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TECHNICAL EVALUATION OF SITES LOCATED IN THE ZONE OF SATURATION

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DRAFT FINAL REPORT

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TECHNICAL EVALUATION OF SITES LOCATED IN THE ZONE OF SATURATION

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TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 DRAIN THEORY 4

2.1 Drain Spacing and Head Levels 4

2.1.1 Drains On Impervious Barriers 5
2.1.2 Drains Above Impervious Barriers. 7
2.1.3 Drains On Sloping Impervious Barriers 10

2.2 Drain Pipe Sizing 13

2.2.1 Hydraulic Gradient (i) and Roughness Coefficient 14
2.2.2 Discharge (Q) 14
2.2.3 Pipe Size 21

2.3 Filters and Envelopes 23

2.3.1 Function of Filters and Envelopes 23
2.3.2 Design of Sand and Gravel Filters 24
2.3.3 Design of Sand and Gravel Envelopes 25
2.3.4 Synthetic Filters 26

3.0 DESIGN AND CONSTRUCTION 27

3.1 Hypothetical Site 27
3.2 Sensitivity Analysis 30

3.2.1 Drains On Impermeable Barriers 30
3.2.2 Drains On Sloping Impervious Barriers 34
3.2.3 Drains Above An Impervious Barrier 37

3.3 Application 40

3.3.1 Parameter Estimation 43
3.3.2 Landfill Size 44
3.3.3 Example Problem 44

3.4 Application 47

3.4.1 Construction Inspection 47
3.4.2 Drain System Maintenance. 48
3.4.3 Future Operating Conditions 49

TABLE OF CONTENTS
(Continued)

4.0 FLOW NET ANALYSIS 50

4.1 Introduction 50
4.2 Initial Conditions 52
4.3 Parameter Analysis 55

5.0 MODELS 60

5.1 Release Rate Models 60

5.1.1 Fundamentals 60

5.1.1.1 Leachate Generation 60
5.1.1.2 Leachate Constituent Concentrations. . 66

5.1.2 Selected Release Rate Models 69

5.2 Solute Transport Models 76

5.2.1 Fundamentals 76

5.2.1.1 Analytical Models 80
5.2.1.2 Numerical Models 80

5.3 Model Limitations 90

6.0 RECOMMENDATIONS 92

7.0 BIBLIOGRAPHY 94

LIST OF TABLES

2-1 Values for the Equivalent Depth d of Hooghoudt 11

2-2 Minimum Hydraulic Gradients for Closed Pipes 15

2-3 Drain Grades for Selected Critical Velocities 16

3-1 Range of Site Variables 29

3-2 Drain Length Spacing (m) for Drains on an Impermeable Barrier 31

3-3 Values of h/L for Various $C=q/k_1$ and Angles a 35

3-4 Drain Spacing (m) for Head Maintenance Levels of 2 Meters 38

3-5 Drain Spacing (m) for Drains Above an Impermeable Barrier 41

3-6 Example Data Set 45

5-1 Major Factors Affecting Leachate Generation 62

5-2 Factors Affecting Leachate Constituent Concentrations . 67

5-3 Factors Affecting Leachate Release 68

LIST OF FIGURES

1-1	Typical Stratigraphic Column of Zone of Saturation Sites in Wisconsin	2
2-1	Drains Resting on an Impervious Barrier	6
2-2	Flow to Drains Above an Impervious Barrier	9
2-3	Drains on Sloping Impermeable Barrier	12
2-4	Flow Components to a Landfill	17
2-5	Division of a Symmetrical Drawdown Drain Problem Into Two Equivalent Fragments	20
2-6	Capacity Chart $n = 0.013$	22
3-1	Hypothetical Zone of Saturation Landfill	28
3-2	Drain Length versus Flow Rates for Head Levels Equal to 1 Meter	32
3-3	Plot of h/L versus h/I = tan a for Drain and Sloping Impervious Layers	36
3-4	Drain Length (L) versus $c - q/k_1$	39
3-5	Drain Spacing (L) versus Inflow rates (q)	42
4-1	Cross Section of Typical Zone of Saturation Landfill	51
4-2	Inflow With Typical Landfill Cell	54
4-3	Flow Nets With Different Leachate Levels	56
4-4	Flow Nets With Different Vertical Gradients	57
4-5	Flow Nets With Different Numbers of Drains	58
5-1	Major Components of Groundwater Transport Equations	78

1.0 INTRODUCTION

The State of Wisconsin and much of EPA Region V are located in a portion of the country characterized by thick surficial deposits of saturated glacial till underlain by fractured sedimentary or crystalline rock. A typical stratigraphic column for Wisconsin is shown in Figure 1-1. Landfill site design in this part of the country is complicated by the hydrology of the zone of saturation. At these sites, the glacial clay deposits into which the landfills are built act as limited unconfined aquifers. Hydraulic conductivities of the clays range from 10^{-5} cm/sec, where the clays are fractured, to 10^{-7} cm/sec, where the clays are unfractured. Underlying the clays are thick units of dolomite and sandstone which act as confined aquifers. Confining pressures in these units can bring groundwater to the land surface in drilled wells. Recharge of the underlying units occurs at or near outcrops.

Siting landfills in this region, especially those for accepting hazardous wastes, is a problem because the base grade of the facility is typically below the water table (i.e., in the zone of saturation). As a result of having the base grade below the groundwater table, the potential for accelerate leachate generation and contaminate release are greatly enhanced. To alleviate this problem, landfill operators are required to manage groundwater and leachate in the landfills so that inward hydraulic gradients are constantly maintained, thereby limiting the possibility of contaminant escape. The method utilized to maintain inward hydraulic gradients is a drainage collection system on the base of the landfill. The drainage system allows for the maintainance of landfill head levels which are lower than the natural groundwater table (i.e., inward hydraulic gradients).

The purpose of this study was to perform a theoretical evaluation of the validity of the presently used landfill management schemes for groundwater and leachate at sites located in the zone of saturation. This evaluation included flow net and parameter sensitivity analyses. The key parameters that were evaluated include:

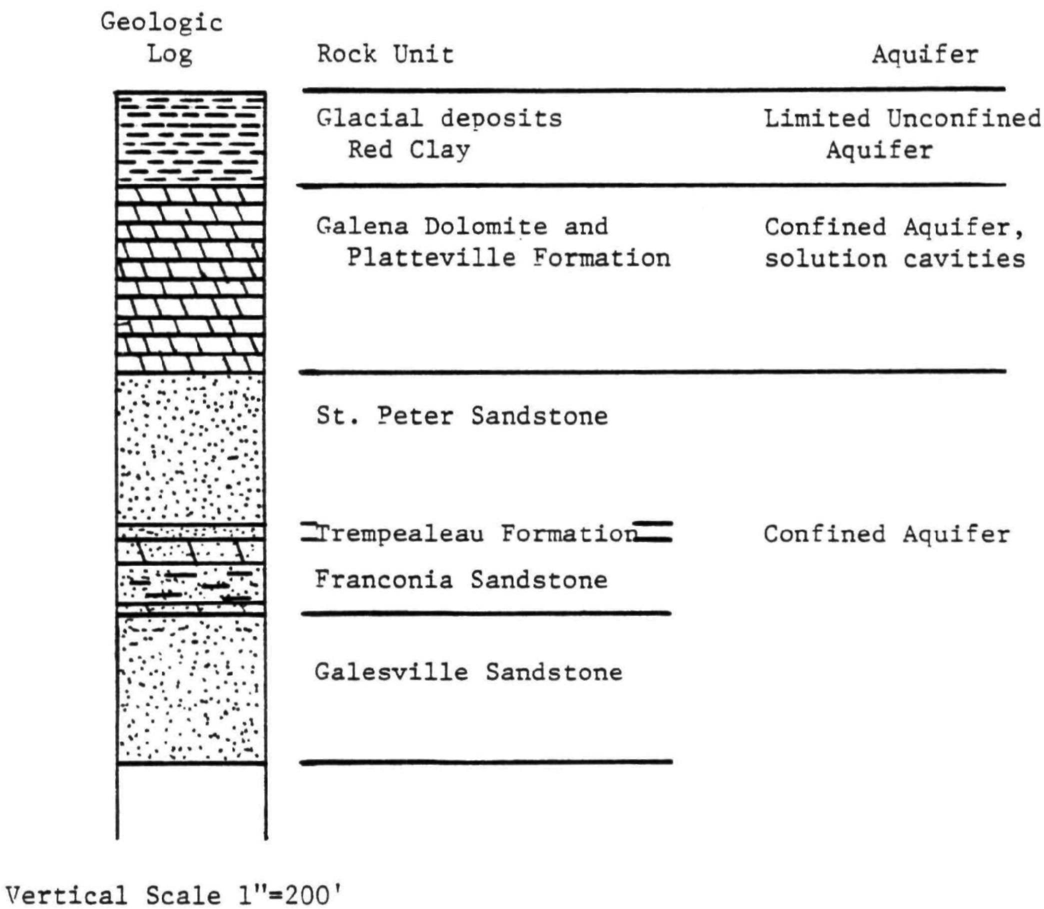

Figure 1-1. Typical Stratigraphic Column of Zone of Saturation Sites in Wisconsin (City of Fond du Lac)

- o Drain spacing
- o Hydraulic conductivities of the landfill and natural soils surrounding the site
- o Inflow rates resulting from groundwater infringement and leachate generation
- o Head maintenance levels within the landfill
- o Pipe sizing
- o Drainage blanket use.

Other parameters addressed include landfill dimensions, construction inspections, and future operating conditions. Drainage theory and selected models for predicting release rates and so solute transport are also described.

The results of this study should assist permit writers in determining engineering design modifications and site monitoring requirements, as well as aid in establishing a basis for future design protocols for zone of saturation landfills.

2.0 DRAIN THEORY

There are three major elements to consider in the design of a subsurface drainage system suitable for zone of saturation landfills:

- The drain spacing required to achieve the desired head maintenance levels
- The hydraulic design of the conduit including the pipe diameter and gradient
- The properties of the drain filter and envelope.

This section briefly describes the principles involved in determining a deisrable drain slope and spacing, and in selecting appropriate drain materials.

2.1 DRAIN SPACING AND HEAD LEVELS

There are numerous analytical solutions and models that have been developed for estimating the drain spacing required to maintain head levels at a predetermined height. This section presents the analytical solutions for determining drain spacing based on maintenance head levels, permeabilities, and flow rates for:

- Drains resting on an impermeable barrier
- Drains installed above an impermeable barrier
- Drains resting on an impermeable barrier that slopes symmetrically at an angle to the drains.

The equations presented here assume that steady state conditions exist, recharge distribution and leachate generation over the area between the drains is uniform, and the soil is homogeneous. Most real world situations do not fully meet these criteria, therefore, results obtained should be considered approximate. In using the equations for designing a landfill drain system, a conservative approach should be taken to ensure that head maintenance levels are at or below the desired height.

2.1.1 Drains on Impervious Barriers

Groundwater flow to drains resting on a horizontal (flat) impervious barrier can be represented by the equation:

$$L = [(8KDh + 4Kh^2)/q]^{0.5} \tag{1}$$

where:

- L = drain spacing (m)
- K = hydraulic conductivity of the drained material (m/day)
- D = distance between the water level in the drain line and the impermeable barrier(m)
- h = water table height above the drain levels at the midpoint between two drains (m)
- q = leachate generation rate (m/day)

Figure 2-1 illustrates the relationship between these terms. When two parallel drain lines are installed properly, each line exerts a drawdown curve that, in theory, will intersect midway between the two drain lines. In solutions to gravity flow problems, the drain spacing (L) is the distance from the drain to a point where the drawdown can be considered insignificant. This distance, L, is commonly referred to as the "zone of influence" of the drain.

For a pipe drain resting on an impermeable barrier, the parameter D approximately equals the radius of the pipe and hence is very small in comparison to h (the water table height above the drain). This allows equation 1 to be simplified to:

$$L = [(4Kh^2)/q]^{0.5} \tag{2}$$

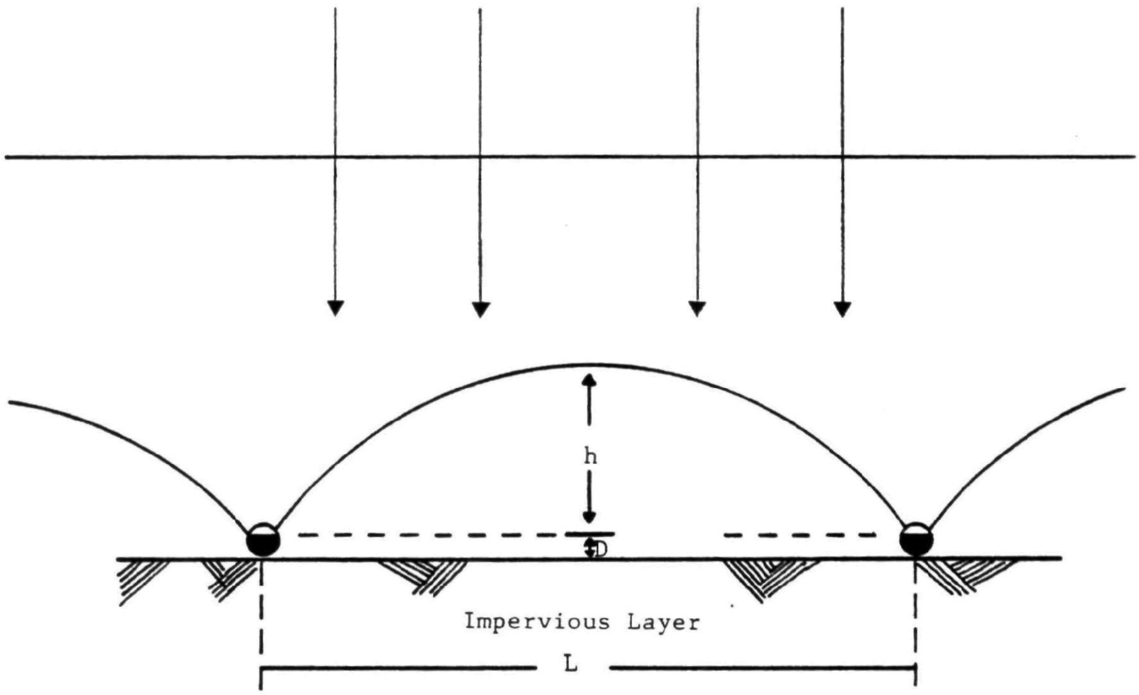

Figure 2-1 Drains Resting on an Impervious Barrier

where:

$$8KDh \approx 0$$

Equation 2 represents horizontal flow to the drains above the drain level.

Drain spacing (L) and hydraulic head level (h) in the equations are interdependent design variable which are a function of the leachate generation rate (q) and hydraulic conductivity (K) of the drained material. Assuming a constant leachate generation and hydraulic conductivity, the closer two drains are spaced the more their drawdown curves will overlap and the lower the hydraulic head levels between the drains will be. Therefore, in order to space the drains at the required distance to achieve the desired head maintenance levels, the hydraulic conductivity of the landfill material and the quantity of leachate generated must be determined to a reasonable degree of accuracy.

2.1.2 Drains Above Impervious Barriers

Equations 1 and 2 are suitable for estimating drain spacing and head levels if the drains are located on an impervious barrier, as is the case with most landfill operations. In using drainage system design equations a layer is generally considered impervious if it has a permeability less than 0.1 of the overlying layer (i.e., $K_{below}/K_{above} \leq 0.1$). The clay base of a landfill may not act as an impermeable layer in the design equattions if:

- Clays are not adequately compacted to produce the desired permeability
- Clays are fractured (naturally or during placement)
- Clays are not uniform (e.g., contain sandy zones)
- Landfill material has a permeability that is close to the clay liner.

Where drains are not installed on an impermeable barriers, flow to the drains is radial (as illustrated in Figure 2-2). In this case the drains are considered to be installed at the interface of a two layered soil with permeabilities of K_1 and K_2 (as shown in Figure 2-2). Substituting the permeability (K_2) of the material below the drain into the first term of the right hand side of equation 1 compensates for radial flow to the drain system, as given by:

$$L = [(8K_2Dh + 4K_1h^2)q]^{0.5} \tag{3}$$

Radial flow to the drains causes a convergence of flow lines near the drains which in turn causes the flow lines to lengthen. This process results in a more than proportional loss of hydraulic head because the flow velocity in the vicinity of the drains is larger than elsewhere in the flow region. Consequently, the elevation of the water table and drain spacing would be larger than those predicted using equation 3.

To account for the extra resistance caused by radial flow, Hooghoudt (1940) introduced a reduction of the depth, D, to a smaller equivalent depth, d. The equation that was developed to take into account radial flow can be rewritten as:

$$L = [(8K_2dh + 4K_1h^2)/q] \tag{4}$$

where the new terms are defined as;

d = equivalent depth (m)
K_1 = hydraulic conductivity of the layer above the drain (m/day)
K_2 = hydraulic conductivity of the layer below the drain (m/day)

Because the drain spacing, L, is dependent on the equivalent depth, d, which in turn is a function of, L, equation 4 can not be solved explicitly in terms of L. The use of this equation as a drain spacing formula involves either a trial and error procedure of selecting d and L until both sides

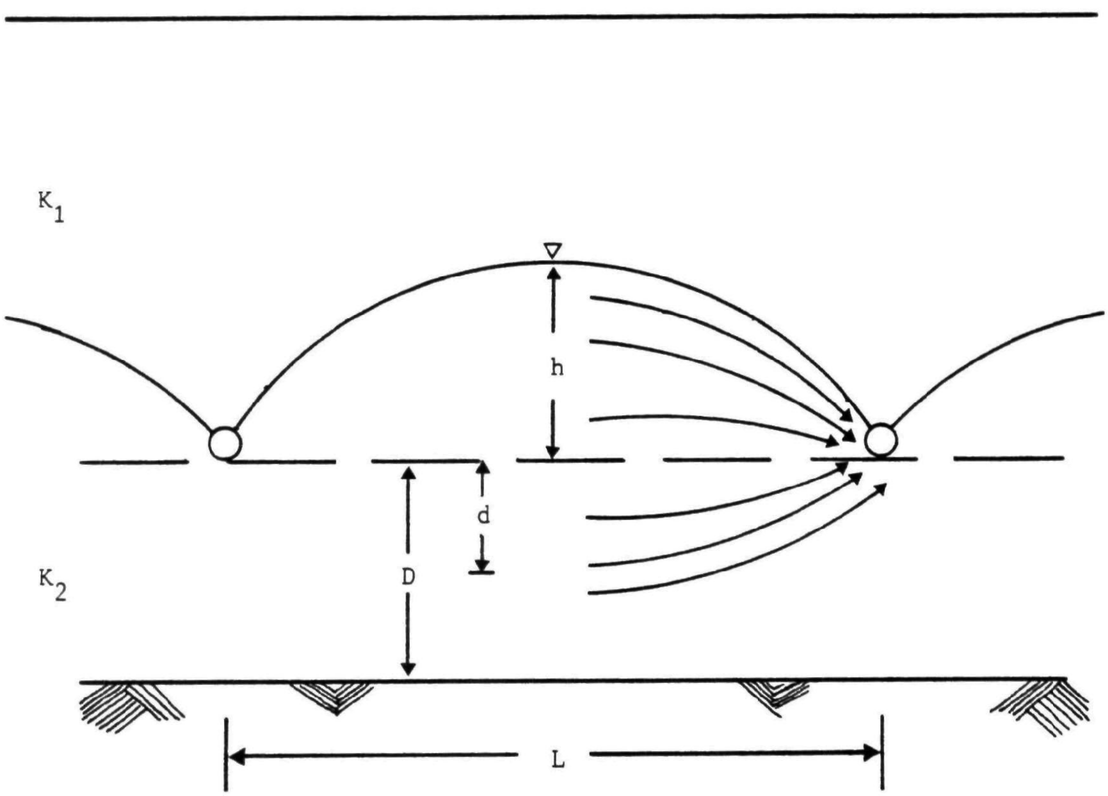

Figure 2-2. Flow to Drains above a Impervious Barrier

of the equation are equal or the use of nomographs which have been developed specifically for equivalent depth and drain spacing. Table 2-1 gives values for the equivalent depth (d) as a function of drain spacing (L) and depth below the drains (D), where r_o, the radius of the drain pipe equals 0.1 meter. Similar tables have been prepared for other values of r_o. For depth (i.e., D valves) greater than 10 meters, the equivalent depth can be calculated from drain spacing using the following equation:

$$d = 0.057 (L) + 0.845 \tag{5}$$

This equation was developed by linear regression form the values given in Table 2-1.

