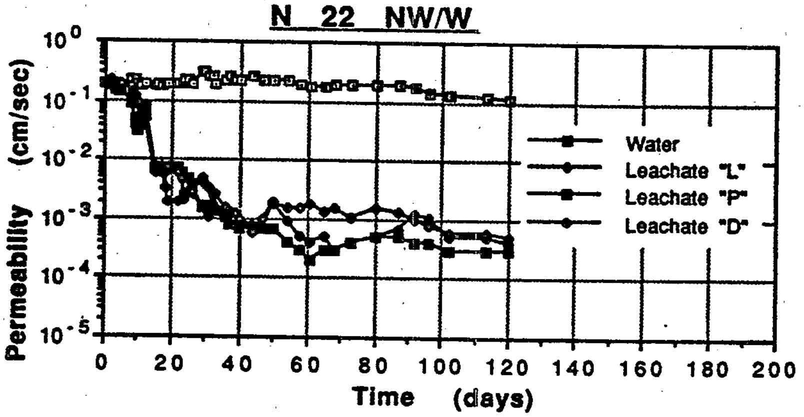


Figure 28 - Response Curves of Experimental Geotextiles After 120 Days of Leachate Permeation

(1.0 gal/week) to 0.95 l/week (0.25 gal/week). These tests were designated as an "intermediate" flow rate, recall Table 13. Again, similar behavior was observed to the higher flow rates, with equilibrium times being still longer and equilibrium permeability values becoming still higher. The curves for this intermediate flow rate are also given in Appendix "B" with accompanying text in Reference 10.

6.6 Comparison of Results Using Different Flow Rates

With equilibrium permeability values for all twelve filters (2 soils and 10 geotextiles) at very high, high and intermediate flow rates, a characterization and comparison of the results can be made. Table 14 gives the transition time, i.e., the onset of equilibrium permeability, and the equilibrium permeability value itself for the 12 filters at the three different leachate flow rates. As seen in the table there are considerable differences in behavior between the various combinations of filters and leachate conditions. Some observations follow:

- There was the tendency for all three leachates to reach similar transition times and equilibrium flow rates. Thus the generalized table represents average values for the three leachates.
- The water permeant used as the control was worthwhile but did not give any succinct information except that leachate behaves very differently from water.
- The concept of an accelerated flow rate test seems reasonable on the basis of results of these tests. By simply passing greater quantities of leachate through the filter, progressively shorter transition times to flow equilibrium were required and progressively lower equilibrium permeability values resulted.
- Comparing the different filter responses to one another was difficult except for those which perform best and poorest. The N22 NW/W geotextile resulted in higher equilibrium permeability than any other geotextile or soil filter. The N32W was also very high in its equilibrium permeability, with a large number of geotextiles and Ottawa sand close behind. Conversely, the A10W and T4HBNW geotextiles resulted in the lowest equilibrium permeabilities.

6.7 Summary of Laboratory Permeability Testing

Presented in this section was the long term permeability testing of four permeants, on 12 different filters at three accelerated flow rates. The resulting 144 tests took 120 to 300 days to reach equilibrium. The results of the very high flow rates are included in this section, while the high and intermediate flow rate results are given in Appendix "B".

For all 144 columns, the water flow tests acted as an good control liquid. The permeability data remained quite constant and, furthermore, gave an indication of the type of fluctuation in the ASTM D1987 flow columns. The stability of the test results was encouraging, as was the functioning of this entire experimental test set-up.

Insofar as the leachate flow tests are concerned to obtain allowable permeability values for a given filter, it took anywhere from 25 to 100 days for the test results to reach equilibrium, or to a point where equilibrium could be estimated. The equilibrium flow rates for the various types of

Table 14 - Evaluation of Laboratory Permeability Test Results using Leachate at Different Flow Rates

Type of Filter	"Very High" Flow Rate		"High" Flow Rate		"Intermediate" Flow Rate	
	Transition Time (days)	Equil. Perm. (cm/sec)	Transition Time (days)	Equil. Perm. (cm/sec)	Transition Time (days)	Equil. Perm. (cm/sec)
(a) Soil Filters						
Ottawa Sand	70	2×10^{-4}	140	2×10^{-3}	180	3×10^{-2}
Composite Sand	100	4×10^{-5}	90	3.5×10^{-3}	180	1×10^{-2}
(b) Woven Geotextile Filters						
N7W	25	8×10^{-6}	90	2×10^{-4}	120	1×10^{-3}
N14W	30	7×10^{-4}	65	7×10^{-3}	90	2×10^{-2}
N32W	30	2.5×10^{-4}	80	5×10^{-3}	120	1.5×10^{-2}
A10W	45	5×10^{-6}	60	1×10^{-3}	60	1.0×10^{-2}
(c) Nonwoven Geotextile Filters						
H4NPNW	45	6×10^{-6}	140	7×10^{-4}	180	4×10^{-3}
H8NPNW	55	1.2×10^{-5}	160	7×10^{-4}	180	5×10^{-3}
H16 NPNW	50	2×10^{-5}	160	8×10^{-3}	180	8×10^{-3}
T4HBNW	45	4×10^{-6}	110	8×10^{-5}	180	9×10^{-5}
P6NPNW	40	1.5×10^{-5}	160	1×10^{-3}	150	6×10^{-3}
(d) Composite Nonwoven/Woven Geotextile Filter						
N22NW/W	60	3.5×10^{-4}	70	4×10^{-2}	90	1.5×10^{-2}

filters are also given in Table 14 in the order of highest to lowest values of permeability for each filter group. Here it is seen that the range of permeabilities is from 7×10^{-4} to 4×10^{-6} cm/sec, i.e., greater than 100 times different from highest to lowest. Clearly, the selection of a specific type of filter is important in the performance of a landfill leachate collection system. It should once again be mentioned that the permeability variation as a function of the type of leachate was not significant. This was probably due to the fact that over time, the leachates from these three different landfills came close together, particularly insofar as sediment loading is concerned.

Using the data of Table 14 we are in a position to graph the results of each filter at the three different flow rates evaluated. This behavior is shown in Figure 29. Figure 29(a) illustrates the behavior of the 2 sand soils, Figure 29(b) the 4 woven geotextiles and Figure 29(c) the 6 nonwoven geotextiles. The composite nonwoven/woven geotextile is included in this group.

All 12 filters are shown together in Figure 29(d) where one can observe a trend toward a somewhat leveling of the curves as the flow rate decreases. It is suggested that by back extrapolation to a site specific flow rate, an allowable permeability for a particular filter can be obtained. This value of allowable permeability, or "k_{allow}", for a given filter is the essential value that will be used in the design method to follow.

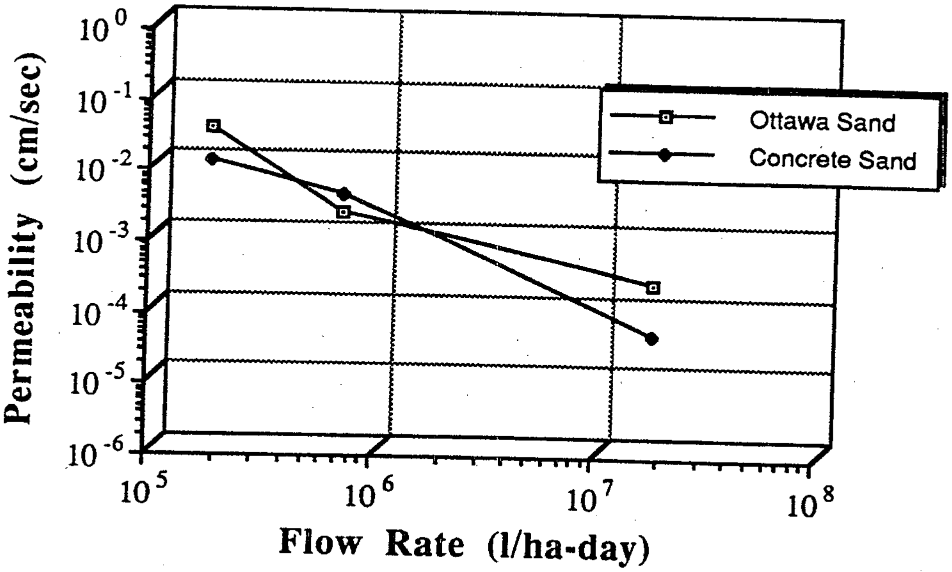


Figure 29(a) - Flow Rate Curves for Sand Soils

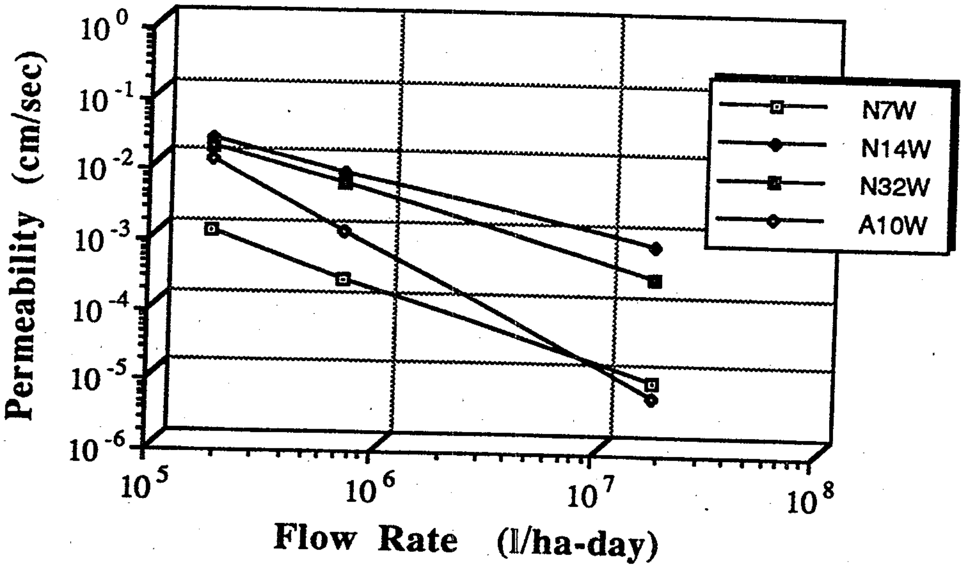


Figure 29(b) -Flow Rate Curves for Woven Geotextiles

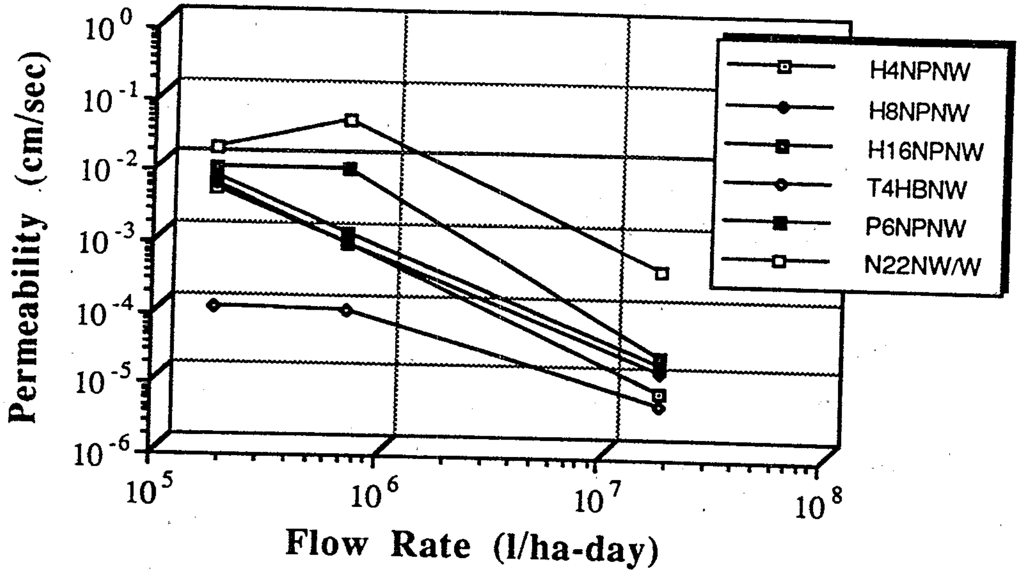


Figure 29(c) - Flow Rate Curves for Nonwoven Geotextiles

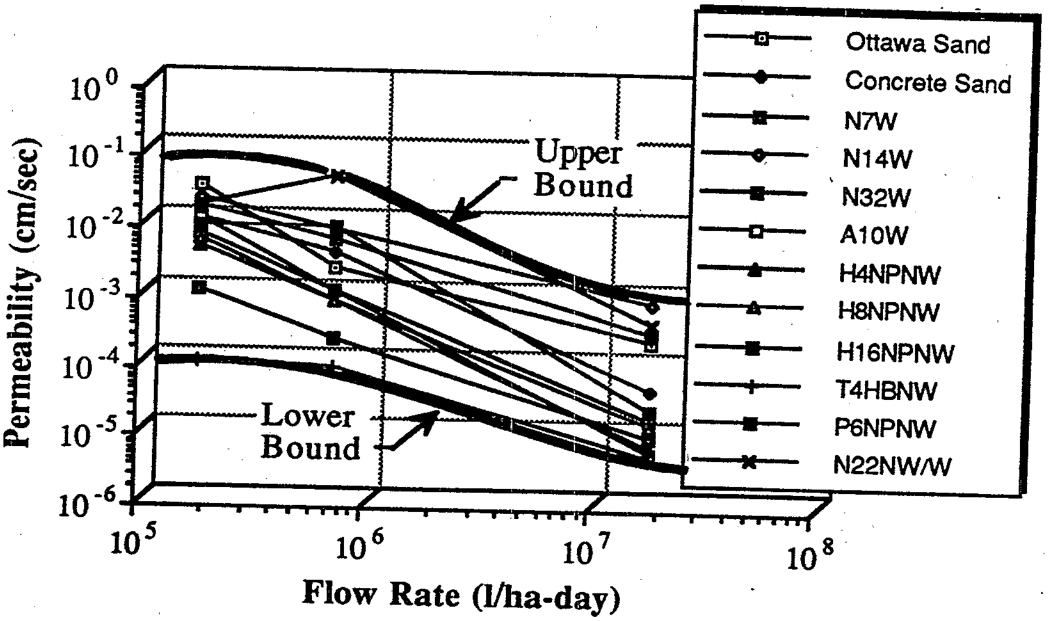


Figure 29(d) - Master Curves for all Twelve Filter Evaluated (2 sands and 10 geotextiles) and Estimated Upper and Lower Bounds for Extrapolation Back to "Typical" Site Specific Flow Rate Values

7. Modeling for Required Flow Rates

In order to compare the allowable permeability of a given filter to a site-specific required permeability an analytic design model is necessary. This section uses the "Hydrologic Evaluation of Landfill Performance," or HELP, model developed by Schroeder, et al. [12] for the purpose of obtaining a value for "k_{reqd}".

7.1 Overview of the HELP Model

The HELP model was developed by the U.S. Army Corps of Engineers for the U.S. Environmental Protection Agency. The primary purpose of the model was to enable a comparison of landfill design alternatives as judged by an assessment of liquid flow through the solid waste material. The model can be characterized as a quasi-two dimensional, deterministic, liquid-routing model for solving liquid balance situations. The liquid can be leachate or water depending on whether it flows through solid waste or soil.

The model accepts weather, waste, soil and site specific design data and uses solution techniques that primarily account for the effects of surface storage, infiltration, evapotranspiration and percolation. Figure 30 is a conceptual design of the HELP model simulation process.

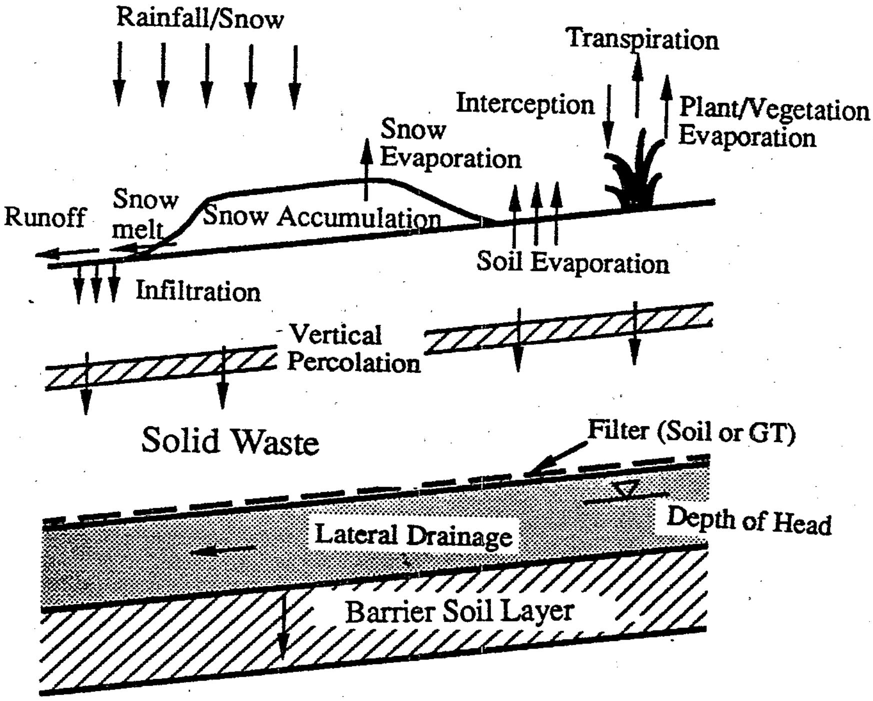


Figure 30 - HELP Model Simulation Process

Results are expressed as daily, monthly, annual and long-term peak or average liquid quantities at any point or location in the landfill. Version 3 of the model developed in 1994 was used for this investigation, Schroeder, et al. [13].

The required permeability was obtained by imputing the site specific weather, waste, soil and geometric data and sequentially varying the filter permeability. The permeability value was varied from 1.0 to 1 to 10^{-7} cm/sec. By tracking the peak daily discharge response with respect to the varying filter permeability a threshold was established. The threshold condition was established when the filter began to limit the flow into the underlying drainage layer. This threshold was defined as the value of k_{reqd}.

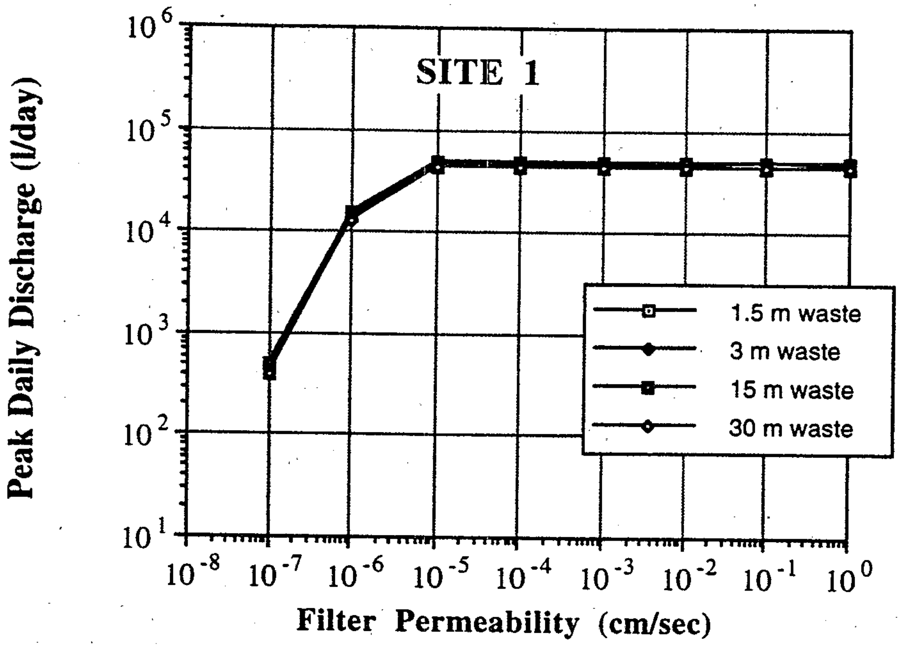
7.2 Use of the HELP Model for Exhumed Sites

Using the site specific configuration of the four exhumed landfills described in Section 5, coupled with the hydrologic data for the closest city to the site, the HELP model was used to generate the required permeability of the filter located above the leachate collection drainage layer. Note that the model does not preferentially distinguish between a geotextile or a natural soil material. Some of the more relevant input data for the four exhumed sites is given in Table 15. Here it is seen that many geometric values are required in addition to the hydrologic data. Also important are characteristics of the waste insofar as its moisture content and density are concerned. Thus the model has a large number of input parameters. This situation is heightened for our exhumed sites since they are unknown in regard to some of their details. For this reason default values were used in some situations.

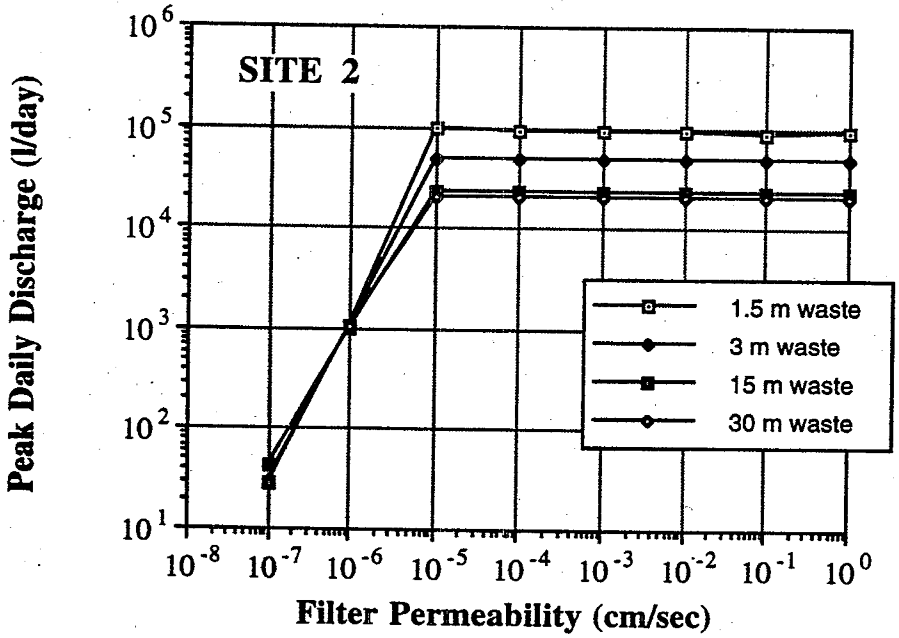
Table 15 - Input Data of Exhumed sites for Use in HELP Model to Obtain the Required Filter Permeability

Site No.	Cell Area		Base Slope	Pipe Spacing		$K_{Drainage\ Stone}$
	(ha)	(Acre)	(%)	(m)	(ft)	(cm/sec)
1	2.8	7	1.5	61	200	0.01
2	2.8	7	1.5	61	200	0.3
3	2.9	7.3	2.0	61	200	0.3
4	5.6	13.8	1.5	31	100	0.3

The curves generated to obtain the required geotextile permeability for each of the four exhumed sites are given in Figure 31. The results for Site 1, shown in Figure 31(a), indicate that the peak daily discharge decreases significantly when the filter permeability falls below 1 × 10^{-5} cm/sec. All waste heights produce similar trends. Below this value of permeability, the filter is essentially starving the underlying drainage layer and it (the filter) becomes the controlling material. Thus the required permeability of the filter for this site is at least 1 × 10^{-5} cm/sec. The situation is similar for Site 2 with the required permeability again being at least 1 × 10^{-5} cm/sec, see Figure 31(b). The results for Site 3, shown in Figure 31(c), indicates a slightly different trend with the break in the curves being somewhat higher than the first two sites. The required

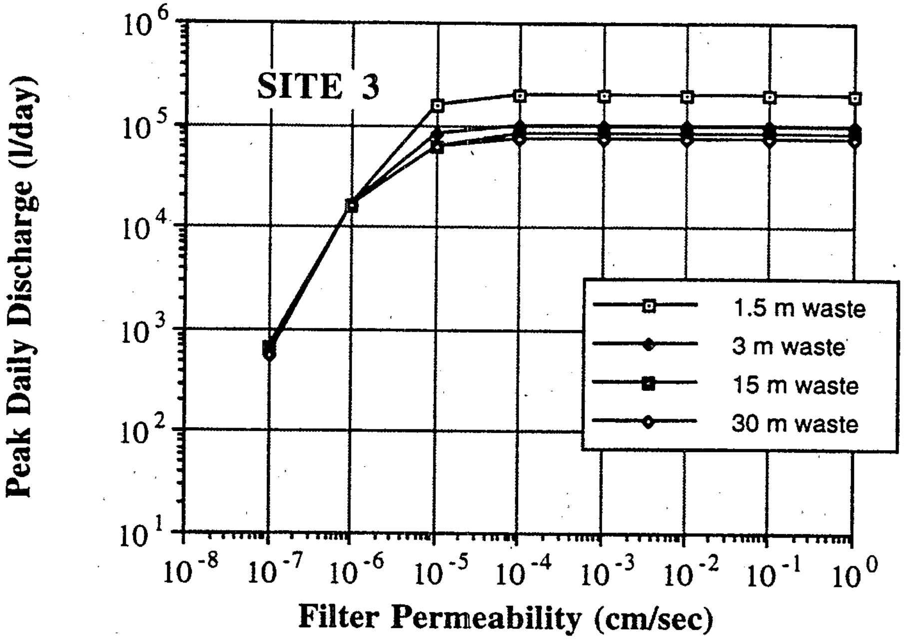


(a) - Site No. 1

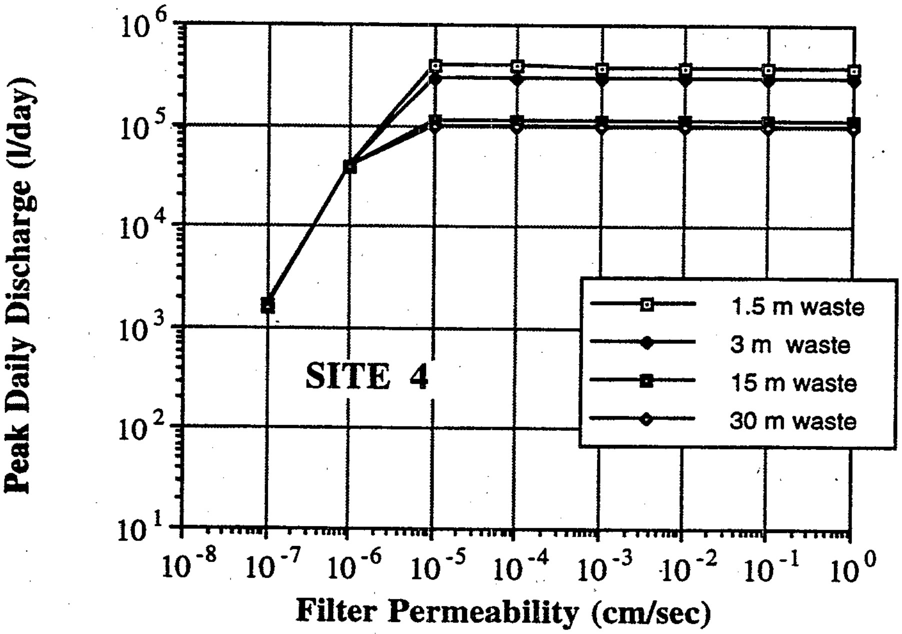


(b) - Site No. 2

Figure 31 - HELP Model Generated Values of Filter Permeability for Exhumed Sites



(c) - Site No. 3



(d) - Site No. 4

Figure 31 - (Continued)

permeability of the filter material is estimated to be at least 5 × 10^{-5} cm/sec. Lastly, the results for Site 4, shown in Figure 31(d), again give a clear break in the curves at 1 × 10^{-5} cm/sec, which is the minimum required filter permeability. In the design to follow we will use the above listed values as the required permeability of the filters without increasing them in any arbitrary manner, e.g., by a partial factor-of-safety. The factor-of-safety will be included in a traditional manner in the next section.

It should be noted that all of the curves are quite similar with required filter permeability values between 1 × 10^{-5} and 5 × 10^{-5} cm/sec. This is not particularly surprising since all four landfills were in the northeastern part of the United States and share many similar site specific conditions. Also to be mentioned is that Sites 1 and 2 were at the same location, only with slightly different geometric configurations.

7.3 Summary of HELP Model Utilization

As illustrated by the use of the HELP model in this section, design related values of required permeability of the filters at the exhumed sites have been obtained. They are as follows:

Site No. 1: $k_{reqd} = 1 \times 10^{-5}$ cm/sec

Site No. 2: $k_{reqd} = 1 \times 10^{-5}$ cm/sec

Site No. 3: $k_{reqd} = 5 \times 10^{-5}$ cm/sec

Site No. 4: $k_{reqd} = 1 \times 10^{-5}$ cm/sec

Also as mentioned, the similarity was somewhat expected but nonetheless the HELP model was further investigated. The sensitivity of the model to numerous input parameters was evaluated with the results given in Appendix "C". The parameters under investigation were the following:

- Site location.
- Thickness of the lateral drainage layer.
- Slope of the base of the lateral drainage layer.
- Evaporation depth.
- Runoff curve number.
- Permeability of the lateral drainage layer.
- Height of the waste material.
- Permeability of the waste material.

These parameters were investigated by observing their sensitivity on the peak daily discharge of the drainage system and (most importantly for the purposes of this study) the permeability of the filter. The entire parametric study is presented in Appendix "C" with the relevant conclusions.

The HELP model is a useful tool for comparison of design alternatives. It is used by all landfill regulators in the United States. Hence (by default), it must be used by the consulting engineering community and landfill owner communities as well. The model does not seem to be extremely sensitive to subtle changes to weather, soil or site specific design data. However, it does appear to yield logical response trends to systematically varied input parameters.

The results generated from the HELP model tend to be conservative. This implies that estimates of leachate generated at the base of a landfill generally overpredict the amount that is actually collected. Confidence in the HELP model is greatly increased when it is calibrated with actual field data. This can be obtained from estimates of leachate generation rates in the open literature or from the adjacent cell in a landfill expansion situation. Version 3 of the HELP model is very user-friendly and is a powerful tool to access the hydrologic performance of landfills.

8. Design Method and Substantiation

At this point we are in position to not only develop a design methodology (utilizing both the laboratory and design sections of the report), but also to substantiate the design method using the results of the field exhumed sites. Hence, the results of Sections 5, 6 and 7 will be brought together in this section, per the project flow chart of Figure 7.

8.1 Design Methodology and the Drainage Correction Factor

A general method for designing all engineering materials is the formulation of a factor-of-safety (FS). It is the ratio of an allowable material property to that of the site specific required property. The value must be greater than one, since it is necessary to compensate for any uncertainties in the testing and/or design processes. While the actual target value is site specific and ultimately the decision of the designer, it is felt that the recommended value for a landfill filter design should be considerably in excess of one, e.g., ten or higher.

The value of factor-of-safety for filtration is formulated by comparing the allowable filter property for the candidate material with the required filter property for the specific site under consideration. For the case of filter designs involving permeability this ratio is expresed as follows:

$$FS = \frac{k_{allow}}{k_{reqd}} \tag{4}$$

where

FS = factor-of-safety
k_{allow} = allowable permeability
k_{reqd} = required permeability

The allowable permeability is determined from laboratory testing as described in Section 6. More specifically, Figure 29 illustrated its evaluation for the twelve filter materials and their respective leachates. The required permeability is generally obtained using the HELP model as was described in Section 7 for the four sites which were exhumed.