2.1.3 Drains on Sloping Impervious Barriers

Typically, landfill cells are designed so that the compacted base of the cell slopes symmetrically at an angle towards the drains. A cross-section of such a design is shown in Figure 2-3. By designing the drain system on sloping barriers, the flow of water towards the collection system is accelerated thus decreasing the steady-state head maintenance levels. This allows either the drains to be spaced further apart or the heads to be lowered if the other parameters in the equation are held constant.

Head levels and drain spacing can be calculated for landfills designed with an impervious layer sloping towards the drains at an angle of α by the following equation:

$$1/L = (c^{0.5}/2\, h_{max})[(\tan^2\alpha/c) + 1 - (\tan\alpha/c)\,(\tan^2\alpha + c)^{0.5}] \tag{6}$$

where:

$c = q/K$ (dimensionless)
α = slope angle (degrees)

TABLE 2-1. VALUES FOR THE EQUIVALENT DEPTH d OF HOOGHOUDT (r_o = 0.1 m, D and L in m)

D \ L →	5 m	7.5	10	15	20	25	30	35	40	45	50
0.5 m	0.47	0.48	0.49	0.49	0.49	0.50	0.50				
0.75	0.60	0.65	0.69	0.71	0.73	0.74	0.75	0.75	0.75	0.76	0.76
1.00	0.67	0.75	0.80	0.86	0.89	0.91	0.93	0.94	0.96	0.96	0.96
1.25	0.70	0.82	0.89	1.00	1.05	1.09	1.12	1.13	1.14	1.14	1.15
1.50		0.88	0.97	1.11	1.19	1.25	1.28	1.31	1.34	1.35	1.36
1.75		0.91	1.02	1.20	1.30	1.39	1.45	1.49	1.52	1.55	1.57
2.00			1.08	1.28	1.41	1.5	1.57	1.62	1.66	1.70	1.72
2.25			1.13	1.34	1.50	1.69	1.69	1.76	1.81	1.84	1.86
2.50				1.38	1.57	1.69	1.79	1.87	1.94	1.99	2.02
2.75				1.42	1.63	1.76	1.88	1.98	2.05	2.12	2.18
3.00				1.45	1.67	1.83	1.97	2.08	2.16	2.23	2.29
3.25				1.48	1.71	1.88	2.04	2.16	2.26	2.35	2.42
3.50				1.50	1.75	1.93	2.11	2.24	2.35	2.45	2.54
3.75				1.52	1.78	1.97	2.17	2.31	2.44	2.54	2.64
4.00					1.81	2.02	2.22	2.37	2.51	2.62	2.71
4.50					1.85	2.08	2.31	2.50	2.63	2.76	2.87
5.00					1.88	2.15	2.38	2.58	2.75	2.89	3.02
5.50						2.20	2.43	2.65	2.84	3.00	3.15
6.00							2.48	2.70	2.92	3.09	3.26
7.00							2.54	2.81	3.03	3.24	3.43
8.00							2.57	2.85	3.13	3.35	3.56
9.00								2.89	3.18	3.43	3.66
10.00									3.23	3.48	3.74
∞	0.71	0.93	1.14	1.53	1.89	2.24	2.58	2.91	3.24	3.56	3.88

D \ L →	50	75	80	85	90	100	150	200	250
0.5	0.50								
1	0.96	0.97	0.97	0.97	0.98	0.98	0.99	0.99	0.99
2	1.72	1.80	1.82	1.82	1.83	1.85	1.00	1.92	1.94
3	2.29	2.49	2.52	2.54	2.56	2.60	2.72	2.70	2.83
4	2.71	3.04	3.08	3.12	3.16	3.24	3.46	3.58	3.66
5	3.02	3.49	3.55	3.61	3.67	3.78	4.12	4.31	4.43
6	3.23	3.85	3.93	4.00	4.08	4.23	4.70	4.97	5.15
7	3.43	4.14	4.23	4.33	4.42	4.62	5.22	5.57	5.81
8	3.56	4.38	4.49	4.61	4.72	4.95	5.68	6.13	6.43
9	3.66	4.57	4.70	4.82	4.95	5.23	6.09	6.63	7.00
10	3.74	4.74	4.89	5.04	5.18	5.47	6.45	7.09	7.53
12.5		5.02	5.20	5.38	5.56	5.92	7.20	8.06	8.68
15		5.20	5.40	5.60	5.80	6.25	7.77	8.84	9.64
17.5		5.30	5.53	5.76	5.99	6.44	8.20	9.47	10.4
20			5.62	5.87	6.12	6.60	8.54	9.97	11.1
25			5.74	5.96	6.20	6.79	8.99	10.7	12.1
30							9.27	11.3	12.9
35							9.44	11.6	13.4
40								11.8	13.8
45								12.0	13.8
50								12.1	14.3
60									14.6
∞	3.88	5.38	5.76	6.00	6.26	6.82	9.55	12.2	14.7

Source: Wesseling, 1973.

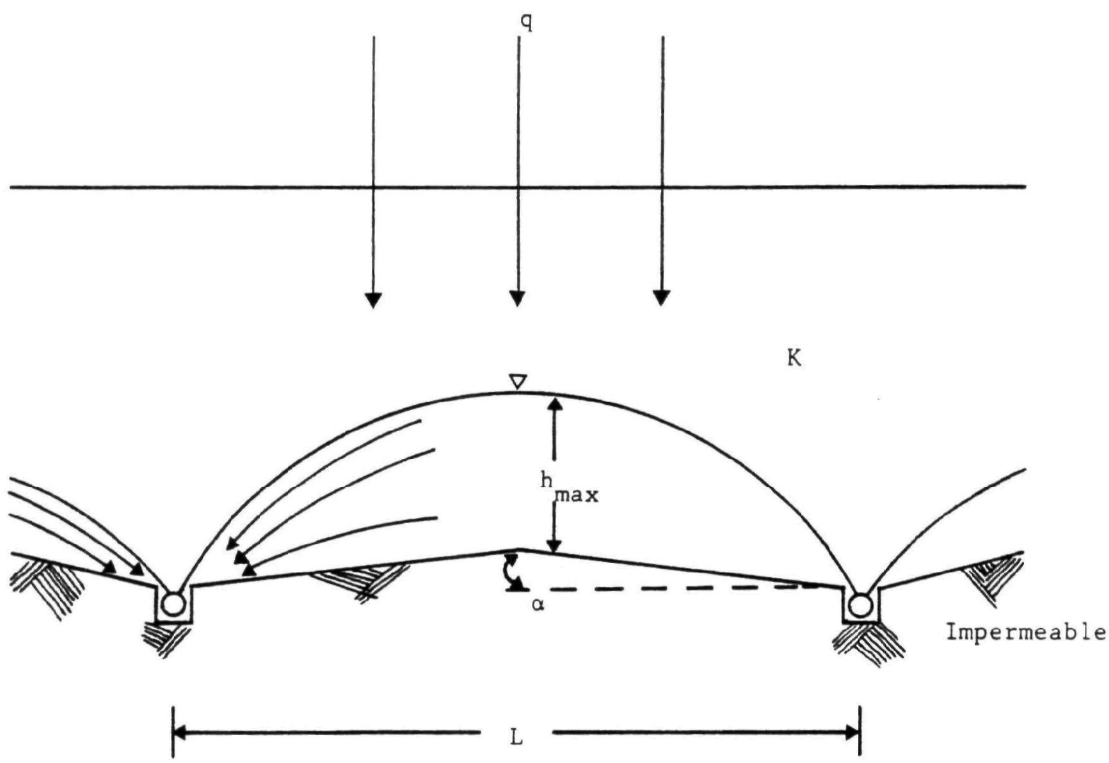

Figure 2-3. Drains on Sloping Impermeable Barriers

As in the previous equations, h_{max} is the hydraulic head above the impervious layer, but this level does not occur midway between the drains. Rather it occurs at some point along the sloping impervious layer.

These drainage systems design equations assume that the drain pipe will accept the drainage water when it arrives at the drainline and that the drain pipe will carry away the water without a buildup in pressure. To meet the second assumption, the pipe size and drain slope must be adequate to carry away the water after it enters the drain pipe. The following sections describe the methods utilized to ensure that these assumptions are valid.

2.2 DRAIN PIPE SIZING

The design diameter of a drain pipe is dependent on the flow rate, the hydraulic gradient, and the roughness coefficient of the pipe. The roughness coefficient, in turn, is a function of the hydraulic resistance of the drain pipe. The formula for the hydraulic design of a drain pipe is based on the Mannings formula for pipes which is:

$$Q_T = (R^{0.67})(i^{0.5})/n \qquad (7)$$

where:

Q_T = design discharge (m^3/sec)
R = hydraulic radius of the pipe (m) which is equal to the wetted cross sectional area divided by the wetted perimeter or is equal to ¼ of the diameter of a full flowing pipe
i = hydraulic gradient (dimensionless)
n = roughness coefficient (dimensionless).

Each of the above factors is described in further detail in the following sections.

2.2.1 Hydraulic Gradient (i) and Roughness Coefficient (n)

Subsurface drains are generally installed on a gradient (i) that is sufficient to result in a non-silting water velocity within the pipe, but is less than the velocity which will cause turbulent flow. Past experience has shown that non-silting velocities are about 1.4 feet per second (Soil Conservation Service, 1973). In situations where silting may be a problem and velocities are less than 1.4 ft/sec, filters and traps can be utilized to prevent the drains from clogging. The minimum hydraulic gradients required to prevent siltation in three sizes of closed pipe are listed in Table 2-2. However, steeper gradients are generally desirable provided they are less than the gradients which would result in turbulent flow.

To prevent turbulent flow, the hydraulic gradients should result in velocities that are less than critical velocities. Table 2-3 gives critical velocities for various drain sizes, gradients, and roughness coefficients. For smooth perforated concrete or plastic pipes, roughness coefficients can be assumed to be equal to 0.013 (Soil Conservation Service, 1973). Knowing the velocity which results in siltation and that which results in turbulent flow, the design engineer can select a gradient which results in a velocity some where between the two extremes.

2.2.2 Discharge (Q)

The design discharge of a pipe, Q_T is equal to the sum of the individual discharges which impinge upon the drain. Figure 2-4 shows the various flow components that could contribute to a drain discharge within a landfill. These flow components can be broken into two major categories-- flow from within the site and flow from the surrounding aquifer.

TABLE 2-2 MINIMUM HYDRAULIC GRADIENTS FOR CLOSED PIPES

Pipe Diameter Inches	Cm	Grade %
4	10.2	0.10
5	12.7	0.07
6	15.2	0.05

Scource: Soil Conservation Service, 1973

TABLE 2-3. DRAIN GRADES FOR SELECTED CRITICAL VELOCITIES

Drain Size Inches	1.4 fps[1]	3.5 fps	VELOCITY 5.0 fps	6.0 fps	7.0 fps	9.0 fps
			Grade - feet per 100 feet			
			For drains with "n" = 0.011[2] Clay Tile, Concrete Tile, and Concrete Pipe (with good alignment)			
4	.28	1.8	3.6	5.1	7.0	11.5
5	.21	1.3	2.7	3.9	5.3	8.7
6	.17	1.0	2.1	3.1	4.1	6.9
8	.11	0.7	1.4	2.1	2.8	4.6
10	.08	0.5	1.1	1.5	2.1	3.5
12	.07	0.4	0.8	1.2	1.6	2.7
			For drains with "n" = 0.013 Clay Tile, Concrete Tile, and Concrete Pipe (with fair alignment)			
4	.41	2.5	5.2	7.5	10.2	16.8
5	.31	1.9	3.9	5.6	7.7	12.7
6	.24	1.5	3.1	4.4	6.0	10.0
8	.17	1.0	2.1	3.0	4.1	6.8
10	.12	.8	1.6	2.2	3.0	5.0
12	.09	.6	1.2	1.8	2.4	3.9
			For drains with "n" = 0.015 Corrugated Plastic Pipe			
4	.53	3.3	6.8	9.8	13.3	21.9
5	.40	2.5	5.1	7.3	9.9	16.6
6	.32	2.0	4.0	5.8	7.9	13.2
8	.21	1.3	2.7	3.9	5.3	8.8
10	.16	1.0	2.0	2.9	4.0	6.6
12	.13	.8	1.6	2.3	3.1	5.1

(1)—Feet per second
(2)—"n" is the roughness coefficient

Source: Soil Conservation Service, 1973.

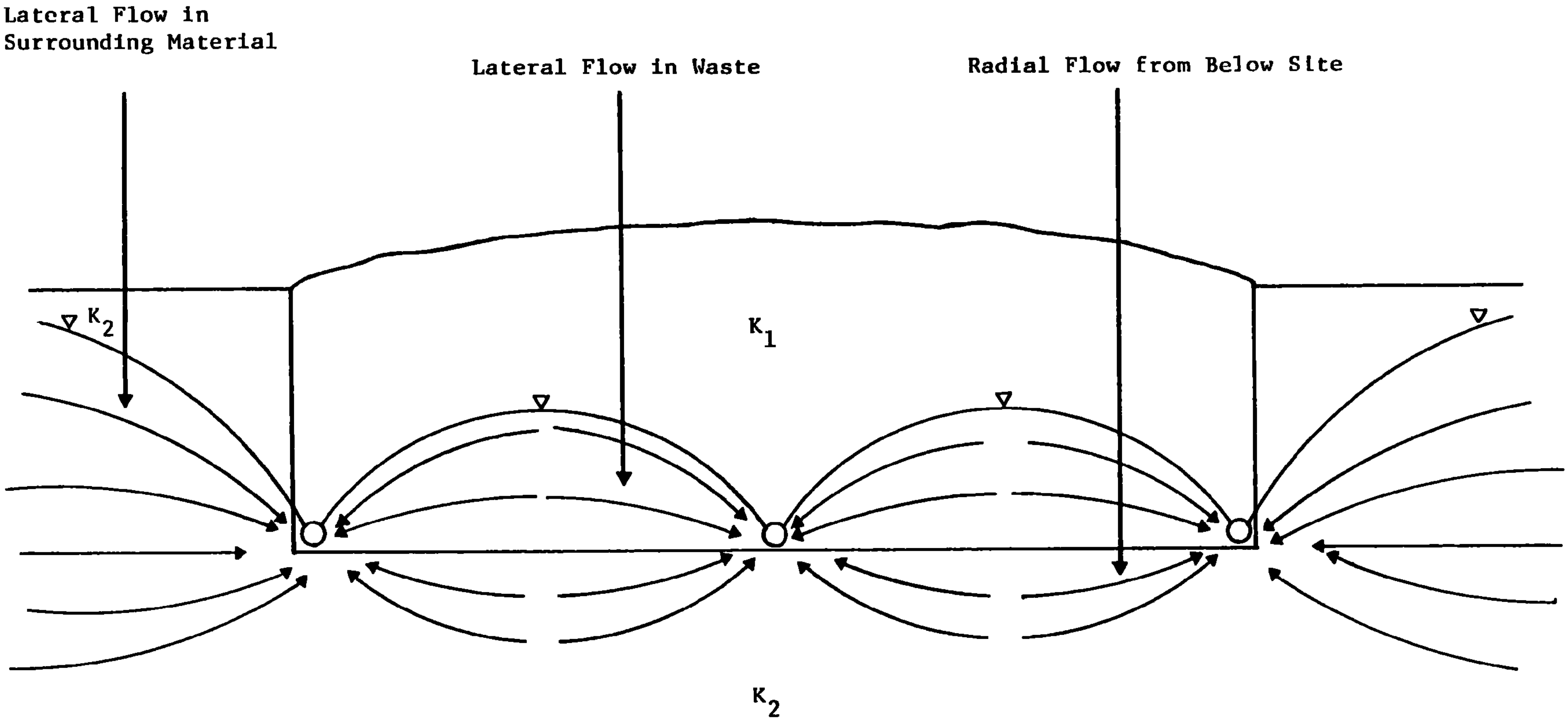

Figure 2-4. Flow Components to a Landfill

In most instances the flow rates are estimated for design purposes so that the drain spacing can be determined using the previously presented equations. Estimates of discharge can be obtained using two simplified methods--the water balance method and the method of fragments. The water balance method is used to calculate the amount of percolation that can recharge the water table between the lines of drains. This flow must be removed to maintain steady stae conditions. A simple water balance equation is as follows:

$$q_p = P-RO-ST-ET \tag{8}$$

where:

q_p = percolation rate: amount of water that must be removed by the drainage system (m/day)
P = precipitation (m/day)
RO = surface water runoff (m/day)
ST = Change in soil moisture storage (m/day)
ET = Evapotranspiration rate (m/day)

Once the percolation rate has been calculated, discharge can be obtained by multipling the percolation rate by the drainage area (i.e. Q_p (m^3/day) = q_p (m/day) Area (m^2)).

When using the water balance method to calculate flow rates or discharges for landfills, the following points should be considered:

- Precipitation values for those time periods with high intensity rainfalls should be used to ensure percolation values are maximized and drainage design is adequate to handle these discharges

- Soil moisture storage changes can be significant as new refuse is placed into the landfill; once field capacity of the materials is attained the ST term can be considered zero o Variations in cover depths and the absence of vegetation can have significant effects on percolation rates; these effects will probably be greatest during the active operational phase when shallow covers are presented and drainage is inadequate.

The method of fragments is used to calculate the flow rates that are derived from the aquifer that broader the outermost drains. This flow component is not considered in the drain spacing equations but could signicantly affect the pipe sizing of the boarder drain. Discharge to the boarder drains can be derived exclusively from horizontal flow or through a combination of horizontal and radial flow. Figure 2-5 shows the division of the boarder drain flow into two fragments that can be calculated separately and summed.

The quantity of flow into fragment 1 (Figure 2-5) can be estimated from the equation:

$$Q_{G1} = K(h^2)x/2(R_d-b) \tag{9}$$

where:

Q_{G1} = pipe discharge (m^3/day)
K = hydraulic conductivity (m/day)
h = height of the water table above the drain (m)
x = length of the drain (m)
R_d = distance of the drain's influence (m)
b = half the width of the drain and the trench (m).