In dealing with geotextile filters, an equivalent term for permeability is the permittivity, "ψ". It is defined as the traditional permeability value "k" divided by the geotextile thickness "t":

$$\psi = \frac{k}{t} \tag{5}$$

Thus an equivalent equation for factor-of-safety is as follows:

$$FS = \frac{\psi_{allow}}{\psi_{reqd}} \tag{6}$$

where

FS = factor-of-safety

ψ_{allow} = allowable geotextile permittivity

ψ_{reqd} = required geotextile permittivity

Consistency with design methods in all engineering materials (including geosynthetics) suggests that these equations be used for filters of leachate collection systems, which is the topic of this study. Unfortunately, the field exhuming efforts described in Section 5 suggest the need for a modification. Field Sites 1, 3 and 4 were all cases where the geotextile filter was wrapped directly around perforated drainage pipes. Thus only a small portion of the geotextile was available for flow since it was limited to those small areas directly adjacent to the holes in the pipe. To a lesser extent field Site No. 2 also had a flow area less than the entire footprint of the landfill cell. It will also have to be suitably accommodated. As a result of such restrictions in available drainage area in all four of the field exhumed cases, a compensating term for drainage areas less than the full footprint of the landfill cell must be formulated. The term we have selected is a drainage correction factor, or "DCF", which is formulated into Equations 4 and 6 as follows:

$$FS = \frac{k_{allow}}{k_{reqd} \times DCF} \tag{7}$$

or

$$FS = \frac{\psi_{allow}}{\psi_{reqd} \times DCF} \tag{8}$$

where

DCF = drainage correction factor

The value of DCF is defined as the ratio of the entire landfill or cell area (i.e., the footprint) divided by the actual flow area that is available beneath the filter(s) for drainage. The DCF is site specific and determined via a geometric ratio taking into account the area of influence with respect to the size and configuration of the drain.

Expressed as an equation, the DCF is as follows:

$$DCF = \frac{A_{FP}}{A_{DS}} \tag{9}$$

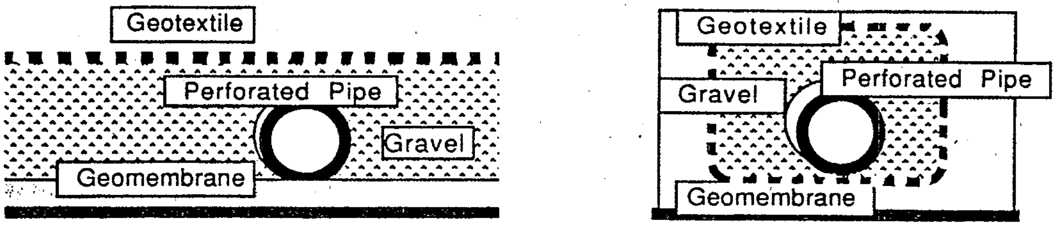
where

DCF = drainage correction factor

A_{FP} = footprint area of the landfill or cell

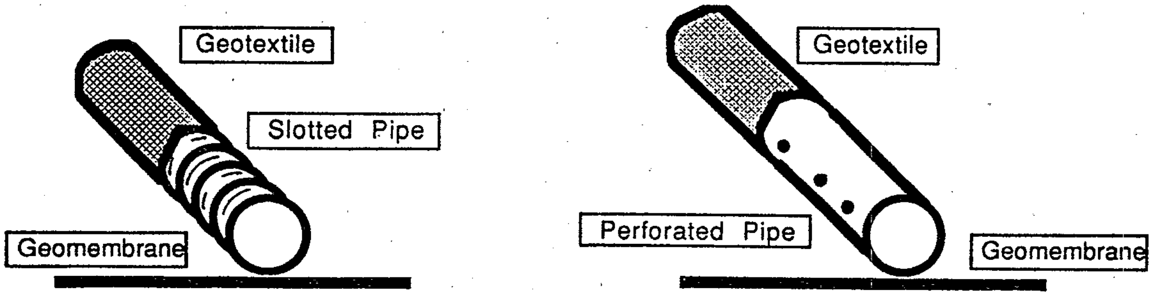
A_{DS} = area of actual downstream drainage system

Four typical field situations have been encountered and are illustrated in Figure 32.



(a) Areal Filter
DCF = 1.0

(b) Trench Wrap
DCF = 10 to 40



(c) Socked Corrugated Pipe
DCF = 64 to 260

(d) Socked Smooth Wall Pipe
DCF = 7,500 to 24,400

Figure 32 - Various Leachate Collection Filter Configurations with the Associated Range of Drainage Correction Factors

(a) For an areal filter placed over the entire footprint of the landfill or cell, the DCF is equal to 1.0 since the footprint area and drain system area are equal, see Figure 32(a). Note that this could be considered conventional design practice which would eliminate the need for a DCF term.

(b) For a geotextile wrapped gravel, or trench wrap, the DCF varies from 10 to 40 depending upon the spacing of the drains and the size of the drainage gravel cross sectional area, see Figure 32(b). Note that the gravel must be sized appropriately with respect to the holes or slots in the perforated pipe and vice versa, but this is conventional design, see Cedegren [14].

(c) A geotextile wrapped around a corrugated pipe with slots in the valleys of the corrugations results in typical DCF values of 60 to 260, see Figure 32(c). The value depends on the pipe spacing and the diameter of the pipes. Approximately 50% of the surface area of the pipe is available for leachate flow using corrugated pipe, i.e., the area above the valleys of the corrugations. This value divided into the landfill footprint area is the numeric value of the DCF.

(d) A geotextile wrapped around smooth wall pipe with holes perforated in it results in DCF values of 7,500 to 24,000, see Figure 32(d). The value depends on the pipe spacing and the number and diameter of holes in the pipe. Since very little area is available for flow, the resulting DCF values are extremely large.

Table 16 gives some additional insight into the influence of the different designs illustrated in Figure 32 and the impact of drain spacing, drain size, hole size and number of holes on the numeric value of DCF.

Table 16 - Selected Values of Drainage Correction Factors (DCF) for Use in Calculating the Factor-of-Safety of a Leachate Collection Filter

Drain Configuration	Drain Spacing		Drain Size		Hole Size		Number of Holes		Drain Correction Factor (DCF)
	(m)	(ft)	(mm)	(in.)	(mm)	(in.)	(per m)	(per ft)	
(a) Areal Coverage	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	1
(b) geotextile wrapped around gravel (i.e., a trench wrap)	15	50	450 × 300	18 × 12	n/a	n/a	n/a	n/a	10
	30	100	450 × 300	18 × 12	n/a	n/a	n/a	n/a	20
	45	150	450 × 300	18 × 12	n/a	n/a	n/a	n/a	30
	60	200	450 × 300	18 × 12	n/a	n/a	n/a	n/a	40
(c) geotextile around corrugated pipe (i.e., socked pipe)	15	50	150	6.0	n/a	n/a	n/a	n/a	60
	30	100	150	6.0	n/a	n/a	n/a	n/a	130
	45	150	150	6.0	n/a	n/a	n/a	n/a	190
	60	200	150	6.0	n/a	n/a	n/a	n/a	260
(d) geotextile around smooth wall pipe (i.e, socked pipe)	15	50	150	6.0	12	0.5	1.8	6	7,500
	30	100	150	6.0	12	0.5	1.8	6	12,000
	45	150	150	6.0	12	0.5	1.8	6	18,000
	60	200	150	6.0	12	0.5	1.8	6	24,000

note: n/a = not applicable

8.2 Substantiation of Design Methodology

The implication of the high DCF values in Table 16 on the resulting factor-of-safety values in Equations 7 or 8 is obvious. The numeric impact will be illustrated on the basis of the field exhumed sites that were described in Section 5.

Table 17 gives a summary of the findings from the four exhumed sites. Included are k_{allow} values from the laboratory testing of the same type of geotextile used at each site, and k_{reqd} values for the site-specific hydrologic and waste conditions from the HELP model. Also included in Table 17 is the DCF for the geometry of the geotextile filters that were actually used at each of the exhumed field sites.

Using the design formula recommended in Equation 7, it is seen that the calculated factor-of-safety values are drastically different for the four different sites. Clearly, the three sites that showed evidence of excessive clogging, i.e., Sites 1, 3 and 4, could readily have been predicted as failures (FS = 0.0003, 0.18 and 0.53, respectively) and major modifications should have been made in the design stage. Also, Site No. 2 was still functioning and was substantiated using this formulation although its factor-of-safety is certainly not excessive (i.e., 7.1) in light of the required long-term performance of the filter.

Obviously, the factor-of-safety must be greater than one, and the higher the value the more conservative is the design. As far as the selection of a recommended value for the factor-of-safety, the decision is really that of the design engineer. Of course, it must be approved by the reviewer of the permit application as being acceptable for the particular site in question. Based upon observations at the the four exhumed sites it appears that the factor-of-safety should have been in excess of ten (10) since the sites are either practicing leachate recycling or are accepting sludge as part of the waste stream. This issue as well as providing design guidance as to index properties of specific geotextile filters will be given in the conclusions and recommendations section.

Table 17 - Corroboration of the Newly Modified Factor-of-Safety Equation (Equation 7) as Applied to Four Exhumed Field Sites

Site	Observed performance	k_{allow} (cm/s)	k_{reqd} (cm/s)	Value of DCF	Calculated factor-of-safety value	Predicted performance
1	Terrible	6×10^{-4}	1×10^{-5}	24,000	0.0003	Failure
2	Good	1×10^{-2}	1×10^{-5}	140	7.1	Acceptable
3	Terrible	9×10^{-3}	5×10^{-5}	990	0.18	Failure
4	Poor	9×10^{-3}	1×10^{-5}	1,700	0.53	Failure

9. Conclusions and Recommendations

The project around which this report has been written focused on various types of landfill leachate collection filters. It was felt that filters were the most likely component of a leachate collection system to become excessively clogged. Such excessive clogging would eventually starve the downstream drain (gravel or geonet) and pipe system of leachate, thereby rendering the liquids management practice ineffective. Current options of liquids management practices in this regard are either leachate withdrawal on demand or leachate recycling.

It should be clearly recognized that the design of the filter of a landfill leachate collection system is considerably more challenging than filter design in transportation and geotechnical engineering applications. This is due to the nature of the liquid (leachate versus water), the required lifetime before excessive clogging occurs (generally decades) and the filter's inaccessibility after the waste is placed (cost of exhuming and environmental concerns). The situation is exacerbated if leachate recycling is the liquid management strategy at the landfill site.

With these thoughts in mind and having the benefit of an earlier EPA financed study to bear upon, a five-part study has been reported herein. The individual parts, and their respective conclusions, were as follows.

(a) The *field exhuming* work described in Section 5 was particularly rewarding. It gave insight as to "ground truth" and although only four sites were exhumed they set the tone for much of the work to follow:

- In the three sites with little to no leachate flowing out of the collection pipes, a geotextile filter was the culprit with clear indications of excessive clogging.
- In all three cases, the geotextile filter was wrapped directly around the perforated removal pipe, i.e., it was so-called "socked pipe". This configuration must simply cease to be designed and installed.
- The fourth site exhumed was still functioning. It had a geotextile filter wrapped around a gravel trench with a perforated pipe within the gravel. This was a more acceptable filter configuration than was socked pipe.
- The exhuming of these four sites points to what was felt to be the preferred filter design configuration, i.e., one in which the filter covered the entire footprint of the landfill or cell. This was called an "areal filter".
- Either geotextile or soil filters could be used in areal filter configurations, however, geotextiles will generally be favored due to savings in the initial installed cost and greater available landfill volume due to reduction in thickness over natural soil filters.
- Regarding the leachate itself, it appeared as though either high suspended solids or high microorganism content, or a combination of both, were important insofar as excessive filter clogging was concerned.
- These two properties of leachate were best quantified by the total suspended solid (TSS) content and the biochemical oxygen demand (BOD) as characterized by the 5-day measurement, i.e., BOD_5.

- Although very subjective, leachates with TSS ≥ 2500 mg/l and/or BOD_5 ≥ 2500 mg/l were considered harsh and were of special concern.

(b) Long-term *laboratory permeability* tests described in Section 6 focused on ten geotextile filters and two soil filters. The tests were performed at constant flow rates using four different permeants. Water was the control and its response was counterpointed against three different municipal solid waste leachates. The flow rates were accelerated over typical field flow rates and were categorized as "very high", "high" and "intermediate". Data for the very high flow rates were given in Section 6 and for the latter two flow rates were given in Appendix "B".

- The ASTM D1987 test method can be used as a performance test to determine allowable filter permeability using most types of liquids, including leachate, as the permanent.
- The test can be conducted for geotextile or soil filters, under aerobic or anaerobic conditions, using falling head or constant head measurements, for as long of a period of time as is necessary to reach a conclusion about terminal (or equilibrium) permeability.
- The test results can be presented as permeability (k), permittivity (ψ) or flow rate (q).
- The twelve different filters examined in these tests came to approximate equilibrium between 120 and 300 days.
- The equilibrium flow rates varied by two orders of magnitude between the different types of filters investigated.
- The highest equilibrium flow rates were the high denier, thick, nonwoven geotextiles, open woven geotextiles and sand, i.e., the filter materials with the largest voids.
- Utilizing accelerated quantities of leachate flow (i.e., higher than field anticipated values) passing through the various filters allowed for back-extrapolation to the leachate quantity anticipated at a given landfill site. This procedure resulted in a site specific value of "k_{allow}", "ψ_{allow}" or "q_{allow}" for the candidate filter.
- The three different municipal solid waste leachates used for these long term laboratory tests gave remarkably similar flow rate results. This was surprising since one leachate was selected for its high TSS content, another for its high BOD_5 content and the third for a combined high TSS and high BOD_5 content.
- The use of water as a control permanent was important in illustrating that filters for leachate are considerably more challenging than water with respect to excessive clogging. The use of water as a control also signified that the laboratory test setups were properly functioning.

(c) An effort was made to utilize the HELP model developed under EPA sponsorship. Such modeling was necessary from a design perspective since filter design requires a predicted, or required, flow value of "k_{reqd}", "ψ_{reqd}" or "q_{reqd}" for utilization in arriving at a factor-of-safety value. In this part of the study (described in Section 7 and presented as a parametric study in Appendix "C") the following conclusions were reached.

- For no waste in the cell and the drainage system acting as a dewatering system, flow rates decreased with decreasing the drainage layer permeability from 1.0 to 1×10^{-3} cm/s, i.e., the filter was of no consequence.
- Furthermore, the flow rate was not altered until the filter permeability became less than approximately 1×10^{-5} cm/s.
- As waste was placed in the facility, drainage layer permeabilities from 1.0 to 1×10^{-3} cm/s only moderately decreased flow rates from the collection system.
- Furthermore, for filter permeabilities from 1.0 to 1×10^{-5} cm/s, the flow rate exiting from the system only decreased moderately.
- For filter permeabilities less than 1×10^{-5} cm/s, however, the drainage system flow rates decreased rapidly.
- For the "no filter" case presented in Appendix "D", the HELP model indicated that the drainage layer permeability directly influenced the amount of leachate drained for the no waste condition. For accumulated waste, the flow of leachate did not vary greatly for drain layer permeabilities between 1.0 and 1×10^{-5} cm/s. Lower values, however, began to rapidly decrease the exiting flow rates.
- The HELP model was felt to be successful in predicting the required flow rates from the four exhumed field sites for subsequent analysis in the design model substantiation.

(d) The *design method formulation and substantiation* part of the study described in Section 8 was successful in utilizing each of the previous sections and arriving at a final design formulation. Furthermore, the findings of the field exhumed sites were used to challenge the validity of the design model. More specifically, the following items were concluded.

- The traditional factor-of-safety model of comparing allowable permeability with required permeability was not appropriate if the site specific drain configuration limited the available flow area to less than that of the landfill or cell footprint area.
- In light of the above statement a drainage correction factor, "DCF", was included in the denominator of the traditional factor-of-safety formulation, i.e.,

$$FS = \frac{k_{allowable}}{k_{required} \times DCF} \tag{7}$$

- Values of DCF varied enormously. They ranged from 1.0, where the filter covered the entire footprint of the landfill or cell, to 24,000 for the case of geotextile socked smooth wall perforated pipe.
- Other filter configuration strategies resulted in DCF values that were between these two extremes.
- The implication of high DCF values on the calculated value of filter factor-of-safety was obvious; high DCF values resulted in disastrously low factors-of-safety values.

- These low factor-of-safety values were associated with geotextile socked pipe of both the smooth wall and corrugated wall variations. Neither type should be used for the filter of a leachate collection system.
- The above formulated design equation was utilized to predict the factor-of-safety of the filters in the four sites that were exhumed. A direct correspondence to the observed behavior was seen to exist. For the three sites where leachate was not flowing, the predicted factor-of-safety values were 0.0003, 0.18 and 0.53, respectively. All are obviously unacceptable. For the one site where leachate was still flowing the predicted factor-of-safety was 7.1. This value was felt to be marginally acceptable.
- While the "ground truth" data of the four exhumed sites was sparse, we feel that the design formulation embodied in Equation 7 was substantiated and should be used for all landfill leachate collection system filters.

(e) Long-term laboratory permeability tests were performed on the "no filter" design strategy as described in Appendix "D". In this strategy, "select" solid waste was placed directly on drainage soil (gravel or sand) with no filter used whatsoever. Eight long term "no filter tests", 4 on gravels and 4 on sands, were conducted for up to 1000 days. Flow rates though the permeameters were typical of leachate field rates, i.e., these tests were not accelerated flow rate tests. The following conclusions were reached.

- For the gravel tests, the solid waste above the drainage layer became the controlling flow material. The system permeability was essentially constant for the duration of the tests.
- For the sand tests, the solid waste again dominated the flow behavior. However, there was a trend that the system permeability was decreasing over time which was due to particulate and/or microorganisms clogging of the sand drainage material.
- It was felt that if the "no filter" strategy is used, the drainage material should be gravel with a permeability of 1.0 cm/sec, or higher.
- Furthermore, if a "no filter" strategy was proposed and the leachate had high TSS and/or high BOD_5 (e.g., either value greater than 2,500 mg/l), then long-term laboratory tests should have been conducted to substantiate the design feasibility.

Insofar as *recommendations* from this study are concerned, it is felt that the optimal strategy for the filter of a leachate collection system is to place a geotextile over the entire footprint of the landfill. In this way, the drainage correction factor (DCF) in Equation 7 is 1.0 and all of the filters we have investigated during the course of this study result in acceptably high long term flow rates for the sites that were evaluated. Even further, we can suggest that for municipal solid waste leachates where the TSS and BOD_5 are not excessively high, the types of geotextile filters listed in Table 18 can be recommended.

Table 18 - Recommended Geotextile Filters(1) for use with Relatively Mild Landfill Leachates Which Have Low TSS and Low BOD_5 Values, e.g., Less than 2500 mg/l(2)

Type of Geotextile	Granular Soil Protection Layer Over Filter	Select Waste(4) Placed Directly Over Filter
(a) Woven Monofilament(3)		
mass per unit area, g/sq. m (oz/sq. yd.)	170 (5.0)	200 (6.0)
percent open area, %	10 ——	10 ——
grab tensile strength, N (lb)	1100 (250)	1400 (310)
trapezoidal tear strength, N (lb)	250 (55)	350 (80)
puncture strength, N (lb)	400 (90)	500 (110)
burst strength, kPa (lb/sq. in.)	2700 (390)	3500 (510)
(b) Nonwoven Needle Punched(3)		
mass per unit area, g/sq. m (oz/sq. yd.)	200 (6.0)	270 (8.0)
apparent opening size, mm (sieve size)	0.212 (#70)	0.212 (#70)
grab tensile strength, N (lb)	700 (160)	900 (200)
trapezoidal tear strength, N (lb)	250 (55)	350 (80)
puncture strength, N (lb)	250 (55)	350 (80)
burst strength, kPa (lb/sq. in.)	1300 (190)	1700 (250)

Notes:

1. Laboratory test data and the requisite design may result in less conservative filters than listed in the table. Properly designed they are acceptable.
2. Low TSS and BOD_5 refers to ≤ 2500 mg/l, for higher values of TSS and/or BOD_5, the procedures and details given in this report should be followed.
3. The values of strength listed in the above table are in approximate agreement of the Class 2 and Class 1 values per the proposed AASHTO M288 specification for transportation facilities in the high and very high survivability ratings, respectively [15].
4. Select waste cannot contain any hard or coarse material which can damage the geotextile. For hard or coarse waste the strength requirements of the geotextile must be increased at the discretion of the design engineer.

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APPENDIX "A"

BEHAVIOR OF BIOCIDE TREATED GEOSYNTHETICS

A-1 Introduction

In light of the concern over excessive clogging of leachate collection filters, an attempt at using biocides in the various flow systems was undertaken. This was done under the assumption that the biocide would kill the microorganisms that came into contact with it and that the non-viable (i.e., dead) matter would pass through the system in much the same way that fine particles or sediment moves through any other drainage system. Furthermore, the introduction of the biocide was felt to be best achieved when delivered on a long-term basis rather than as one bulk dosage. Thus, biocide was added to the polymer compound during fabrication of selected geonets or geotextiles. The reasoning for this approach was that the biocide would time release, via molecular diffusion, through the polymer structure and migrate to the surface of the ribs or filaments over a long period of time. If the approach was seen to be of value, calculations could then be made as to the long-term time release behavior. This Appendix to the report describes our attempts to increase the flow rates of landfill leachate filters and drainage systems using biocide treated geonets and geotextiles.

A-2 Type of Biocide

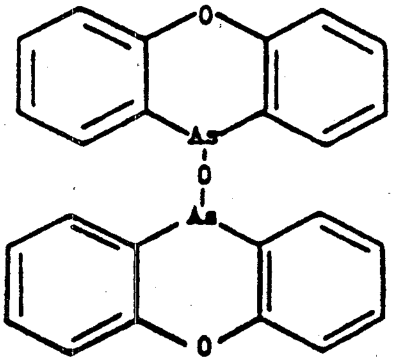
The biocide used in this study is Vinyzene® SB-1 PR manufactured by Morton Thiokol, Inc. of Danvers, Massachusetts. Vinyzene SB-1 PR is a concentrate of 10, 10' - oxybisphenoxarsine (OBPA) in a polypropylene resin carrier. The product, is supplied as a homogeneous solid in pelletized form measuring approximately 3.5 mm by 2.5 mm. It is recommended for use in polyolefins and other polymeric compositions requiring preservation against fungal and bacterial deterioration. The manufacturer states that "low levels of Vinyzene SB-1 EEA (a similar product but in an ethylene acrylic acid copolymer resin carrier) will provide long term preservation against fungal and bacterial attack and will help prevent surface growth, permanent staining, embrittlement and premature product failure...Vinyzene can be incorporated into the polymer compound at any convenient stage of the manufacturing process. The product can be fed into an extrusion operation in much the same way as pelletized color concentrates."

In 1976, EPA placed OBPA on its list of suspect pesticides that might be hazardous to human health. EPA's review of animal and other studies on OBPA, however, indicated that it was not as hazardous as originally suspected. On May 4, 1979 the U.S. Environmental Protection Agency decided that the pesticide, OBPA, which is used in a wide variety of plastic consumer products to protect them from fungal and bacterial damage, did not pose a threat to human health or the environment if used in accordance with label instructions. This decision meant that OBPA was restored to its former place on EPA's list of currently registered pesticides.

Materials containing OBPA include swimming pool liners, wall coatings, vinyl roofs on cars, marine upholstery, awnings, industrial fabrics, and caulking for tubs, sinks, weatherstripping and gutter repair. The EPA registration number for Vinyzene® is 2829-115 and Morton Thiokol's patent number is 4,086,297.

Some selected physical and chemical properties of 10, 10'-oxybisphenoxarsine (OBPA) are as follows.

• Molecular Formula:	$C_{24}H_{16}As_{2}0_{3}$
• Molecular Weight:	502.2
• Structural Formula:	



• Specific Gravity:	1.40 - 1.42
• Appearance:	White to off-white crystalline solid
• Melting Point:	185 - 186°C.
• Vapor Pressure:	$< 10^{-6}$ torr @ 21°C
	$< 10^{-6}$ torr @ 100°C
	1.0×10^{-3} torr @ 150°C
	3.0×10^{-3} torr @ 200°C
	8.5×10^{-3} torr @ 250°C
• Thermal Decomposition Range:	300 – 380°C
• Solubility:	5 ppm in H_20
	2.75 gm/100 gm of 95% ethanol
	2.30 gm/100 gm of isopropanol
	2.78 gm/100 gm of xylene

A-3 <u>Incorporation of the Biocide into Different Geosynthetics</u>

The biocide was shipped to the respective geosynthetic fabrication facilities for inclusion into the candidate geonet or geotextile. After the dosage was decided upon (it varied from 1 to 8% by weight), it was added to the standard compound, suitably mixed and extruded into ribs (for geonets) or filaments (for geotextiles).

In the first series of tests, either 1, 2 or 4% biocide levels were introduced into the compound to produce a 6.3 mm (0.25 in.) thick, high density polyethylene (HDPE) geonet. For control purposes, the same type of geonet was produced without the addition of any biocide. The cross section of the columns for these tests consisted (from the top down) of sand/geotextile/geonet/gravel and they were coded as Series "A", see Table A-1. Tests were conducted both saturated at all times (hence anaerobic) and allowed to air dry between readings (thus aerobic). All tests in this series were run for 444 days.

The second series of tests, i.e. Series "B", utilized the biocide in a geotextile and did not use a geonet. The cross section consisted of sand/geotextile/gravel. The geotextile was a nonwoven needle punched polypropylene and it contained either 2% or 4% biocide. The biocide was introduced at the fabrication facility along with the manufacturers standard compound of resin, carbon black (or other antidegradant) and processing package. As seen in Table A-1, there were also geotextiles included with no biocide to act as control materials. Tests were conducted under both constantly saturated conditions (hence anaerobic) and intermittently saturated, then air dried conditions (thus aerobic). All tests in this series were performed for 444 days.

Test Series "C" consisted of biocide treated geotextiles and no geonets; but unlike the previous series, three different types of geotextiles were evaluated. The geotextiles were nonwoven needle punched (as before) and also two types of woven monofilament geotextiles with different opening sizes, see Table A-1. The tests were also different in that gravel was used above the geotextile instead of sand. Thus the flow column consisted of gravel/geotextile/gravel, with the geotextiles treated with 2, 4 or 8% biocide. Again, the biocide was introduced at the manufacturing facility. In this series, which lasted 121 days, all tests were kept saturated, thus anaerobic.

A-4 Field Testing and Evaluation Procedures

Flow rate testing for each of the columns with biocide treated geosynthetics utilized the 100 mm (4.0 in.) diameter incubation and test permeameters described in the main body of the report. There were 8 columns in Series "A", 8 columns in Series "B" and 16 columns in Series "C". All permeameters were made and conducted according to the ASTM D1987 test method. Series "A" and "B" were evaluated over a 444 day duration and Series "C" were evaluated for 121 days.

All of the tests in this biocide study used leachate from the same municipal solid waste landfill site. This particular leachate had the highest concentration of COD, TS and BOD_5 of the six landfill leachates which were evaluated during the course of the project. The approximate properties of the leachate were as follows:

- pH = 5.8
- COD = 40,000 mg/l
- TS = 17,000 mg/l
- BOD_5 = 24,000 mg/l

Table A-1 - Conditions Within Flow Columns for Biocide Study

Coding(1)	Soil Above	Geotextile Type(3)	Geotextile AOS		Geotextile Pct. Biocide	Geonet(4) Pct. Biocide	Soil Below	Condition
			Size (mm)	Sieve No.				
A0-AN	Sand(2)	N-N-PP	0.15	100	0	0	Gravel(5)	Anaerobic
A0-A	Sand	N-N-PP	0.15	100	0	0	Gravel	Aerobic
A1-AN	Sand	N-N-PP	0.15	100	0	1	Gravel	Anaerobic
A1-A	Sand	N-N-PP	0.15	100	0	1	Gravel	Aerobic
A2-AN	Sand	N-N-PP	0.15	100	0	2	Gravel	Anaerobic
A2-A	Sand	N-N-PP	0.15	100	0	2	Gravel	Aerobic
A4-AN	Sand	N-N-PP	0.15	100	0	4	Gravel	Anaerobic
A4-A	Sand	N-N-PP	0.15	100	0	4	Gravel	Aerobic
B0-AN1	Sand	N-N-PP	0.15	100	0	-	Gravel	Anaerobic
B0-AN2	Sand	N-N-PP	0.15	100	0	-	Gravel	Anaerobic
B0-A1	Sand	N-N-PP	0.15	100	0	-	Gravel	Aerobic
B0-A2	Sand	N-N-PP	0.15	100	0	-	Gravel	Aerobic
B2-AN	Sand	N-N-PP	0.15	100	2	-	Gravel	Anaerobic
B2-A	Sand	N-N-PP	0.15	100	2	-	Gravel	Aerobic
B4-AN	Sand	N-N-PP	0.15	100	4	-	Gravel	Anaerobic
B4-A	Sand	N-N-PP	0.15	100	4	-	Gravel	Aerobic

Notes

(1) Test Series "A" and "B" lasted 444 days; Test Series "C" lasted 121 days.
(2) Sand was a #40 Sieve Subrounded Ottawa Sand in a 100 mm (4.0 in.) thick layer above the geotextile
(3) N-N-PP = nonwoven needle punched polypropylene
W-M-PP1 = woven monofilament polypropylene AOS = #70 sieve
W-M-PP2 = woven monofilament polypropylene AOS = #40 sieve
(4) Geonet was 250 mil HDPE
(5) Gravel was 25 to 37 mm (1.0 to 1.5 in.) Subrounded Gravel

Table A-1 - Continued

Coding(1)	Soil Above	Geotextile Type(3)	Geotextile AOS Size (mm)	Geotextile AOS Sieve No.	Geotextile Pct. Biocide	Geonet(4) Pct. Biocide	Soil Below	Condition
C1-0-AN1	Gravel	N-N-PP	0.15	100	0	-	Gravel	Anaerobic
C1-0-AN2	Gravel	N-N-PP	0.15	100	0	-	Gravel	Anaerobic
C1-2-AN1	Gravel	N-N-PP	0.15	100	2	-	Gravel	Anaerobic
C1-2-AN2	Gravel	N-N-PP	0.15	100	2	-	Gravel	Anaerobic
C1-4-AN1	Gravel	N-N-PP	0.15	100	4	-	Gravel	Anaerobic
C1-4-AN2	Gravel	N-N-PP	0.15	100	4	-	Gravel	Anaerobic
C1-8-AN1	Gravel	N-N-PP	0.15	100	8	-	Gravel	Anaerobic
C1-8-AN2	Gravel	N-N-PP	0.15	100	8	-	Gravel	Anaerobic
C2-0-AN1	Gravel	N-N-PP	0.15	100	0	-	Gravel	Anaerobic
C2-0-AN2	Gravel	N-N-PP	0.15	100	0	-	Gravel	Anaerobic
C2-2-AN	Gravel	N-N-PP	0.15	100	2	-	Gravel	Anaerobic
C2-4-AN	Gravel	N-N-PP	0.15	100	4	-	Gravel	Anaerobic
C3-2-AN	Gravel	W-M-PP-1	0.21	70	2	-	Gravel	Anaerobic
C3-4-AN	Gravel	W-M-PP-1	0.21	70	4	-	Gravel	Anaerobic
C4-2-AN	Gravel	W-M-PP-2	0.42	40	2	-	Gravel	Anaerobic
C4-4-AN	Gravel	W-M-PP-2	0.42	40	4	-	Gravel	Anaerobic

Fresh leachate was used for each test since it was taken directly out of the sump at the low elevation of the landfill or from the nearest underground storage tank.

The tests were of the falling head variety which measured the time of flight for a high head of leachate to reach a predetermined lower value. The protocol for the test itself is included in the ASTM D1987 test method. Calculations allowed for the determination of a "system" hydraulic conductivity, or permeability value, which was in units of cm/sec. Note, however, that the permeabiliy being measured was the permeability of the composite system including all components which may retard flow. In this Appendix, the "permeability" value will be used on a comparative basis, with the original value being the highest that the system could possibly achieve.

A-5 Results of Test Series "A"

As indicated in Table A-1, this test series consisted of a sand/geotextile/geonet/gravel cross section with the biocide having been introduced into the geonet during its manufacture. The biocide levels were at 0, 1, 2 and 4% and tests were conducted under both anaerobic and aerobic conditions.

The system permeability results for test Series "A" are shown in Figures A-1 and A-2 representing anaerobic and aerobic conditions, respectively. Separate curves are presented for the control and each biocide level in the geonet. Comparison of these two figures indicates that there is essentially no difference in the flow characteristics from the anaerobic to the aerobic state. Within the curves of each figure a nominal improvement in permeability from using 2 or 4% biocide in the geonet was evidenced at the conclusion of the 444 day test period. However, because the improvement in flow is nominal at the end of testing and flow improvement is not evidenced throughout the entire testing period, statistical variation in the data may influence the behavioral trends. Our general feeling was that using biocide in the geonet was simply not logical since the flow rate in the geonet was relatively high. Thus the biocide probably did not have adequate residence time to be effective.

A-6 Results of Test Series "B"

As indicated in Table A-1, Test Series "B" consisted of a sand/geotextile/gravel cross section with the biocide having been introduced into the geotextile. The biocide levels were at 0, 2, and 4% and tests were conducted under anaerobic and aerobic conditions. The rational for this change from the previous test series was that flow in the geotextile would be much lower than in the geonet due to its significantly smaller void spaces. The decreased flow rate in the geotextile would possibly allow for the biocide to have a greater contact time with the microorganisms in the leachate and hence be more effective.

Figures A-3 and A-4 provide a comparison of anaerobic and aerobic conditions. A replicate for the geotextile with 0% biocide was provided for each condition and the data was averaged for plotting. A comparison of these figures reveals little difference in flow characteristics from anaerobic to aerobic conditions. This same trend was seen previously with the geonet tests. Generally, the geotextile with 4% biocide provided slightly higher flow rates with the exception of the anaerobic conditions in which the geotextile clogged severely beyond 400 days. As with Series

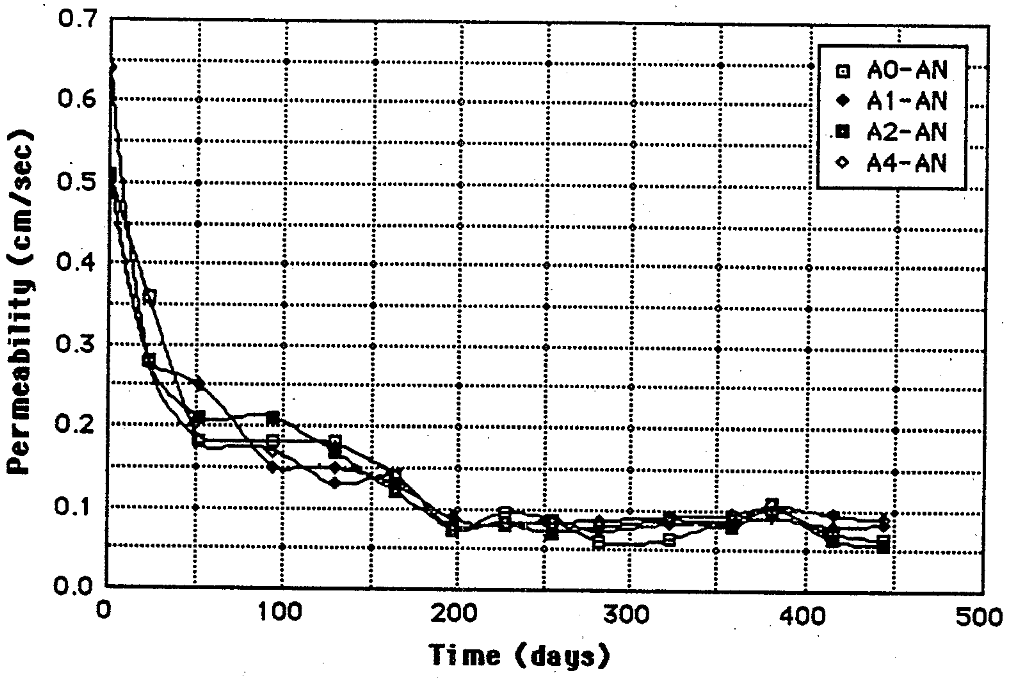


Figure A-1 - Effect of Geonet Biocide Content on System Permeability under Anaerobic Conditions (Test Series "A")

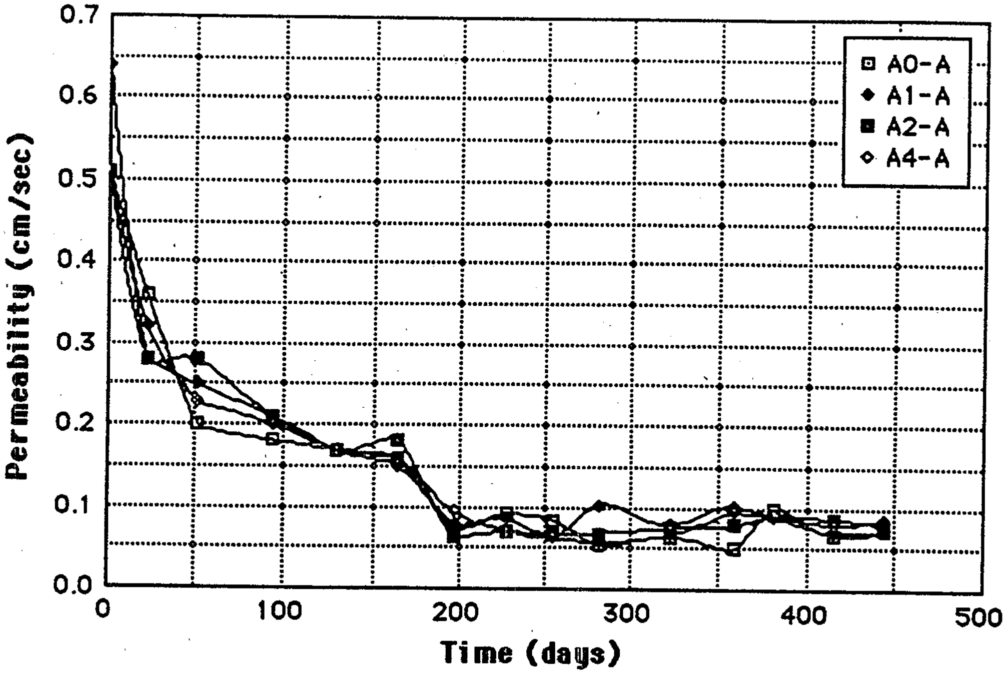


Figure A-2 - Effect of Geonet Biocide Content on System Permeability under Aerobic Conditions (Test Series "A")

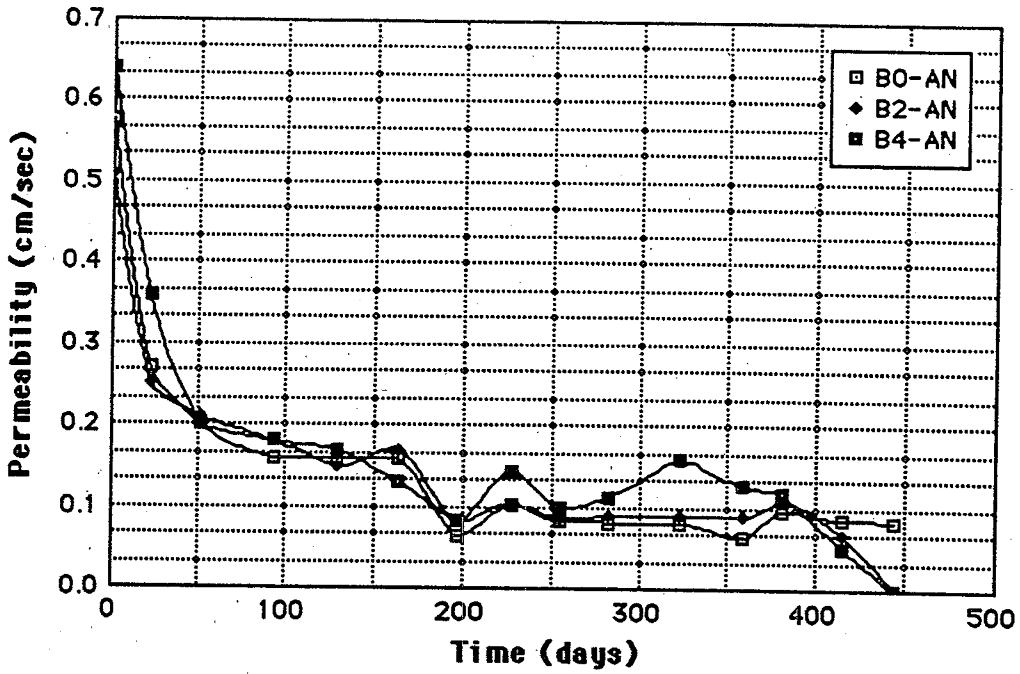


Figure A-3 - Effect of Geotextile Biocide Content on System Permeability under Anaerobic Conditions (Test Series "B")

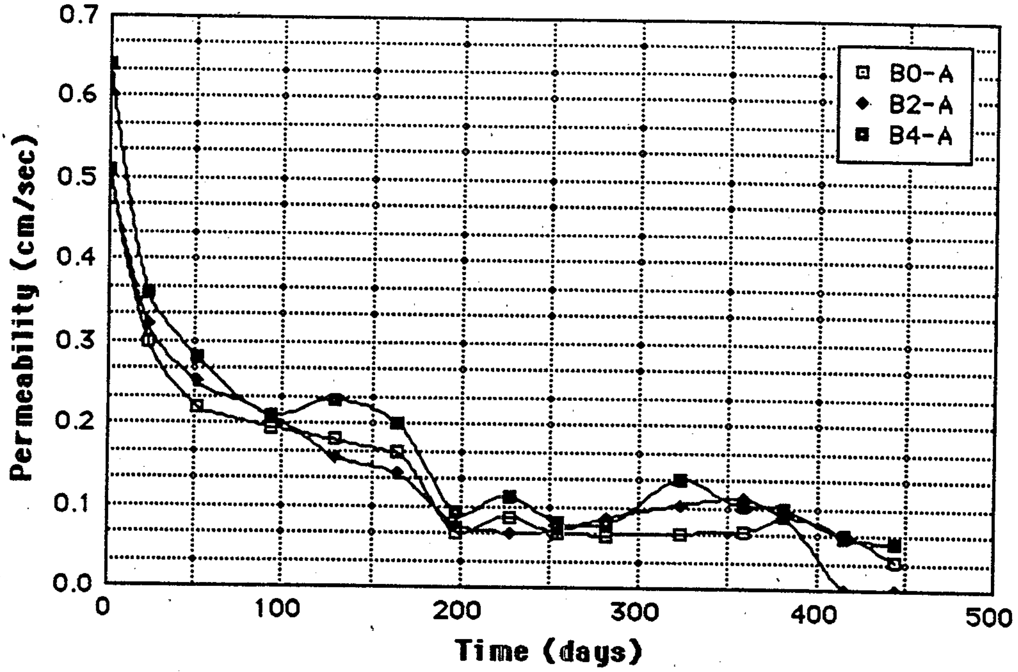


Figure A-4 - Effect of Geotextile Biocide Content on System Permeability under Aerobic Conditions (Test Series "B")

"A" tests, statistical variation played a significant role. Two test specimens in Series "B", B0-AN1 and B4-A, were resin set and dissected to visually determine the extent of clogging. In each specimen it appeared as though the Ottawa sand clogged within the upper 25 to 50 mm (1.0 to 2.0 in.) of the specimen. The upper layer of soil was completely bonded together while the soil beneath it and above the geotextile was loose. The biofilm apparently did not reach the level of the geotextile indicating that either the biocide was too far from the biofilm itself or that the grain size distribution of the sand was sufficiently small to create its own clogged layer.

A-7 Results of Test Series "C"

After evaluating the behavior of the flow column trends of Test Series "A" and "B", it was apparent that a clearly defined flow improvement resulting from biocide activity was not being observed. It was considered likely that the biofilm layer was occurring in the upper portion of the sand, hence the biocide in the geonet (Series "A") and in the geotextile (Series "B") was too far away from the clogged layer to be effective. Thus, it became necessary to assemble an additional series of 16 columns without overlying sand, which was the thrust of Series "C".

Series "C" columns consisted of a cross section of gravel/geotextile/gravel. The gravel was 25 to 37 mm (1.0 to 1.5 in.) in size and was negligible insofar as retarding flow was concerned. The geotextiles were treated with varying amounts of biocide, from 0 to 8% (recall Table A-1) and all columns were evaluated in the anaerobic condition. This latter decision was made since there was little difference in the anaerobic and aerobic flow rates in the previous tests and anaerobic conditions were felt to better simulate landfill leachate conditions. The geotextiles in this series varied considerably. Those used were the following:

- nonwoven needle-punched with an opening size of 0.15 mm
- woven monofilament with a opening size of 0.21 mm
- woven monofilament with an opening size of 0.42 mm

The first part of Series "C" tests consisted of the nonwoven needle punched polypropylene geotextile with 0, 2, 4 and 8% biocide within the fabric. A replicate set was constructed so that the values used in graphing are the average of two data sets. The permeability behavior under varying biocide contents is displayed in Figure A-5. There appeared to be little difference in flow at the onset of testing, however, there was an improvement in flow with 8% biocide at the completion of testing 121 days later. The use of 8% biocide, however, may affect the strength characteristics of the geotextile and currently EPA has restricted biocide content of this type in other media to 4%. As with the other test series, statistical scatter was significant.

In the second part of Series "C" testing, a different manufacturer's nonwoven needle punched polypropylene geotextile with 0, 2 and 4% biocide was used. A replicate was constructed for the control, i.e., 0% biocide, and graphs were plotted using the average of the data sets. To compare the two different products, Figure A-6 is presented. In the first month of testing, there was little difference in flow rates. As the test progressed, the 2 and 4% biocide geotextiles tended to give better flow rates with a large improvement in flow at 121 days. However, statistical scattering and the short duration of the testing were concerns with respect to the significance of the

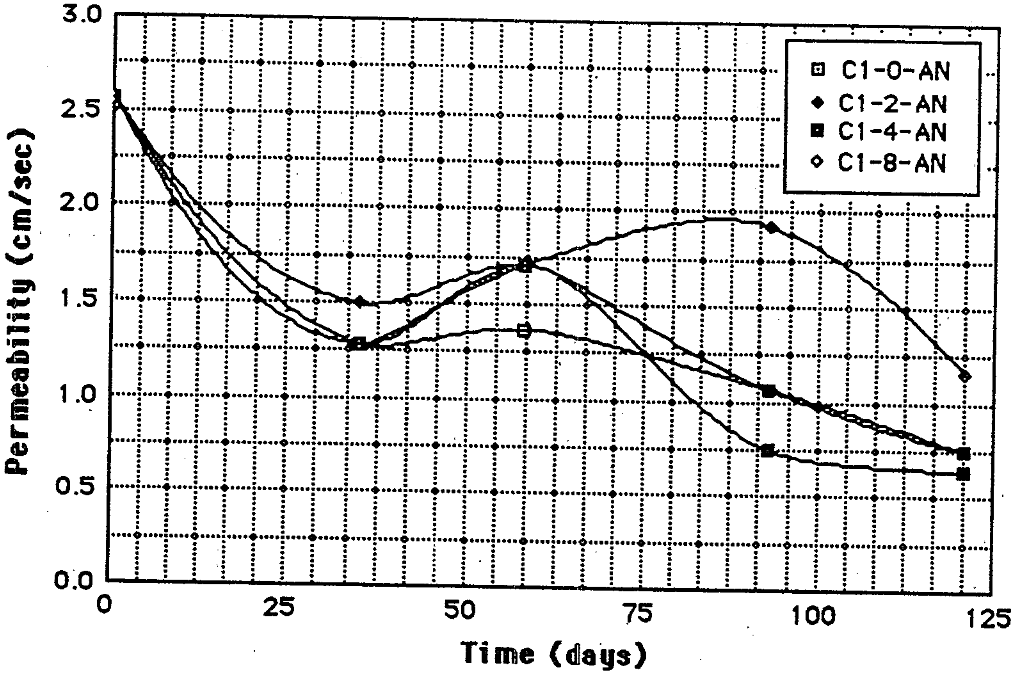


Figure A-5 - Nonwoven Needle-Punched Effect of Geotextile Biocide Content on System Permeability (Test Series "C")

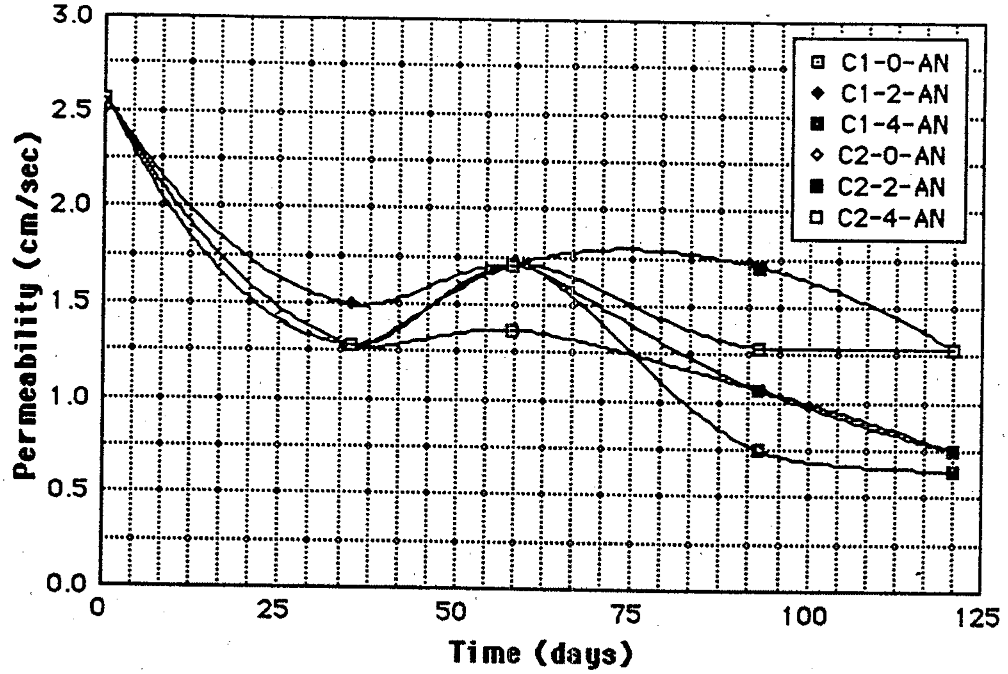


Figure A-6 - Comparison of Two Nonwoven Needle-Punched Geotextiles with Varying Biocide Content (Test Series "C") data.

Two woven monofilament polypropylene geotextiles were used for the third part of the Series "C" tests. The apparent opening sizes and other relevant test conditions are provided in Table A-1. Each geotextile was tested with 2 and 4% biocide content. A control with 0% biocide, was not used for this test set. To compare the effects of opening size on clogging, Figure A-7 was prepared. From the graph it was seen that the larger opening size geotextile provided a measurable increase in flow rate with the 4% biocide content giving better results in general. The smaller opening size geotextile with 4% biocide excessively clogged two months into the test. The four samples in this series were then epoxy resin set and dissected in the same manner as the Series B specimens. While difficult to observe visually, it was obvious that the larger opening size (0.42 mm) allowed more epoxy to flow through the geotextile indicating that it was indeed providing better flow than the 0.21 mm opening size geotextile.

A-8 Conclusions of the Biocide Study

From the results of the Series "A" tests (biocide in geonets) and Series "B" tests (biocide in geotextiles) it was concluded that the location of the biocide vis-a-vis the initial formation of a biofilm layer is critical. This conclusion was tentatively reached during the conducting of these tests. It was confirmed at the termination of the 444 day tests after setting the test columns with epoxy and cutting them apart. Clearly the biofilm layer was occurring at the top of the sand column some 50 to 75 mm (2 to 3 in.) above the biocide treated geosynthetics, recall Figure A-5. While there may have been some flow rate improvement due to high concentrations of biocide, it was very subtile (at best) and was masked by the inherent scatter in the test data. There was essentially no difference between flow rates in anaerobic versus aerobic conditions.

These findings led to Series "C" tests which contained no sand above the biocide treated geotextile and forced the leachate to interface directly with the biocide. Rather than use a single type of geotextile, three different types of geotextiles were utilized. They had opening sizes varying from 0.15 mm (the nonwoven needle punched styles used in test Series "B"), to 0.21 mm (a woven monofilament), to 0.42 mm (another woven monofilament). Quite clearly, the flow rates through the largest opening size geotextiles, i.e. the 0.42 mm, were the highest. This suggested that microorganisms (dead or alive) must be able to pass through the system. Whenever these microorganisms reside on, or within, the small pores of a filter (either natural soil or a geotextile) there was a possibility of partial, or even excessive, clogging.

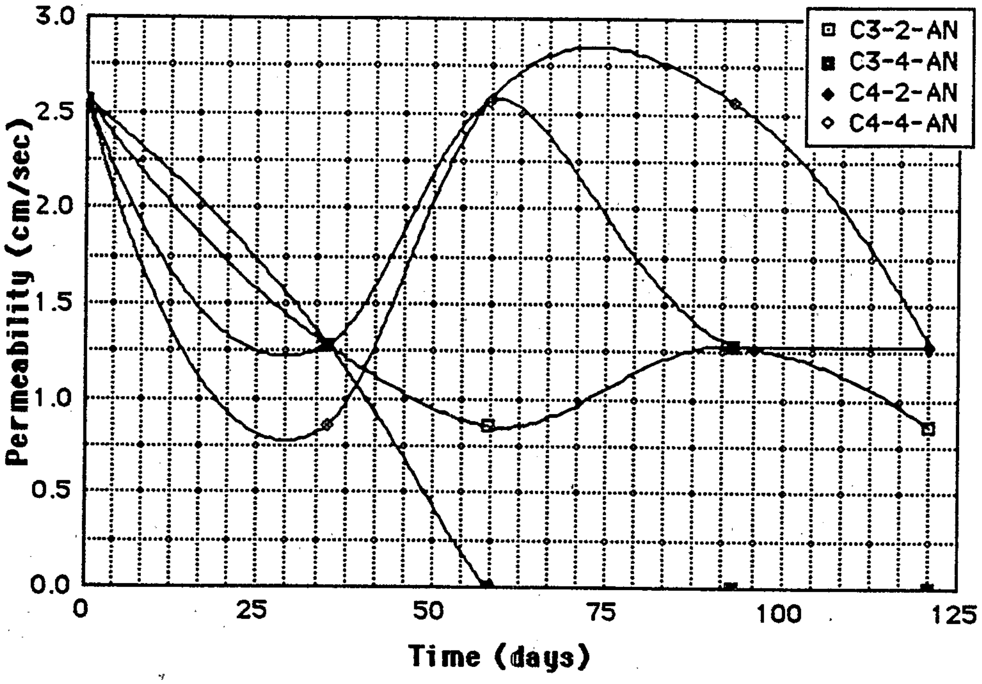


Figure A-7 - Comparison of Woven Monofilament Geotextile Opening Size with Varying Biocide Content (Test Series "C")

APPENDIX “B”

PERMEABILITY RESULTS FOR “HIGH” AND “INTERMEDIATE” FLOW RATES

(NOTE THAT THE “VERY HIGH” FLOW RATE RESULTS ARE GIVEN IN THE BODY OF THE REPORT IN SECTION 6)

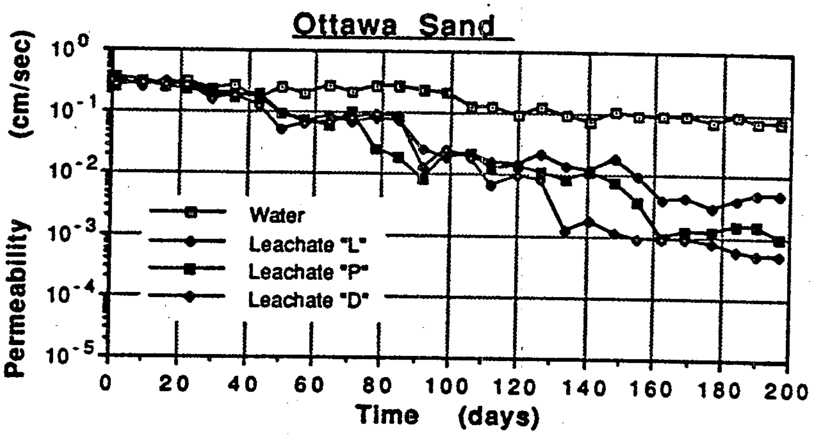


Figure B-1 - Permeability Response Curves for Ottawa Sand Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

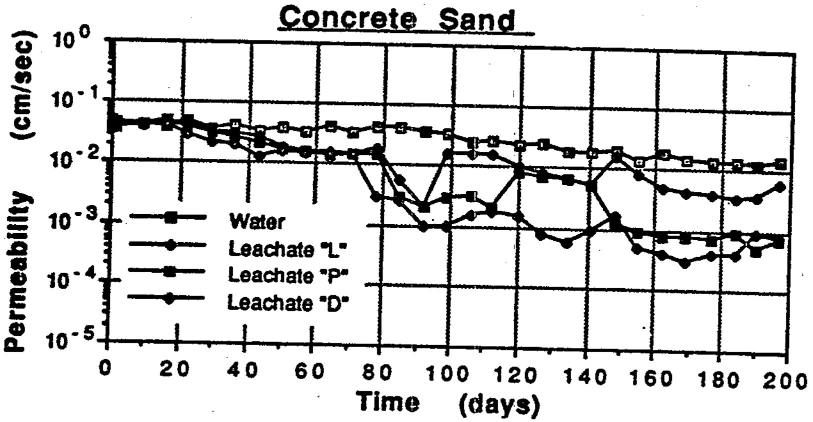


Figure B-2 - Permeability Response Curves for Concrete Sand Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

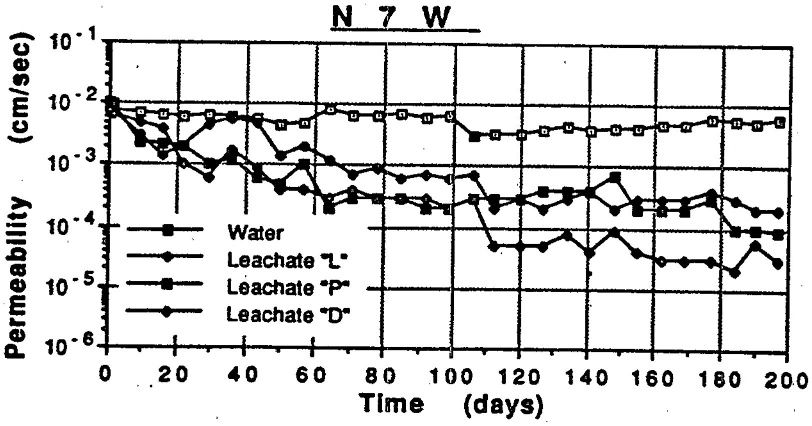


Figure B-3 - Permeability Response Curves for Geotextile Filter N 7 W Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

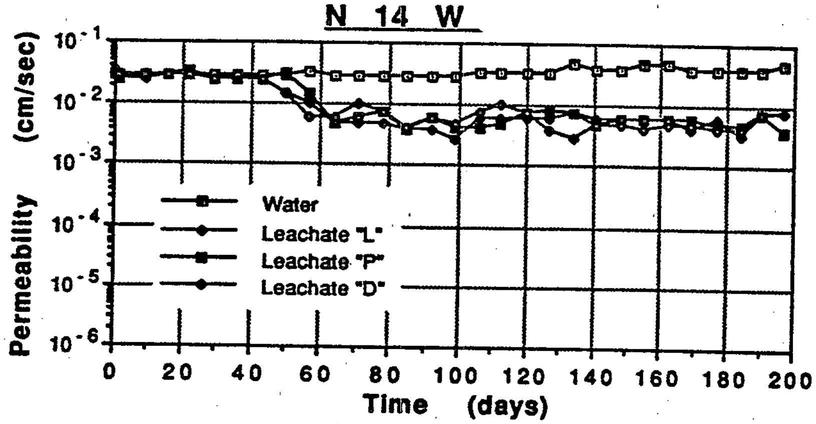


Figure B-4 - Permeability Response Curves for Geotextile Filter N 14 W Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

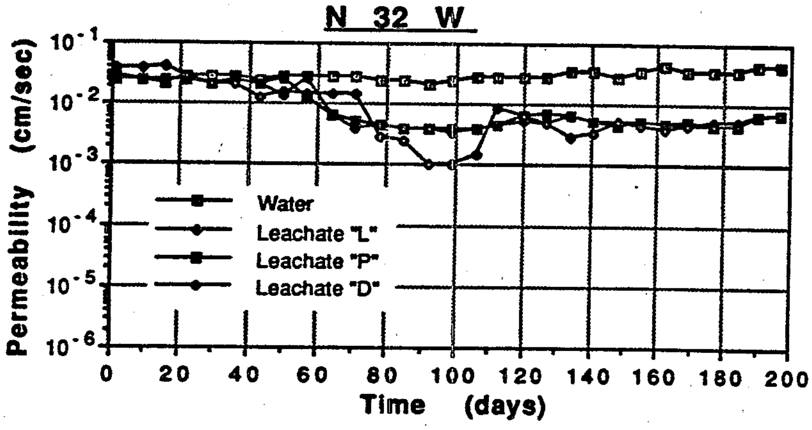


Figure B-5 - Permeability Response Curves for Geotextile Filter N 32 W Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

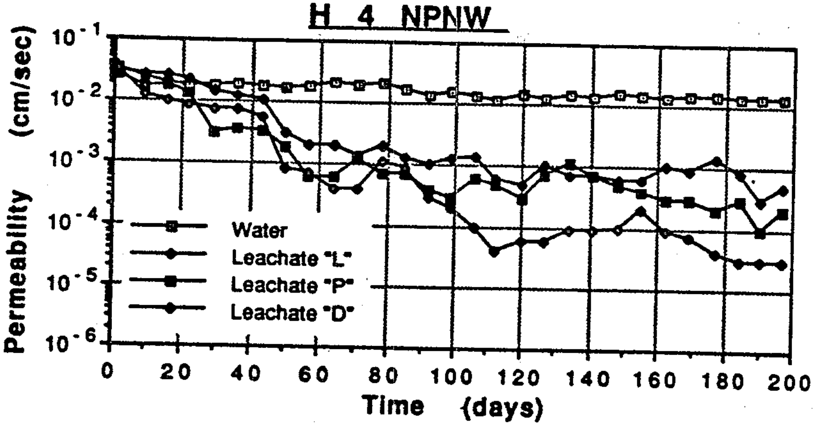


Figure B-6 - Permeability Curves for Geotextile Filter H 4 NPNW Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

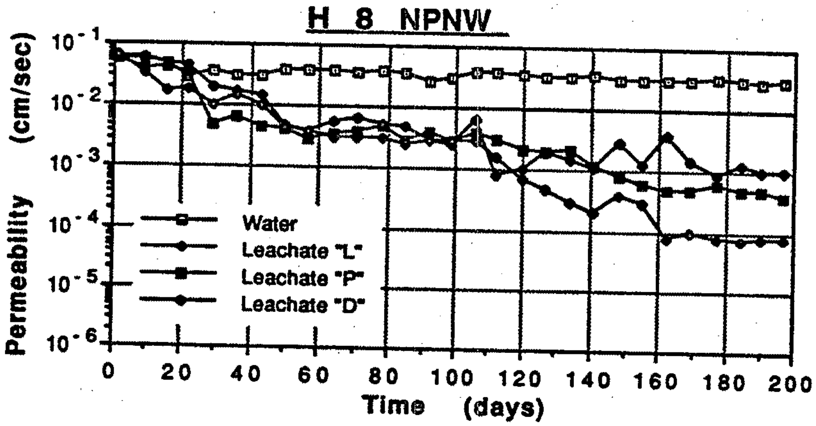


Figure B-7 - Permeability Curves for Geotextile Filter H 8 NPNW Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