In order to solve the equation, the value of R_d must be known or estimted. Typically the value of R_d is estimated using an equation such as the Sichardt (1940) equation:

$$R_d = 3(h)(K)^{0.5} \tag{10}$$

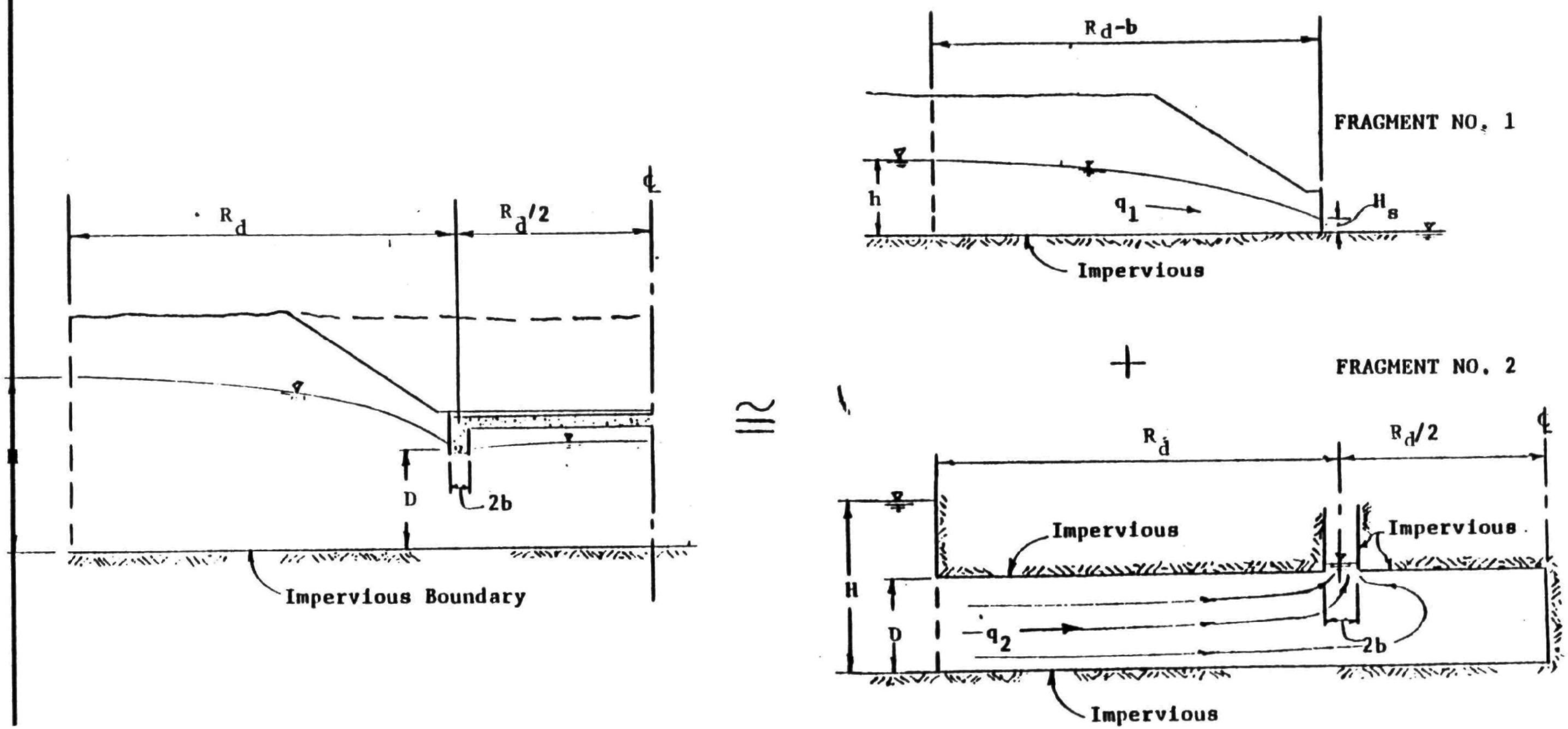

Figure 2-5. **Division of a Symmetrical Drawdown Drain Problem Into Two Equivalent Fragments** (Moulton, 1979).

The quality of flow into fragment 2 can be estimated from the equation:

$$Q_{G2} = (K(H-D)x)/[(R_d/D)-(1/\pi)(\log 0.5(\sinh \pi /D)] \quad (11)$$

where the new terms are:

H = height of the water table above impervious barrier (m)

D = height of the drain above impervious barrier (m)

Once the individual discharges for the segments are calculated, the total discharge is the sum of the individual discharges. For sites on impervious barriers, the total discharge is equal to Q_{G1}; for sites with drains above an impermeable barrier the total discharge is the sum of Q_{G1} plus Q_{G2}.

2.2.3 Pipe Size

Once the total discharge (Q_T) has been determined, an appropriate grade selected, and the appropriate roughness coefficient determined, the minimum drain diameter can be determined. Nomographs such as the one shown in Figure 2-6 are typically utilized to obtain pipe diameters. Because the nomographs are based on the Manning formula (Equation 7), this formula can be used directly to obtain pipe size. Rearranging Equation 7, pipe diameter (D) can be found from:

$$D = 4(Q_T n/i^{0.5})^{1.5} \quad (12)$$

A margin of safety is usually incorporated in the selected drain diameter which will account for the reduction in drain capacity caused by siltation over time and for any discharge in excess of the design capacity. Because the nomographs and Manning formula estimate drain diameter without accounting for a margin of safety, the drain diameter is typically chosen as the next larger size.

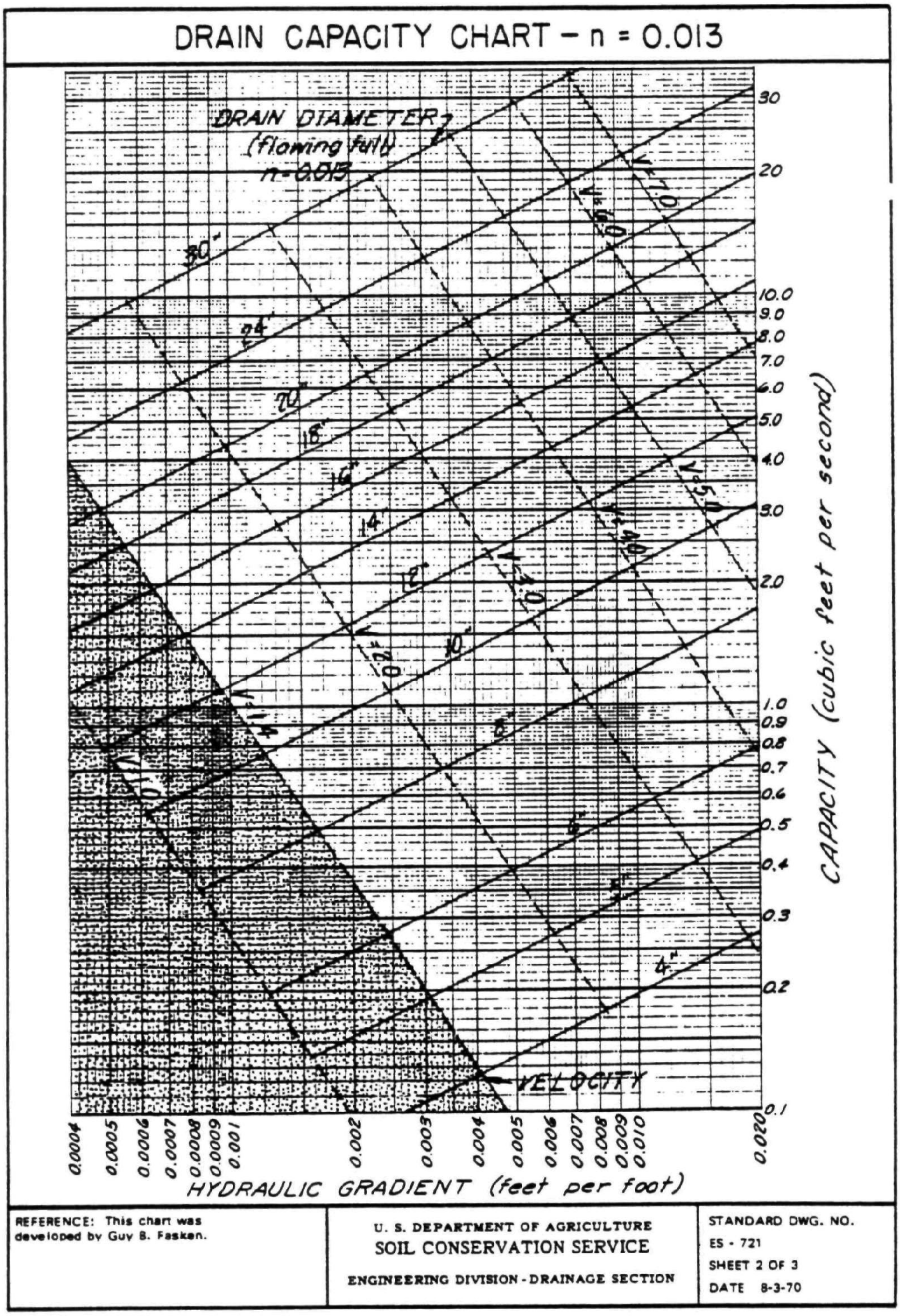

Figure 2-6. Capacity chart - n = 0.013

Source: Soil Conservation Service, 1973.

2.3 FILTERS AND EVELOPES

Performance of a drainage system is based on the assumption that the pipe will accept all inflow without a pressure build up. Filters and envelopes are used to ensure that this requirement is met

2.3.1 Function of Filters and Envelopes

The primary function of a filter is to prevent soil particles from entering and clogging the drain. The function of an envelope is to improve water flow into the drains by providing a material that has a higher permeability than the surrounding soil. Envelopes may also be used to provide suitable bedding for a drain and to stabilize the soil material on which the drain is being placed. The filter's function and the envelope's function are somewhat contradictory, whereas, filtering is best accomplished by fine materials, coarse materials are more appropriate for envelopes.

As water approaches a subsurface drain, the flow velocity increases as a result of convergence towards the perforations or joints in the pipe. This increase in velocity is accompanied by an increase in hydraulic gradient. As a result, the potential for soil particles to move towards the drain is increased. By using a highly permeable envelope material around the pipe, the number of pore connections at the boundary between the soil and the envelope will increase, thereby decreasing the hydraulic gradient.

A filter should prevent the entry of soil particles, which could result in sedimentation and clogging of the drains, blocking of perforation or tile joints, or blocking of the envelope. The filter materials should not, however, be so fine that they prevent all soil particles from passing through. If silts and clays are not permitted to pass through. they may clog the envelope resulting in increased entrace resistance which can cause the water level to rise above the drain.

Although filter and envelopes have different distinct functions, it is possible to meet the requirements of both a filter and an envelope by using well graded sands and gravels. The specifications for granular filters, however, are more rigid than those for envelopes. It is usually necessary for filter materials to be screened and graded to develop the desired gradation curves. Envelope materials, on the other hand, may have a wide range of allowable sizes and gradings (SCS, 1971).

2.3.2 Design of Sand and Gravel Filters

Detailed design procedures are available for both gravel and sand envelopes. SCS (1971) has distinct design criteria for filters and envelopes whereas the Bureau of Reclamation (1979) has developed one set of standards for a well graded envelope which meets the requirements of both a filter and an evelope. The separate SCS design criteria will be considered below for the following reasons:

- o Site specific condtions may warrant the use of only a filter or an envelope, but not both
- o Where both a filter and an envelope are needed, the SCS design criteria for a filter can generally be used
- o It may be desirable to use a fabric filter with a gravel envelope.

The approach recommended by SCS is first to determine whether the drainage system needs a filter and second to determine the need for an envelope. Generally, this sequence is performed because a well graded filter can also function as an envelope.

The general procedure for designing a gravel filter is to: (1) make a mechanical analysis of both the soil and the proposed filter material; (2) compare the two particle distribution curves; and (3) decide by some set of criteria whether the envelope is satisfactory. The Corps of Engineers and the Soil Conservation Service (1973) have adopted similar criteria which set size limits for a filter material based on the size of the base material. These limits are:

$$\frac{\text{50percent grain size of the filter}}{\text{50 percent grain size of the base}} = 12 \text{ to } 58$$

$$\frac{\text{15 percent grain size of the filter}}{\text{15 percent grain size of the base}} = 12 \text{ to } 40$$

Multiplying the 50 percent grain size of the base material by 12 and 58 gives the limits the 50 percent grain size of the filter should fall within. Multiplying the 15 percent grain size of the base material by 12 and 40 gives the limits the 15 percent grain size of the filter should fall within.

All of the filter material should pass the 1.5 inch sieve, 90 percent of the material should pass the 0.75-inch sieve, and not more than 10 percent of the material should pass the No. 60 sieve. The maximum size limitation aids in preventing damage to drains during placement, and the minimum size limitation aids in preventing an excess of fines in the filter which can clog the drain. When the filter and base materials are more or less uniformly graded, a generally safe filter stability ratio of less than 5 is recommended.

$$\frac{\text{15 percent filter grain size}}{\text{85 percent filter grain size}} = \text{less than } 5$$

Consideration must also be given to the relationship between the grain size of the filter and the diameter of the perforations in the pipe. In general, the 85 percent grain size of the filter should be no smaller than on-half the diameter of the perforations. SCS recommends a minimum filter thickness of 8 cm (3 inches) or more for sand and gravel envelopes (Soil Conservation Service, 1973).

2.3.3 Design of Sand and Gravel Envelopes

The first requirement of sand and gravel envelopes is that the envelope have a permeability higher than that of the base material. SCS (1973) generally recommends that all of the envelope material should pass the 1.5-inch sieve, 90 percent should pass the 0.75-inch sieve, and not more than 10 percent should pass the No. 60 sieve (0.25 millimeter). This minimum limitation is the same for filter materials, however, the gradation of the envelope is not important since it is not designed to act as a filter.

The optimum thickness of envelope materials has been a subject of considerable debate. Theoretically, by increasing the diameter of the pipe, the inflow is increased. If the permeable envelope is considered to be an extension of the pipe, then the larger the envelope's thickness the better. There are, however, practical limitations to increasing envelope thickness. The perimeter of the envelope through which flow occurs increases as the first power of the diameter of the envelope, while the amount of the envelope material required increases as the square of the diameter. Doubling the diameter of the envelope (and consequently decreasing the inflow velocity at the soil-envelope interface by half) would require four times the volume of envelope material. Recommendations for drain envelope thickness have been made by various agencies. The Bureau of Reclamation (1978) recommends a minimum thickness of 10 centimeters (4-inch) around the pipe. SCS (1973) recommends an 8 centimeter (3-inch) minimum thickness.

2.3.4 Synthetic Filters

For synthetic materials, the suitability of a filter can be determined from the ratio of the particle size distribution to the pore size of the fabric. The accepted design criterion for geotextile filters is:

$$\frac{\text{P85 (85\% pore size of the filter fabric)}}{\text{D85 (85\% grain size of the subgrade material)}} \leq 1$$

or PB85 $\leq$ D85

Using this equation, the P85 of the filter fabric can be determined from the D85 of the subgrade soil. Manufacturers of geotextile fabrics can then be consulted to select the proper filter type (DuPont, 1981).

3.0 DESIGN AND CONSTRUCTION

This chapter presents an analysis of a hypothetical zone of saturation landfill site that is based on data provided by the State of Wisconsin. A sensitivity analysis was performed for the site (based on the equations presented in Chapter 2) using the following parameters:

- Groundwater leachate generation rates, q
- Hydraulic conductivity of waste materials, K_1 and native soils, K_2
- Head maintenance levels within the site, h
- Drain spacing, L

Also included in this chapter are design and construction considerations which may be incorporated into a zone of saturation landfill.

3.1 HYPOTHETICAL SITE

A hypothetical zone of saturation landfill site was developed using available data from similar sites located in Wisconsin. This site is presented in Figure 3-1. Table 3-1 presents some of the typical ranges of values that may be encountered at a site. The hypothetical site and the accompanying site data are used in the next section as the basis for the sensitivity analysis. Assumptions made to simplify the site conditions include:

- Landfill materials and soils are homogeneous and isotropic
- Water tables within and outside the site are drawndown to drain level
- Groundwater system is at equilibrium (steady state) conditions
- Groundwater pressures in the underlying aquifer do not affect groundwater movement into the bottom of the pit

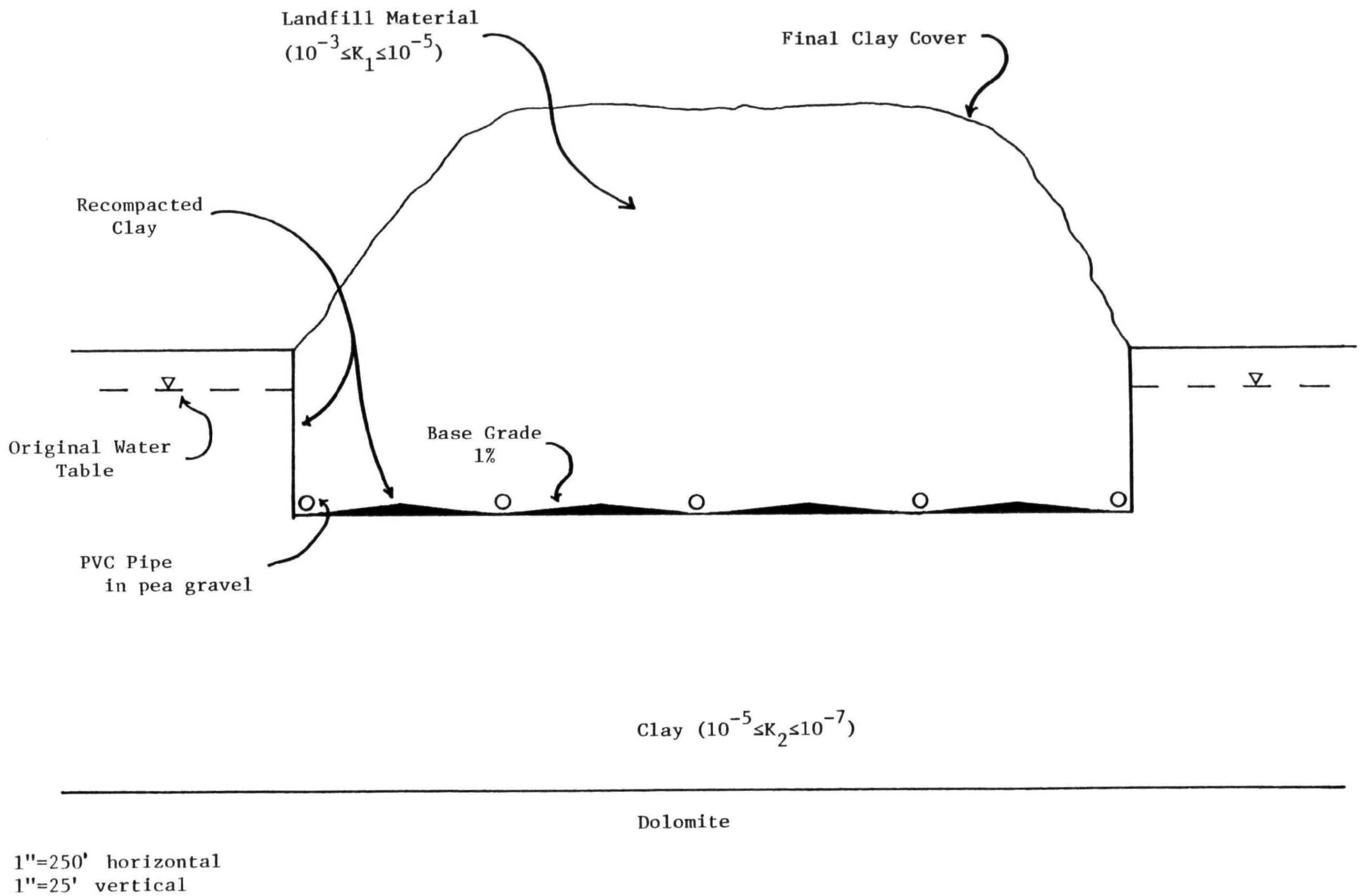

Figure 3-1. Hypothetical Zone of Saturation Landfill

TABLE 3-1 RANGE OF SITE VARIABLES

Parameter	Range of Values
Cell Dimensions	
o Depth of cell	20 to 30 feet average up to 60 feet
o Thickness of compacted clay beneath drains	3 to 5 feet
o Thickness of compacted clay sidewalls	5 feet average
o Thickness of clay till below base grade	20 to 30 feet minimum usually greater than 50 feet
o Drain space	200 to 400 feet
Hydraulic Data	
o Depth to water table	10 feet average
o Permeability of:	
Clay till	10^{-5} to 10^{-7} cm/sec
Refuse	10^{-2} to 10^{-5} cm/sec
Compacted clay	$\geq 10^{-7}$ cm/sec
o Drain diameters	4 to 6 inches
o Gradient of base	1 % average

If these assumptions are invalid, the drain equations will predict drain spacings that are too large. The degree to which these equations over predict will be directly related to the degree the assumption are invalid.