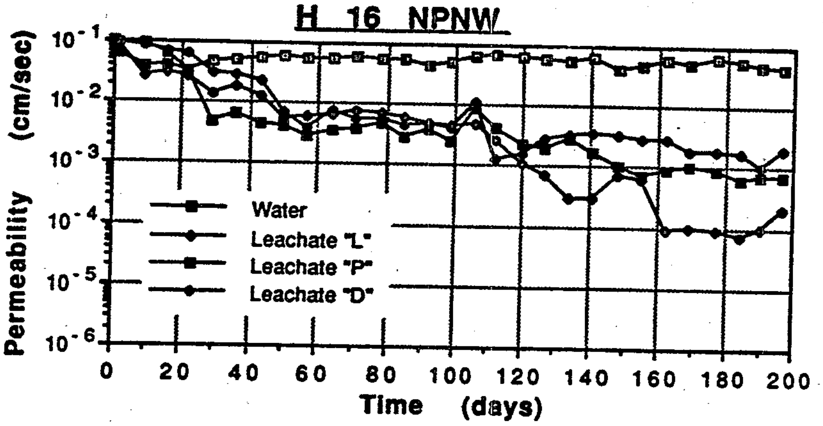


Figure B-8 - Permeability Curves for Geotextile Filter H 16 NPNW Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

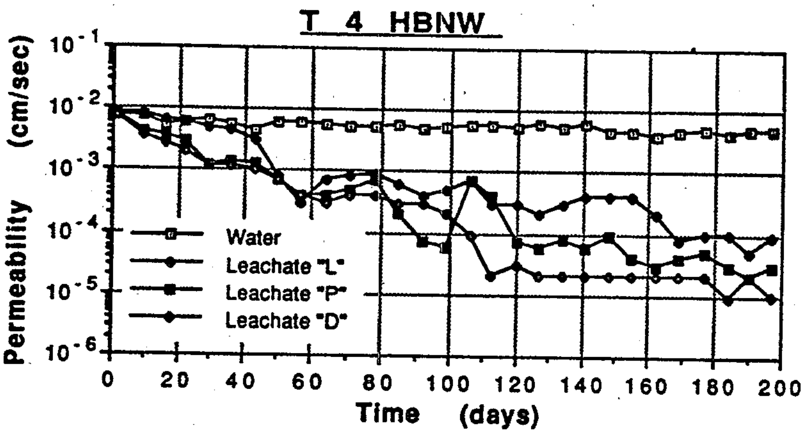


Figure B-9 - Permeability Curves for Geotextile Filter T 4 HBNW Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

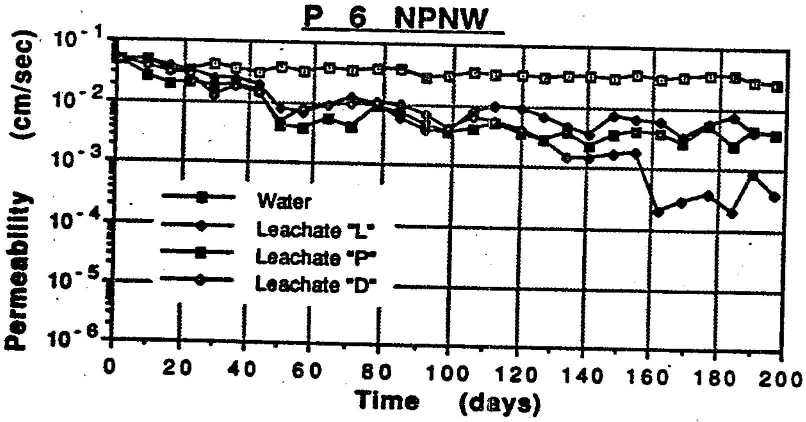


Figure B-10 - Permeability Curves for Geotextile Filter P 6 NPNW Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

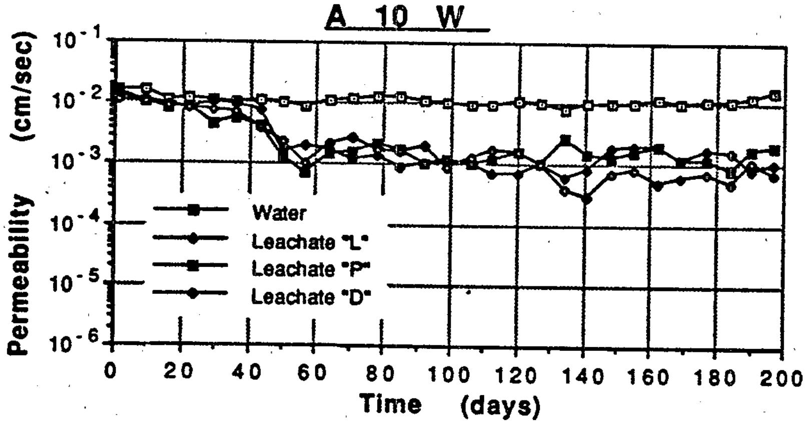


Figure B-11 - Permeability Curves for Geotextile Filter A 10 W Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

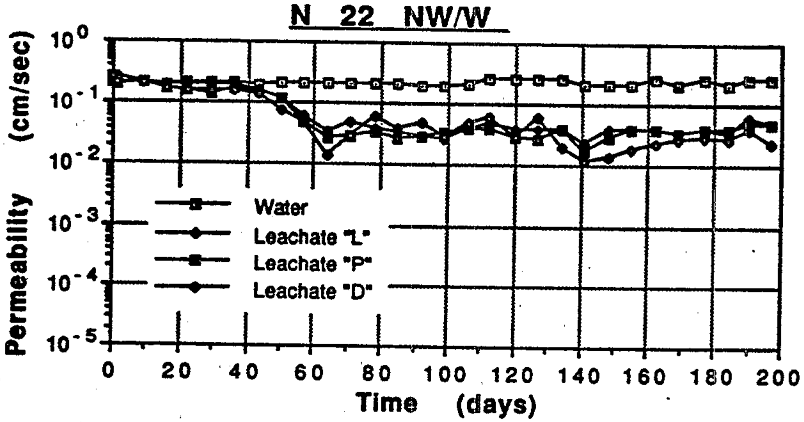


Figure B-12 - Permeability Curves for Geotextile Filter N 22 NW/W Permeated with 4 Fluids at a Fast Flow Rate of 3.8 l/week (1.0 gal/week)

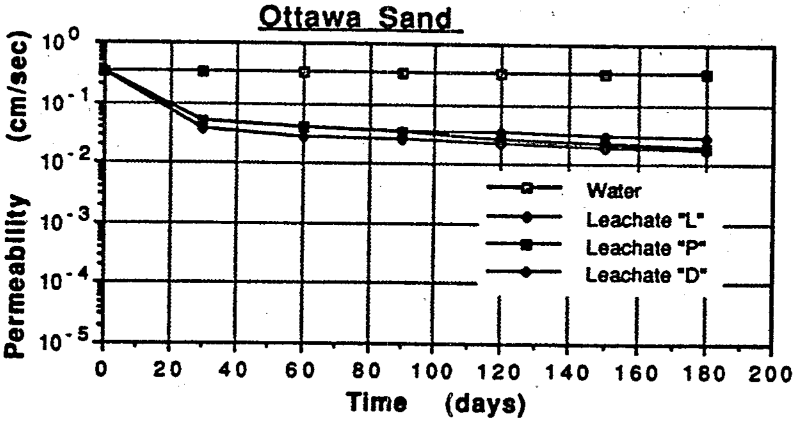


Figure B-13 - Permeability Response Curves for Ottawa Sand Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

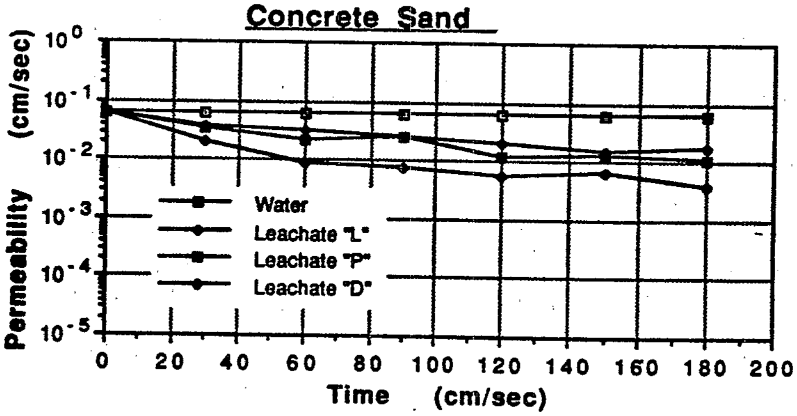


Figure B-14 - Permeability Response Curves for Concrete Sand Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

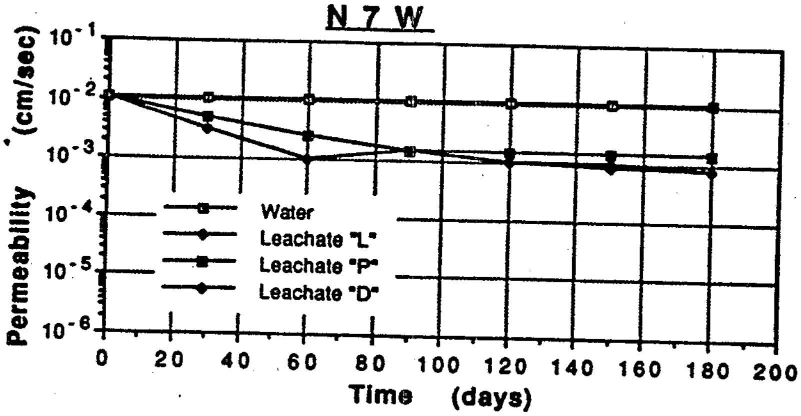


Figure B-15 - Permeability Response Curves for Geotextile Filter N 7 W Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

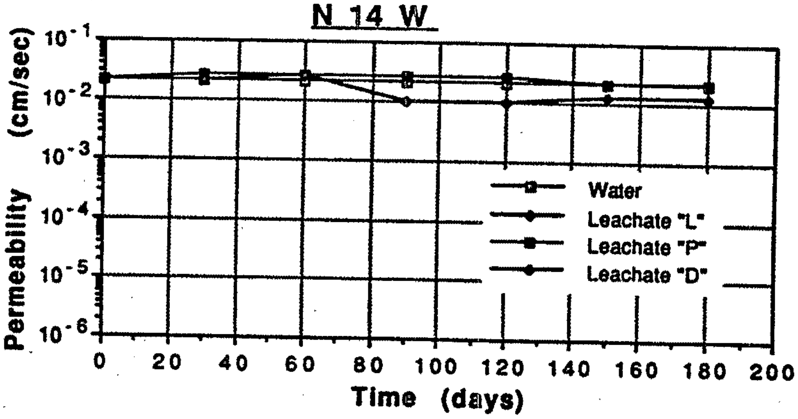


Figure B-16 - Permeability Response Curves for Geotextile Filter N 14 W Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

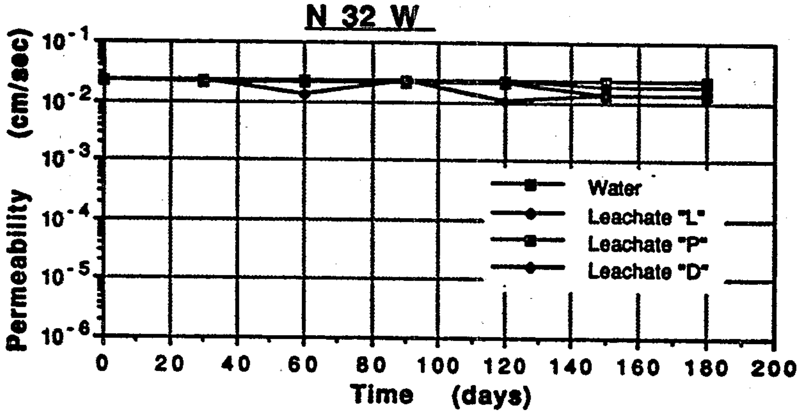


Figure B-17 - Permeability Response Curves for Geotextile Filter N 32 W Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

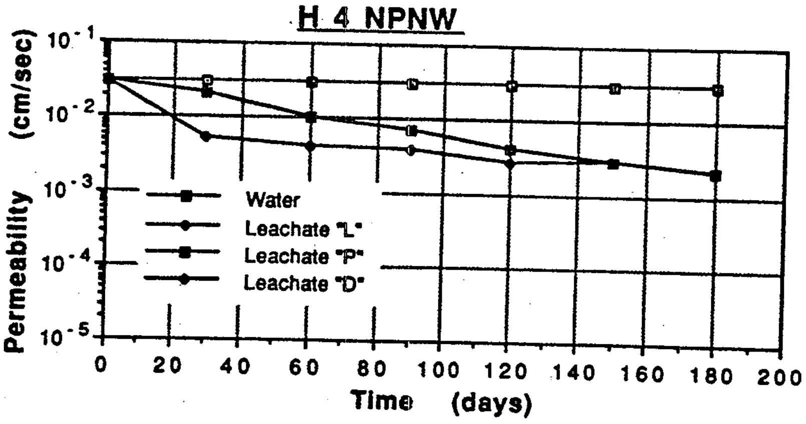


Figure B-18 - Permeability Curves for Geotextile Filter H 4 NPNW Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

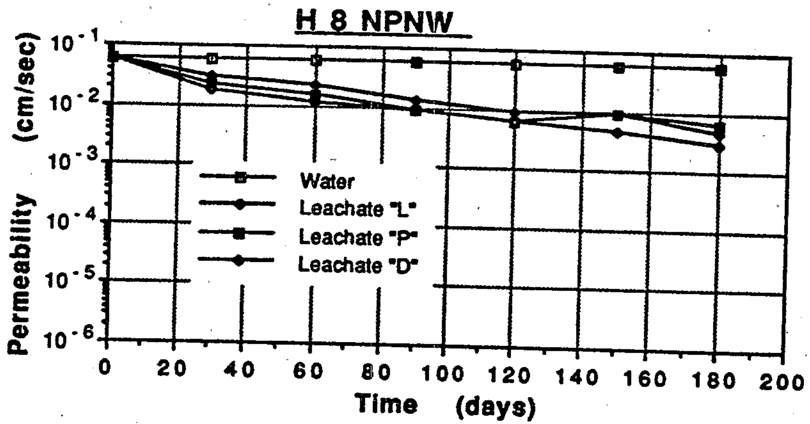


Figure B-19 - Permeability Curves for Geotextile Filter H 8 NPNW Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

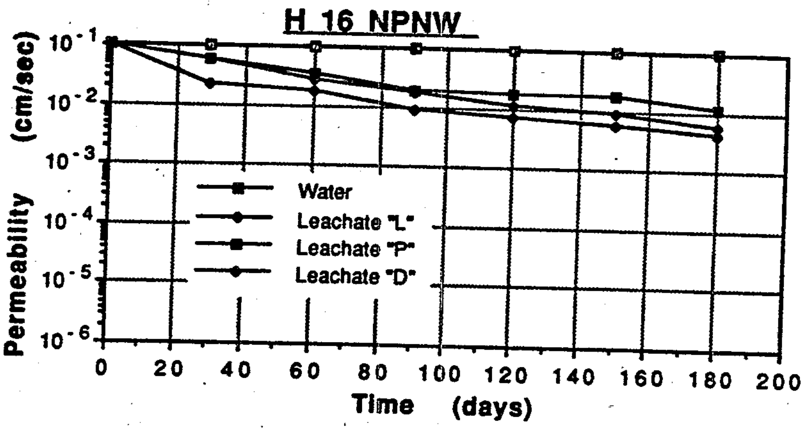


Figure B-20 - Permeability Curves for Geotextile Filter H 16 NPNW Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

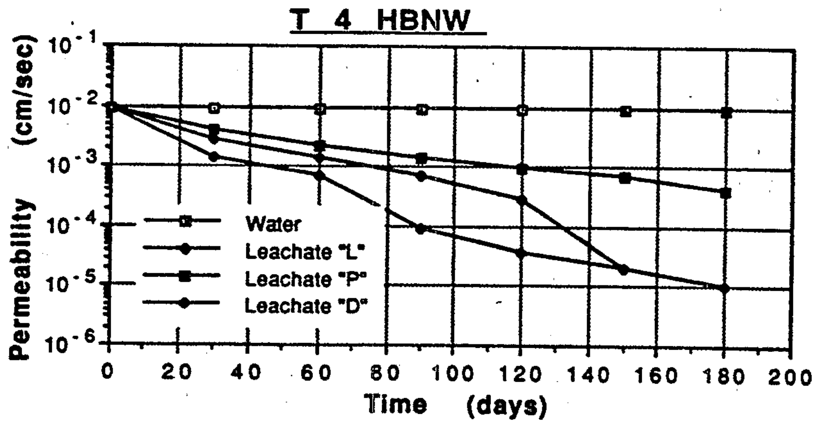


Figure B-21 - Permeability Curves for Geotextile Filter T 4 HBNW Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

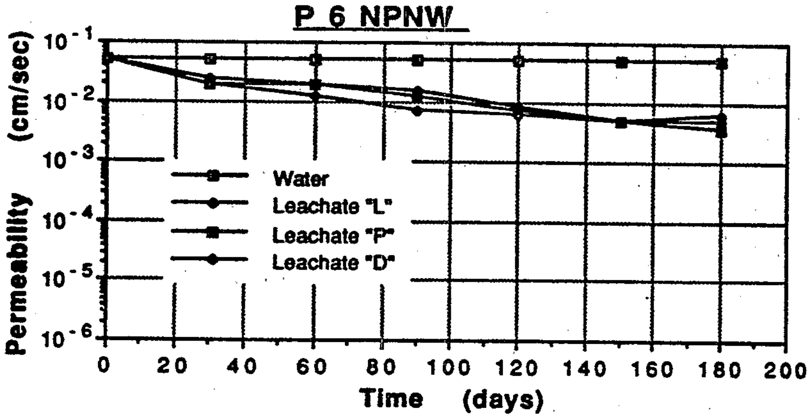


Figure B-22 - Permeability Curves for Geotextile Filter P 6 NPNW Permeated with 4 Fluids at a Fast Flow Rate of 0.95 l/week (0.25 gal/week)

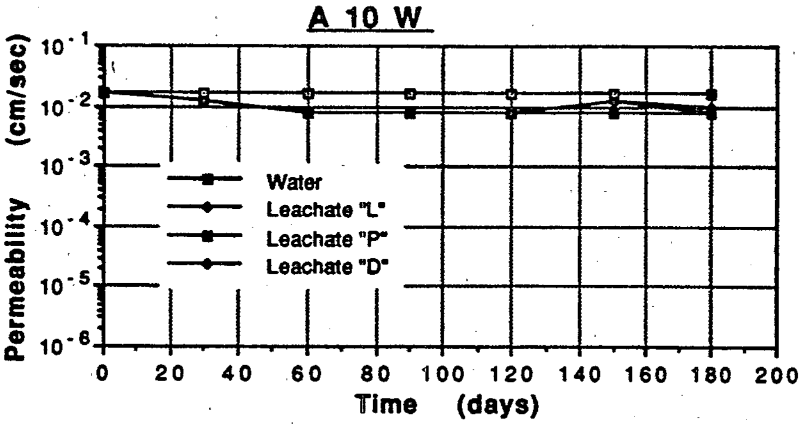


Figure B-23 - Permeability curves for Geotextile Filter A 10 W Permeated with 4 Fluids at a Intermediate Flow Rate of 0.95 l/week (0.25 gal/week)

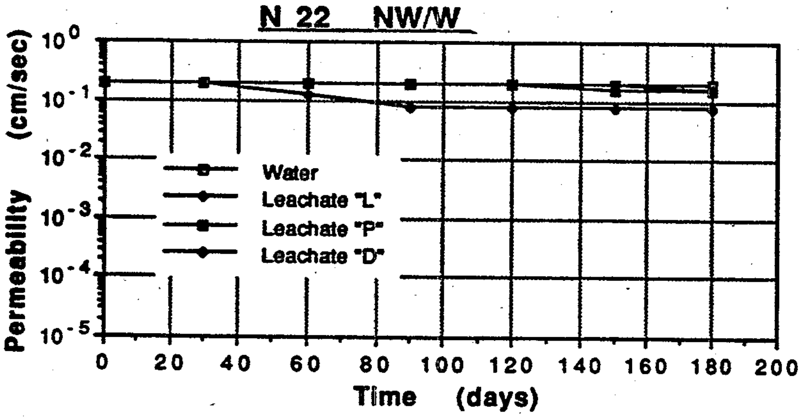


Figure B-24 - Permeability Curves for Geotextile Filter N 22 NW/W Permeated with 4 Fluids at a Intermediate Flow Rate of 0.95 l/week (0.25 gal/week)

APPENDIX "C"

HELP MODEL PARAMETRIC ANALYSIS

C-1 Overview

The HELP model which is generally used to follow and apportion the liquids flowing into and through a solid waste landfill mass was briefly described in Section 7. It was utilized to determine the site specific "k_{reqd}" values for the four landfills that were exhumed. This data was then used directly in the design method to substantiate the results of the field exhuming activities. In addition to this direct use of the model, considerable effort also went into assessing the latest version of the model insofar as a parametric analysis was concerned. The results of this ancillary effort are presented in this appendix.

The purpose of the present effort was to perform a parametric evaluation of items that are particularly important in estimating the leachate flow through a landfill and that need to be defined as input data in the HELP model. The parametric study was performed to determine the effects of these major design parameters on the amount of leachate collected at the bottom of a landfill and to highlight the likely variations in leachate flow rates due to changes in the value of a specific parameter.

The study examined the general effect of various parameters for two different landfill configurations. In both cases, final closure of the landfill had not occurred. The first configuration consisted of the waste material underlain by a lateral drainage layer (without a filter) and then an impermeable geomembrane. This is the "no filter" scenario. The second configuration assumed that a filter was placed between the waste material and the lateral drainage layer. Again, an impermeable geomembrane was beneath the lateral drainage layer. This second case is particularly important in estimating the required flow rate (permeability) of a filter layer; either geotextile or natural soil material.

C-2 Site Description

The analysis was carried out for a landfill in Philadelphia, Pennsylvania. The reason being that all four exhumed landfills described earlier were within a 320 km (200 mile) radius of Philadelphia. However, to simulate different climatic conditions the investigation was extended to include both a dry and a wet site. Phoenix, Arizona and Seattle, Washington were arbitrarily chosen to simulate the dry and wet climates, respectively. For all three cities default climatologic data for the year 1974 was used. Mean monthly temperatures and precipitation values contained in the program for 1974 are given below. Note that the HELP model is generally used in standard units, thus dual units will not be given in this section except for permeability which is in the customary units of cm/sec.

Philadelphia, Pennsylvania

Normal Mean Monthly Temperatures (F)

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
31.2	33.1	41.8	52.9	62.8	71.6	76.5	75.3	68.2	56.5	45.8	35.5

Average Monthly Precipitation Values (inches)

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
2.95	2.14	4.91	2.77	3.21	4.43	2.08	3.83	4.68	1.93	0.81	4.04

Average Annual Precipitation (inches)	37.78
Peak Daily Precipitation (inches)	1.7

Seattle, Washington

Normal Mean Monthly Temperatures (F)

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
39.1	42.8	44.2	48.7	55.0	60.2	64.8	64.1	60.0	52.5	44.8	41.0

Average Monthly Precipitation Values (inches)

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
7.76	3.98	5.83	2.37	1.34	1.24	1.51	0.01	0.2	1.97	8.17	8.33

Average Annual Precipitation (inches)	42.71
Peak Daily Precipitation (inches)	1.48

Phoenix, Arizona

Normal Mean Monthly Temperatures (F)

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
52.3	56.1	60.6	68.0	77.0	86.5	92.3	89.9	84.6	73.4	60.6	53.3

Average Monthly Precipitation Values (inches)

JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
0.57	0.02	1.37	0.01	0.00	0.00	0.84	1.15	1.07	2.12	0.44	0.59

Average Annual Precipitation (inches)	8.18
Peak Daily Precipitation (inches)	1.04

C-3 Landfill Description

The landfill under consideration was assumed to be a 5 acre cell measuring 330 ft wide by 600 ft long as shown in Figure C-1. Two different leachate collection system configurations were used; these are shown in Figures C-2(a) and C-2(b). Note that the filter in Figure C-2(b) is identified as a filter/separator since it is serving in a dual role and is properly identified as such. The leachate being transmitted within the lateral drainage layer was collected by a perforated feeder

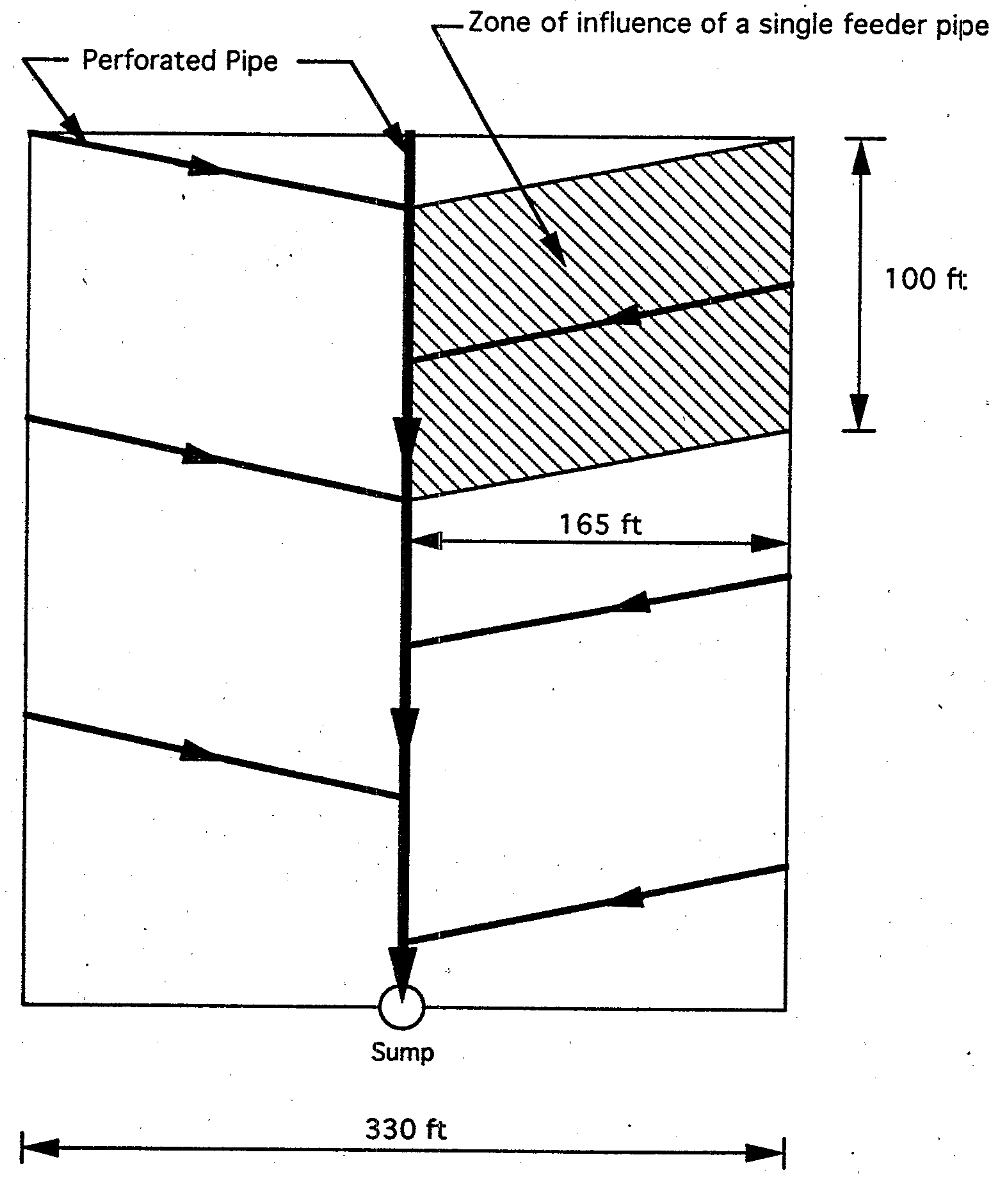
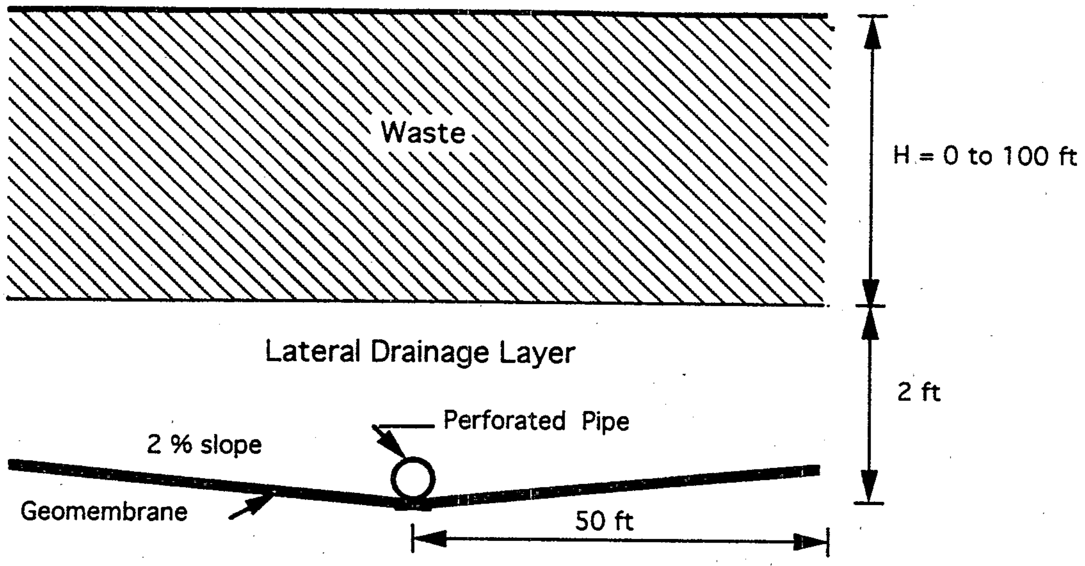
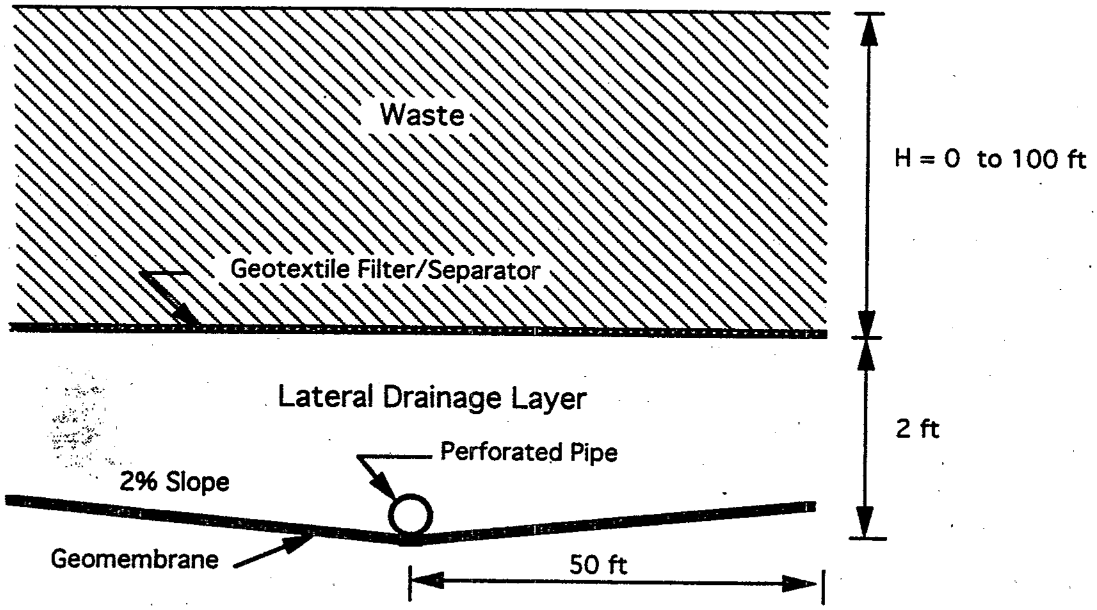


Figure C-1 - Plan View of 5 Acre Landfill Cell Showing Perforated Pipe Network for Leachate Removal



(a) LANDFILL WITH NO GEOTEXTILE FILTER / SEPARATOR



(b) LANDFILL WITH A GEOTEXTILE FILTER / SEPARATOR

Figure C-2 - Idealized Cross Section of Leachate Collection and Removal System Used in the Present Study pipe placed at the bottom of this layer, serving an area equal to 165 ft × 100 ft. The properties of the different layers shown in each configuration are given in Table C-1.