3.2 SENSITIVITY ANALYSIS

Sensitivity analyses were performed on the drain equations for:

- Drains on impermeable barriers
- Drains on sloping impermeable barriers
- Drains above impermeable barriers.

The variables in the sensitivity analyses include head maintenance levels, hydraulic conductivities, flow rates, drain spacing and barrier slope.

3.2.1 Drains on Impermeable Barriers

The equations for drains placed on an impermeable barrier is (Equation 1, Chapter 2):

$$L = (4K_1h^2/q)^{0.5} \qquad (2)$$

When using this equation, a barrier is usually considered impermeable if the barrier material has a permeability (K_2) less than 0.1 of the overlying material (K_1) such that K_1/K_2 is less than or equal to 0.1. This situation typically occurs in landfills that have recompacted clay bases.

Table 3-2 presents the results obtained when equation 2 is solved for drain spacing (L) using various values of flow rates (q), head maintenance levels (h), and hydraulic conductivities (K_1) of the landfill material. A log-log plot of drain length versus flow rate for four hydraulic conductivities (holding the head level constant at one meter) is shown in Figure 3-2. Table 3-2 and Figure 3-2 show how the proper drain spacing

TABLE 3-2. DRAIN LENGTH SPACING (m) FOR DRAINS ON AN IMPERMEABLE BARRIER

q (m/day)	h=6(m)				h=4(m)				h=2(m)				h=1(m)				h=0.5(m)			
	K(cm/sec)																			
	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-2}	10^{-3}	10^{-4}	10^{-5}
0.5	157	50	'6	5	105	33	11	3	52	17	5	2	26	8	3	1	13	4	1	>1
0.01	353	111	35	11	235	74	23	7	118	37	12	4	59	19	6	2	29	9	3	1
0.005	499	157	50	16	333	105	33	11	166	52	17	5	83	26	8	3	42	13	4	1
0.001	1115	353	111	35	744	235	74	23	371	118	37	12	186	59	19	6	93	29	9	3
0 0005	1577	499	157	50	1052	333	105	33	525	166	52	17	263	83	26	8	131	42	13	4
0.0001	3527	1115	353	111	2352	744	235	74	1174	371	118	37	588	186	59	19	294	93	29	9
0 00005	4989	1577	499	157	3328	1052	333	105	1661	525	166	52	831	263	83	26	416	131	42	13

---------- Isobar equal to 122 meters (400 ft): maximum L for theoretical site

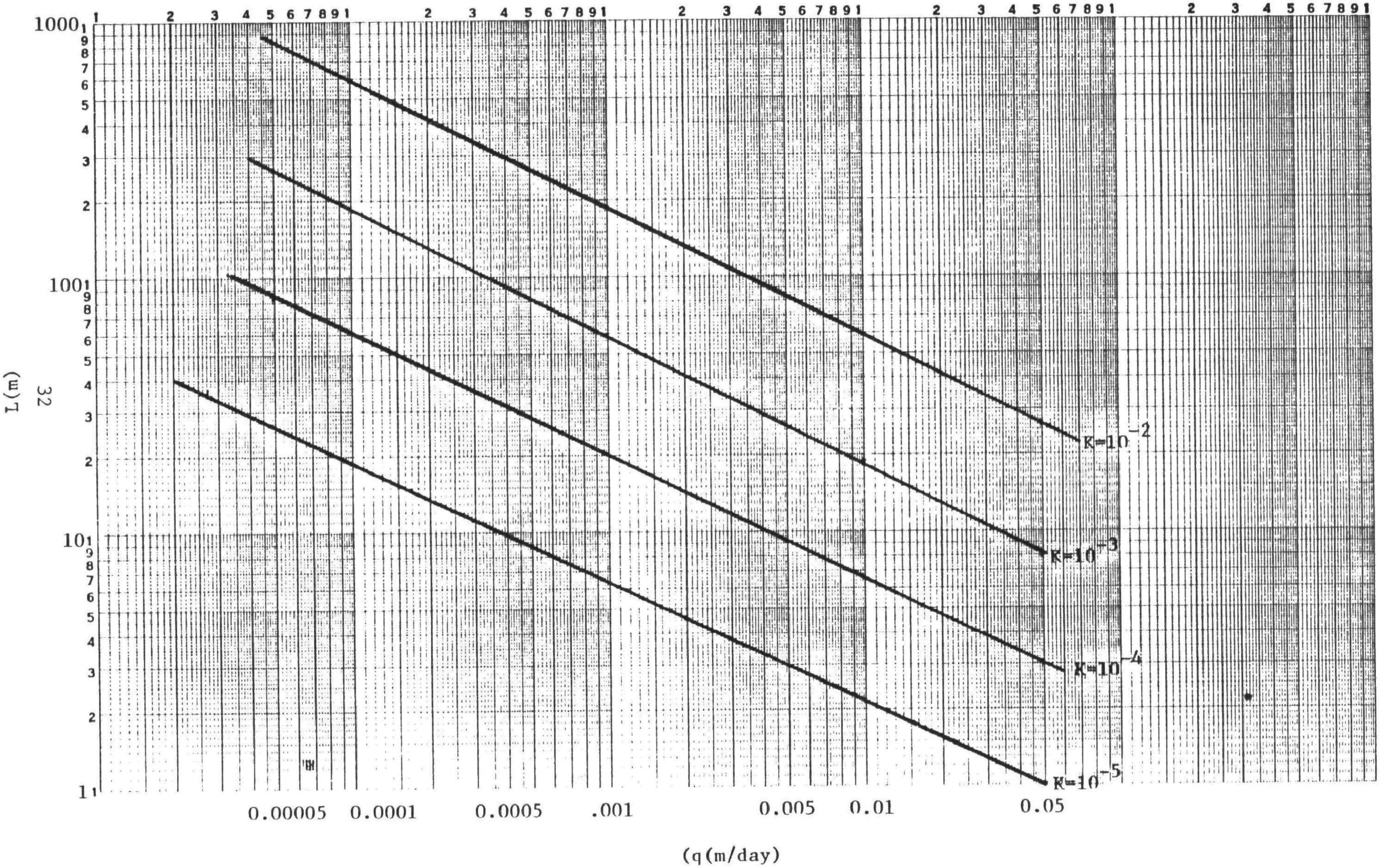

Figure 3-2. Drain Length versus Flow Rates for Head Levels Equal to 1-meter

(L) is directly related to head maintenance levels (h), inversely related to the square root of the leachate generation rate (q), and directly related to the square root of the landfill permeability (K_1).

Leachate generation rate (q) and landfill permeability (K_1) are the most important parameters to determine accurately when designing a landfill not because they are the most sensitive but because they are the hardest to determine with precision. Head maintenance levels are usually predetermined and therefore are not sensitive even though they potentially can have the greatest affect on drain spacing.

One apsect of the equation for drains on impermeable barriers that is not readily apparent is that this equation can also be used for determining depth of drainage blankets. When the equation is utilized for this purpose, the height of the water table is designed to remain within the blanket. Permeabilities of the drainage blanket materials are also generally known with some precision, which is not the case with most landfill materials. Table 3-2 can be used to demonstrate the use of the drain equation for blanket design. For example, if the leachate generation rate, q, is determined to be 0.001 m/day, the landfill material has a hydraulic conductivity, K_1 of 10^{-5} cm/sec, and the maximum head levels are chosen to be 4 meters, the drain spacing required would be 23 meters. By installing a drainage blanket with a depth of 0.5 meter (i.e., also maximum height of head levels) and hydraulic conductivity of 10^{-2} cm/sec the drain spacing can be increased to 93 meters. Installing a drainage blanket not only allows for spacing drains further apart but also reduces the head pressures on the confining layer or compacted base. Reducing these head levels minimizes the possibility for contaminant release from the landfill especially if the landfill is above the water table.

Table 3-2 also shows the combination of flow rates, hydraulic conductivities, and head maintenance levels that yield drain spacings that are equal to or less than the maximum spacing used in the theoretical site (i.e., 400 ft). Generally, when flow rates are large and hydraulic conductivities are low, the theoretical upper limit of 400-feet on drain spacing is too large to accomplish the intended design. It is, therefore, important to quantify the values associated with leachate generation rates and landfill permeabilities to determine drain spacing; drain spacing should not be specified arbitrarily.

3.2.2 Drains on Sloping Impervious Barriers

Drains that are placed on sloping impervious barriers are governed by:

$$1/L = (c^{0.5}/2h)[(\tan^2\alpha/c) + 1 - (\tan\alpha/c)(\tan^2\alpha+c)^{0.5}] \quad (6$$

where:

$$c = q/K_1$$

As discussed in the previous section, the underlying drain material is typically considered impermeable if its permeability is 0.1 of the permeability of the overlying material. This situation is typical of landfills that have compacted clay bases.

Table 3-3 gives solved values of h/L for selected values of $c = q/K_1$ and barrier slope angles, α. Figure 3-3 presents a plot of h/L versus $I = \tan\alpha$ for selected values of $c = q/K_1$. This graph shows how h/L is indirectly related to the barrier slope angle, α, and directly related to $c = q/K_1$. Generally, the greatest decrease in h/L occurs when the barrier angle, α, increases from zero to five degrees (i.e., approximately 10% slope). Angles greater than five degrees cause decreasingly smaller changes in h/L. For design purposes, this means that increasing the angle above five degrees has little affect on head maintenance levels (h) and drain spacing (L).

Using the values of h/L presented in Table 3-3, drain spacing was solved while holding head maintenance levels (h) constant at two meters

TABLE 3-3 VALUES OF h/L FOR VARIOUS $C=q/K_1$ AND ANGLES α

$c=q/K_1$	α (degrees)					
	0	0.5	1	2	3	5
5.787	1.203	1.198	1.194	1.186	1.177	1.161
0.5787	0.380	0.376	0.372	0.364	0.356	0.341
0.05787	0.120	0.116	0.112	0.105	0.101	0.092
0.005787	0.038	0.034	0.031	0.027	0.024	0.022
0.0005787	0.012	0.009	0.008	0.007	0.006	0.006

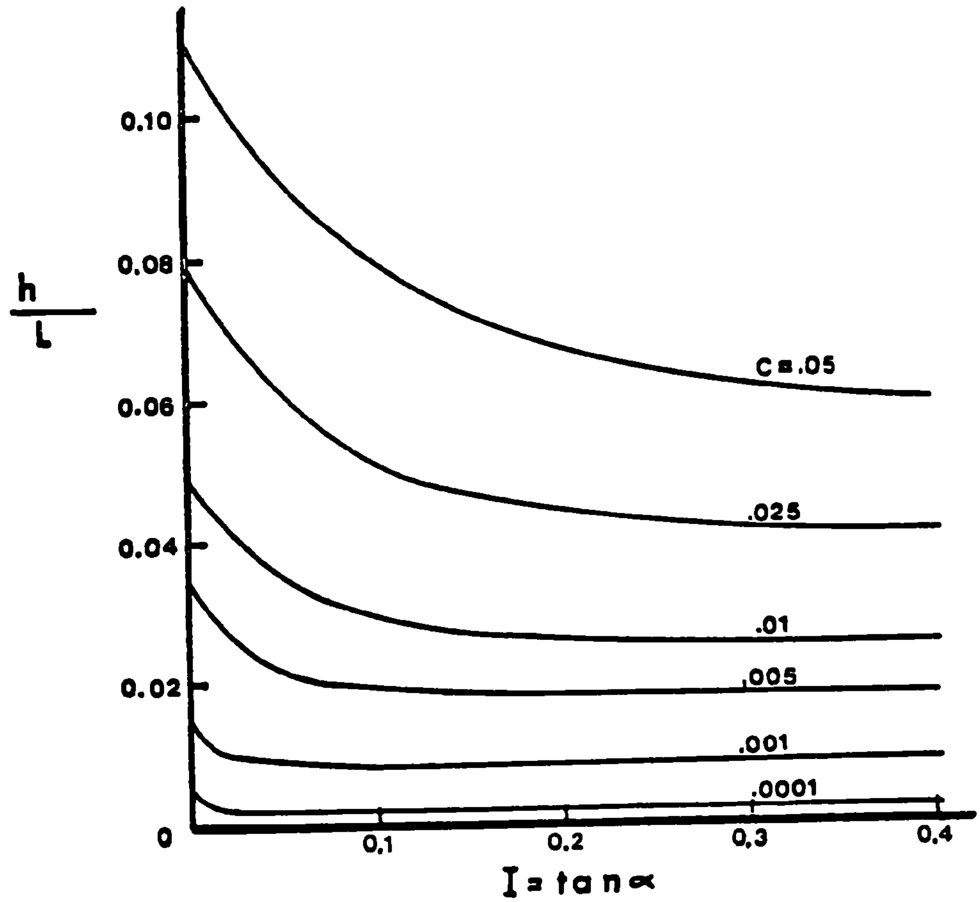

Figure 3-3. Plot of h/L versus I=tan α for drain and sloping impervious layers (Moore, 1980)

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Table 3-4 presents the results of this analysis, and Figure 3-4 shows a plot of drain length versus $c = q/K_1$ for barrier angles of 0°, 1° and 5°. This data shows that if h is held constant, barrier angle, α, has the greatest effect on increasing drain spacing length at lower values of $c = q/K_1$ (i.e., low leachate generation rates divided by high permeabilities).

The equation for drains on an impermeable sloping barrier can also be utilized for determining the thickness of a drain blanket by substituting h for the thickness of the drain blanket (i.e., so that the maintenance head level is designed to be within the blanket) and K_1 for the permeability of the drain blanket. For example, a landfill without a drainage blanket has the following parameters: q = 0.0005 m/day, K_1 = 0.00864 m/day (i.e., 10^{-5} cm/sec), $\alpha = 1^\circ$, and h = 2 meters. Based on these figures, drains would have to be spaced at intervals of L = 18 meters to maintain a 2-meter head level. If a drainage blanket that has a permeability of K_1 = 0.864 m/day (10^{-3} cm/sec) is installed at the site and the values of q and L remain unchanged, the thickness of the blanket and hence the corresponding height of the head levels would be 0.14 meters. This is a subtantial reduction in head levels for a relatively thin drain layer. The advantage to lowering the head level within the site is that less leachate is likely to be released from the site.

3.2.3 Drains Above an Impervious Barrier

The governing equation for a drain system above an impervious barrier is:

$$L = [(8K_2dh+4K_1h^2)/q]^{0.5} \tag{3}$$

Where the parameters are as defined previously in Chapter 2. This equation takes into account radial flow to the drains through the material underlying the drain.

TABLE 3-4 DRAIN SPACING (m) FOR HEAD MAINTENANCE LEVELS OF 2-METERS

$c=q/K_1$	α (degrees)			
	0	1	3	5
5.787	1.66	1.68	1.70	1.72
0.5787	5.26	5.38	5.62	5.87
0.05787	16.67	17.86	19.80	21.74
0.005787	52.63	64.52	83.33	90.91
0.0005787	166.67	250.00	317.40	326.60

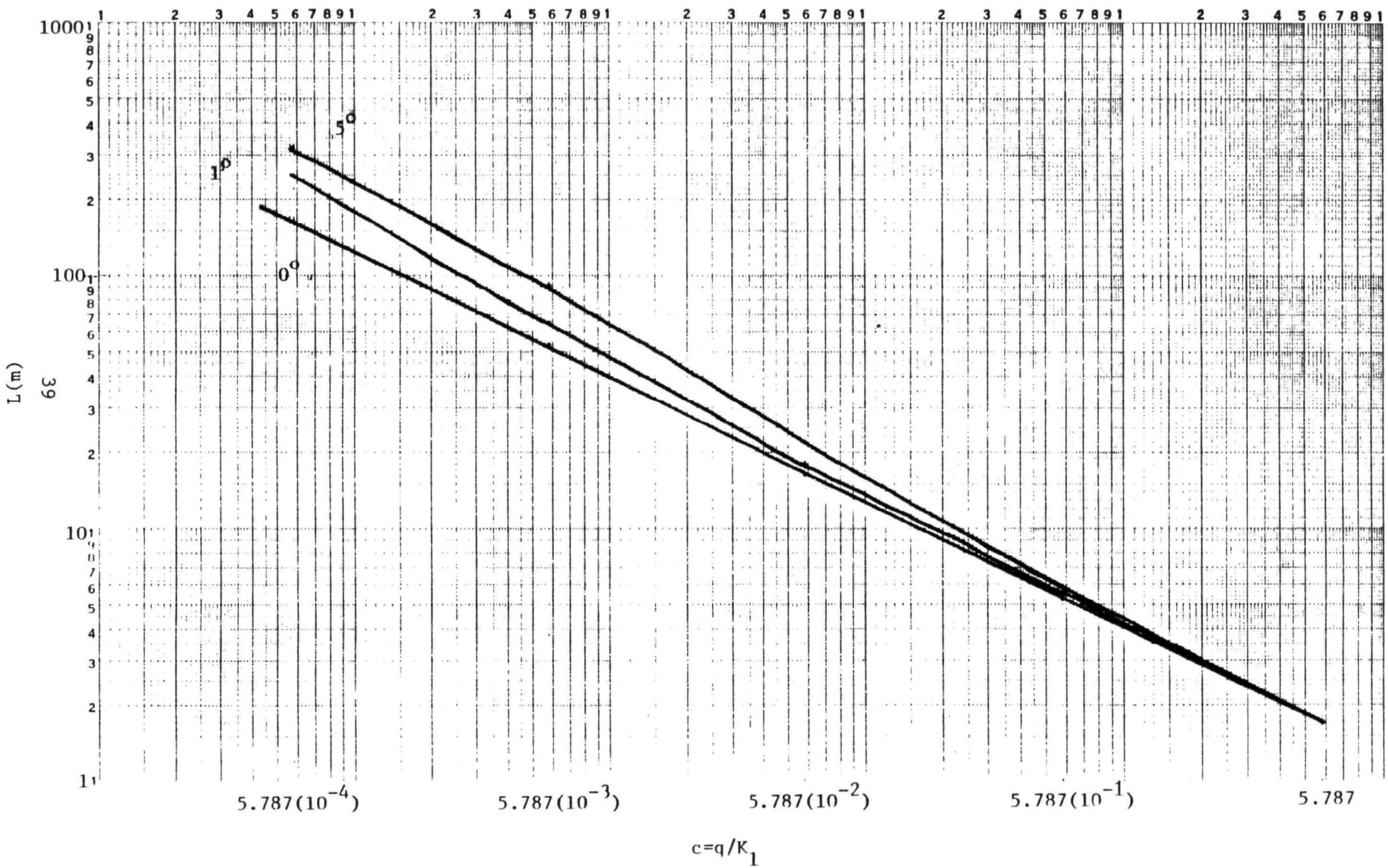

Figure 3-4. Drain Length (L) versus $c=q/K_1$ for α equal to 0^o, 1^o, and 5^o.