Table C-1 - Description and Soil Properties of the Different Layers

Case 1. Landfill with no geotextile filter/separator

Layer No.	Soil Type* (USCS)	Permeability (cm/sec)	Height (in.)	Layer Type**
1	Waste	0.0002	0 to 1200	1
2	Soil 1, Gravel	1.0	24	2
3	Soil 21, Barrier	1×10^{-10}	1	4

* Default soil characteristics given by the HELP model

** Legend :- Layer type 1 - Vertical percolation layer
Layer type 2 - Lateral drainage layer
Layer type 3 - Barrier soil layer
Layer type 4 - Barrier soil layer with an impermeable liner

Case 2. Landfill with a geotextile filter/separator

Layer No.	Soil Type*(USCS)	Permeability (cm/sec)	Height (in.)	Layer Type**
1	Waste	0.0002	0 to 1200	1
2	Soil 1, Geotextile	0.1 to 1×10^{-7}	0.5	1
3	Soil 1, Gravel	1.0	24	2
4	Soil 21, Barrier	1×10^{-10}	1	4

In the analysis it was assumed that the landfill had not reached its permitted height and was not finally capped. If it were, it would have been necessary to define the fraction of infiltration that passed through leaks in the geomembrane in the cover of the landfill as well as drainage off of the surface of the geomembrane. In the present analysis these two factors were not necessary.

C-4 Landfill With No Geotextile Filter/Separator

The analysis was divided into two parts; a sensitivity analysis and a parametric study.

C-4.1 Sensitivity Analysis

The sensitivity of the HELP model to variations in the values of selected input parameters is illustrated in this subsection. This will aid one to assess the influence of each of these parameters in controlling the amount of leachate collected in the leachate collection system.

As this study was mainly a sensitivity analysis, it is carried out only for the Philadelphia, Pennsylvania site. The parameters selected for the sensitivity analysis are given in Table C-2.

Table C-2 - Parameters Selected for the Sensitivity Analysis

Parameter	Range		
Thickness of the lateral drainage layer (in.)	6	to	36
Slope at the base of the lateral drainage layer (%)	0.5	to	5
Evaporation depth (in.)	6	to	18
Runoff curve number	20	to	100

As previously mentioned, this investigation was concerned with the evaluation of the effect of different parameters on the amount of leachate gathered at the base of the landfill. Accordingly, the value of the peak daily discharge from the lateral drainage layer (Q in units of ft^3/day) was chosen for the current comparison. Most of the sensitivity analyses were carried out for the no waste condition (where the amount of liquid is a maximum) and for a 10-foot deep waste condition (the typical thickness of the first lift of waste). The comparison carried out between these two conditions clarified the role of the waste material in controlling the liquid balance as the presence of waste resulted in greater retention, higher evaporation values, and lesser amounts of leachate arriving at the underlying leachate collection system.

(a) Effect of the thickness of the lateral drainage layer

Figure C-3 illustrates the effect of the thickness of the lateral drainage layer, assuming a permeability of 1.0 cm/sec, on the value of the peak daily discharge, Q. It can be seen that the value of Q, for the no waste condition, was not affected by the thickness of the lateral drainage layer for values ranging between 12 and 36 inches. However, for very small layer thicknesses the value of Q was reduced. For example, for a 6 inch thick layer, Q was reduced by 12%. On the other hand, when waste was added, the value of Q seemed to be unaffected by the variation in the thickness of the lateral drainage layer.

(b) Effect of the slope at the base of the lateral drainage layer

To study the effect of the slope at the base of the drainage layer on the values of Q, slope values ranging between 0.5% and 5.0 % were considered for the no waste and 10 ft waste conditions. The results plotted in Figure C-4 indicate that in this range of slope values the amount of peak daily discharge was not affected except for the no waste condition at a very small slope value of 0.5 %, where Q was reduced by 13.5 %.

(c) Effect of the evaporation depth

In Figure C-5 the effect of the evaporation depth on the value of Q is presented. It is seen that for the no waste condition, Q had a constant value for evaporation depths ranging between 9 and 15 inches. It should be mentioned here that the default value of the evaporation depth suggested by the HELP model, for a landfill with a bare ground surface constructed in Philadelphia; Pennsylvania, is 9 inches. For evaporation depths less than 9 inches Q was slightly reduced. For example, at the evaporation depth of 6 inches Q was reduced by 10 %. On the other hand, as the evaporation depth was increased from 15 to 18 inches, the value of Q increased by

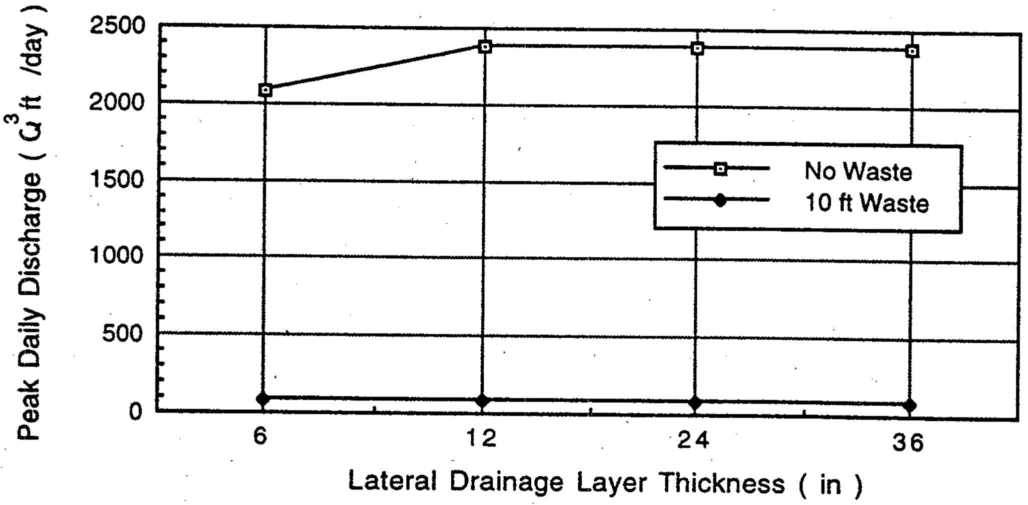


Figure C-3 - Effect of Thickness of Lateral Drainage Layer on Peak Daily Discharge (No Geotextile)

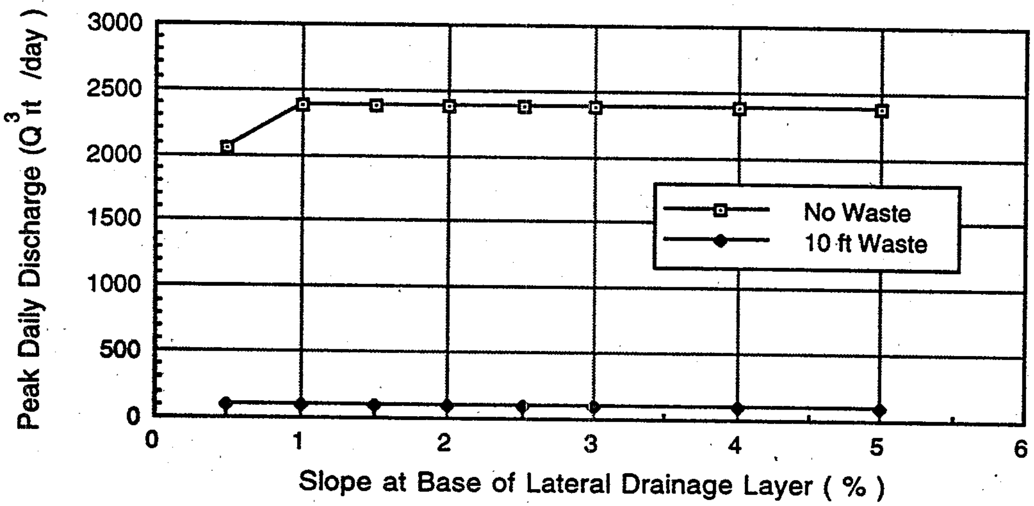


Figure C-4 - Effect of the Slope at the Base of the Lateral Drainage Layer on Peak Daily Discharge (No Geotextile)

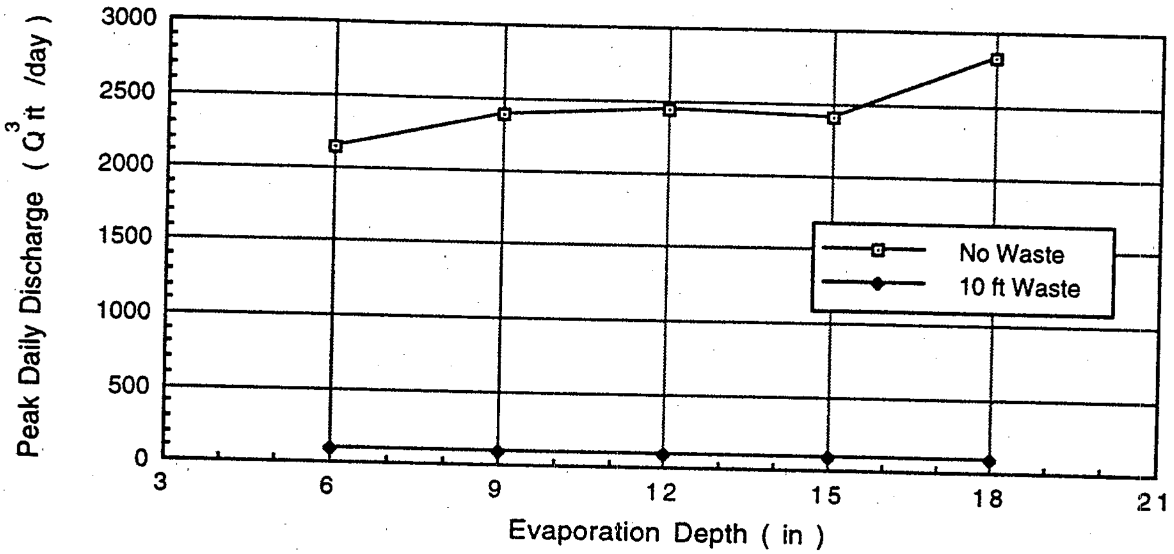


Figure C-5 - Effect of Evaporation Depth on Peak Daily Discharge
(No Geotextile)

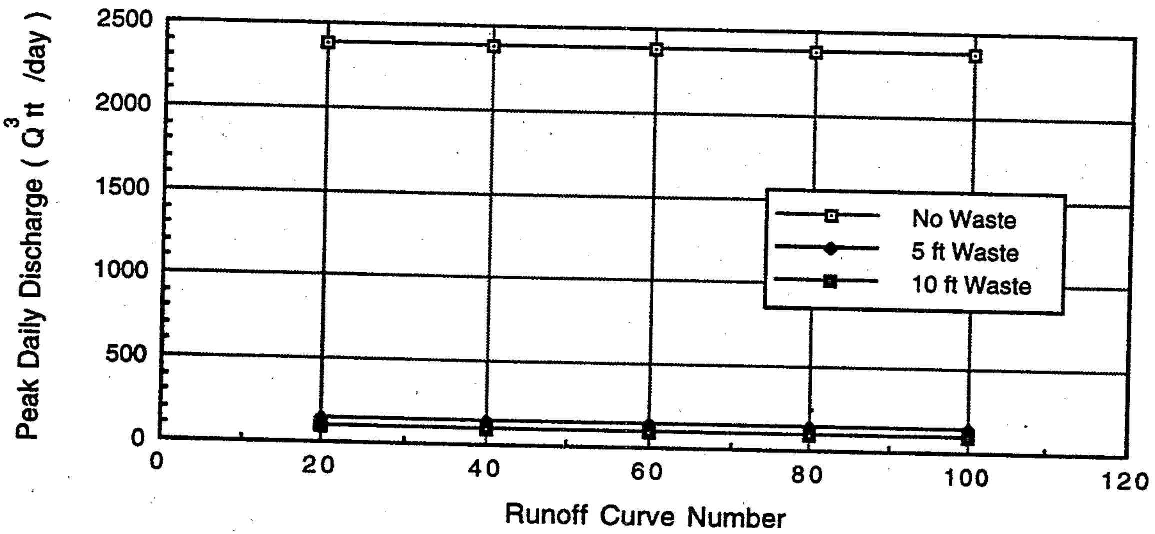


Figure C-6 - Effect of Runoff Curve Number on Peak Daily Discharge
(No Geotextile)

18%. When waste was added, the value of Q did not vary with the variation in the evaporation depth for values between 6 and 18 inches.

(d) Effect of the runoff curve number

The runoff curve number is a number ranging between 10 and 100 and is used to partition incoming rainfall or snowmelt between runoff and infiltration. It is required to specify the runoff curve number at one stage in the program, however, if the landfill is open, the control is passed to another subroutine where it is required to specify the fraction of the total potential runoff that actually drains from the surface of the top layer. Accordingly, as this fraction was always given a zero value in the present study, it was expected that the runoff curve number would have no effect on the value of Q. This is clear from Figure C-6 where it is shown that Q is independent of the runoff curve number and has a constant value depending on the waste height.

C-4.2 Parametric Study

This section reports an investigation carried out to clarify the importance of the parameters given in Table C-3 in controlling the amount of leachate collected in the bottom of an open landfill.

Table C-3 - Parameters Selected for the "No Filter" Parametric Study

Parameter	Symbol	Range
Permeability of the lateral drainage layer (cm/sec)	K	1.0 to 1×10^{-6}
Height of the waste material (ft)	H	0.0 to 100
Permeability of the waste material (cm/sec)	–	0.02 to 0.00002

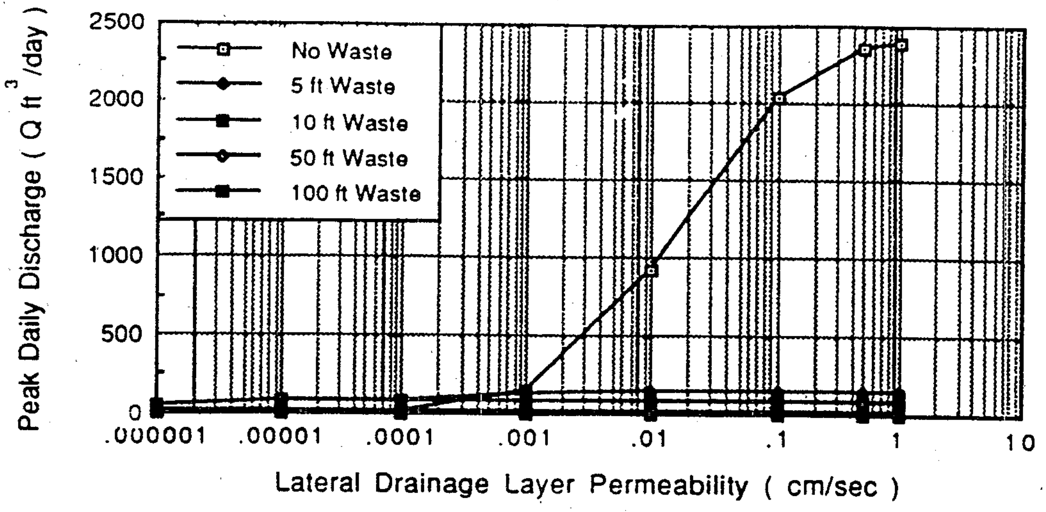
(a) Effect of the permeability of the lateral drainage layer

Figure C-7 illustrates the relationship between the lateral drainage layer permeability, K, and the peak daily discharge, Q, for different waste heights. For the no waste condition it can be seen that Q decreased greatly with the decrease in K for values ranging between 1.0 and 0.001 cm/sec. At lower permeabilities the rate of the decrease in Q becomes much smaller. The relationship between K and Q on a log-log scale could be divided into two straight lines, the slope of the first line, representing K values greater than 0.02 cm/sec, was much smaller than the slope of the second line which simulated K values less than 0.02 cm/sec.

When waste was placed above the drainage layer the value of Q was tremendously reduced even for small waste heights. Clearly the leachate collection system for an unfilled solid waste cell acts as a dewatering system until waste is placed. For a given waste height, the value of Q was slightly affected by the variations in the permeability values. The reduction in Q occurs at K values less than 1×10^{-4} cm/sec.

(b) Effect of the height of the waste material

The decrease in the value of the peak daily discharge, Q, due to the increase in the waste height, H, is plotted in Figure C-8 for landfills with lateral drainage layers of different



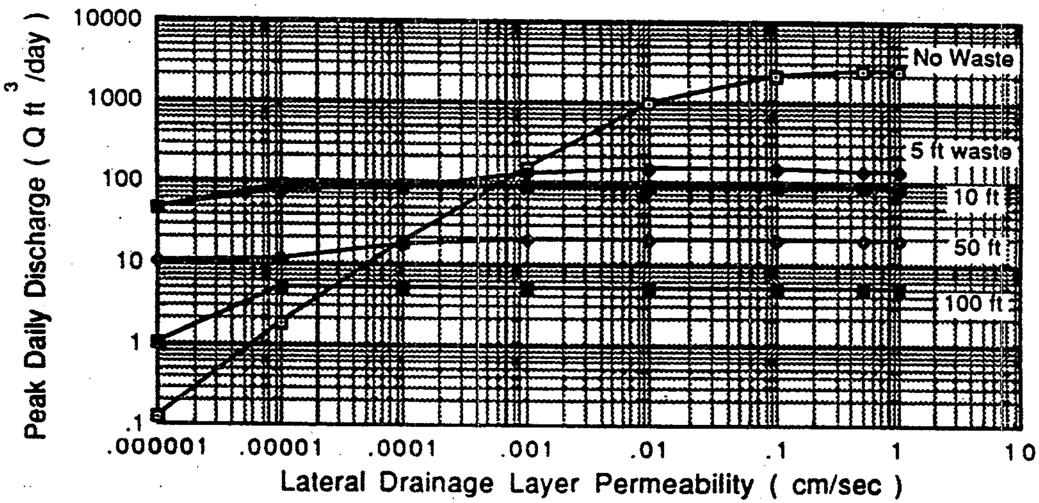
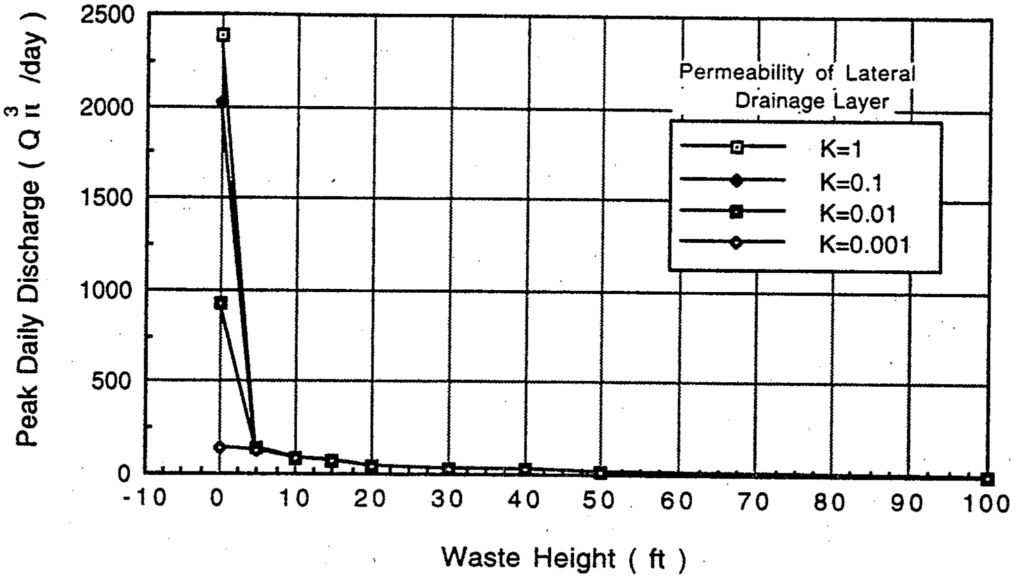


Figure C-7 - Effect of Lateral Drainage Layer Permeability on Peak Daily Discharge (No Geotextile)



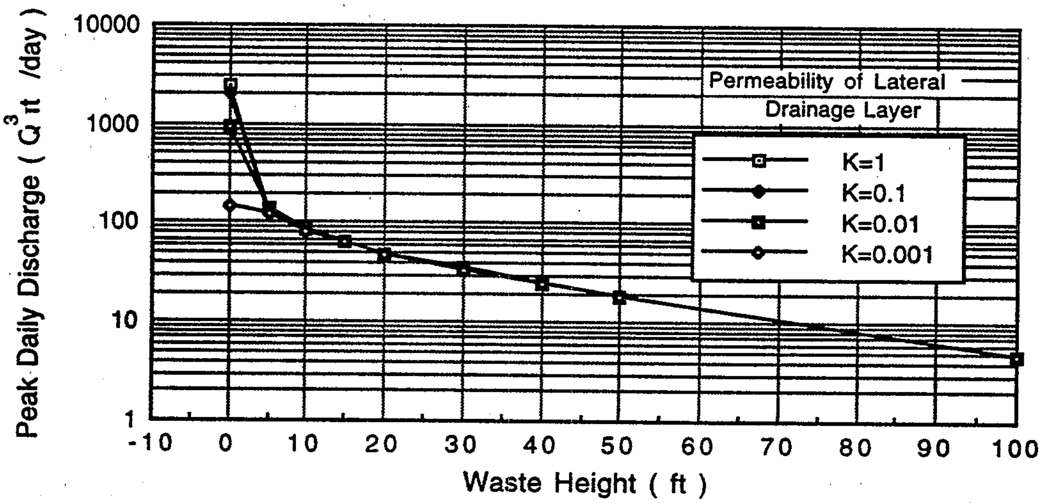


Figure C-8 - Effect of Waste Height on Daily Discharge for Different Lateral Drainage Layer Permeabilities (No Geotextile) permeabilities. The results indicated that the value of Q depended greatly on the height of the waste, H. The permeability of the lateral drainage layer, K, was less significant. As soon as waste was placed in the landfill, curves corresponding to different values of K coincided forming a single curve. As shown in Figure C-8, for H values greater than zero, the relationship between H and Q plotted on a semi-log scale can be represented by two linear portions. The first portion corresponds to H values between 5 and 20 feet, while the second portion represents H values between 20 and 100 feet and has a smaller inclination than the first portion. This indicated that the rate of the decrease in Q was much greater at the early stages of waste disposal. However, as waste accumulated in the landfill, the rate of the reduction in Q became much smaller.

(c) Effect of the permeability of the waste material

The effect of the permeability of the waste material on the amount of the peak daily discharge, Q, is illustrated in Figure C-9 for different waste heights. As shown in Table C-3 the value suggested by the default data for the waste permeability is 0.0002 cm/sec. The results indicated that at small waste heights, e.g., 5 and 10 ft, the values of Q corresponding to waste permeability values ranging between 0.002 and 0.00002 cm/sec were not significantly altered. For the 5 feet waste condition the variation in Q value was about 16.4%, whereas it was only 13% for the 10 feet of waste condition. However, if the value of the permeability of the waste material was increased to 0.02 cm/sec, the corresponding values of Q for 5 ft and 10 ft waste conditions were 71% and 51% higher than those calculated using the default values. In the case of waste heights of 50 to 100 ft, the amounts of leachate corresponding to the default permeability value were very small. In this case, even though the values of Q increased at a relatively high rate with the increase in the waste permeability, the actual values still remained comparatively low.

C-5 Landfill with a Geotextile Filter/Separator

A parametric study was performed to assess the variations in the computed leachate values due to the placement of a geotextile filter/separator layer beneath the solid waste material and above the lateral drainage layer as illustrated in Figure C-2(b). The analysis is the same for a natural soil (sand) filter since the program does not differentiate between geotextile and natural soil filters. The different parameters selected for the present investigation are listed in Table C-4.

Table C-4 - Parameters Selected for the Parametric Study with Filter

Parameter	Symbol	Range
Permeability of the geotextile layer (cm/sec)	k	0.1 to 1 × 10^{-7}
Permeability of the lateral drainage layer (cm/sec)	K	1.0 to 1 × 10^{-3}
Height of the waste material (ft)	H	0 to 100
Permeability of the waste material (cm/sec)	–	0.02 to 0.0002

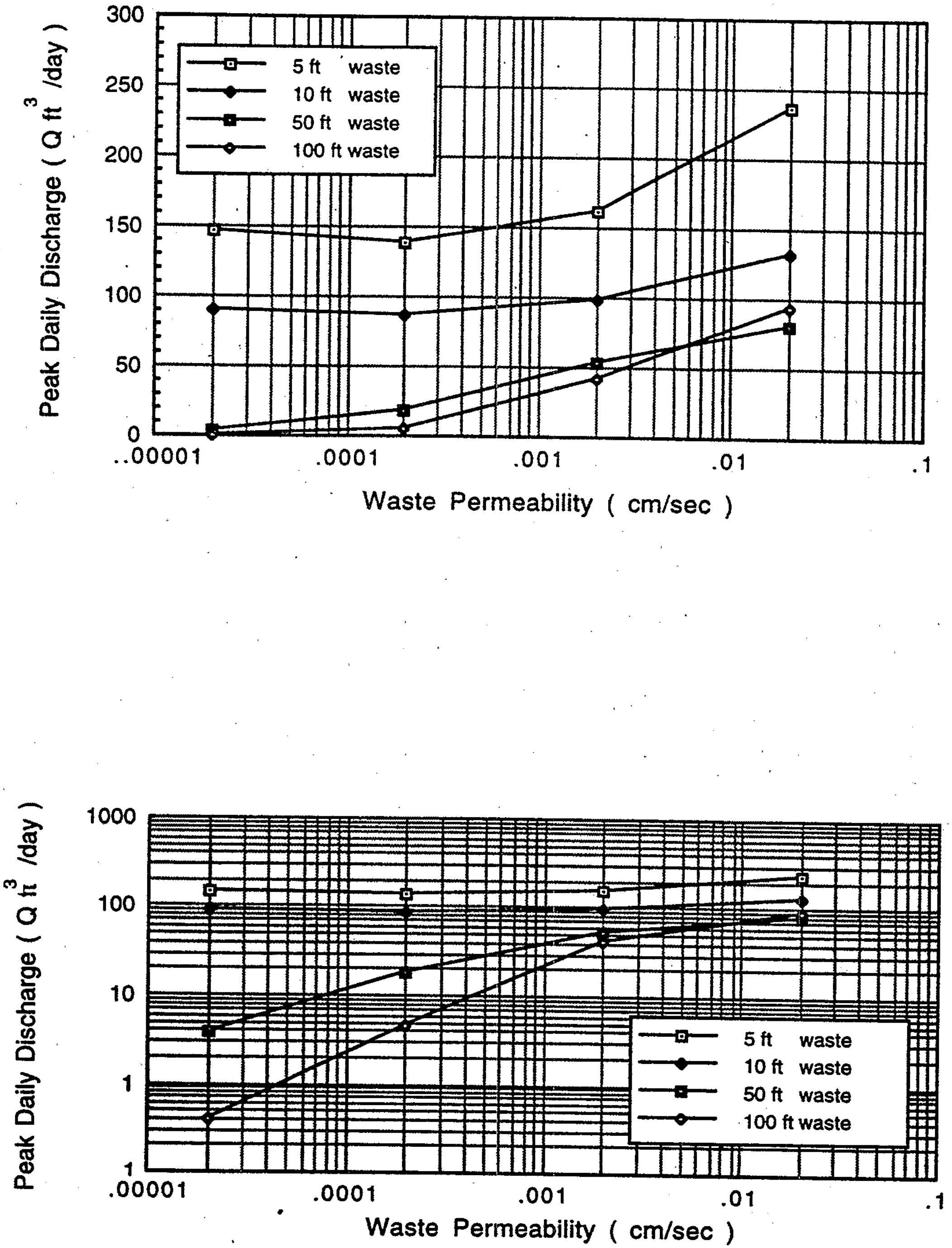


Figure C-9 - Effect of Waste Permeability on Peak Daily Discharge
(K Lateral Drainage Layer = 1 cm/sec, No Geotextile)

The analysis was divided into two parts. The first part highlighted the effect of the different parameters given in Table C-4 on the amount of leachate collected at a landfill in Philadelphia, Pennsylvania. The second part of the investigation aimed at pointing out the variations in the amounts of leachate that accumulated in different climates. Accordingly, the investigation was extended to include both a wet and a dry site. Arbitrarily selected were Seattle, Washington and Phoenix, Arizona, for wet and dry locations respectively. In this second part of the investigation only the major parameters affecting the amount of leachate were taken into consideration.

C-5.1 Parametric Study for a Landfill in Philadelphia; Pennsylvania

The effect of the different parameters listed in Table C-4 are demonstrated in the following subsections.

(a) Effect of the permeability of the geotextile layer

The relationships between the geotextile permeability, k, and peak daily discharge, Q, are given in Figures C-10 through C-13 for different lateral drainage layer permeabilities and different waste heights. In the analysis it was assumed that the geotextile layer was a vertical percolation layer. For each of the lateral drainage layer permeabilities it was noticed that for the no waste condition, Q had a constant value if the geotextile permeability, k, exceeded 1×10^{-4} cm/sec. When the geotextile permeability was less than 1×10^{-4}, the value of Q was greatly reduced with any reduction in the value of k.

As waste was added, the value of Q decreased as the height of the waste increased. At any given height of waste the value of Q was not affected by the decrease in the geotextile permeability down to a value of 1×10^{-5} cm/sec. If the geotextile permeability was reduced lower than this value, for example due to progressive clogging, the amount of discharge decreased with the decrease in the geotextile permeability. As the geotextile permeability decreased to a value of 1×10^{-7}, the geotextile became excessively clogged and the amount of leachate passing through it became extremely small, i.e., it then became a de-facto liner.

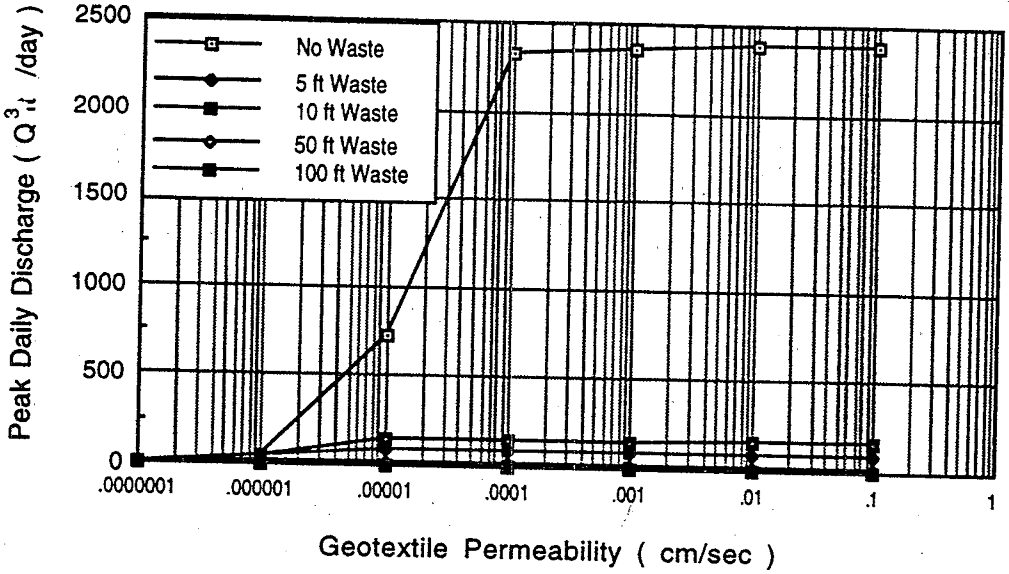
It is worth noting that the vertical flow submodel of the HELP program is based on Darcy's law which is given by the following equation:

$$Q_v = k\,(dh/dl) \qquad \text{(C-1)}$$

where

Q_v = rate of vertical flow, in./day.
k = permeability of layer, in./day.
h = gravitational head, in.
l = length in the direction of flow, in.

Free outfall was assumed from each layer such that dh/dl was set equal to unity (Schroeder et al., [13]). This assumption was acceptable if the permeability of the profile was constant or increased with depth. Because this assumption was not fulfilled at the interface between the waste material (with a permeability of 0.0002 cm/sec) and the geotextile filter (with a permeability which can be less than 0.0002 cm/sec) a different procedure was employed at the top interface of the



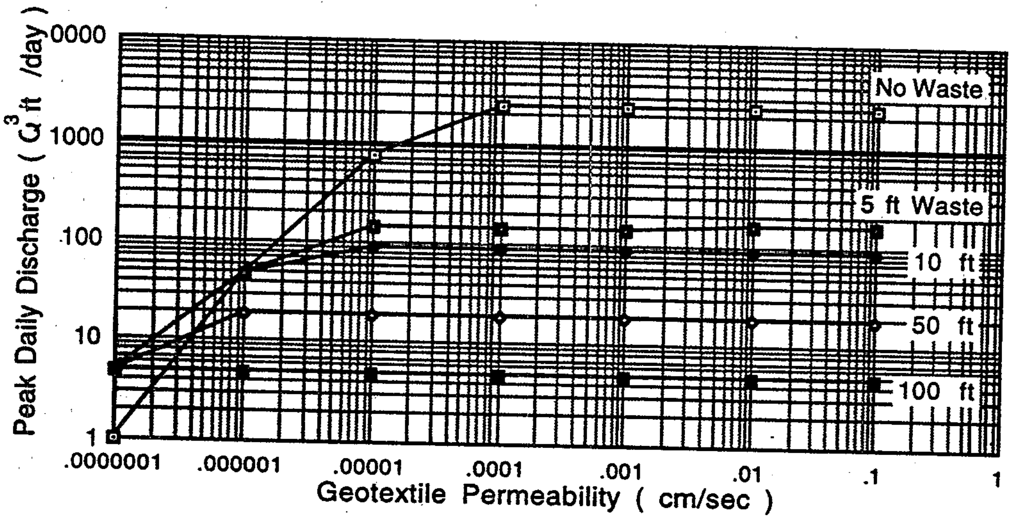


Figure C-10 - Effect of Geotextile Permeability on Peak Daily Discharge (K for Lateral Drainage Layer = 1 cm/sec)

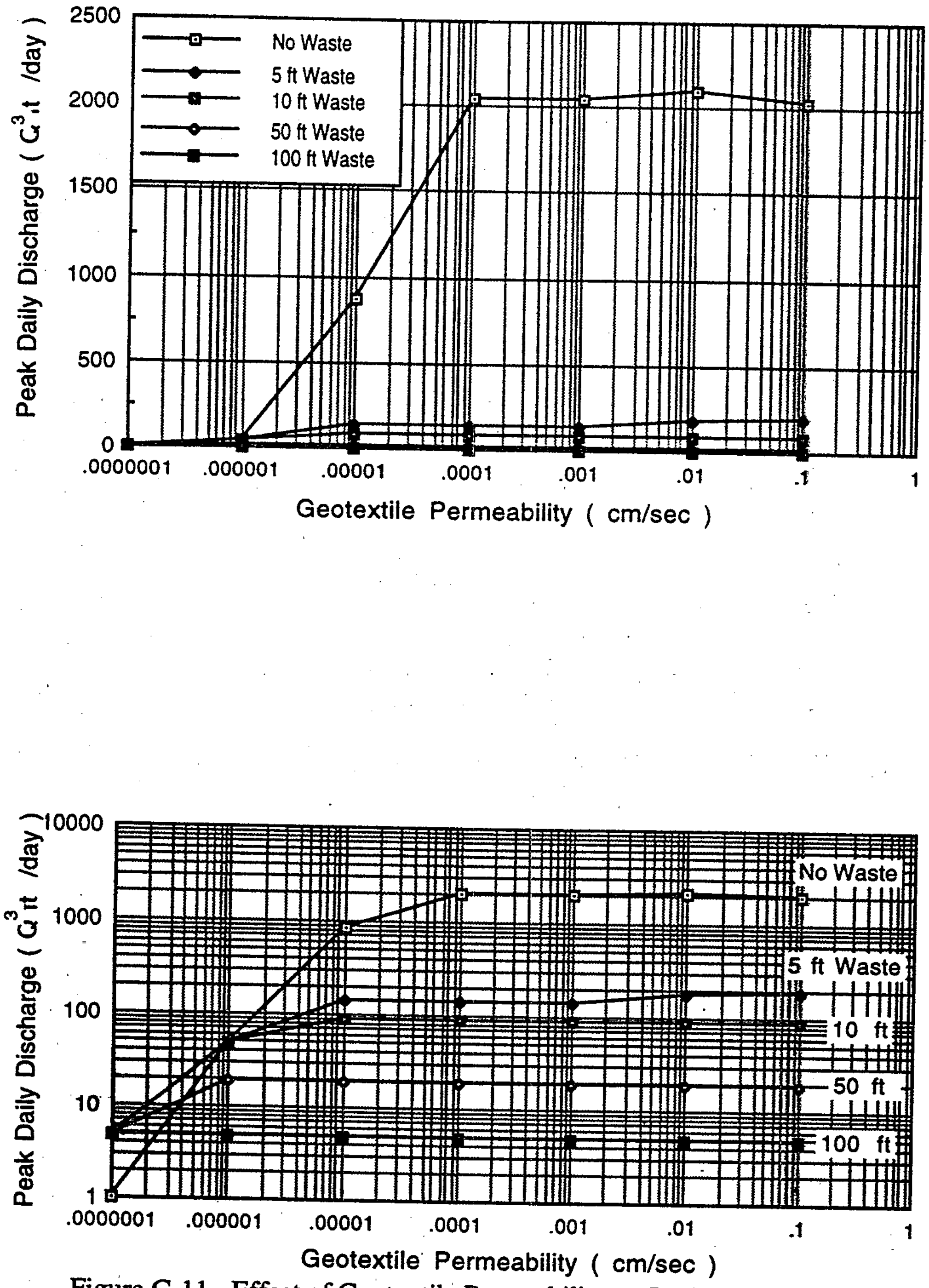


Figure C-11 - Effect of Geotextile Permeability on Peak Daily Discharge
(K for Lateral Drainage Layer = 0.1 cm/sec)

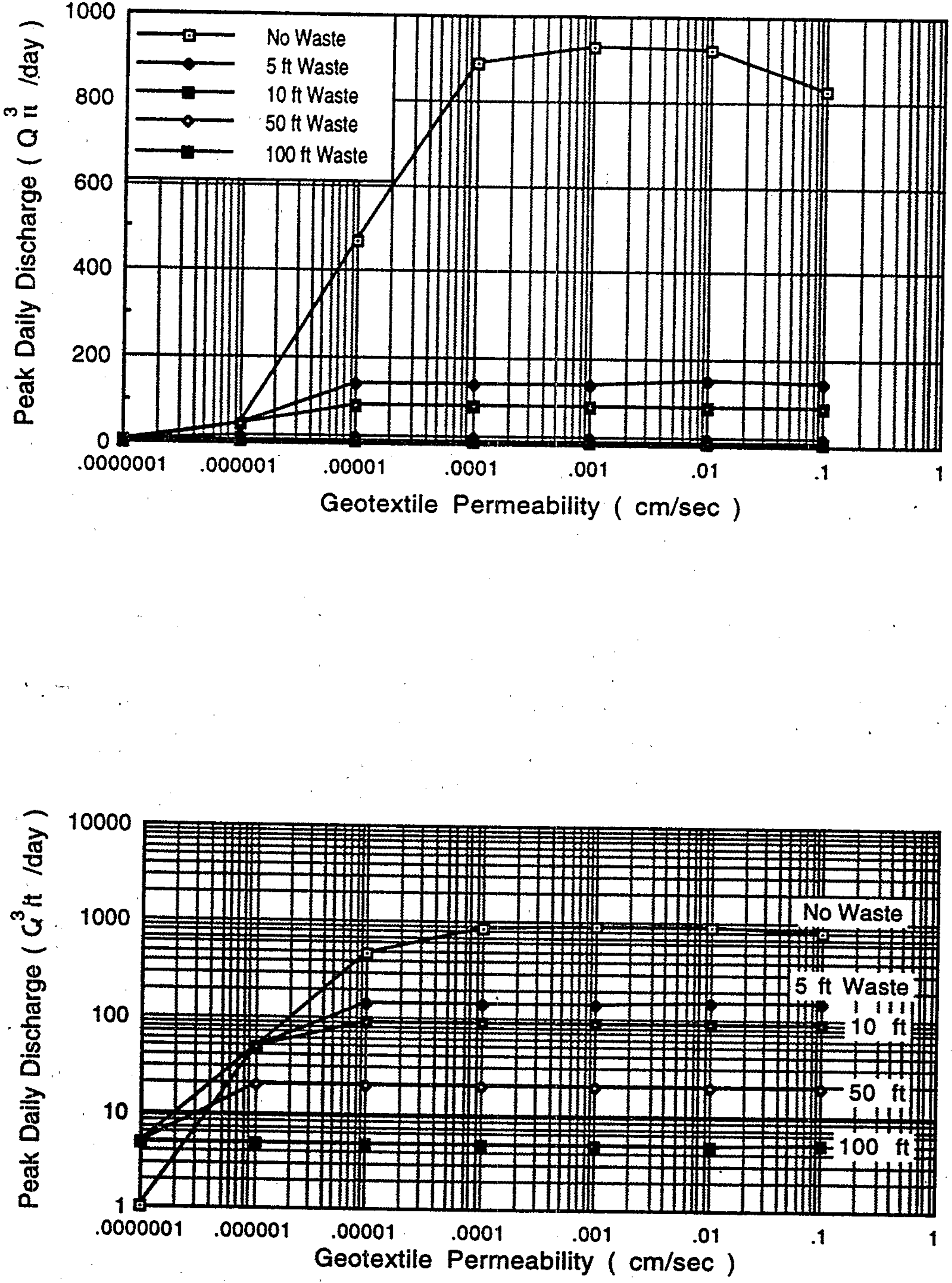
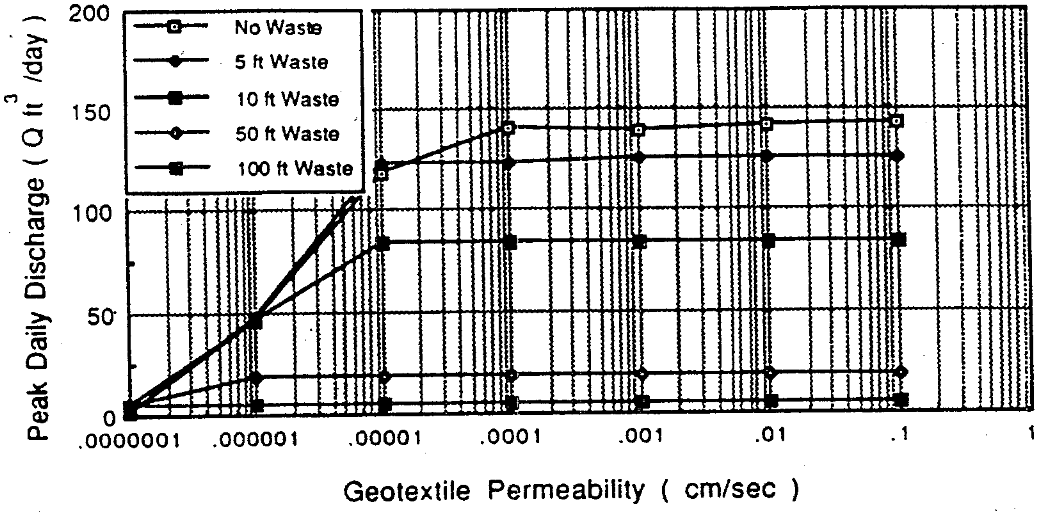


Figure C-12 - Effect of Geotextile Permeability Peak on Daily Discharge
(K for Lateral Drainage Layer = 0.01 cm/sec)



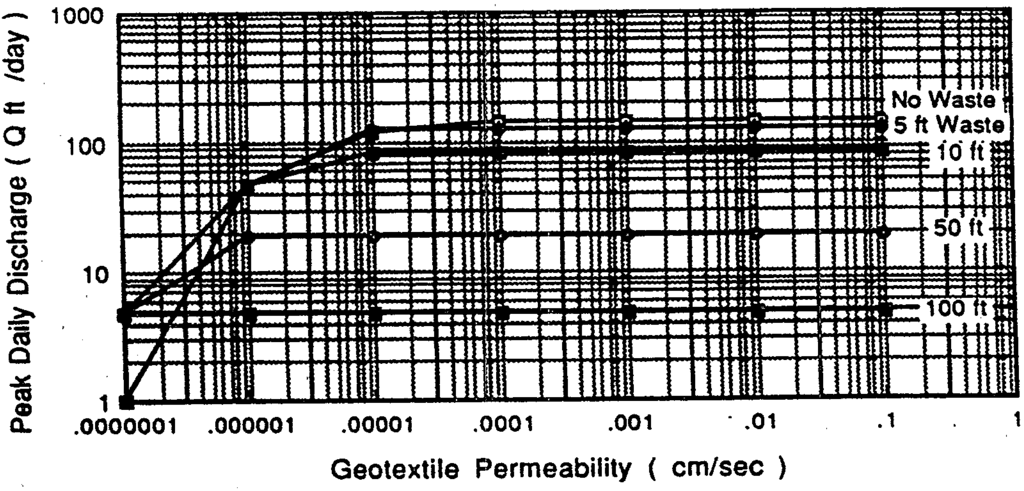


Figure C-13 - Effect of Geotextile Permeability on Daily Discharge
(K for Lateral Drainage Layer = 0.001 cm/sec)

geotextile. In this case, it was felt to be more realistic to assume that a geotextile layer of significantly reduced permeability was acting as a barrier layer. Accordingly, the total head above it was calculated and the amount of liquid percolating through it is given by the equation :

$$Q_p = k_p \ [(TH+Tc) /Tc] \qquad (C\text{-}2)$$

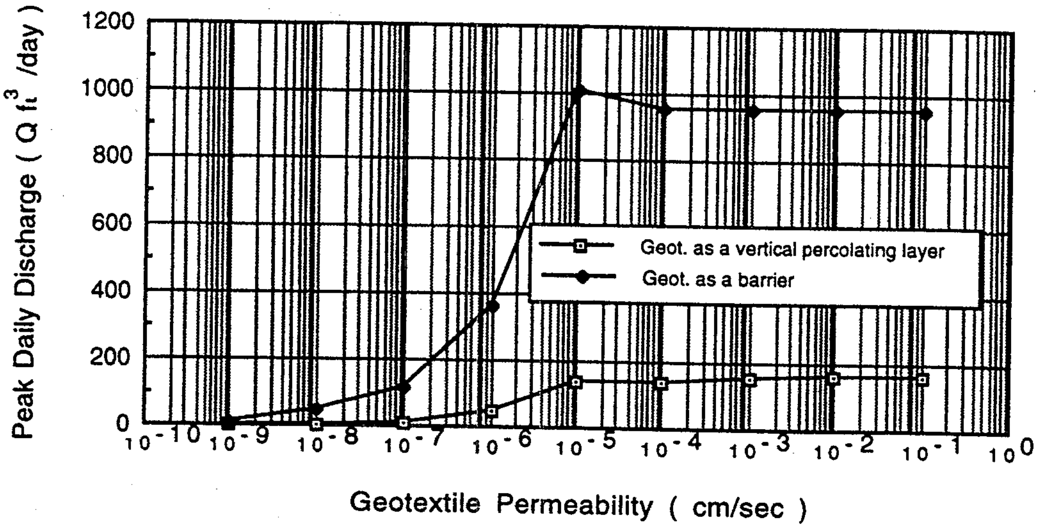
where

Q_p = rate of percolation through the barrier layer.
k_p = permeability of the barrier layer.
TH = total head in the profile above the barrier layer.
Tc = thickness of the barrier layer.

Figure C-14 illustrates the variation in the peak daily discharge for a 5 ft waste condition assuming geotextile permeabilities ranging between 0.1 and $1x10^{-9}$ cm/sec. The permeability of the lateral drainage layer was assigned a value of 1 cm/sec. The geotextile layer was considered to be either a vertical percolation layer or a barrier layer. The results indicate that the values of the peak daily discharge assuming the geotextile to be a barrier layer, were much higher than those corresponding to the geotextile acting as a vertical percolating layer. However, for each case the amount of Q corresponding to geotextile permeabilities higher than or equal to 1×10^{-5} cm/sec were almost equal. The amount of leachate decreased with any further decrease in the geotextile permeability and understandably reached extremely small values at a permeability of 1×10^{-8} cm/sec.

It is worth pointing out that even though the amount of peak daily discharge was greatly affected by the leachate accumulation on top of the geotextile layer, the average annual total amount of leachate was only slightly altered. This is illustrated in Figure C-15 where the relationships between the geotextile permeabilities and the average annual totals are plotted assuming the geotextile to act either as a vertical drainage layer or as a barrier layer. The diagram indicates that the average annual totals are equal for both assumptions down to a geotextile permeability of 1×10^{-6} cm/sec. If the effect of the head on top of the geotextile layer was neglected, the calculated amount of leachate appears to be greatly reduced at geotextile permeabilities less than 1×10^{-6} cm/sec. However, as the effect of liquid accumulation on top of the geotextile (due to its low permeability) was taken into consideration, the average annual amount of leachate was almost the same for different geotextile permeabilities down to a value of 1×10^{-7} cm/sec. As the geotextile permeability was reduced to 1×10^{-9} cm/sec, the leachate could hardly pass through and in this case the geotextile acted as a barrier material rather than as a filter.

To highlight the effect of the accumulation of leachate on top of the geotextile, the relationships between geotextile permeability and head on geotextile, peak daily discharge, and average annual totals are plotted for different waste heights in Figures C-16, C-17 and C-18, respectively. The results show that waste heights up to 100 ft have no effect on the amount of leachate for this situation. In this case the major parameter affecting the amount of leachate is the



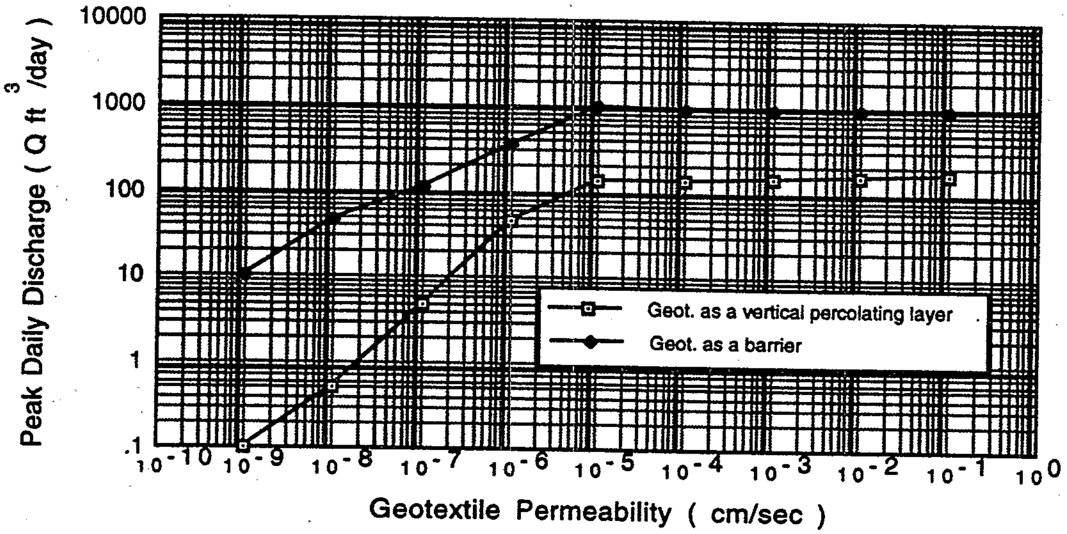


Figure C-14 - Variation in Peak Daily Discharge Assuming Geotextile to be a Vertical Percolating Layer or a Barrier Layer for 5 ft Waste

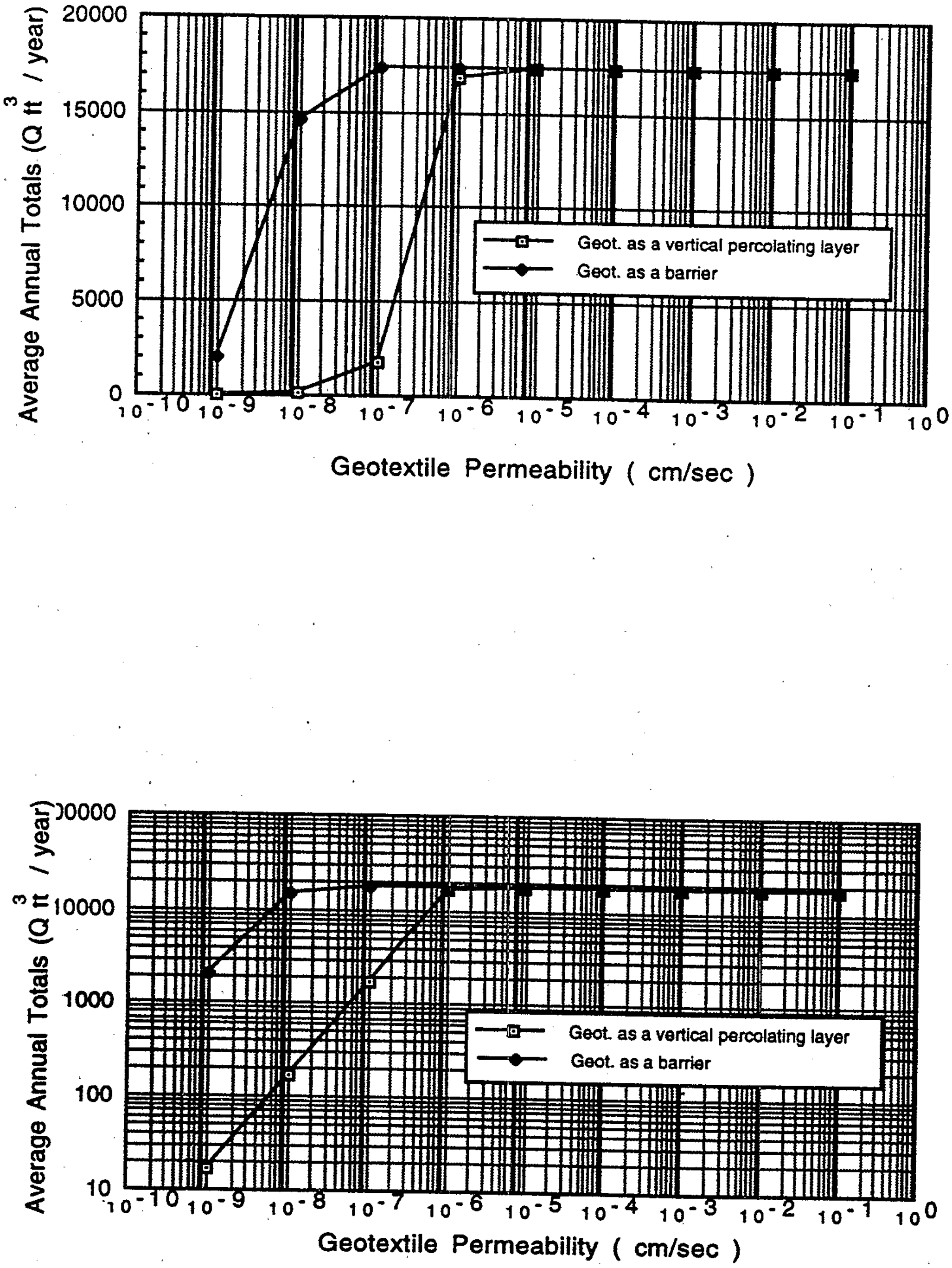


Figure C-15 - Variation in Average Annual Totals Assuming Geotextile to be a Vertical Percolating Layer or a Barrier Layer for 5 ft Waste

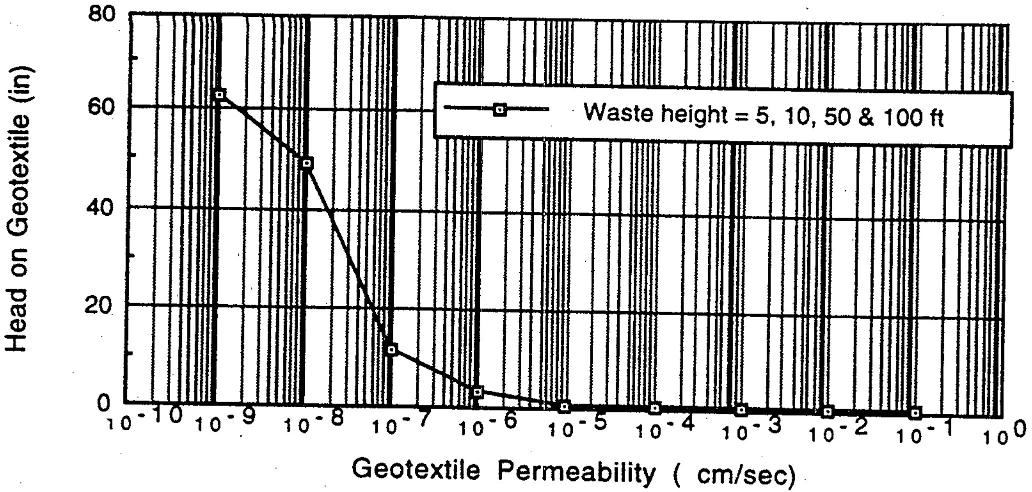


Figure C-16 - Head on Geotextile Assuming it to be a Barrier Layer for Different Waste Heights up to 100 ft

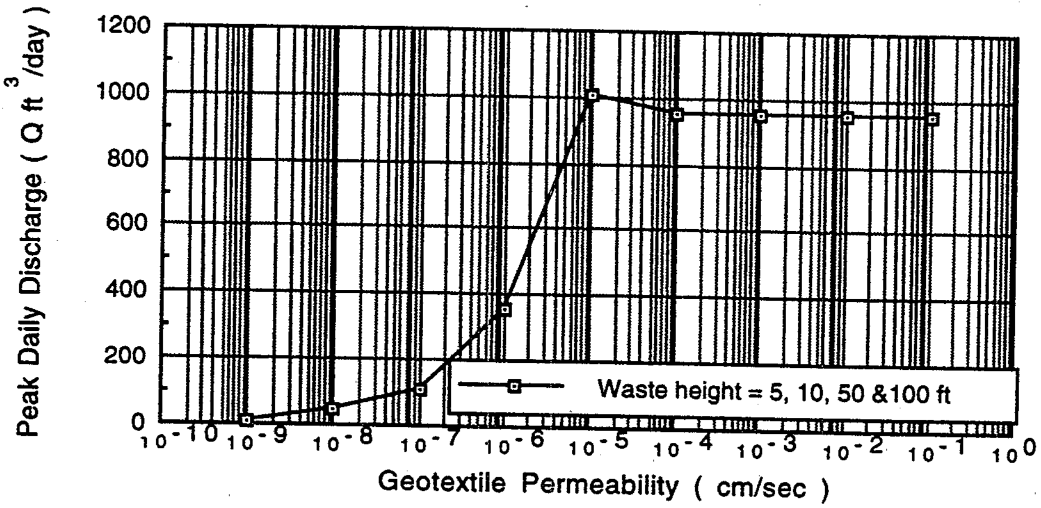


Figure C-17 - Peak Daily Discharge Assuming the Geotextile to be a Barrier Layer for Different Waste Heights up to 100 ft

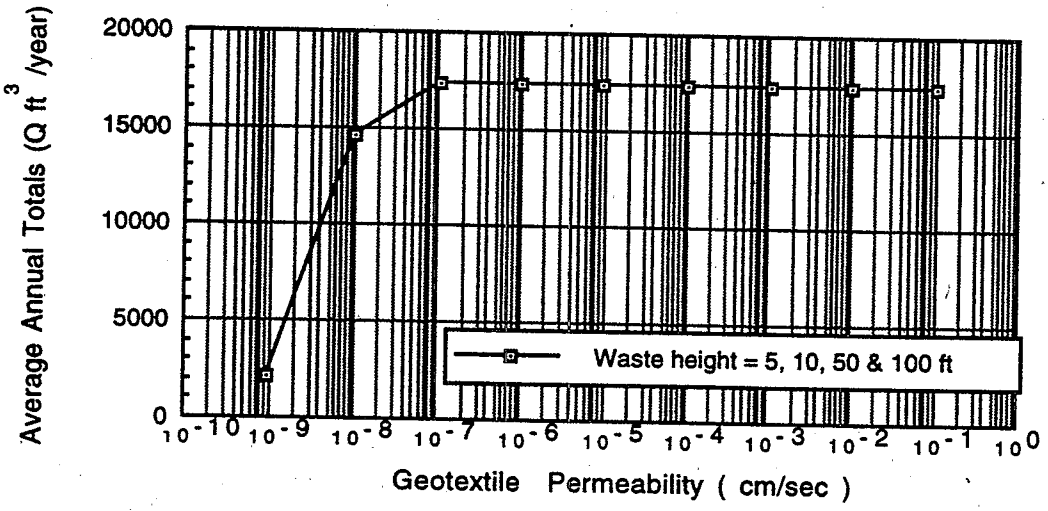


Figure C-18 - Average Annual Totals Assuming the Geotextile to be a Barrier Layer for Different Waste Heights up to 100 ft geotextile permeability. As the geotextile permeability became less than the waste permeability, which was assumed to be 0.0002 cm/sec, leachate started to accumulate and the head on top of the geotextile layer started to increase as indicated in Figure C-16. Also, as the geotextile permeability was reduced to values lower than the waste permeability, the value of the peak daily discharge started to decrease as shown in Figure C-17. However, Figure C-18 indicates that the variations in the amount of the average annual rainfall above a geotextile permeability of 1 × 10^{-8} cm/sec are small.

(b) Effect of the permeability of the lateral drainage layer

To illustrate the role of the value chosen for the lateral drainage layer permeability on the amount of leachate, the no waste and the 5 feet waste conditions were taken into consideration. The results for these two cases are plotted in Figures C-19 and C-20 respectively. Figure C-19 shows that for the no waste condition the value of the peak daily discharge, Q, was reduced as the permeability of the lateral drainage layer decreased. It is worth pointing out that these results are all based on the assumption that the geotextile was a vertical percolating layer.

When waste was added on top of the geotextile layer, even at very small thicknesses, the value of the daily discharge was no longer dependent on the value of the soil permeability as shown in Figure C-20. It became mainly dependent on the waste height and the geotextile permeability being less or greater than $1\text{x}10^{-5}$ cm/sec as previously mentioned.

(c) Effect of the height of the waste material

Figures C-21 and C-22 show the effect of the waste height, H , on the value of Q, for different geotextile permeabilities, k, assuming the geotextile to be a vertical percolating layer. To illustrate the effect of this parameter two conditions were considered, i.e., lateral drainage layer permeabilities of 0.1 and 0.01 cm/sec. In Figures C-21 and C-22 the pattern illustrating the effect of the waste height on the variations in Q was the same, however, the value of Q was dependent on the permeability of the lateral drainage layer.