Solutions to the equation are shown in Table 3-5 for a head level (h) of two meters, a permeability (K_1) of the landfill material of 0.00864 m/day (i.e., 10^{-5} cm/sec), and a depth to the impermeable layer (D) of 30.4 meters. Figure 3-5 shows a plot of drain spacing (L) versus inflow rates (q) for various permeabilities of underlying material (K_2). These presentations show that when the permeabilities of the overlying (K_1) and the underlying (K_2) material are the same, the drain spacing (L) increases. This phenomenon is caused by the introduction of radial flow to the drains rather than straight lateral flow.

When the permeability of the underlying material (K_2) is an order of magnitude (0.1) less than the overlying material (K_1), the calculated drain spacing does not differ significantly from a drain on an impermeable barrier. If the underlying material has a permeability that is two orders of magnitude (0.01) less than the overlying material, the drain spacing is identical to a drain on an impermeable barrier. This occurs because the term $8K_2dh$ does not significantly impact the results of the drain spacing equation. Consequently, the "rule of thumb" for designing drains is that if the underlying layer has a permeability of 0.1 of the overlying material, the underlying material can be considered impermeable.

3.3 APPLICATION

The basic premise behind the use of a landfill located in the zone of saturation is that if the head maintenance levels within the site are less than the groundwater levels outside the site, then hydraulic gradients should be into the site thus minimizing the likelihood of contaminant release. Basic hydraulic principles show this to be the case as long as (1) the landfill materials and soils are homogeneous and istropic, (2) water tables are drawn down to drain levels, and (3) the groundwater system is at equilibrium. In reality these conditions rarely exist at a landfill site. For these situations, the Wong (1977) model can be used to predict releases.

TABLE 3-5 DRAIN SPACING (m) FOR DRAINS ABOVE AN IMPERMEABLE BARRIER (h=2m, K_1=0.00864 m/day (10^{-5}cm/sec), D=30.4m)

q (m/day)	K_2 (cm/sec)			
	10^{-5}	10^{-6}	10^{-7}	Impermeable
0.05	2	2	2	2
0.01	5	4	4	4
0.005	7	5	5	5
0.001	19	12	12	12
0.0005	31	18	17	17
0.0001	114	43	37	37
0.00005	212	64	52	52

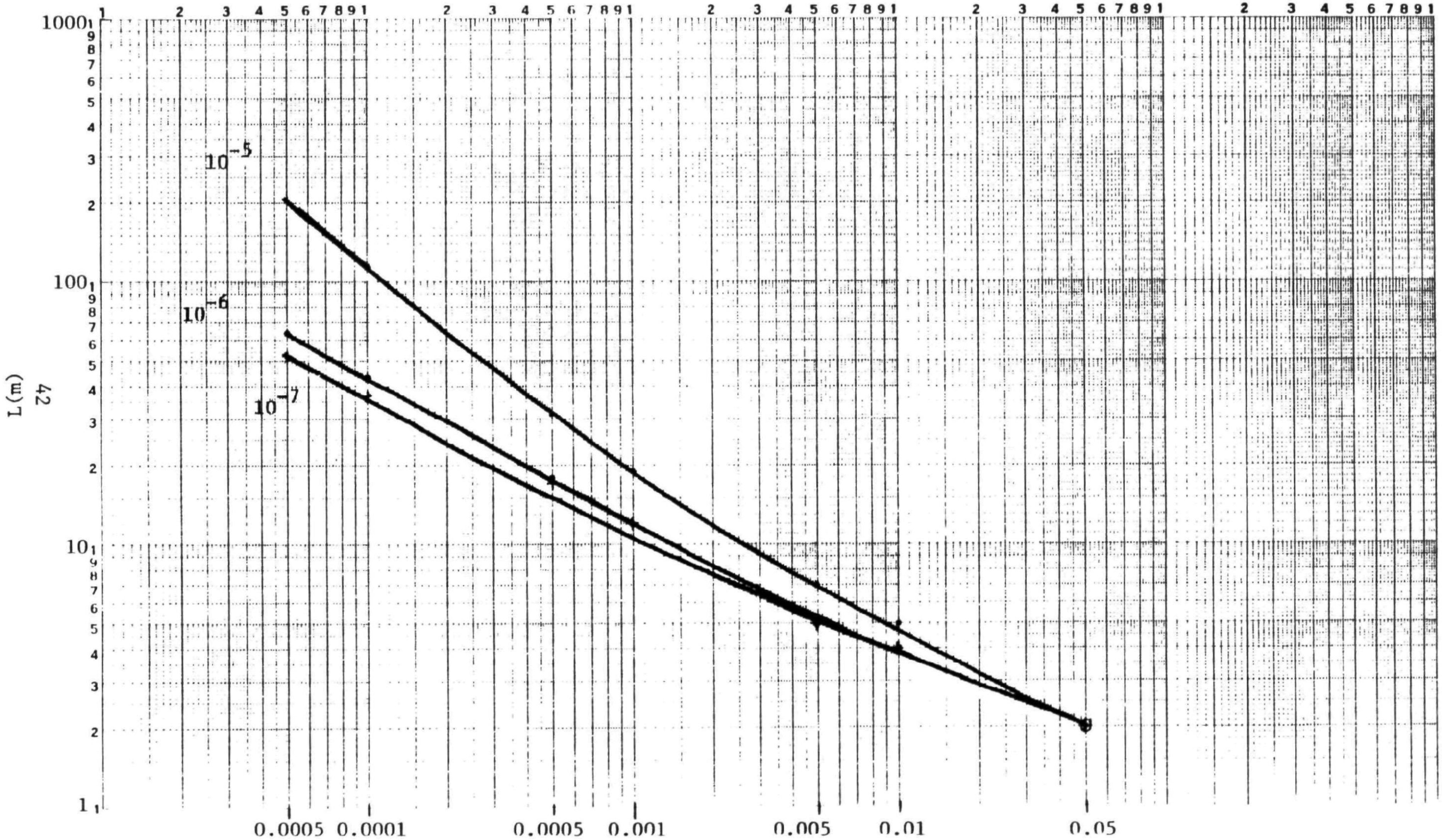

Figure 3-5. Drain Spacing (L) versus inflow rates (q) where h=2 meters and $K_1=10^{-5}$ cm/sec.

3.3.1 Parameter Estimation

In order to utilize the drain spacing equations, the input parameters must be known with some accuracy. The two parameters that are the hardest to obtain accurate estimates of are leachate generation rates, q, and hydraulic conductivities, K. Head maintenance levels, h, and barrier slope angles, α, are typically chosen by the designer and are not estimated.

Total inflow rates to the drains can be estimated through the use of water balance equations (equation 8) and the method of fragments (equations 9 and 10). These equations however do not take into account the volume of liquid that is added to the site as landfill material (e.g., paper waste sludges). Estimating this volume is very specific to a landfill. Once the total volume is estimated, the value should probably be increased to take into account variations that were not anticipated (e.g., acceptance of more liquids, unseasonally high precipitation) and to add a margin of safety to the design.

Determining the hydraulic conductivity of the landfill material with any accuracy is very difficult because typically, the material is a mixture of wastes and daily covers tend to create discrete cells. Waste mixtures tend to cause the landfill to be very heterogeneous and anisotropoic making estimations of permeabilities difficult. Here as before, the hydraulic conductivity selected should be the lowest conductivity for the waste materials. The problems associated with daily cover can be minimized if the cover is removed each day, allowing old and new fill materials to be hydrologically connected.

The use of drainage blankets in a landfill effectively eliminates the problem associated with determining conductivities. Hydraulic conductivities for the drainage blanket can be determined easily in the lab and most blanket materials are homogeneous and isotropic. A blanket layer covering the bottom of the cell and the sidewalls provides a hydraulic connection between the leachate generated and the drains. A drainage blanket also aids in the prevention of leachate pooling, accelerates leachate removal times, and allows for lower head maintenance levels with the same drain spacing.

3.3.2 Landfill Size

The limits of depth, width, and length of a landfill in the zone of saturation appeared to be of concern when landfills are designed. Technically a landfill is not depth limited in a zone of saturation setting because theory shows that the lower the base grade of the fill the higher the hydraulic gradients will be into the site. These higher gradients will result in larger quantities of groundwater that must be removed from the site. Problems can arise, however, if the base grades are extended deep enough to cause quick conditions in the base or if the base intercepts an aquifer that has a lower head level than the overlying aquifer. If quick conditions occur, extensive dewatering may be necessary to maintain a stable base. If the base extends into an underlying aquifer with a lower head level, contamination of the aquifer is likely when the pit is filled. The areal extent of the landfill is generally not considered limiting if the drainage system is functioning properly.

3.3.3 Example Problem

An example of the use of the drain equations for the hypotetical site developed in Section 3.2 is present here for illustrative purposes. Table 3-6 presents the data utilized in the example. For the example, the recompacted clay base and sidewalls were assumed to have the same permeabilities as the saturated clay aquifer.

To calculate the drain spacing needed at the site, the equation for drains on sloping barriers (equation 6) can be used because the underlying clay material has a permeability of 0.01 of the fill material. Solving equation 6 for drain spacing (L):

$$1/L = (c^{0.5}/2h)[(\tan^2\alpha/c) + 1 - (\tan\alpha/c)(\tan^2\alpha+c)^{0.5}]$$

TABLE 3-6 EXAMPLE DATA SET

Parameter	Value
Cell Dimensions	
o Width	1000 ft (304m)
o Length	1000 ft (304m)
o Depth	30 ft (9.1m)
Clay Thickness Below Base	50 ft (15.2m)
Hydraulic Conductivities	
o Fill (K_1)	10^{-5} cm/sec (0.00864 m/day)
o Clay (K_2)	10^{-7} cm/sec (0.0000864 m/day)
- compacted or natural	
Base Grade	1% (0.6 degrees)
Water Table (b.l.s.)	10 ft (3.0m)
Head Maintenance Levels	10 ft (3.0m)
Inflow (q) From Percolation	10 in/yr (0.0007 m/day)

where:

$$c = q/K_1$$
$$= 0.0007(m/day)/0.00864(m/day)$$
$$= 0.08102$$

$$1/L = (0.08102^{0.5}/2(3.0))[(\tan^2 0.6/0.08102)+1-(\tan 0.6/0.08102)(\tan^2 0.6+0.008102)^{0.5}]$$

$$1/L = (0.04744)[(0.00135)+1-(0.12926)(0.28483)]$$

$$1/L = 0.0458$$

$$1/L = 21.85m(71.89\ ft)$$

The value obtained for drain spacing using these parameters is significantly less than the currently specified allowable upper limit (i.e., 200 to 400 ft). This solution would not be considered conservative for an actual site if (1) conductivities of the fill were not a uniform 10^{-5} cm/sec, (2) inflow rates were higher than expected such as when the site is not capped, and (3) materials with high water content are landfilled.

An appropriate pipe size can be selected by calculating the system's expected discharge and considering the factors described in Section 2. Total flow along any single collector pipe can be calculated by:

$$Q_1 = qA$$

where q, inflow, is 0.0007 m/day and A, area, is 22m x 304m (drain spacing times length)

$$Q_1 = (0.0007\ m/day)(22m)(304m)$$
$$= 4.7\ m^3/day\ (0.002\ ft^3/sec)$$

Using Figure 2-5 for plastic pipes with a roughness coefficient of n = 0.013 and a design velocity of 1.4 ft/sec, a 4-inch drian pipe should be sufficient to handle the discharge. Total discharge from the main header pipe is:

$$
\begin{aligned}
Q_T &= qA \\
&= (0.0007 \text{ m/day})(304\text{m})(304\text{m}) \\
&= 64.7 \text{ m/day } (0.03 \text{ ft}^3\text{/sec or } 1241 \text{ gal/day})
\end{aligned}
$$

Using Figure 2-5 again, A 4-inch pipe should be sufficient to handle the flows. A larger size pipe may be selected as a margin of safety.

3.4 CONSTRUCTION ASPECTS

Previous discussions related the application of zone of saturation landfill design criteria for leachate collection systems to a hypothetical site. Some aspects of construction of these sites are important but do not fall within the focus of the other sections. These considerations are described in the following sections and include:

- Construction inspection
- Collector drain maintenance
- Future operating conditons

3.4.1 Construction Inspection

Construction inspections are important to ensure the specified design criteria are implemented in the field. Some criteria which are important for zone of saturation landfills are:

- Moisture of clays when compacted on bases and sidewalls
- Placement of drain pipe and filter envelopes, and their protection while exposed
- Compliance with specified design criteria for slopes, and drain blanket thickness and gradation

The clay tills associated with the zone of saturation landfills can easily exceed the moisture content at which maximum density can be achieved. Trying to compact these excessively wet clays on the base and sidewalls of the landfill is a wasted effort, and in many instances, causes the soil to shear. Shearing of the clays may result in planes of weakeness and cracks throughout the layer which can easily transmit leachate out of the cell. Construction inspections can be used to identify when the soils exceed the optimum moisture content for compaction and prevent their placement until the clays have dried to within an acceptable moisture content range.

Construction inspections should be performed during drain pipe and filter envelope placement. Inspections would ensure that these operations are performed as specified in the designs and that:

- Drain pipes are not damaged during placement
- Filter materials are kept free of fines and are graded and installed properly
- Drains are protected after placement so that fines do not clog drains before wastes are placed

Slopes specified within a cell are generally just enough to prevent pooling of leachate on the compacted clay base (i.e., 1% slope). With these gentle slopes, there is a very small margin for error that would still ensure that leachate moves freely towards the drain. Inspections performed during the grading of the base would ensure proper construction. Inspections should also be performed during the placement of drain blanket to ensure adequate blanket thickness and proper gradiation of the blanket material.

3.4.2 Drain System Maintenance

Proper maintenance of a drain system is a critical element in ensuring its continued performance. The initial design of the system must allow for adequate access to the drain system components both for inspections and cleanings. Regular maintenance inspections should be performed to assess the system's performance and to plan cleaning and repairing activities. Additionally, maintenance inspections should be performed when an unexplained reduction in flow occurs to sumps or increased head levels are observed within the cell.

3.4.3 Future Operating Conditions

Landfill designs should anticipate and account for the type of waste disposal operations planned. As mentioned previously, conductivities of the wastes placed in a cell are critical to the design of the leachate collection system. Conductivities used for design purposes are frequently estimates based on the experience of the designer or are averages that represent the range of conductivities that could be found throughout the cell. If the landfill will be accepting balings, shreddings, large solid objects, or separations of particular wastes during its operational life, the conductivities should be adjusted accordingly.

For zone of saturation landfills the total quantity of water that must be extracted can vary widely. Waters can be derived from percolating rainfall, groundwater intrusion, and from sludges or liquids brought into the site. Underestimating the amount of water that must be removed will result in rising interval head levels and the eventual release of generated leachate. Therefore, care must be exercised to ensure that design water removal capacities realistically represent the eventual landfill operations.

4.0 FLOW NET ANALYSIS

4.1 Introduction

Water flow through porous media is governed by several physical relationships (e.g., Darcy's Law) that can also be represented mathematically. The equations used to represent this flow can be arranged so that they apply to certain physical conditions. The solutions to the equations can then be obtained through numerical or graphical means; the graphical solutions are called flow nets. In this case, the flow nets are graphical representations of two-dimensional equations of continuity for water flow.

The generation and application of a flow net requires certain assumptions and simplifications. Most of these are common to any representation (model) of a natural physical system and include the assumptions of aquifer homogeneity, water incompressibility and laminar Darcian flow. Consideration of site conditions at each of the three landfills being studied reveals several features which are common to all, so that the flow nets can be constructed for a generic site that incorporates these common features (Figure 4-1) and also includes some simplifications. These include:

a) elimination of small heterogeneities in the clay around the site, since they are discontinuous and not well-mapped.

b) treatment of the underlying dolomite as an impermeable boundary. It is recognized that recharge to the clay probably does occur from the dolomite; however, the vertical permeability of the clay is low enough that this recharge can be considered negligible for the purposes of constructing a flow net.

c) treatment of all landfill surfaces as orthogonal to ground surface.

d) assuming that the landfill excavation can be treated as a circular well so that water flows radially to it.

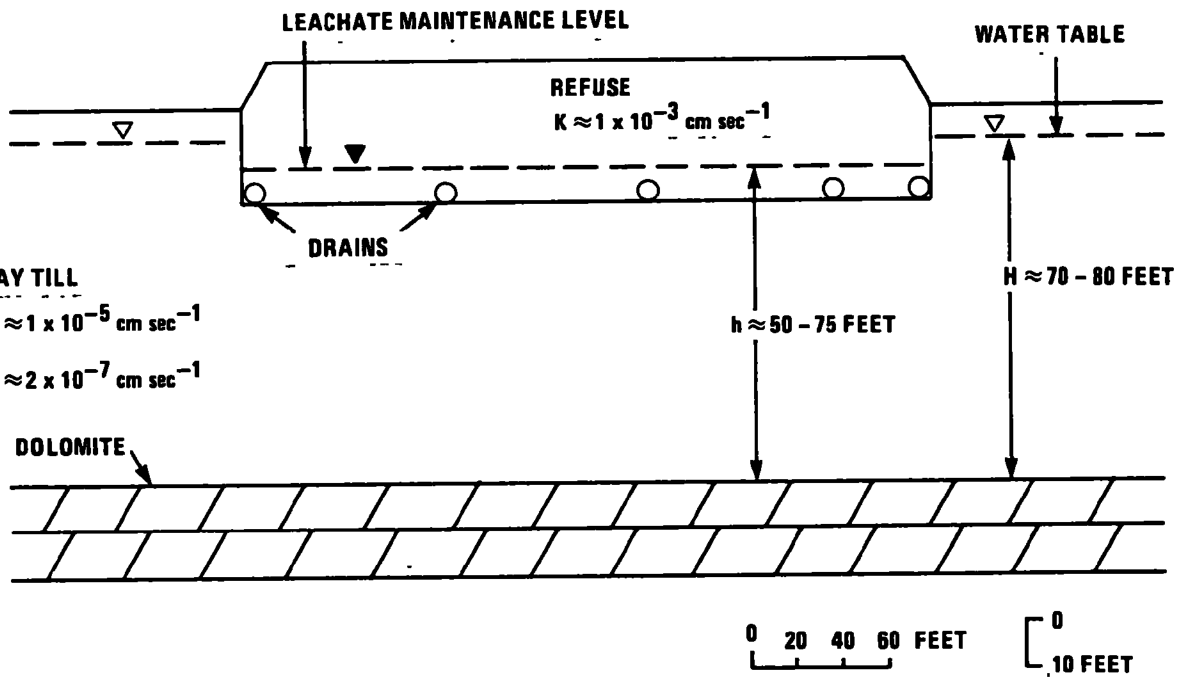

FIGURE 4-1. CROSS SECTION OF TYPICAL ZONE-OF-SATURATION LANDFILL

e) assuming that the water table aquifer can be treated as a line source of seepage beyond 250 feet from the landfill.

The flow nets are composed of two families of lines that intersect at right angles. Flow lines show paths along which water can flow; equipotential lines represent lines of equal hydraulic head. The flow nets constructed for the generic site indicate the approximate arrangement of flow lines and equipotential lines around the landfill for different physical conditions, as explained below.

4.2 Initial Conditions

Three points need to be made about the flow nets before examination. First, by treating the excavation as a circular well, some idea of the radius of influence (R_o) and equivalent well radius (R_w) can be obtained. These are defined by:

$$R_w = \sqrt{\frac{LW}{\pi}}$$ where L = length of landfill
W = width of landfill

Values of L and W were chosen equal to 300 ft. so that R_w = 169 ft.

$$R_o = C\Delta h\sqrt{\frac{K}{10^{-4}}}$$ where C = a constant ranging from 1.5 to 3
Δh = drawdown expected in "well"
K = hydraulic conductivity in $cm\text{-}sec^{-1}$

Values chosen for Δh (30 feet) and K (1×10^{-5} cm sec^{-1}) represent typical values for these parameters at each site. Several references used suggested setting C = 2.0 for typical results. This gives R_o = 19 feet.

Second, there is a difference between the horizontal and vertical hydraulic conductivities at these sites because of the layered nature of the till deposits. Average values for horizontal K and vertical K used here were 1×10^{-5} cm sec^{-1} and 2×10^{-7} cm sec^{-1}, respectively. Construction of flow nets for anisotropic media requires that the horizontal and/or vertical dimensions be transformed to offset the effect of different horizontal and vertical hydraulic conductivities.

The factor used is $\sqrt{K_V/K_H}$, which resulted in a vertical exaggeration of approximately 7 times in the cross sections which have not been "re-transformed". This exaggeration must be kept in mind when interpreting the flow nets.

Third, during the construction of extremely accurate flow nets, each four-sided figure developed by the intersection of the two families of lines should approximate a square or curvilinear square. This convention has been attempted in these sketches, but some variances were allowed to arrive at reasonable figures without an undue expenditure of time.

The plan view flow net (Figure 4-2) indicates nearly symmetrical inflow, as would be expected from any roughly square excavation in an aquifer acting as a line source of seepage on all sides. Due to the low hydraulic conductivity, the amount of inflow is relatively small. This can be approximated (using the parameters given in Figure 1) by:

$$Q = \frac{\pi K(H^2-h^2)}{\ln(R/r_w)}$$

where H = total saturated thickness of clay = 75 feet

h = saturated thickness beneath "well" = 65 feet

R = radius of influence = 188 feet

r_w = effective radius of "well" = 169 feet

K = hydraulic conductivity in ft-sec^{-1} = 3.3×10^{-7}

$= 1.4 \times 10^{-2}$ cfs (about 6 gpm)

However, this formula assumes that the radius of the well is small (not the case here) so that inflow through the landfill floor has been underestimated. This may be estimated by:

$$Q = K_v iA$$

where K_v = vertical hydraulic conductivity $= 6.6 \times 10^{-9}$ ft sec^{-1}

i = hydraulic gradient = 10

A = surface area = 90,000 ft^2

$= 5.9 \times 10^{-3}$ cfs (about 2.7 gpm)

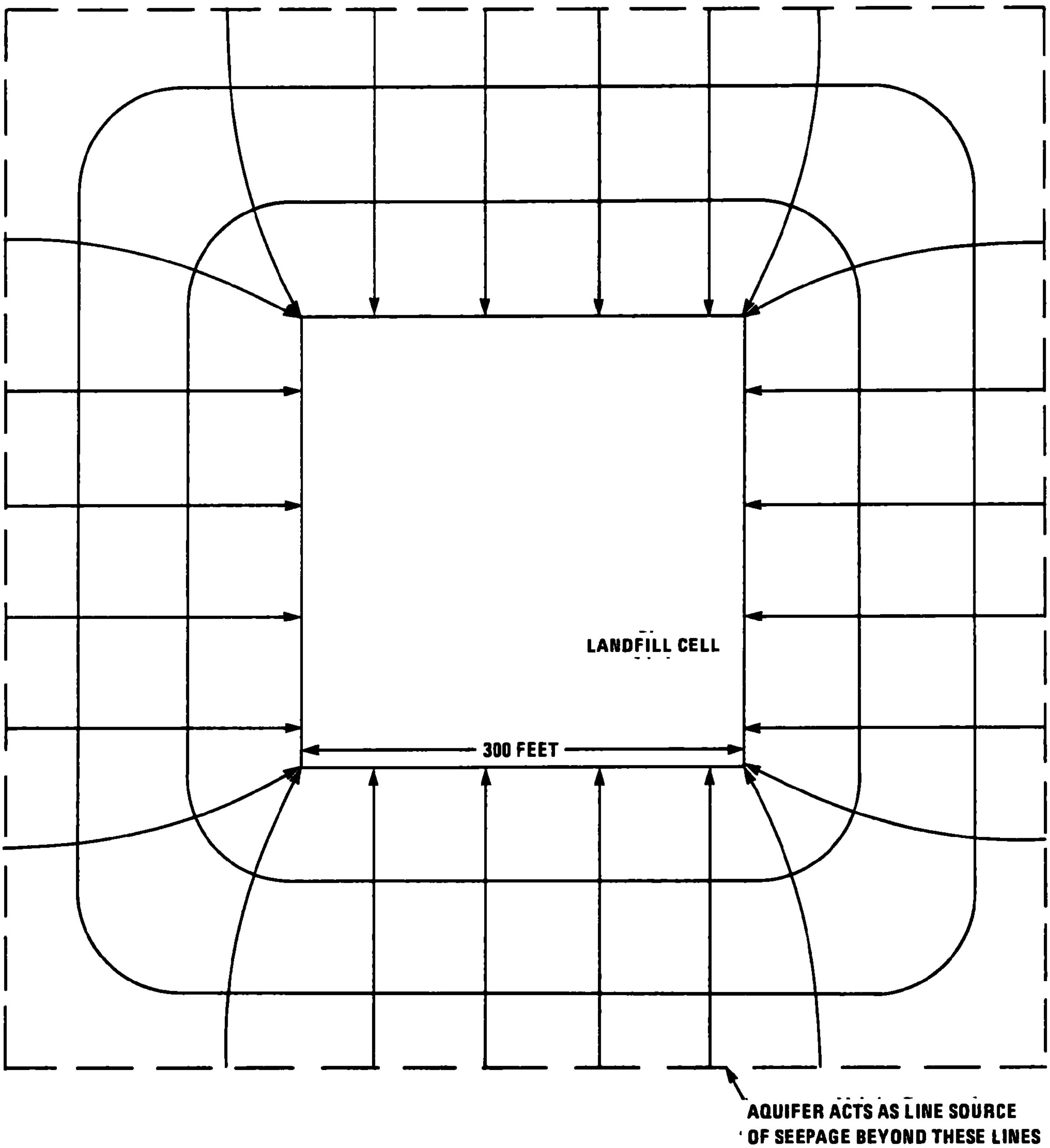

FIGURE 4-2. INFLOW WITH TYPICAL LANDFILL CELL (PLAN VIEW)

These values are for a 10 - foot head difference and would be reduced by about half if the head difference were reduced to 5 feet.

4.3 Parameter Analysis

Examination of the effect of leachate maintenance levels (Figures 4-3A, B, C) indicate that the most favorable situation in terms of collecting lower amounts of leachate is when the maintenance level is relatively high. This causes redistribution of the equipotential lines in the aquifer around the drains such that the contribution of inflow from the aquifer is small compared to that from the leachate. However, this does not leave much of a safety margin and also enhances the susceptibility for outward leackage along permeable sandy lenses in the sides of the landfill.

Analysis of the effect of vertical gradients (Figures 4-4A, B) indicates that, in comparison to sidewall inflow and leachate from the refuse, the amount of inflow at the base will be small regardless of the direction of gradient. This is predominantly due to the low vertical hydraulic conductivity of the clay till in contrast to the more permeable sidewalls.

Analysis of the effect of drain spacing (Figure 4-5) shows that the drains tend to depress equipotential lines between their centers, bringing higher head levels closer to the bottom of the excavation. In the presence of a local downward gradient, this might enhance the possibility of diffusion of contaminants through the base of the fill, although most movement would be horizontal because of aquifer anisotropy. As the number of drains increases, this effect becomes less noticeable, and at some point would be offset by inflow through the base of the excavation.

The effect of the depth of excavation below the water table can also be examined by using some of the assumptions and site dimensions given earlier and treating the excavation as a well. If the leachate

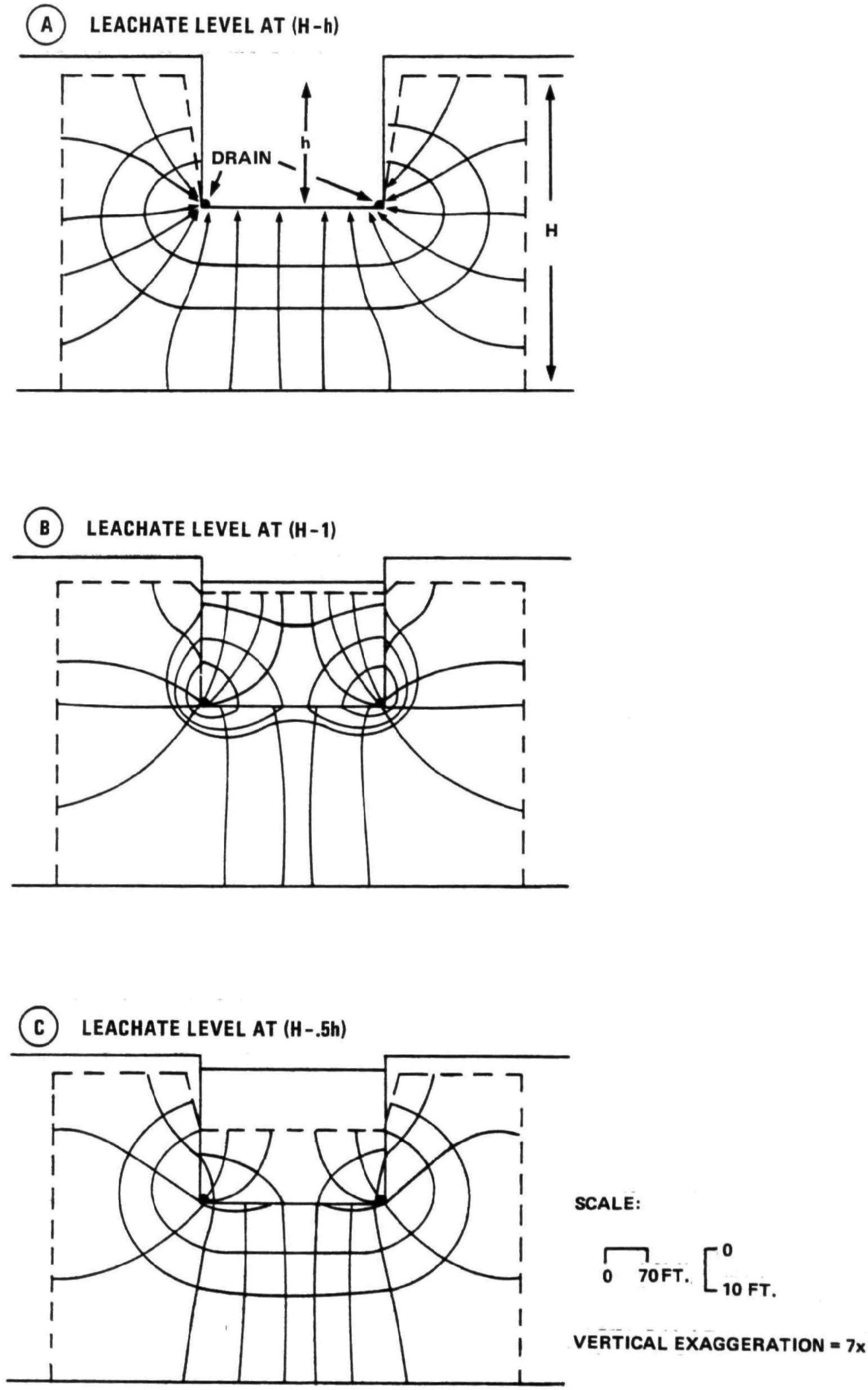

FIGURE 4-3. FLOW NETS WITH DIFFERENT LEACHATE LEVELS

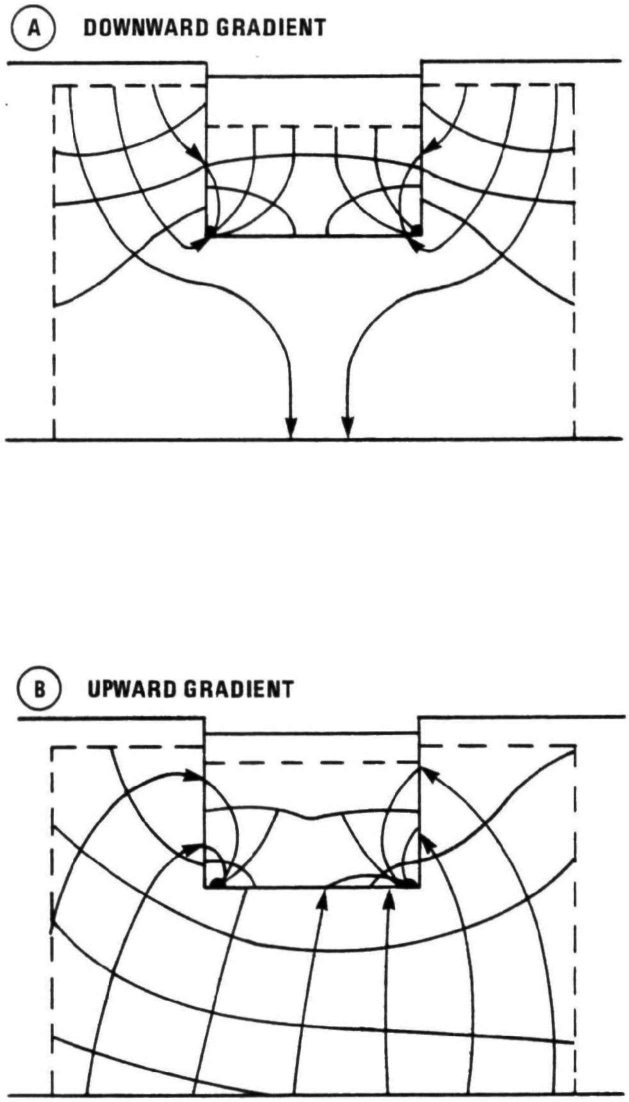

FIGURE 4-4. FLOW NETS WITH DIFFERENT VERTICAL GRADIENTS

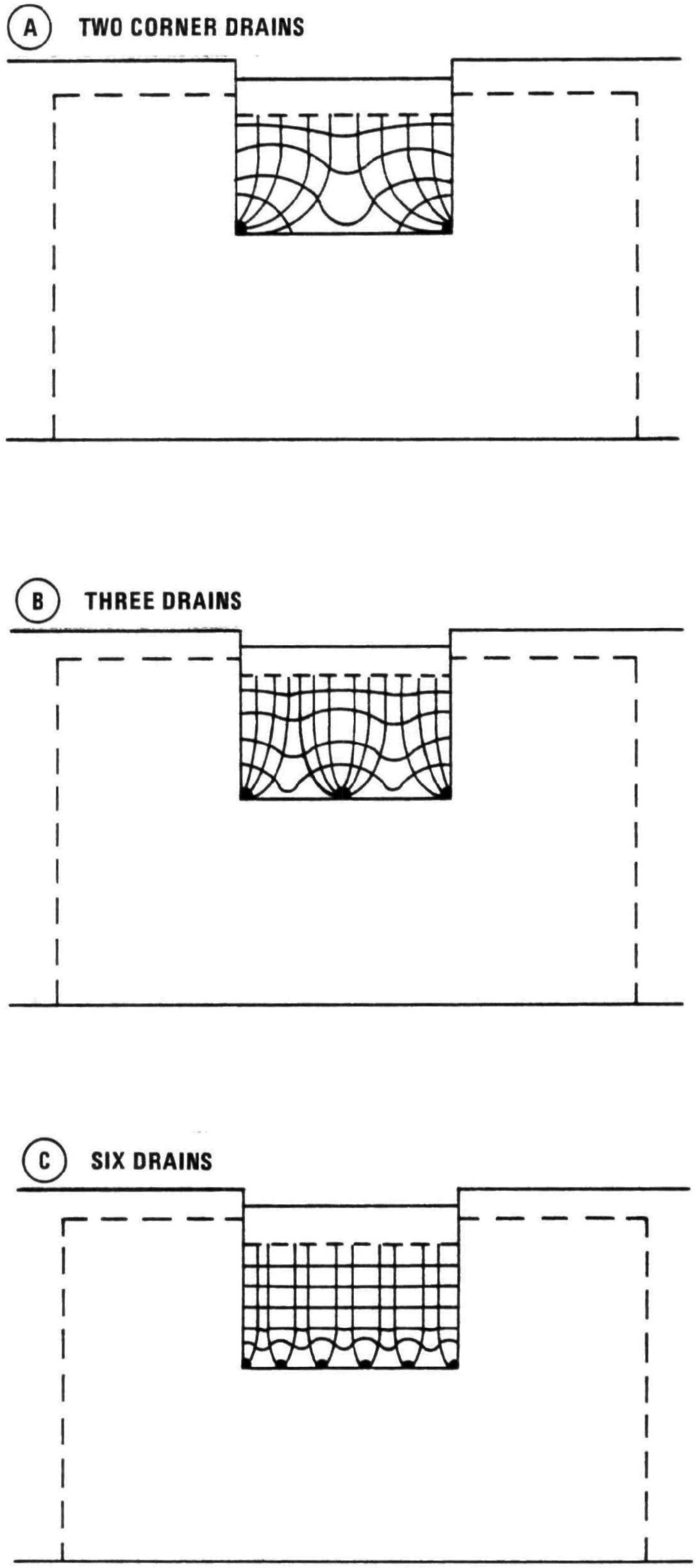

FIGURE 4-5. FLOW NETS WITH DIFFERENT NUMBERS OF DRAINS

maintenance level is set at the base of the fill, the amount of inflow in relation to excavation depth varies as follows:

Depth Below Water Table Excavated, Ft.	Approximate Inflow, GPM
10	8.8
20	16.7
30	23.8
40	29.8

These inflows need to be considered in comparison to the capacity of the drain system and the possible inducement of heaving or buckling because of the artesian head in the underlying dolomite. Rough calculations indicate that 40 feet is about the maximum depth that should be excavated, less if the remaining depth to the dolomite is below about 50 feet.