For geotextile permeabilities higher than $1\text{x}10^{-5}$ cm/sec the value of Q depended mainly on the waste height . This is indicated by the three curves corresponding to geotextile permeabilities equal to 0.1 , 0.001 and 0.00001 cm/sec as they coincide forming one curve at H values greater than 5 ft . For geotextile permeability, k , equal to 1 × 10^{-6} cm/sec the value of Q was slightly less than that corresponding to geotextiles with higher permeabilities. As the waste height reached 50 ft, the value of Q coincided with that corresponding to geotextiles with higher permeabilities . When the geotextile permeability reached a value of 1 × 10^{-7} the waste height had no effect on the value of Q. This was due to the fact that in this case the geotextile acted as if it were an impermeable barrier layer permitting only small amounts of leachate to percolate through it.

(d) Effect of the permeability of the waste material

The changes in the amount of peak daily discharge due to the increase in the waste permeability are shown in Figure C-23 for the 5 ft waste condition. These results are also based on the assumption that the geotextile was a vertical percolating layer. The diagram shows that the

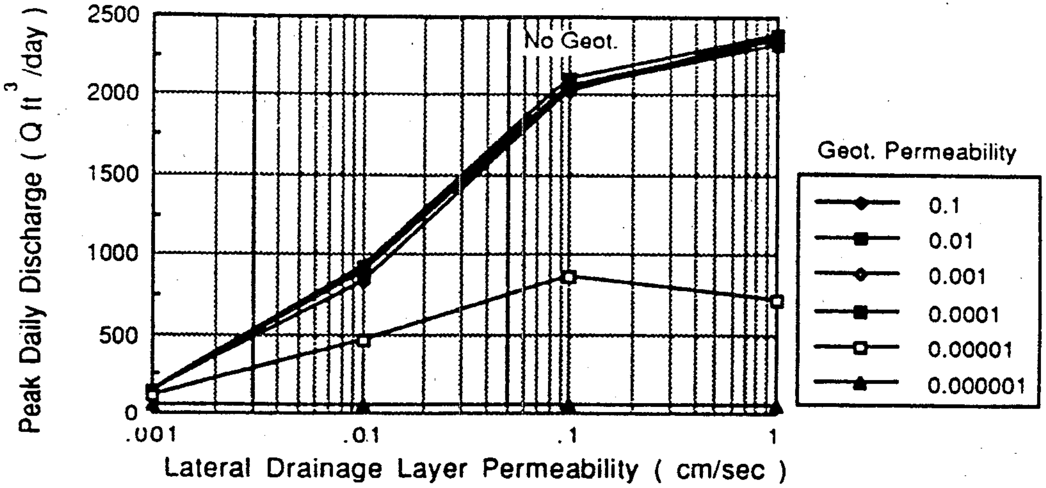


Figure C-19 - Effect of Lateral Drainage Layer and Geotextile Permeability on Peak Daily Discharge (For No Waste Condition)

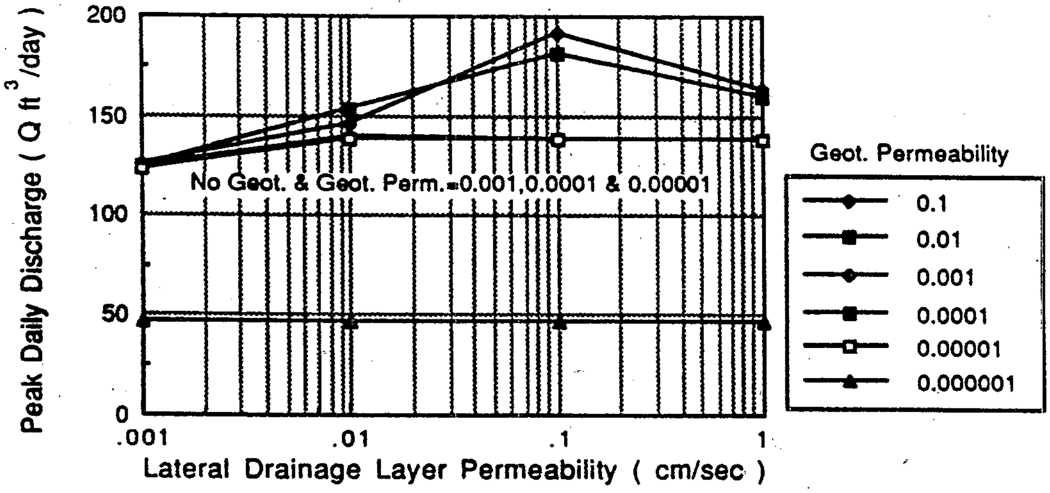


Figure C-20 - Effect of Lateral Drainage Layer and Geotextile Permeability on Peak Daily Discharge (For 5 ft Waste Condition)

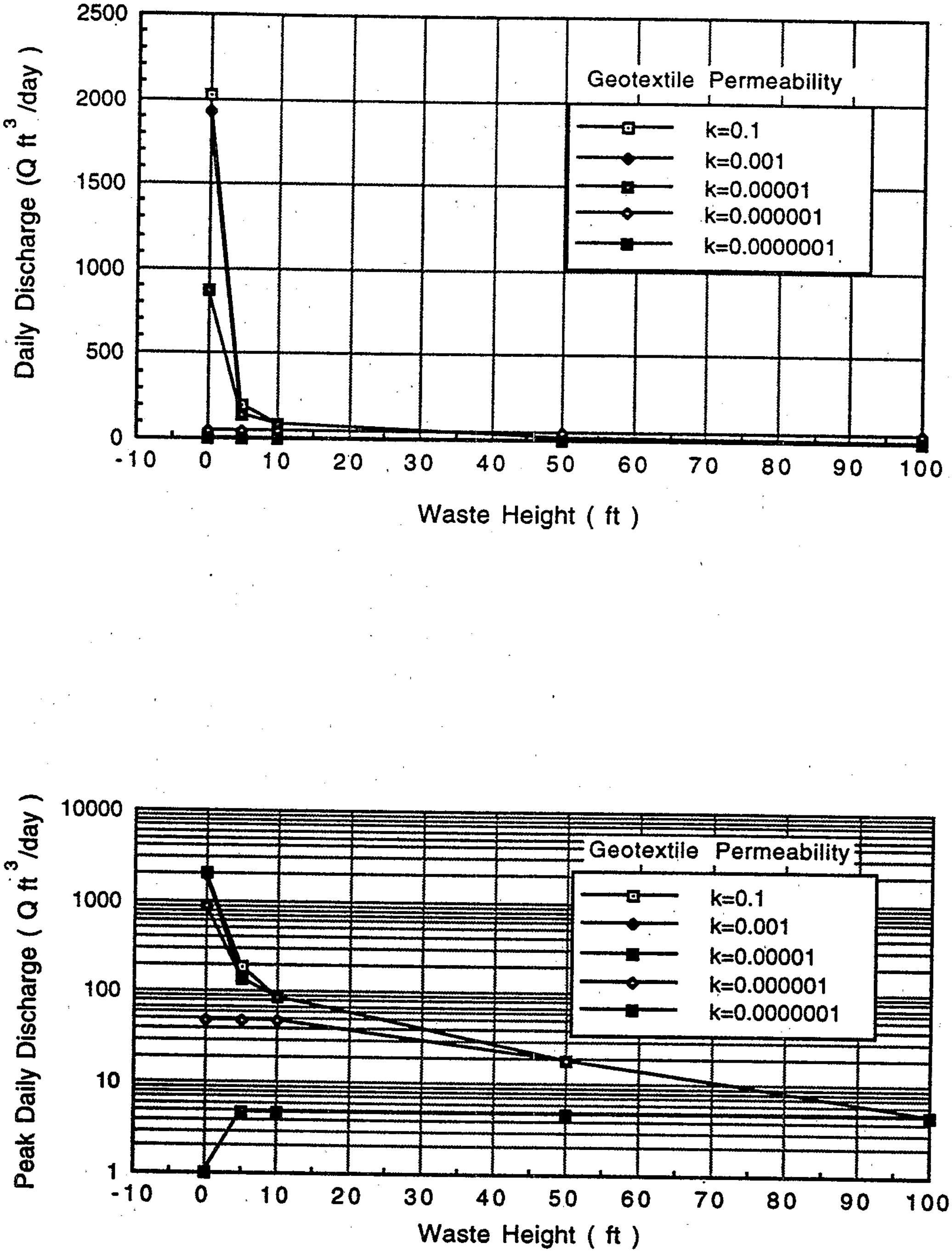
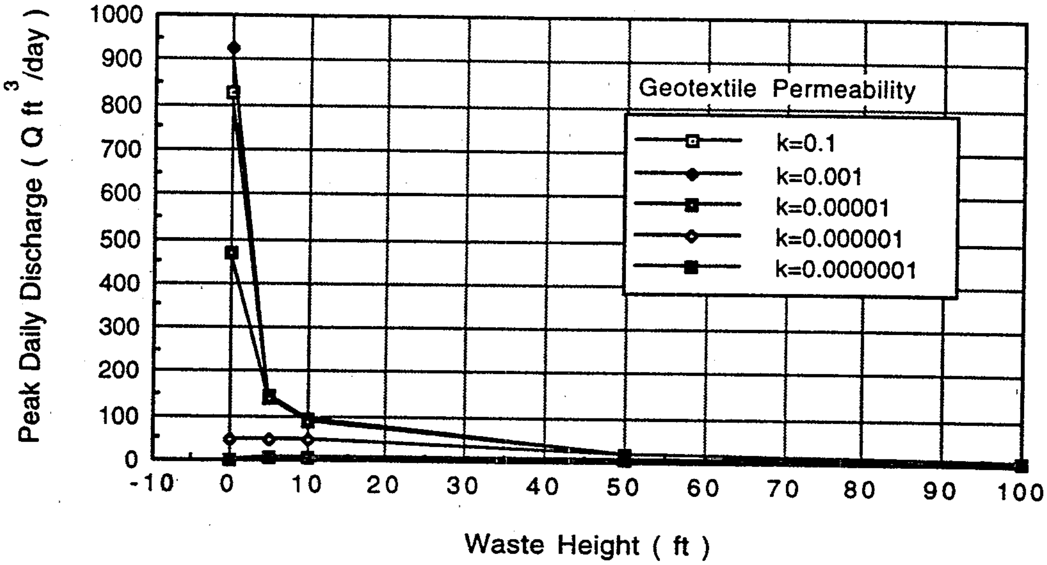


Figure C-21 - Effect of Waste Height on Daily Discharge for Different Geotextile Permeabilities (K for Lateral Drainage Layer = 0.1 cm/sec)



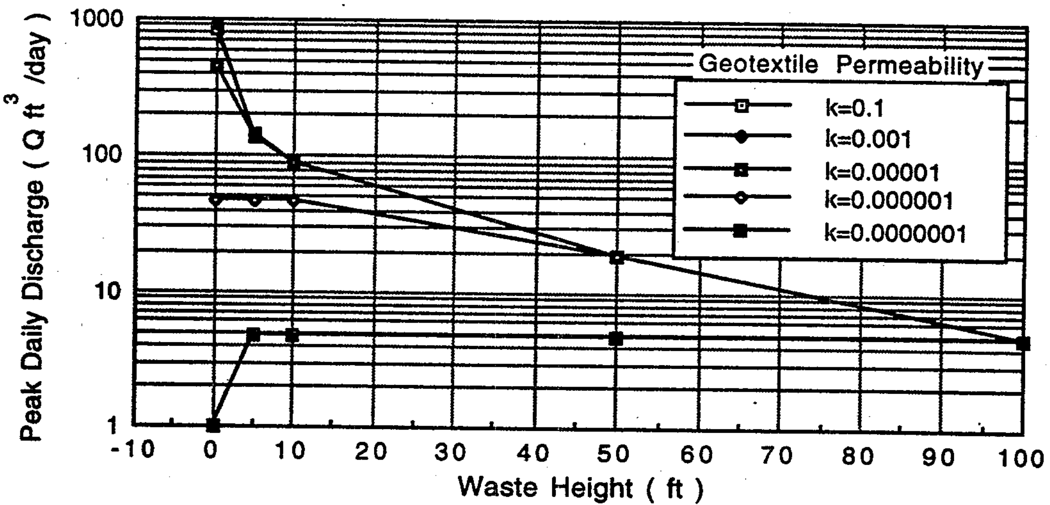


Figure C-22 - Effect of Waste Height on Daily Discharge for Different Geotextile Permeabilities (K for Lateral Drainage Layer = 0.01 cm/sec)

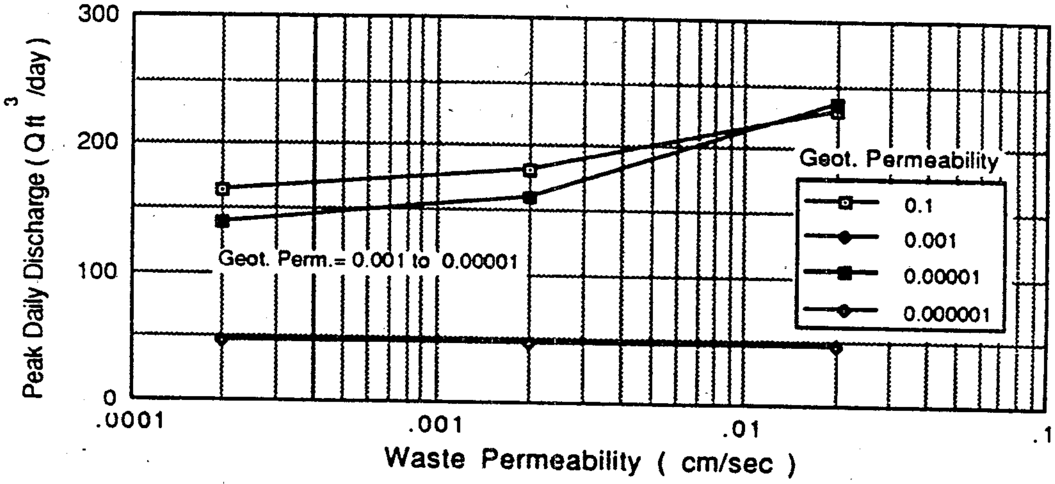


Figure C-23 - Effect of Waste and Geotextile Permeability on Peak Daily Discharge for 5 ft Waste (K for Lateral Drainage Layer = 1 cm/sec) value of Q was slightly altered due to the changes in the geotextile permeability up to a value of 1 × 10^{-5} cm/sec for the different waste permeabilities. If the waste permeability increased from 0.0002 to 0.002 cm/sec the increase in the value of Q did not exceed 15.5 %. However, if the waste permeability was increased to a value of 0.02 cm/sec the increase in the values of Q ranged from 41% to 71% depending on the geotextile permeability. For geotextile permeabilities greater than 1 × 10^{-6} cm/sec the value of Q did not vary with the variation in the waste permeability.

C-5.2 Parametric Study With Geotextiles - Different Climates

So as to highlight the effect of different climatic conditions on the amount of peak daily discharge, two cities other than Philadelphia were taken into consideration. These were Seattle, Washington and Phoenix, Arizona which represent wet and dry climates, respectively. Mean monthly temperatures and average monthly precipitation for the different cities were given in Section C-2. To illustrate the effect of climatic variations only the major parameters affecting the amount of leachate were considered. These are the geotextile permeability and the waste height as shown previously in Table C-4.

(a) Effect of the permeability of the geotextile layer

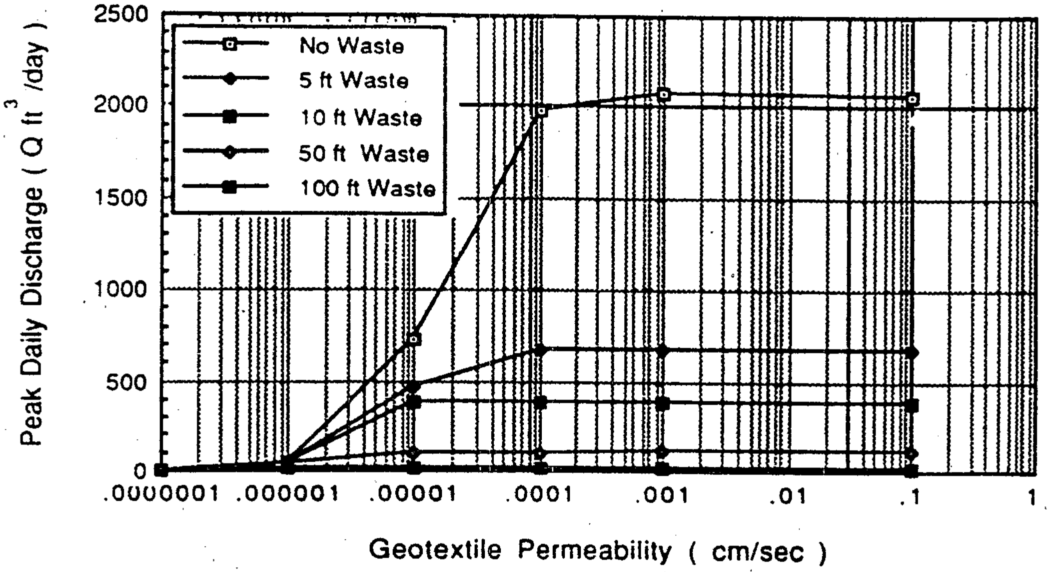
Figure C-24 illustrates the effect of the geotextile permeability, k, on the amount of peak daily discharge, Q, for Seattle, Washington. For the no waste condition it is clear that the value of Q was almost constant for k values higher than 1×10^{-4} cm/sec. At a value of 1 × 10^{-5} cm/sec, a reduction in the value of Q was noticed. As the value of k was reduced to 1 × 10^{-6} cm/sec the amount of Q decreased at a very high rate to reach very small values.

As waste accumulated in the landfill, the value of Q decreased with the increase in the waste height. At any specific waste height, the amount of Q was almost unaffected by the reduction in geotextile permeability up to a value of 1 × 10^{-5} cm/sec. However, as the geotextile permeability reached a value of 1 × 10^{-6} cm/sec, the amount of leachate was greatly reduced. At a geotextile permeability equal to 1 × 10^{-7} cm/sec, Q reached an extremely low value.

The relationship between k and Q for Phoenix, Arizona is given in Figure C-25. It is clear that Q had significant values only before waste was placed in the landfill. In this case the variations in Q values were of small magnitudes for geotextile permeabilities greater than 1 × 10^{-5} cm/sec. As the geotextile permeability decreased, the value of Q diminished.

(b) Effect of the height of the waste material

Figures C-24 and C-25 also show the effect of the waste height, H, on the values of Q. It is clear that the increase in the height of the waste was one of the major factors that resulted in a large decrease in the amount of discharge. In Seattle, the values of Q even after the placement of the waste were still measurable, however, in Phoenix the values of Q become negligible as soon as waste was placed even at small heights. For geotextile permeabilities between 0.1 and 1 × 10^{-5} cm/sec the values of the peak daily discharge in Seattle, Washington corresponding to waste heights of 5, 10, 50, and 100 ft were about 670, 390, 110 and 14 ft^3/day, respectively. On the



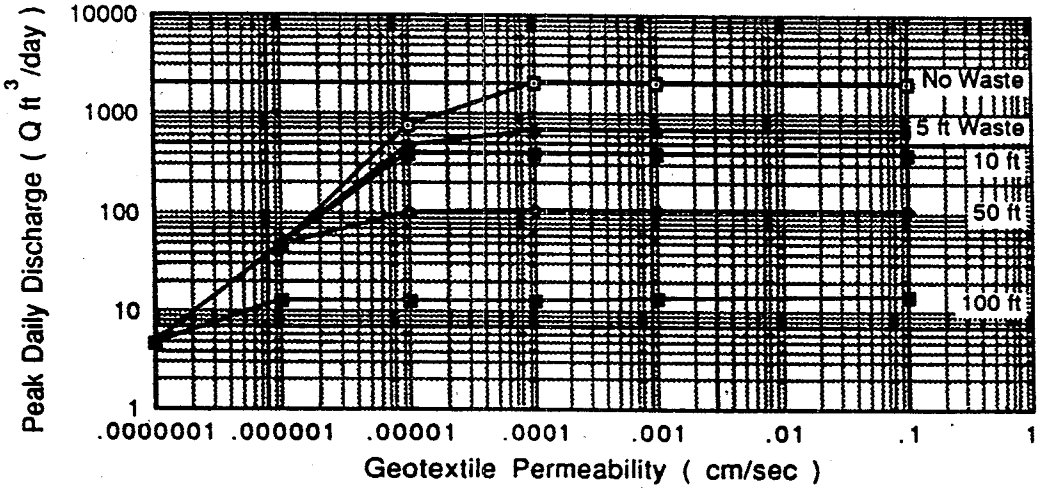
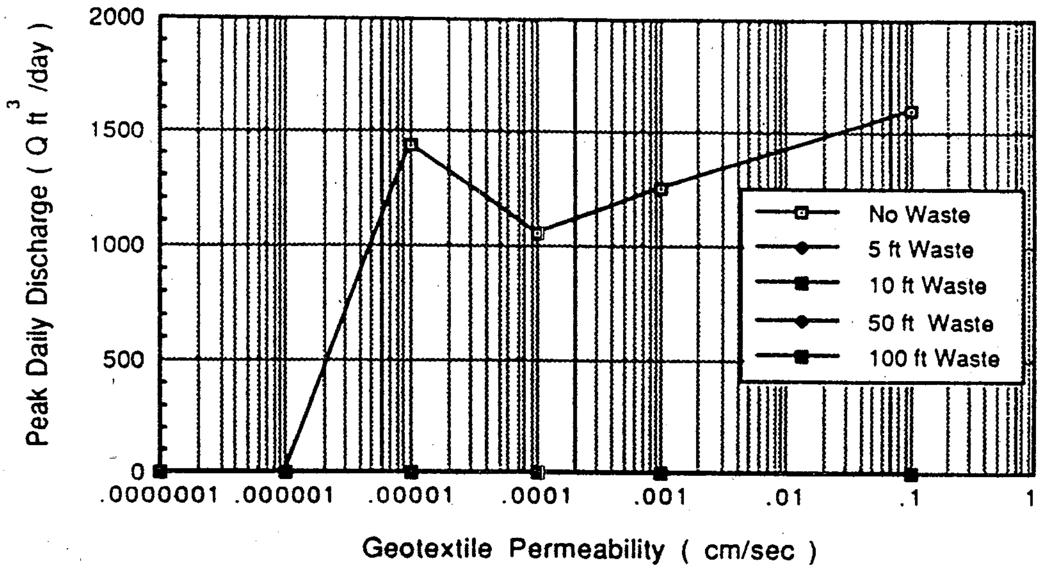


Figure C-24 - Effect of Geotextile Permeability on Daily Discharge at Seattle, Washington (K for Lateral Drainage Layer = 1 cm/sec)



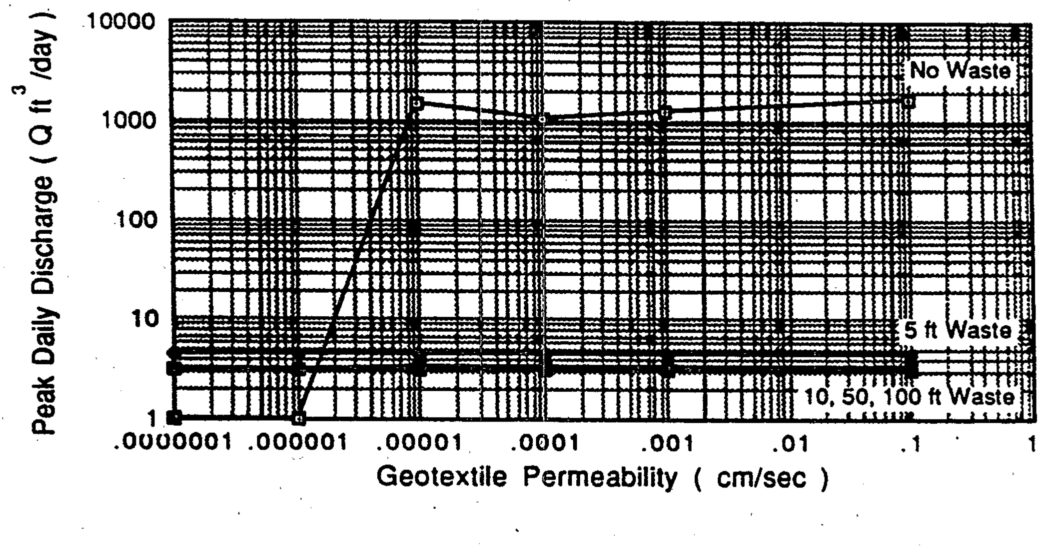


Figure C-25 - Effect of Geotextile Permeability on Daily Discharge at Phoenix, Arizona (K for Lateral Drainage Layer = 1 cm/sec) other hand, in Phoenix, Arizona, the values of the discharge corresponding to 5, 10, 50 and 100 ft of waste were 4.5, 3.5, 3.3 and 3.3 ft ^{3}/day, respectively.

(c) Effect of the climatic conditions

A comparison between the three different cities (Philadelphia, Seattle and Phoenix) is shown in Figures C-26 to C-30. For the no waste condition, illustrated in Figure C-26, the variation in Q values corresponding to the different climates were not extremely large, the values of Q being 2390, 2070 and 1600 ft ^{3}/day for Philadelphia, Seattle and Phoenix, respectively.

As waste was added, the variation in the values of the peak daily discharge between the different locations became pronounced. The value of Q for Seattle was much greater than that for Philadelphia and Phoenix. For example, in Philadelphia and Phoenix the amount of peak daily discharge after applying 5 feet of waste was only 7 % and 0.3 % of that corresponding to the no waste condition. On the other hand, in Seattle the amount of Q after adding 5 feet of waste was 32% of that obtained for the no waste condition. The curves plotted in Figures C-27 through C-30 indicate that the pattern of the relationships between k and Q were almost the same for different waste heights, even though the discharge magnitudes progressively decreased as waste heights increased.

C-6 General Comments

This section of the report was concerned with the evaluation of the major parameters affecting the leachate movement in a landfill as predicted using Version 3 of the "Hydrologic Evaluation of Landfill Performance" (HELP) computer model. The study examined the general effects of various input parameters, which must be identified by the user, on the amount of leachate collected at the base of a landfill in which the waste had not reached its full height and final closure of the landfill had not occurred.

The study was divided into two main parts. The first part was a sensitivity analysis to highlight the effect of some of the input parameters on the calculated output values. The second part was a parametric study to evaluate the effect of the major parameters which are particularly important in estimating the percolation through the landfill and hence, need to be well defined by the user. To achieve this goal two landfill configurations were considered. In the first configuration the waste was placed directly on top of the lateral drainage layer (i.e., the "no filter" situation) which was underlain by an impermeable liner. In the second configuration it was assumed that the waste was placed on top of a geotextile filter/separator which covered the lateral drainage layer. Again, an impermeable liner was considered to be placed beneath the drainage layer.

The leachate movement in the landfill was calculated using the default climatologic data for Philadelphia, Pennsylvania for the year 1974. To compare the variations in the amount of leachate due to different climates, both a wet and a dry site were considered. These were arbitrarily chosen to be Seattle, Washington and Phoenix, Arizona, respectively.

From the results obtained the following comments can be drawn:

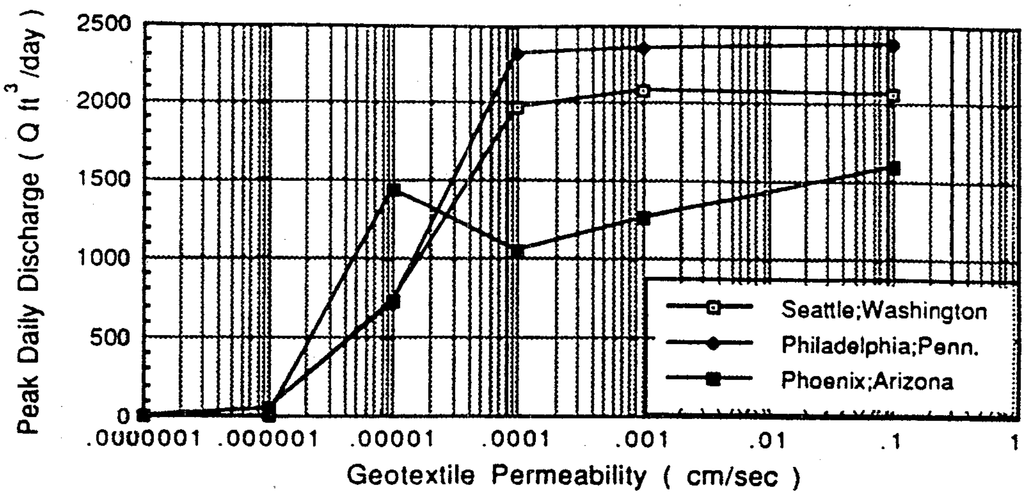


Figure C-26 - Effect of Geotextile Permeability on Daily Discharge For Different Cities (For No Waste Condition)

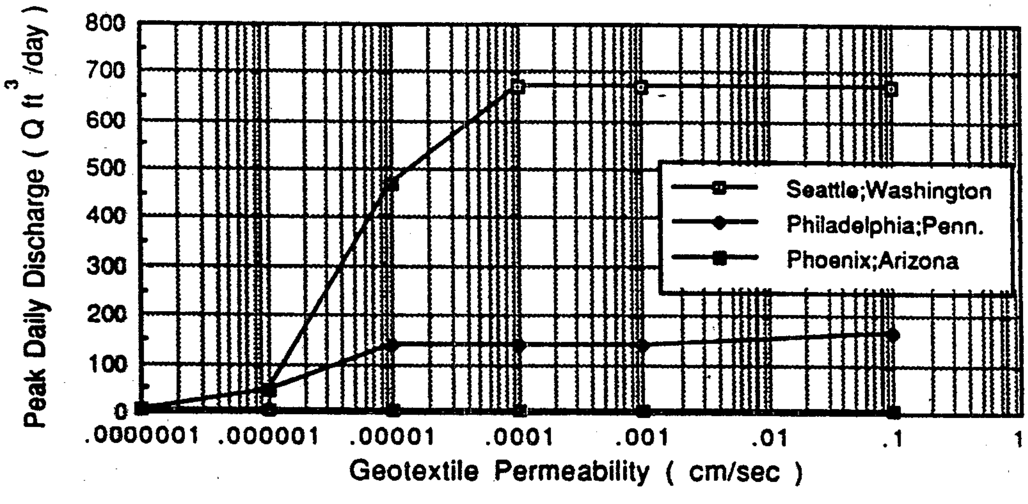


Figure C-27 - Effect of Geotextile Permeability on Daily Discharge for Different Cities (For 5 ft Waste Condition)

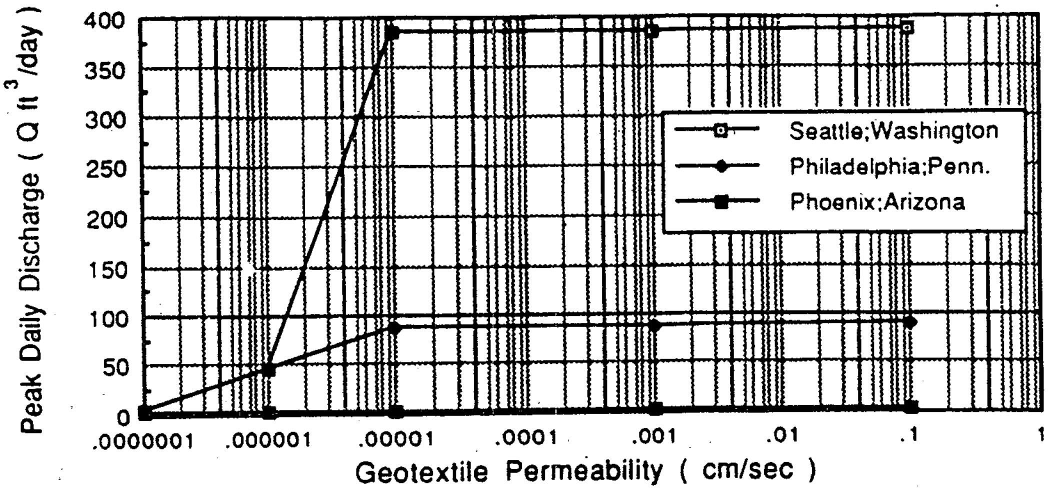


Figure C-28 - Effect of Geotextile Permeability on Daily Discharge For Different Cities (For 10 ft Waste Condition)

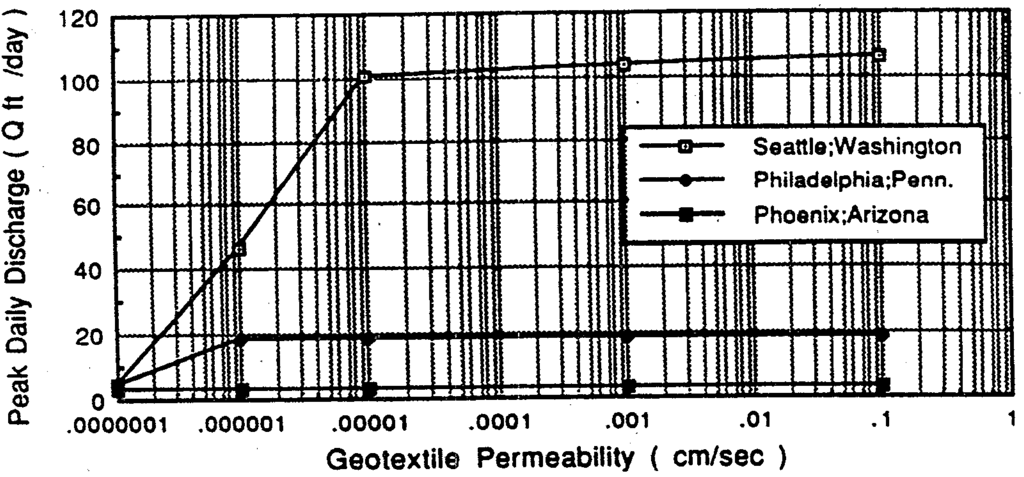


Figure C-29 - Effect of Geotextile Permeability on Daily Discharge For Different Cities (For 50 ft Waste Condition)

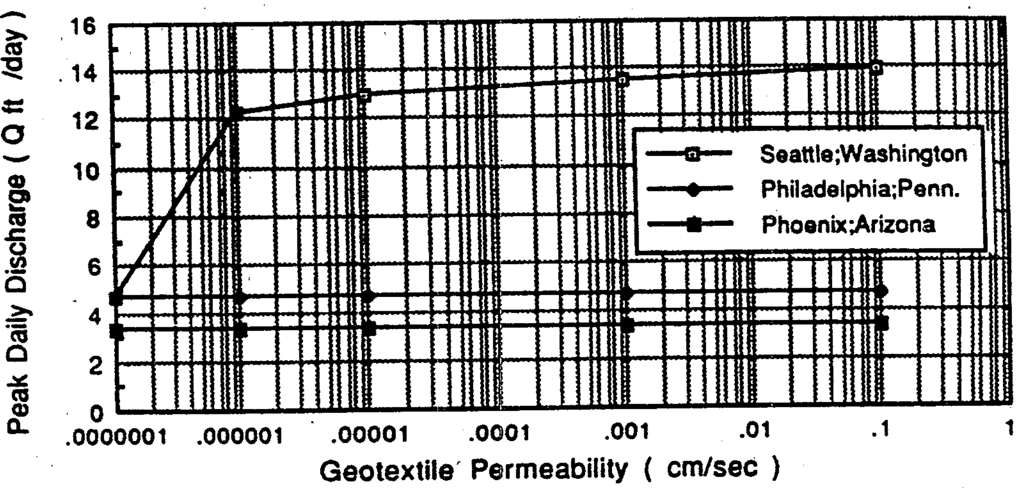


Figure C-30 - Effect of Geotextile Permeability on Daily Discharge For Different Cities (For 100 ft Waste Condition)

Case 1. Landfill with No Geotextile Filter / Separator above Drainage Layer

(1) Sensitivity Analysis

The different factors taken into consideration in this part of the study were thickness of the lateral drainage layer, slope at the base of the lateral drainage layer, evaporation depth and runoff curve number for a landfill located in Philadelphia, Pennsylvania considering a no waste condition as well as a 10 ft waste condition.