5.0 MODELS

This chapter characterizes some commonly accepted models that can be utilized in the design and performance evaluation of waste disposal sites, and in the tracking of pollutants released from these sites. These models are divided into two major groups--the release rate models and the solute transport models. Typically estimates of leachate quantity and quality released from a site are obtained from a release rate model and used as input to a solute transport model. The theory behind some of the models is very complex and readers should refer to other sources such as Bachmat et. al. (1980), Mercer and Faust (1981), Anderson (1979), and Weston (1978) for indepth discussions of modeling.

5.1 RELEASE RATE MODELS

The first and probably the most crucial step in waste site modeling is to obtain accurate estimates of the quantity and quality of leachate that will be released into the subsurface environment. Only after adequate determination of this source term can a solute transport model be performed. This section briefly describes the theory behind release rate models and present those models that can potentially be utilized in obtaining release rates from landfills.

5.1.1 Fundamentals

Most release rate models are based on dividing the problem of prediction into three separate components--leachate generation, constituent cocentrations and leachate release rates from the site. Combining the three separate components allows for prediction of the quantity and quality of leachate that can be expected to be released from the site.

5.1.1.1 Leachate Generation

Leachate generation refers to the quantity of fluid within the site available to leach and transport waste constituents. The major factors that directly influence leachate generation are listed in Table 5-1. Probably the most important factors for zone-of-saturation landfills are infiltrating precipitation, and groundwater instrusion. There are currently two main approaches for predicting leachate generation--the water balance approach and the use of bounding assumptions.

Fenn, Hanley and DeGeare (1975) pioneered the use of water balances to predict leachate generation from solid waste disposal facilities based on the earlier work of Thornthwaite (1955). Several authors have since updated and modified Fenn's work for application to other types of waste disposal sites. Basically, water balances numerically partition the amount of fluid moving into, around, and through the cap of a land disposal facility utilizing the equation:

$$Perc = P - RO - ST - ET$$

where:

Perc = percolation rate; the portion of precipitation which infiltrates the surface and is not taken up by plants or evaporated

P = precipitation rate

RO = surface water runoff; the portion of the precipitation which does not infiltrate into the ground but instead moves overland away from the site

ST = change in soil moisture storage

ET = Actual evapotranspiration; the combined amount of water returned to the atmosphere through direct evaporation from surfaces and vegetative transpiration.

Values for the parameters needed for such an analysis can be found in a variety of sources or estimated using a variety of techniques. Precipitation values for a given location are available from a number of sources including the National Weather Service. Runoff is very site specific and difficult to measure. Most release rate models use one of the following methods to estimate runoff:

TABLE 5-1 MAJOR FACTORS AFFECTING LEACHATE GENERATION

Primary Factors	Secondary Factors
Precipitation	quantity, intensity, duration, frequency, seasonal distribution
Liquid Content of Wastes	type, quantity, moisture content, and moisture storage capacity (field capacity)
Liquids from Waste Decomposition	waste composition, waste environment, and micro-organism populations
Groundwater Intrusion	flow rates into pit, seasonal distribution of water table, head levels, liner materials
Soil Moisture Storage	field capacity of materials, seasonal flucuations
Evapotranspiration	temperature, wind velocity, humidity, vegetation type, solar radiation, soil characteristics
Runon/Runoff Control	diversions, crowning of surface cap, permeability and integrity of cap, depression storage
Operation Mode	open versus closed, coverage
Surface/Cap Conditions	permeability, integrity, surface contour, runoff underdrain systems, subsidence

- Rational Formula--utilizes empirical runoff coefficients based on vegetative type, soil type, and slope

- SCS Curve Numbers--utilizes empirical coefficients which relate runoff to soil type, land use, management practices, and daily rainfall

- Green-Ampt Equation--approximates runoff based on soil properties, initial water content and distribution, surface conditions, and accumulative infiltration.

Both evapotranspiration and soil moisture storage can be estimated using empirical soil moisture retention relationships such as those developed by Thornthwaite. Some models require that evapotranspiration be a measured site specific input, while others do not specify a method to obtain values. Some models relate evapotranspiration to physical parameters such as temperature, solar radiation, and the leaf area index (LAI), while others store evapotranspiration and soil moisture information for various locations on a national data base that can be accessed by the model. One model also makes simplifying assumptions to estimate soil moisture storage either by apportioning soil moisture into a "wet" zone and a "dry" zone or by using the method of depth-weighted fractional water content within the soil profile.

Release rate models also allow the user to set surface conditions and cover liner characteristics for the site with varying degrees of flexibility. Some methods allow multiple clay-synthetic liners, others only clay liners, while still another can only be applied to open sites without covers. Cover vegetation, slope, countour, and soil properties can also be set in all but the most simplistic models.

When cover liners are used to impede percolation to waste cells, excess water moves away from the site through subsurface lateral drainage above the liner. Models estimate this lateral drainage by:

- Approximate methods (utilizing correction factors) derived from the Boussinesq equation for lateral saturated flow

- Empirical methods development by Moore (1980) which calculate the maximum hydraulic head above the liner and then the upper bound of the quantity of liquid flowing into tile drains. The liner is assumed to be impermeable for these calculations.

- Empirical methods which calculate percolation through the liner and soil moisture storage; then extrapolate lateral drainage as the remaining excess water.

The next step is to predict the flow rate through the top liner. This is ultimately the major contributing factor in leachate generation. Numerous methods are used to predict this percolation rate and they can be divided into methods for clay liners and those for synthetics. Methods used for clay liners include:

- Darcy's Law for saturated conditions which relate flow velocity to hydraulic conductivity, effective porosity, hydraulic head, and travel distance using the following general equation:

$$V = Kh/nx$$

  where: V is flow velocity, K is hydraulic conductivity, n is effective porosity, h is hydraulic head difference, and x is travel distance

- Approximations of saturated Darcy flow as proposed by Wong (1977)

- Soil storage routing techniques through multiple soil layers which relate liner permeability to inflow rate, time interval, hydraulic conductivity, soil water storage, and evapotranspiration.

- Darcy's Law with provisions to arbitrarily increase liner permeability assuming that certain events occur (e.g., burrowing animals or equipment breech of the liner)

- Prediction of unsaturated flow driven by capillary forces utilizing the Greem-Ampt approximation of the wetting front assuming a constant capillary head.

Methods to predict flow through synthetic liners include:

- Darcy's Law as described above and based on hydraulic head and liner thickness

- Power law relationships for estimating the aging of a liner based on the life expectancy of the liner

- Arbitrary methods such as assuming that the liner will be impermeable for 20 years and then fails completely (i.e., after 20 years the model treats the faciity as if it were unlined)

- Stochastic (Monte Carlo) simulation for liner failure due to aging and installation problems.

The amount of water percolating through the cover liners and into the waste cells is either adjusted according to the moisture content of the wastes and fill materials in the facility, or the wastes and fill materials are assumed to be at field capacity and, therefore, the amount percolating into waste cells is also the total quantity of leachate generated.

Water balances for waste disposal sites produce only relative solutions to leachate generation for comparing different designs or sites. The high degree of uncertainty that exists in these solutions has led to the use of bounding assumptions. Bounding assumptions are based on the knowledge that the quantity of leachate generated at a given facility falls between 0% and 100% of the maximum potential amount, such that upper and lower bounds for leachate generation volume can be established. Using assumptions and empirical data, the bounds can be narrowed to produce best- and worst case scenarios which can, in turn, be used to design the landfill based upon performance goals.

5.1.1.2 Leachate Constituent Concentrations

For a release rate model to be useful, not only must the quantity of leachate produced be estimated but also the quality (i.e., leachate constituent concentrations). The quantitative simulation of the processes and interactions occurring within a landfill to produce leachate would be very complex, and therefore most available models do not attempt to simulate all these processes. Table 5-2 lists some of the factors affecting leachate constituent concentrations that would have to be considered.

Because of the complex interdependent interactions occurring within a disposal site and the lack of knowledge needed to characterize the interactions, simulations of the processes are extremely difficult if not impossible. Consequently, release rate models do not address the factors which govern constituent concentrations. Rather, the models make assumptions to greatly simplify the complexities of the real world. These assumptions are:

- Constituents are at the saturation solubility concentration levels in leachate

- Constituents exist at equilibrium concentrations between the aqueous and sorbed phases

- Bounding assumptions are used in a similar manner as described for leachate generation.

5.1.1.3 Leachate Release

Leachate release is defined as the escape of any contaminants beyond the containment boundary of a land disposal facility. The type and magnitude of release depends upon the presence of a liner system, the type of liner employed, the presence and efficiency of a leachate collection system, and the occurrence and magnitude of any system failures. Table 5-3 lists some of the factors affecting leachate release from a land disposal site.

TABLE 5-2 FACTORS AFFECTING LEACHATE CONSTITUENT CONCENTRATIONS

Major Components	Primary Factors	Secondary Factors
Waste Composition	Volume	
	Constituents	
	Constituent Concentrations	
Physicochemical Properties	Solubility	pH; temperature; composition of liquid phase
	Mobility	Viscosity; temperature; density; sorption; complexation
	Persistence	pH; temperature; presence of catalysts; chemical degradation (e.g. oxidation, reduction, hydrolysis, photolysis), biological degradation
	Volatilization	Fugacity; constituent vapor pressure; temperature
	Phase/State	Temperature; pressure
Contact Time	Conditions of Waste Environment	Flow rates through wastes, fill materials and drain layers; waste permeability; waste porosity; particle size: site heterogeneities: capillary action; piping through wastes: ponding in waste cells; plugging of pore spaces
Chemical Reactions and Interactions	Hydrolysis	pH; temperature; soil pH, catalysts
	Oxidation	Presence and type of oxidants: catalysts: oxygen concentration; pH: temperature
Chemical Reactions and Interactions (continued)	Reduction	Oxygen concentration; complexation state; concentration and type of reducing agents: pH: temperature
	Photolysis	Solar radiation; transmissivity of water; presence of sensitizens and quenchers
	Microbial Degradation	Microbial population; soil moisture content; temperature; pH, oxygen concentration, redox potential
Facility Age	Microbial Acclimation	
	Changes in Waste Environment	ph; temperature; removal of most soluble constituents

TABLE 5-3 FACTORS AFFECTING LEACHATE RELEASE

Major Components	Primary Factors	Secondary Factors
Synthetic Liners	Physical Factors	Aging: human activities; internal loading stresses: hydrogeology; bathtub effect; weather resistence; deep root growth, burrowing animals: installation and design problems (e.g., subsidence from improper siting, improperly prepared seams): uplifting by gasses or liquids under pressure; impingment rate; temperature
	Chemical Factors	Chemical disintegration: weather, ultraviolet radiation, chemical and microbial attack from the soil atmosphere· waste-linear compatability: nature soil chemistry; pH, temperature
	Leachate Collection	Efficiency, maintenance; design
Clay Liners	Physicochemical Factors	Chemical dehydration, flocculation/ dispersion, alteration of shrink/ swell properties; soil piping; leachate characteristics; pore size distribution
	Chemical Factors	Dissolution of chemical species, adsorpsion properties; chemical disintegration native soil chemistry
	Physical Factors	Internal loading stresses; dehydration; hydrogeology; weathering; erosion: bathtub effect; aging; impingment rate; hydraulic head: structural and design considerations (e.g., proper siting and design to handle differential subsidence
	Biological Factors	Microbial population; burrowing animals: deep root growth, human activities
	Leachate Collection	Efficiency; maintenance; design

Prediction of leachate releases involves the estimation of leachate quantity escaping from the site over time, which is combined with constituent concentration. The major drawbacks in predicting releases are obtaining realistic estimates of liner lifetime, estimating the probability of liner failure, and establishing cause and magnitude of failure if it occurs. The methods employed to predict releases parallel those used to estimate flow rates through cover liners (described previously as part of the water balance approach).

5.1.2 Selected Release Rate Models

DRAINMOD/DRAINFIL (Skaggs, 1982)

DRAINMOD (1980) is a compouter model developed to predict the response of water in both the unsaturated and the saturated zones to rainfall, evapotranspiration, specified levels of surface and subsurface drainage, and the use of water table control or subirrigation practices. DRAINFIL (1982) is an adaptation of DRAINMOD for landfills which considers drainage from a sloping layer underlain by a tight clay liner and seepage through the cap. DRAINFIL can also quantify drainage to the leachate collection system and through the underlying clay liner during the time the landfill is open. A water balance for the soil water profile is used to calculate the infilatration rate, vertical and lateral drainage, evapotranspiration, and distribution of soil water in the soil profile using approximate solutions to nonlinear differential equations. The prohibitive cost of using numerical methods to finding solutions to equations of this sort requires that approximate methods be used. Checks of solutions obtained through these methods suggest, however, that satisfactory results can be consistently obtained.

The minimum data required for these models include precipitation (amount, distribution, intensity, and duration), water table elevation, daily potential evapotranspiration (PET), net solar radiation, temperature, humidity, wind velocity, soil moisture content, soil profile depth, surface compaction, vegetation, depth of root zone, hydraulic conductivity (saturated and unsaturated), and pressure head. Most of the data are readily available, though some difficulty in obtaining reliable unsaturated hydrualic conductivity and pressure head data may exist. In addition, to simplify calculations the models assume:

- One-dimensional, saturated flow in the bottom liner
- Infiltration rates for uniform deep soils with constant initial water content are expressed in terms of cumulative infiltration alone, regardless of the rate of application
- Drainage is limited by the rate of soil water movement to the lateral drains and not by the hydraulic capacity of the drain tubes or outlet

DRAINMOD is currently being used in assessing agricultural drainage systems and has been field verified in a variety of locations. DRAINFIL, however, has not yet undergone the final changes needed for its use in assessing infiltration at waste disposal sites, and therefore remains untested.

These models are similar to other release rate models in that they use a water balance approach, do not consider leachate constitutent concentrations, and do not consider any processes occurring within the waste cell that may affect leachate quantity or quality. Some unique features of DRAINMOD/DRAINFIL are their ability to predict the upward movement of water, and the precision of their hydraulic head estimates.

Hydrologic Evaluation of Landfill Performance (HELP/HSSWDS)
(Perrier and Gibson, 1980)

The hydrologic evaluation of landfill performance (HELP, formerly HSSWDS) is a one-dimensional, deterministic water balance model modified and adapted from the CREAMS (Chemical Runoff and Erosion from Agricultural Management Systems) soil percolation model for use in estimating the amount of water that will move through various landfill covers. This model can simulate daily, monthly, and annual values for runoff, percolation, temperature, soil-water chatacteristics, and evapotranspiration with a minimum of data (e.g., percipitation, mean temperature, solar radiation, leaf area index, and characteristics of the cover material). Should data be unavailable, the model provides default values for such parameters as soil-water characteristics, precipitation, mean monthly temperature, solar radiation, vegetative chracteristics and climate based on the location of the site. The model is portrayed by its developers as "no more complex than a manual tabulation of moisture balance."

The HELP model ignores rainfall intensity, duration, and distribution and considers only mean rainfall rates, which could somewhat limit the accuracy of the estimates. It also does not evaluate leachate quality. However, it can estimate percolation through up to eight drainage layers including through the waste cell itself, and estimate lateral drainage through any or all of these layers. Some other features of the HELP model include the ability to provide estimates of the impingement rate of leachate entering the bottom liner collection system, predict the seepage rate through a saturated clay liner and estimate evapotranspiration and runoff using a minimum of data.

The HSSWDS model has been successfully field verified by Gibson and Malone (1982) and many others. Those HSSWDS users contacted for comments and opinions believed that HSSWDS was very useful in comparing sites or cover designs, but that the accuracy or validity of the outputs could not be determined. HELP is currently undergoing refinement and has not yet been tested.

<u>Landfill and Surface Impoundment Performance Evaluation (Moore, 1980)</u>

The Landfill and Surface Impoundment Performance Evaluation (LSIPE) model attempts to determine the adequacy of designs of hazardous waste surface impoundments and landfills in controlling the amount of fluid released to the environment. LSIPE utilizes a series of linearized equations and simplified boundary conditions to evaluate the efficiency of a proposed liner design in terms of:

- Horizontal flow through sand and gravel drain layers
- Vertical flow through low permeability clay liners
- Efficiencies of liner-drain layer systems
- Seepage through the bottom liner.

LSIPE has the advantage of allowing for nonlinear equations and more complex boundary conditions to be employed if needed. Only transport of liquids through a single modular waste cell can be estimated; however, modules can then be arranged in the proposed configuration for analysis. The LSIPE approach also possesses the unique capability of allowing for leachate releases to be measured indirectly through the efficiency of the leachate collection system.

In order to provide estimates of the above mentioned parameters, this approach requires:

- Liquid routing diagram for the site
- Water balance for the site
- Slopes in the routing system
- Hydraulic conductivities
- Service life of any synthetic liners.

The LSIPE model also makes the general assumption that the operating conditions for a waste landfill or surface impoundment meet the basic requirements of good engineering design, including:

- Surface water runon has been intercepted and directed away from the site so that only the rainfall impinging directly on the landfill need be accounted for

- Proper precautions have been taken to prevent erosion of the cover soils which would degrade cover performance

- o Synthetic liners have been installed properly to ensure their integrity for design life.

Post-Closure Liability Trust Fund Model (PCLTF) (U.S. EPA 1982)

This model is being developed to assess the adequacy of the Post-Closure Liability Trust Fund established as the result of the passage of CERCLA in 1980. The fund will provide for liability claims resulting from failure of RCRA permitted sites after proper closure. The analytical approach involves:

- o Assisting failure probabilities to facilities
- o Determining environmental exposure
- o Assessing damage potential
- o Quantifying damages, and assessing costs for clean-up, remedial action, damage to natural resources, personal injury and economic loss.

PCLTF is the only model reviewed which addresses all three components necessary to predict a mass load release from a land disposal site. The model can be applied to open or closed facilities with both clay and/or synthetic liners. The user can specify one of seven generic site types from a variety of cover and bottom liner and leachate collection configurations. The components of the model consists of:

- o Users supplied inputs which characterize the site design and operation, and identify the wastes placed within the fill

- o A data base of physical and chemical characteristics of waste constituents which relate to their solubility, toxicity, persistance, and mobility as well as their effect on synthetic liner performance

- o A data base of climate, soils, and the geology of various regions of the U.S.

- A base line analysis with which to set starting site conditions

- Water movement simulation which uses Monte Carlo simulation techniques to generate values for seepage velocity, effective porosity, dispersion, and liner failure to route leachates through the layers of the landfill, including liners and drain layers. Adjustments to leachate quantity are made based on moisture content of wastes and fill materials.

This model's output is a two-dimensional, uniformly distributed, leachate discharge estimate (concentration and flux) to the unsaturated soil column beneath the site. The output provides the source term for a mass transport module for the unsaturated zone.

<u>Leachate Travel Time Model (Pope-Reid Associates, 1982)</u>

The Leachate Travel Time Model (currently under development by Pope-Reid Associates) combines several analytic techniques and previously developed models to evaluate the performance of landfills of various designs in a variety of climatic settings. The model consists of a monthly, quarterly, or annual hydrologic and waste budget which is used to calculate leachate volume in the active fill area, leachate head in drain layers, containment time and seepage rate through the bottom liner, and travel time and seepage rate in the unsaturated zone below the landfill. The model possesses the unique feature of accounting for the moisture content of wastes and fill materials. Actual measured values or estimations of moisture content can be input. Also the model includes both gravitational and capillary forces to calculate seepage rates through liners.