(i) No waste condition

Before any waste material was deposited in the landfill the model indicated that the above mentioned parameters have no or very small effect on the amount of leachate, especially within ranges of practical values.

(a) Thickness of the lateral drainage layer

The results indicated that for thicknesses ranging between 1 and 3 ft, the amount of leachate, Q, was not altered. However, if the thickness was reduced to 0.5 ft, Q was decreased by 12%. Federal regulations state that the thickness of the lateral drainage layer should not be less than 1 ft.

(b) Slope at the base of lateral drainage layer

The results show that the calculated value for Q was not affected by slopes ranging between 1% and 5%. On the other hand, if the slope was as small as 0.5% (as allowed in most Federal regulations), Q was reduced by 13.5%. Some state regulations recommend that the slope at the base of the lateral drainage layer should not be less than 2%.

(c) Evaporation depth

The default value for the evaporation depth was given as 9 inches. For an evaporation depth of 6 in., Q is reduced by 10%. As the evaporation depth increased to 15 in., Q was equal to that corresponding to 9 in. If the evaporation depth increased to 18 in., Q increased by 18.5%.

(d) Runoff curve number

In the case of open landfills the runoff curve number had no effect on the values of Q. In this case, a fraction of the total potential runoff that actually drained from the surface of the waste layer should be specified by the user as data input.

(ii) 10 ft waste condition

When waste was placed in the landfill, the calculated amounts of leachate were not altered with variations in either the thickness of the lateral drainage layer, the slope at its base or the evaporation depth. As previously mentioned, in the case of open landfills the runoff curve number was not meaningful.

(2) Parametric Study

The parameters chosen for this study were permeability of the lateral drainage layer, height of the waste material and permeability of the waste material for accumulated waste.

(i) No waste condition

For the no waste condition the amount of Q decreased greatly with the decrease in the permeability of the lateral drainage layer, K, from 1.0 to 0.001 cm/sec. When K values became equal to or less than 1 × 10^{-4} cm/sec the corresponding values of Q became very small.

(ii) Waste accumulation

(a) Permeability of the lateral drainage layer

At any specific waste height the decrease in K from 1.0 to 1 × 10^{-6} cm/sec resulted in small variations in the corresponding values of Q.

(b) Height of the waste material

The height of the waste material, H, was one of the major parameters affecting the amount of discharge. As waste was added the values of Q were reduced dramatically compared to those corresponding to the no waste condition.

(c) Permeability of the waste material

The default value suggested for the permeability of compacted waste in the model is 0.0002 cm/sec. If the waste permeability increased with time up to 0.002 cm/sec, e.g., due to waste degradation, or decreased to 0.00002 cm/sec, e.g., due to the deposition of the fine waste particles into the voids between larger waste particles, the change in the value of Q was relatively small. However, if the waste permeability increased to a value of 0.02 cm/sec the increase in Q became pronounced.

Case 2. Landfill with a Geotextile Filter / Separator above Drainage Layer

(1) Parametric study for a landfill in Philadelphia, Pennsylvania

The parameters taken into consideration when assuming the presence of a geotextile filter/ separator on top of the lateral drainage layer were permeability of the geotextile layer, permeability of the lateral drainage layer, height of the waste material and permeability of the waste material.

(i) No waste condition

(a) Permeability of the geotextile layer

The reduction in the geotextile permeability, k, from 0.1 to 1 × 10^{-4} cm/sec had almost no effect on the value of Q. However, for k values less than or equal to 1x10^{-5} cm/sec the values of Q became significantly low.

(b) Permeability of the lateral drainage layer

For a given geotextile permeability, higher than or equal to 1 × 10^{-4} cm/sec, the values of Q decreased greatly with a decrease in the permeability of the lateral drainage layer. However, for k values less than 1 × 10^{-4} cm/sec the decrease in the lateral drainage layer permeability did not affect the values of Q.

(ii) Waste accumulation

(a) Permeability of the geotextile layer

As waste was placed in the landfill the variation in k values between 0.1 and 1 × 10^{-5} cm/sec did not affect the values of Q. However, as the geotextile permeability was reduced to values lower than 1 × 10^{-5} cm/sec, e.g., due to sediment and/or biological clogging, the amount of leachate collected at the bottom of the landfill started to diminish. As the geotextile permeability dropped lower than the solid waste permeability, the geotextile acted more as a barrier layer rather than as a filter. The values of the peak daily discharge assuming the geotextile to be a barrier layer were much higher than those corresponding to the geotextile acting as a vertical percolating layer. However, the average annual totals corresponding to either of the assumptions were the same up to a geotextile permeability of 1×10^{-6} cm/sec. This is due to the fact that the program allows only downward flow in a barrier layer. Thus, any leachate accumulating on a barrier layer will eventually percolate through it. Percolation rate depends upon the head of the liquid above the base of the barrier layer. As the geotextile permeability reached 1×10^{-7} cm/sec the geotextile filter was now excessively clogged, allowing only very small amounts of leachate to percolate through it. Hence, the greatly reduced geotextile permeability became the dominant parameter affecting the amount of leachate collected within the drainage layer.

(b) Permeability of the lateral drainage layer

For a given waste height and for geotextile permeabilities greater than or equal to 1×10^{-5} cm/sec, the variations in the permeability of the lateral drainage layer had very small effect on the amount of leachate. For geotextile permeabilities less than 1×10^{-5} cm/sec the variations in the lateral drainage layer permeability had no influence on the values of Q.

(c) Height of the waste material

For geotextile permeabilities, k, greater or equal to 1×10^{-5} cm/sec the values of Q were only dependent on the height of the waste material. When k was less than 1×10^{-5} cm/sec the values of Q became dependent on both the geotextile permeability as well as the waste height.

(d) Permeability of the waste material

At any specific waste height and for geotextile permeabilities higher than or equal to 1×10^{-5} cm/sec the amount of leachate increased with the increase in the waste permeability. Also, the rate of the increase in Q increased with the increase in the waste permeability. For geotextile permeabilities less than 1×10^{-5} cm/sec any increase in the waste permeability did not affect the values of Q.

(2) Parametric study for landfills in different climates

As it would be expected, the amount of leachate collected in a landfill constructed at a wet site was higher than that collected in a dry site. For different climates, the patterns of the relationships between the amount of leachate, Q, and the various parameters discussed above were primarily the same, even though the values of Q differ depending on the variations in the precipitation and temperature at the different sites.

C-7 Conclusions of the HELP Model Analysis

The conclusions of the parametric and sensitivity study of the HELP model presented in this appendix follow.

(1) Regarding the situation with no filter layer above the lateral drainage layer

(i) For no waste condition:

- The decrease in the permeability of the lateral drainage layer, K, from 0.1 to 1×10^{-3} cm/sec caused a tremendous decrease in the amount of primary leachate. When K became equal to or less than 1×10^{-4} cm/sec the amount of leachate became extremely small.

(ii) For accumulated waste:

- At any specific waste height the amount of leachate did not vary for K values ranging between 1 and 1×10^{-5} cm/sec. Any further decrease in the permeability caused a decrease in the amount of leachate.

(2) Regarding the situation of having a geotextile or natural soil filter layer above the lateral drainage layer:

(i) For no waste condition:

- The leachate flow decreased with the decrease in the lateral drainage layer permeability from 1 to 1×10^{-3} cm/sec.
- The amount of leachate was not altered with the reduction in the geotextile permeability , k, from 0.1 to 1×10^{-4} cm/sec. For k values equal to or less than 1×10^{-5} cm/sec, however, the leachate flow became very low. Thus the permeability of a geotextile equal to 1×10^{-5} cm/sec limited the leachate flow for the underlying drainage system. As the geotextile permeability became smaller, leachate flow rates were proportionally decreased.

(ii) For accumulated waste:

- The decrease in the lateral drainage layer permeability from 1 to 1×10^{-3} cm/sec had a very small effect on the amount of leachate produced.
- The variation in leachate flow for geotextile permeabilities, k, decreasing from 0.1 to 1×10^{-5} cm/sec was negligible. For k values less than 1×10^{-5} cm/sec the amount of leachate became very small. Thus a geotextile permeability of 1×10^{-5} cm/sec began to limit the leachate flow for the drainage system as was also the case for the no waste condition just described. The limiting geotextile permeability appeared to be 1×10^{-5} cm/sec. The exception was when the geotextile permeability dropped lower than the solid waste's permeability. In this case, the geotextile acted as a barrier layer and should be analyzed in the model as such.

(3) On this basis of this parametric and sensitivity study, the HELP model was of great interest to compare alternate design strategies for leachate collection filters. Use of the model gave good insight as to the important and unimportant parameters in a landfill leachate collection system. It is highly recommended for use in this regard.

It was noted that the filter can be a geotextile or natural soil and the model can be used equivalently with either assumption. Additionally, the HELP model can be used to evaluate the sensitivity of the drainage layer's permeability when there is no filter whatsoever.

APPENDIX "D"

THE "NO FILTER" DESIGN SCENARIO

D-1 Overview

It is believed by the authors of this report that by using a properly designed geotextile filter over the entire footprint of a landfill or landfill cell an acceptable long-term strategy for a leachate collection system in a modern landfill will result. Of course, the proper geotextile must be selected and the recommended process is embodied in the design formulation that was presented in Section 8. However, some designers are considering the option of "no filter" at all. Indeed, some facilities are constructed with no filter whatsoever. With the no filter strategy the waste is placed directly on the drainage material which is either gravel or sand. The logic of this design strategy is that the filter contains the smallest voids, which are most likely to experience excessive clogging. If entirely removed, the likelihood of the larger sized drainage material clogging in comparison to that of the filter is distinctly lessened. Furthermore, by using gravel as the drainage material instead of sand (and even large size gravel at that) the likelihood becomes progressively smaller.

D-2 Design Considerations

The thought of using no filter in the leachate collection system leads some to question problems of particle loss into the drainage material. In the no filter scenario, the permeability of the drainage material must be greater than that of the waste but still be capable of retaining the relatively large particles of the waste. In this regard, field pump tests conducted at several different landfills by Oweis [16], suggest that the permeability of municipal solid waste is typically from 1×10^{-2} to 1×10^{-4} cm/sec.

Unfortunately, this value is too low for rapid and efficient collection and removal of leachate. Thus the permeability of the drainage soil is increased, often to a particle size resulting in a permeability of 1.0 cm/sec. This is typical of a quarried gravel of 6 to 45 mm (0.25 to 1.5 in.) particle size, e.g., AASHTO #57 stone. With such soils the situation of particle loss and the accompanying clogging of the drainage gravel must be considered.

In the context of the design Equations 7 and 8 in Section 8, the DCF = 1.0 and the factor-of-safety is formulated accordingly, i.e.,

$$FS = \frac{k_{allow}}{k_{reqd}} \tag{4}$$

where

k_{reqd} = required permeability of the drainage layer

k_{allow} = allowable permeability of the drainage material

The value of "k_{reqd}" comes from the HELP model, and "k_{allow}" is experimentally obtained. A series of "no filter" experiments is described in this appendix.

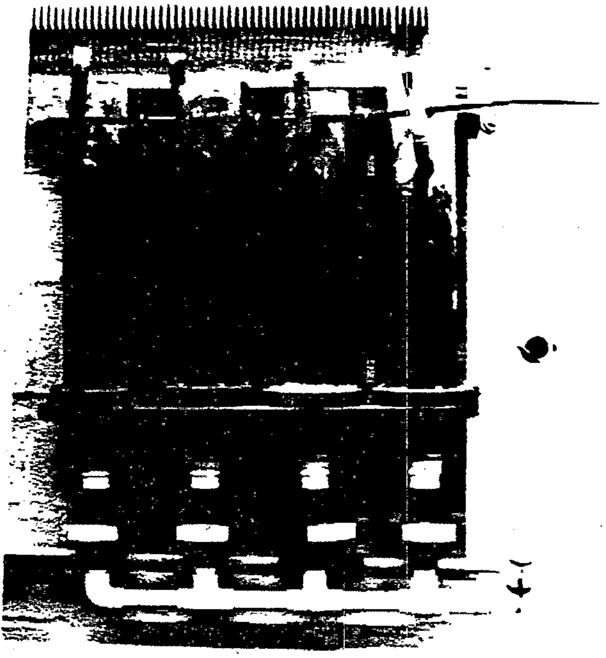
D-3 "No Filter" Experiments and Results

The experimental investigation presented in this appendix involved evaluating the permeability of eight different drainage materials over an extended period of time to determine; (a) if equilibrium flow rates exist, and (b) what are the respective values [17]. The test method used for these tests (as in Section 6) was in accordance with ASTM D1987. The tests were conducted using rigid wall permeameters of 100 mm (4.0 in.) diameter and flow rates were measured under constant head conditions. The resulting permeability values were calculated using Darcy's formula at incremental time periods throughout the duration of the tests. Each of the flow columns were permeated with municipal solid waste leachate at a flow rate of 20,000 l/ha-day (2000 gal/acre-day). This flow rate was chosen on the basis of a New York State survey of its landfills which found that average leachate flow rates are in this approximate range, Phaneuf [11]. Thus these tests are not accelerated flow rate tests in the context of the tests conducted in Section 6, recall Table 13. All tests were conducted under saturated anaerobic conditions and gas production was noted in all of the columns.

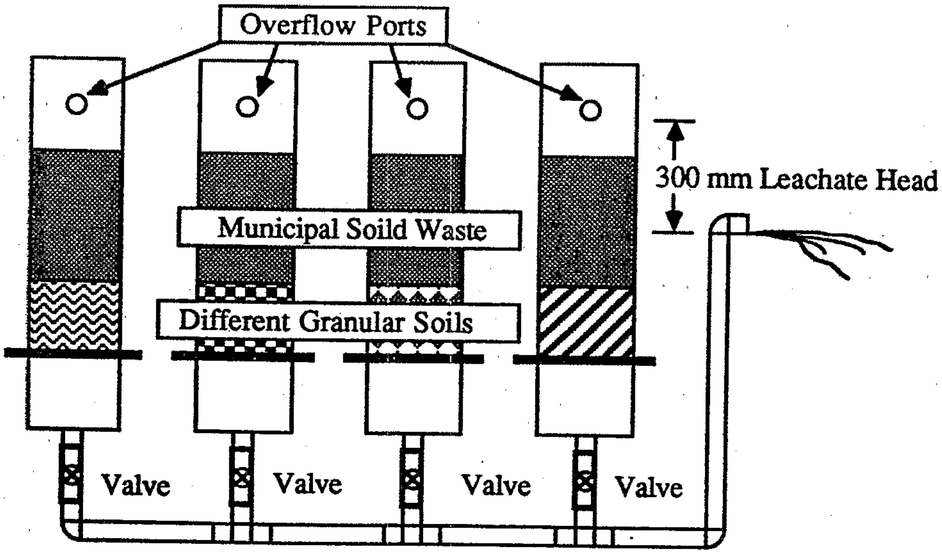
The experimental design consisted of eight flow columns. Figure D-1(a) shows a photograph and Figure D-1(b) a schematic diagram of a 4-unit test setup. Four of the permeameters contained different types of gravel underneath the waste and an additional four columns had different types of sand underneath the waste.

The solid waste, as well as the leachate, was obtained from a local municipal solid waste landfill. The solid waste was excavated out of a region of the landfill that was saturated with leachate for approximately two years. It should be noted that some of the larger pieces of the waste were cut to fit inside the flow columns so that they could be compacted into the 100 mm (4.0 in.) diameter flow permeameters.

Figure D-2 presents the particle size distribution curves of the eight drainage soils selected for this study. They cover a wide range of particle sizes and gradations. Table D-1 is presented to further characterize the various soils. Note that all soils are granular with relatively high permeability values. The reason for selecting these particular soils was that in formulating the draft EPA Leak Detection Rules, the requirement of a minimum 1.0 cm/sec permeability drainage material was regularly discussed. The four gravels selected for this study meet this criterion. All were poorly graded gravels (GP's) under the Unified Classification System. They conformed to sizes between AASHTO #3 and #57 gravels. The mineral composition of the four soils were selected on the basis of the most prevalent types of quarried stone. Thus quartz, gneiss, limestone, and shale gravels were included in the study.



(b) Photograph of a Set of Four Long Term Flow Permeameters



(b) Schematic Diagram of a Set of Four Long Term Flow Permeameters

Figure D-1 - Long Term Flow Permeameters Used for the Testing of Municipal Solid Waste Placed Directly Above Granular Soils Without the Use of Sand or Geotextile Filters

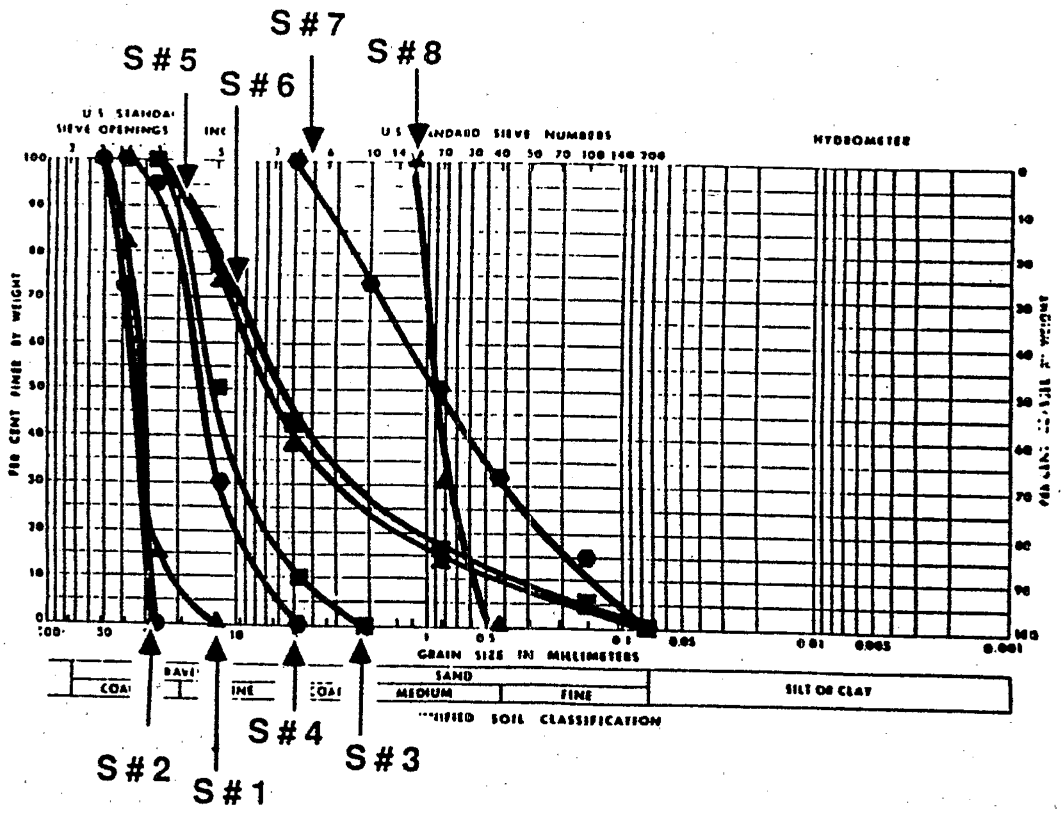


Figure D-2 - Particle Size Distribution Curves of Eight Soils Used in This Study

Table D-1 - Granular Soil Characteristics for "No Filter" Permeability Study

Column	Soil Type	Mineral	Shape	d_{10} (mm)	d_{50} (mm)	d_{60} (mm)	CU	USC Class.	AASHTO Classfication
1	gravel	quartz	rounded	17	32	34	2.0	GP	3
2	gravel	gneiss	angular	27	33	36	1.3	GP	3
3	gravel	limestone	angular	5	16	17	3.4	GP	57
4	gravel	shale	angular	8	17	17	2.1	GP	57
5	sand	limestone	angular	0.4	7	9	22.5	SW	2A
6	sand	quartz	angular	0.4	6	8	20.0	SW	2A
7	sand	quartz	angular	0.2	.9	1.4	8.5	SW	10
8	sand	quartz	rounded	0.6	.9	1.0	1.7	SP	n/a (Ottawa)

Notes

d_{10} = grain size of 10 percent finer by weight

d_{50} = grain size of 50 percent finer by weight

d_{60} = grain size of 60 percent finer by weight

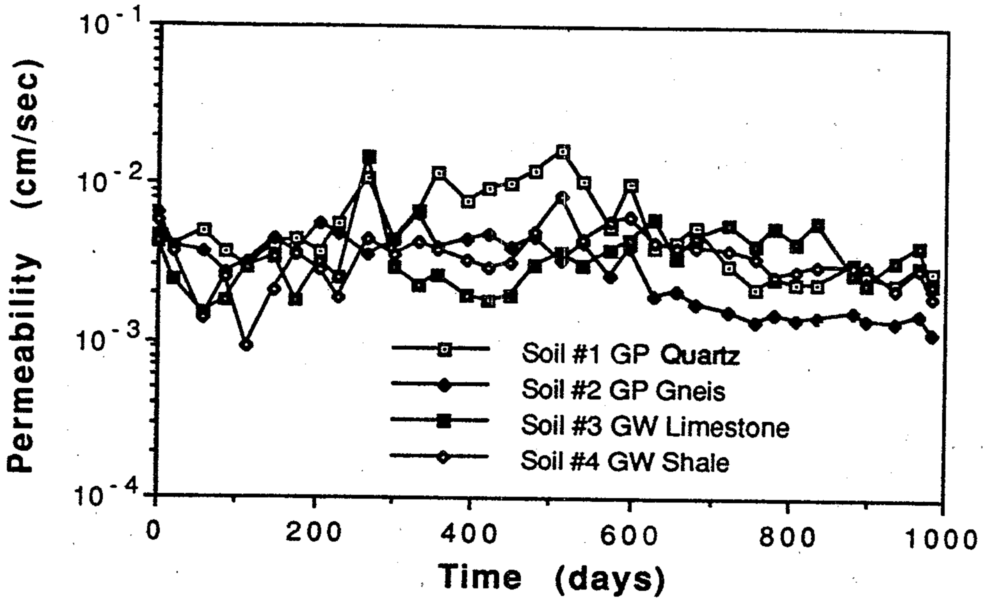
CU = Coefficient of Uniformity = d_{60}/d_{10}

USC = Unified Soil Classification

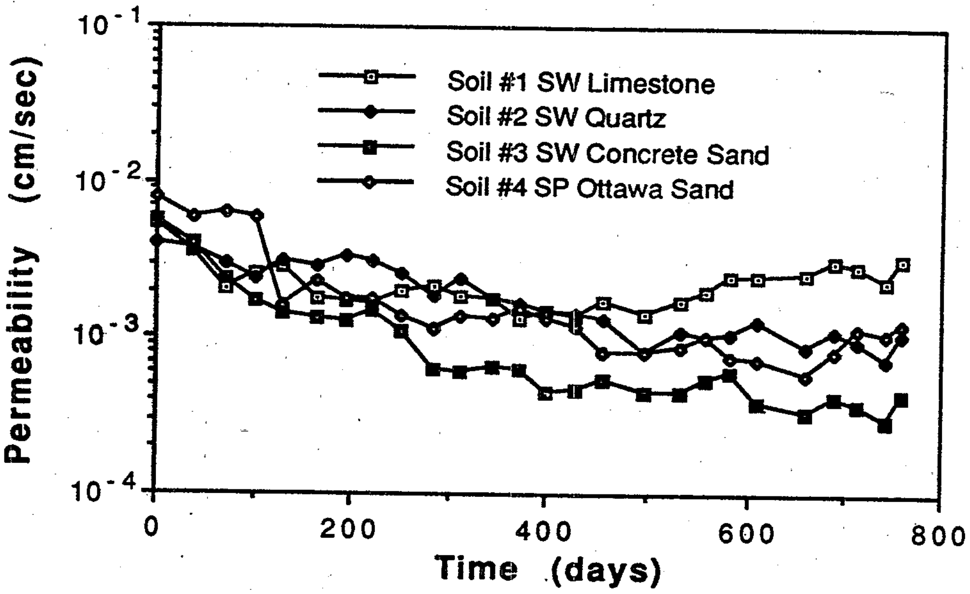
AASHTO = American Association of State Highways and Transportation Officials

When the Leak Detection Rules finally appeared in the Federal Register in the spring of 1992 the permeability requirement did not increase to 1.0 cm/sec but remained at the previously regulated value of 0.01 cm/sec. Thus the need arose for an additional four columns using sand as the drainage soils. Limestone and quartz sandy soils were selected. Both materials were classified by the Unified Classification System as well graded sands (SW) and are designed as AASHTO #2A materials. In addition to these two well graded sands, concrete sand and Ottawa sand were selected to investigate additional alternatives. The concrete sand was classified as a SW in the Unified Soil Classification system and #10 in the AASHTO system. The Ottawa sand was classified as poorly graded sand (SP) in the Unified Soil Classification system and does not fall under any designation in the AASHTO classification system.

Figures D-3(a) and (b) show the graphed results of the system permeability (which includes both solid waste and granular drainage soil) for the eight permeameters that were evaluated. The permeameters with gravel were maintained for nearly 1,000 days while the permeameters with sand were maintained for nearly 800 days. The results of the permeameters with gravel are shown in Figure D-3(a). They indicate that the solid waste was definitely the controlling flow material. There was very little difference in system permeability due to particle shape, gradation or mineral composition of the different gravels. In addition, the system permeability remained relatively constant for all four gravel columns. The slight rise in permeability after 200 days was explained as the start of a trend of piping via loss of fines. Since the permeameters were not recharged with municipal solid waste this could have eventually led to further increases in permeability.



(a) Behavior Using Gravels Beneath Solid Waste



(b) Behavior Using Sands Beneath Solid Waste

Figure D-3 - Long Term Flow Results as per ASTM D1987

The results of the permeameters with sand shown in Figure D-3(b) indicated that the solid waste was again controlling the flow. The permeability values were in the 0.01 to 0.001 cm/sec range which was lower than expected for the sands alone. The sands by themselves had a permeability between 0.05 and 0.10 cm/sec. There was a trend in all of the permeameters with sand that suggested that the system permeability was decreasing over time. This decrease was not pronounced or sudden but it was definitely observable.

In addition to the information presented, the solid waste placed directly on drainage soils was qualitatively analyzed. The flow columns used for this study were made from acrylic which is transparent. In the case of the gravels it appeared that small amounts of fines were migrating through the gravel over time. With the sand columns there was staining of the municipal solid waste-to-sand interface. Additionally, a qualitative observation was made about the columns in which limestone aggregate was utilized. In all cases where limestone was used it had not agglomerated together after being permeated with leachate for 1000 days. Thus the limestone used in these tests had not bound together via a reaction with the leachate. The limestone in this experiment was of a low carbonate content (less than 5%) and did not react with the leachate to any appreciable degree. There is concern over this issue [18] and additional research appears to be warranted in this regard.

D-4 Conclusions of the "No Filter" Study

Landfills are complex and constantly changing bio-reactors. As such, a designer is faced with the challenge of designing a leachate collection system for changing conditions. Each stage of the evolution involves different processes, mechanisms, reactions and microorganisms. Some stages are short-lived and others are nearly permanent. A designer needs to establish the worst-case scenario, or critical condition, during this evolutionary process and design accordingly. While the trends of the curves in Figure D-3 are encouraging, it must be cautioned that they are not accelerated flow rate tests and the 1000 days test duration is a simulated case of only 3 years of typical field flow rates.

Irrespective of the cautions raised in this section, the no filter strategy is being practiced at several landfills. In all cases, the waste placed over the drainage media is considered "select waste". As such, there are no large objects allowed in the select waste. This precaution must be taken so as to eliminate penetration of the drainage material and possible puncture of the underlying liner system. It is absolutely critical that the utmost care be taken in the initial placement of the select waste. The highest level of construction quality control and construction quality assurance must be exercised. For this reason it may be prudent to increase the thickness of the drainage layer to compensate for waste intrusion and also for clogging in the upper 1/3 to 1/2 of the thickness of the layer.