The Leachate Travel Time Model does not include a specific cover liner option, although the user can incorporate a cover liner by altering the hydrologic budget. Like other models, the current program does not address constitutent concentrations of contaminant mass transport, but the authors do intend to incorporate constituent transport in the future. The model does, however, address both leachate generation and release in a relatively simple and easy-to-use program which incorporates many interesting features.

<u>Release Rate Computations for Land Disposal Facilities (SCS Engineers, 1982)</u>

This approach consists of a series of simple calculations to predict the quantity of leachate generated and released form landfills and surface impoundments which will be incorporated into EPA's RCRA Risk/Cost Policy Model Project (ICF Incorporated, Clement Associates, Inc., and SCS Engineers, Inc., 1982). The approach assumes that:

- Synthetic liners last for 20 years, after which liquid moves freely through them
- Clay liners retain their integrity for longer than 100 years
- The only sources of liquids are infiltration from the surface and free liquids in waste. Only saturated flow takes place through the liner in the absence of free liquids
- Infiltration through the cover system after closure is less than or equal to leachate movement through the liner system
- Synergistic effects do not occur.

The time required for leakage to appear beneath the bottom of a clay liner is given by:

$$t = T\P d^2/4D$$

where:

t = time to first appearance of leakage (sec)

d = thickness of clay liner (cm)

D = linearized diffusivity constant, (cm^2/sec) asssumed to be 10^5 cm^2/sec.

The volume of leachate release over time is given by:

$$q = Ks(dh/d)A(\Delta t)$$

where:

q = volume of leachate released over time

Ks = saturated permeability coefficient

$\frac{dh}{dz}$ = hydraulic gradient

A = Area at base of faciity

Δt = length of time over which leachate releases.

5.2 SOLUTE TRANSPORT MODELS

Once the quantity and quality of a release from a land disposal site has been determined, this estimate can be used as the input for a solute transport model. Solute transport models predict the migration of chemical constituents away from a source overtime in one- two-, or three-dimensions. A brief discussion of the principles used in transport modeling and descriptions of several transport models are contained in the following sections.

5.2.1 Fundamentals

The transport models presented in this section are all consider mathematical models, rather than rating or ranking type models (e.g., Mitre Model). The mathematical approach to modeling involves applying a set of equations, based on explicit assumptions, to describe the physical processes affecting pollutant transport from a site. These models can be divided into two types--deterministic and stochastic. Deterministic models attempt to define the shape and concentration of waste migration using the physical processes (e.g., groundwater flow) involved, while stochastic models attempt to define causes and effects using probablistic methods. Models presented in this report are generally deterministic.

Deterministic mathematical models can be further divided into analytical models and numerical models. Analytical models simplify mathematical equations, allowing solutions to be obtained by analytical methods (i.e., function of real variables). Numerical models, on the other hand, approximate equations numerically and result in a matrix equation that is usually solved by computer analysis. Both types of deterministic models address a wide range of physical and chemical characteristics but the analytical models usually simplify the characteristics by assuming steady state conditions. The physical and chemical characteristics considered by these models include:

- Boundary Conditions--hydraulic head distributions, recharge and discharge points, locations and types of boundaries

- Material Constants--hydraulic conductivity, porosity, transmissivity, extent of hydrogeologic units

- Attenuation Mechanisms--adsorption-desorption, ion exchange, complexing, nuclear decay, ion filtration, gas generation, precipitation-dissolution, biodegradation, chemical degradation

- Hydrodynamic Dispersion--diffusion and dispersion (transversee and longitudinal)

- Waste Constituent Concentration--initial and background concentrations, boundary conditions

Both mathematical model types incorporate two sets of equations to define transport; a groundwater flow equation and a mass transport equation. Figure 5-1 illustrates the relationship between these equations.

A general form of the water momentum balance equation for nonhomogeneous anisotropic aquifers is:

$$\frac{\partial}{\partial x_i}\left(K_{ij} \quad \frac{\partial h}{\partial x_j}\right) = S\frac{\partial h}{\partial t} + W$$

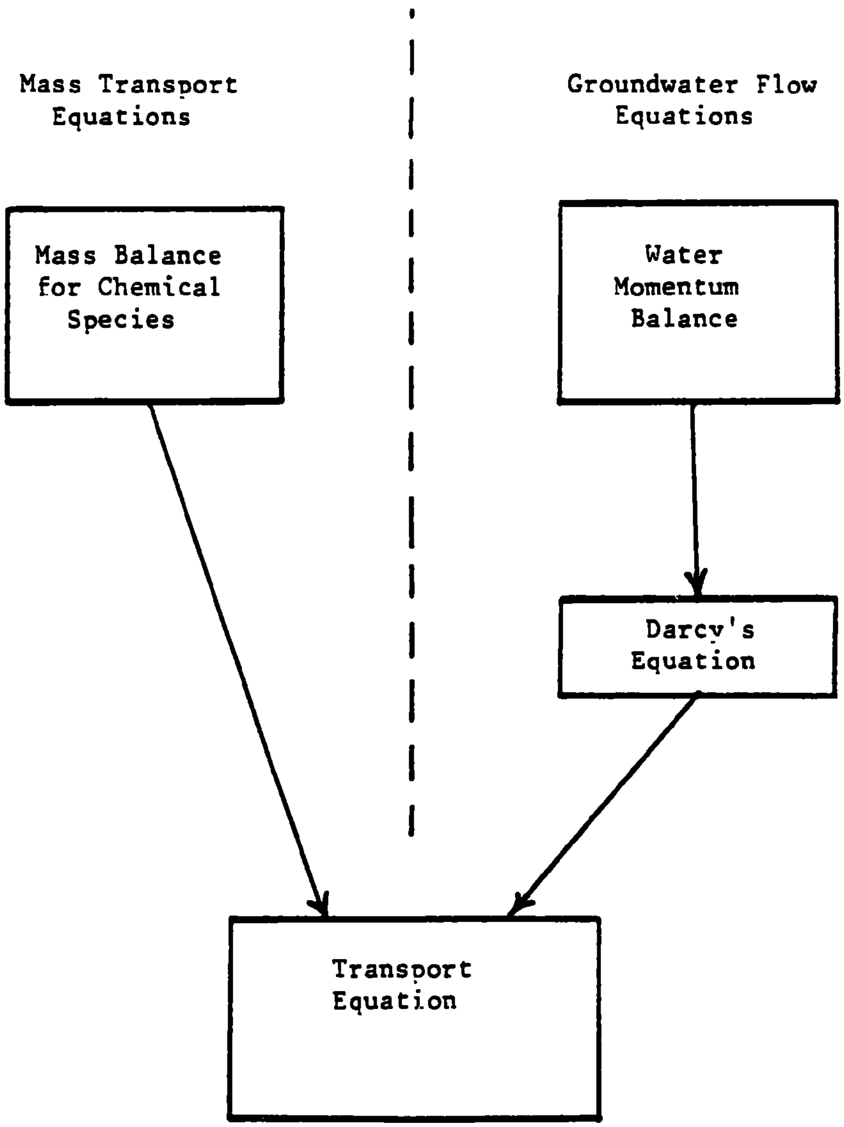

Figure 5-1. Major Components of Groundwater Transport Equation
(after Mercer and Faust, 1981)

where:

- h = hydraulic head
- K = hydraulic conductivity
- S = storage coefficient
- W = volume flux per unit area (e.g., pumping or injection wells, infiltration, leakage)
- x = distance
- t = time

The Darcy equation is generally represented as:

$$V_i = \frac{-K_{ij}}{n}\left(\frac{\partial h}{\partial x_j}\right)$$

where:

- V = groundwater velocity
- n = porosity
- K = hydraulic conductivity

The mass transport side of the model, which describes the concentration of a chemical species in a flow pattern in general form is:

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x_i}\left(D_{ij} \cdot \frac{\partial C}{\partial x_j}\right) - \frac{\partial}{\partial x_i}(CV_i) - \frac{C'W}{n} + R$$

where:

- C' = concentration of solute in the source or sink fluid
- C = chemical species concentration
- D = dispersion tensor (i.e., hydrodynamic dispersion)
- V = groundwater velocity (i.e., convection transport)
- R = rate of chemical species attenuation/transformation

These equations are coupled to provide predictions of solute transport in the groundwater system with chemical reactions considered. For analytical models, these equations are simplified to explicit expression. For either type of model, a sensitivity analysis of model results can be performed by varying the input characteristics singularly or in combination. One type of sensitivity analysis that could be performed involves changing single parameters (within known values of occurrence) to demonstrate the effects that variations in individual parameters have on model output. This analysis helps identify those parameters which have the greatest influence on model results. A second type of sensitivity analysis involves a series of trial runs of the model using an array of input parameters which vary in accordance with the expected errors associated with each parameter (i.e., Monte Carlo simulation techniques). This method provides a general assessment of the overall model sensitivity and intrinsic precision by providing a range of variations of the model outputs as a function of the error bars associated with the input parameters (e.g., mean values, maximum values, minimum values).

5.2.1.1 Analytical Models

Analytical models provide estimates of waste constituent concentrations and distributions using simplified, explicit expressions generated from partial differential equations. The mathematical expressions are usually simplified by assuming steady state conditions relative to fluid velocity, dispersion dynamics, and other physical parameters. For example, groundwater flow equations can be simplified if the aquifer is assumed to have infinite extent. Governing equations characterize both groundwater flow and mass transport, and may also address dilution, dispersion, and attenuation. These models can simulate plume migration from the source to a utilized groundwater system allowing for attenuation and dispersion. The method provides a quick and inexpensive solution with minimal amounts of data as long as the simplifying assumptions do not render results invalid.

5.2.1.2 Numerical Models

Numerical models characterize groundwater contamination processes without the simplification of complex physical and chemical characteristics required by analytical models. The numerical models reduce the partial differential equations to a set of algebraic equations that define hydraulic head at specific points (i.e., grid points). These equations are solved through linear algebra using matrix techniques.

The numerical methods most commonly used to simulate groundwater transport problems can be divided into four groups: finite difference (FD); finite element (FE); method of characteristics (MOC); and discrete parcel random-walk (DPRW). In each method, the governing equations (e.g., groundwater flow eauations) are solved by subdividing the entire problem domain using a grid system of polygons. Every block has assigned hydrogeology properties (e.g., transmissivity) associated with it that define the aquifer. Accompanying each grid as a node point that represents the position of an equation with unknown values (e.g., head). For the finite difference method, the derivatives of the partial differential equations are approximated by linear interpolation (i.e., the differential approach). In the finite element method the partial differential equations are transformed to integral form (functionals) and minimized to solve the dependant variables. The algebraic equations for each node point, derived by the FD or FE methods, are then combined to form a matrix equation which is solved numerically. The FE method is better suited for solving complex two- and three- dimensional boundary conditions than the FD method. When using FD or FE methods for solving contaminant transport problems, results are subject to numerical dispersion or numerical oscillation. Numerical dispersion causes answers to be obscured because of accumulated round-off error at alternating time steps. Numerical oscillation causes answers to overshoot and undershoot the actual solution at alternating time steps. Numerical oscillation is generally associated with FE methods, while numerical dispersion is generally associated with FD methods.

The method of characteristics and discrete parcel random-walk models were developed to minimize the numerical difficulties associated with the FE and FD methods. Both the method of characteristics (MOC) and discrete parcel random walk method analyze temporal changes in concentrations by tracking a set of reference points that flow with the groundwater past a fixed grid point. In the MOC method, points are placed in each finite difference block and allowed to move in proportion to the groundwater velocity at the point and the time increment. Concentrations are recalculated using summed particle concentrations at the new locations. The random-walk method varies from the MOC method because instead of solving the transport equation, a random process defines dispersion. Reference points move as a function of groundwater flow, consistent with a probability function related to groundwater velocity and dispersion (longitudinal and transverse). The methods provide comparable results but the MOC method is reportedly time consuming, expensive, and requires considerable computer storage.

5.2.2 Selected Solute Transport Models

Eight analytical models and nine numerical models are presented in the following sections. Each has characteristics that make it unique from the other models, therefore, selection of a model should be based on making the best use of available data given the desired output.

5.2.2.1 Analytical Models

The eight analytical models characterized in this section do not address cases involving secondary porosity, immiscible liquids, or more than one contaminant. Only one model, AT123D developed by G.T. Yeh (1981), considers both the saturated and unsaturated hydrologic zones; the other models are restricted to modeling only one hydrologic zone.

SESOIL (Bonazountas and Wagner, 1981)

SESOIL, a seasonal soil compartment model, was developed by A.D. Little Inc. for the U.S. EPA Office of Toxic Substances. The model is described as a "user-friendly" statistical/analytical mathematical model designed for long term environmental pollutant fate simulations. Simulations are performed for the unsaturated zone and are based on a three-cycle rationale--the water cycle, the sediment cycle, and the pollutant cycle. SESOIL addresses numerous processes including diffusion, sorption, chemical degradation, biological degradation, and the complexation of metals. The model is presently being updated and is available for limited use although field or analytical verifications have not yet been performed.

PESTAN (Enfield, 1982)

PESTAN was developed at the EPA Robert S. Kerr Environmental Research Laboratory. The model calculates the movement of organic substances in one dimension through the unsaturated zone based on linear sorption and first order degradation (i.e., hydrolysis and biodegradation). Calculated outputs include pollutant velocity, length of the pollutant slug, and contaminant concentrations. Pollutant application rates to the soil surface can be changed to determine the effect of the number of applications, application period, and number of days before reapplication. This model is best classified as a screening model because it provides for a rapid evaluation of chemicals without the sophistication of numerical models. The model is also easy to use and inexpensive. PESTAN can be coupled with PLUME, a saturated zone analytical model. PESTAN has been field verified for the chemicals DDT and Aldicarb, and the model is being used by EPA-Athens.

PLUME (Wagner, 1982)

PLUME is a steady state analytical model developed at Oklahoma State University to model contaminant transport in the saturated zone. The model provides two-dimensional plume traces from a continuous source and allows for first order degradation and linear sorption (i.e., organic pollutants) with dispersion. The model was verified using a case history of groundwater contaminated with hexavalent chromium, although the effects of adsoption and degradation were ignored.

Leachate Plume Migration Prediction (Kent et al. 1982)

The Leachate Plume Migration Model was developed as an analytical technique for the hazard evaluation of existing and potential, continuous source waste disposal sites by predicting plume migration and mixing in the saturated zone. Predictions can be made from nomographs, hand-held calculators, or a large scale computer. The model allows for degradation (i.e., radioactive and biological) of constituents and for the effects of dispersion and diffusion. The predictive methods presented are simplified so that a strong background in mathematics and computer programming are not required for their use. The model has been verified using data from a chromium plume at Long Island and is presently being tested against other case studies.

Cleary Model (Cleary, 1982)

The Cleary Model consists of ten different analytical models that describe mass transport and groundwater flow, with dispersion, under a variety of boundary conditions. The model addresses conservative constituents (i.e., without degradation). The ten available models are:

- 1-dimensional, mass transport; 1st type boundary conditions
- 1-dimensional, mass transport; 3rd type boundary conditions

- 2-dimensional, mass transport; strip boundary, finite width
- 2-dimensional, mass transport; strip boundary, infinite width
- 2-dimensional, mass transport; Gaussian source, infinite width
- 3-dimensional, mass transport; patch source, finite dimensions
- 3-dimensional, mass transport; 5 area Gaussian source
- 2-dimensional, groundwater flow; infinite dimensions, recharge boundary
- 2-dimensional, groundwater flow; finite dimensions, recharge boundary
- 2-dimensional, groundwater flow; infinite dimensions, no recharge.

These models were not available for review and R. Cleary (developer) could not be contacted; information concerning these models was, therefore, very limited.

AT123D (Yeh, 1981)

AT123D, developed by G.T. Yeh at Oak Ridge National Laboratory, is a generalized analytical transient, one-, two-, or three-dimensional computer model for estimating waste transport in both the unsaturated and saturated zones. The model is flexible, providing 450 options: 288 for the 3-dimensional case; 72 for the 2-dimensional case in the x-z plane; 72 for the 2-dimensional case in the x-y plane; and 18 for the 1-dimensional case in the longitudinal direction. AT123D models all of the following options:

- Eight sets of source configurations (i.e., point source; line source parallel to x-, y-, or z-axis; area source perpendicular to the x-, y-, or z-axis; and a volume source)
- Three kinds of source releases (instantaneous, continuous, and finite duration releases)
- Four variations of the aquifer dimensions (finite depth and width, finite depth and infinite width, infinite depth and finite width, infinite depth and infinite width)
- Modeling of radioactive wastes, chemicals, and head levels.

The transport mechanisms addressed are advection, hydrodynamic dispersion, adsorption, decay/degradation, and waste losses to the atmosphere from the unsaturated zone. The model is computer coded and publicly available, making it a potentially valuable tool for preliminary assessment of waste disposal sites. Fifty sample problems (simulations) have been performed but actual field verification appears to be lacking.

Screening Procedure (Falco et al., 1980)

A screening procedure for assessing the transport and degradation of solid waste constituents in the saturated zone as well as surface waters was developed by Falco et al. (1980). The procedure estimates the movement and degradation of chemicals released from landfills and lagoons based on the physical and chemical properties of the compound and a defined range of environmental conditions that the compound would be expected to encounted in groundwater. The procedure developed involves two parts, a mathematical model to obtain quantitative estimates of exposure and a logic sequence that assigns qualitative descriptors of behavior (e.g., low, significant, high) based on the quantitative estimates of exposure. Quantitative estimates are based on hydrolysis, biological degradation, oxidation, and sorption. The results of this study indicate that the procedure provides a means of qualitatively screening organic chemicals when specific process rates are available.

PATHS (Nelson and Schur, 1980)

The PATHS groundwater model is a hybrid analytical/numerical model for two-dimensional, saturated groundwater flow that estimates single contaminant transport under homogeneous geologic conditions. The model also considers the effect of equilibrium ion exchange reactions for a single contaminant at trace ion concentrations. Dispersion effects are not considered by the model. The model provides a fast, inexpensive, first-cut evaluation consistent with the amount of field data usually available for a site. Analytical verifications have been performed but field verifications have not.

5.2.2.2 Numerical Models

The nine numerical models characterized in this section generally address the following characteristics:

- Coupled saturated/unsaturated zones
- Primary porosity
- Heterogeneous, anisotropic aquifers
- Miscible constituents
- Dispersion
- Attenuation/degradation

This group of models represents the most flexible approach to modeling a wide range of hydrogeologic conditions and contaminant types because the governing equations are not simplified as they are for the analytical models. These models generally involve greater costs and require accurate geohydrologic data for a given site.

MMT/VTT/UNSAT1D (Battelle, 1982)

MMT (Multicomponent Mass Transport) is a one- or two-dimensional mass transport code for predicting the movement of contaminants in the saturated or unsaturated zone. The MMT model utilizes the discrete parcel random-walk method and was originally developed to simulate the migration of radioactive contaminants. The model accounts for equilibrium sorption, first-order decay and n-membered radioactive decay chains. A velocity field (i.e., groundwater flow equations) must be input to the model and this is generally accomplished by coupling with the VTT (Variable Thickness Transient) model for the saturated zone and UNSAT1D (One-Dimensional Unsaturated Flow) model for the unsaturated zone. A computer package facilitates interpretation of results by providing graphic data displays. The model has been used at the Hanford, Washington, site to predict tritium concentrations.

CFEST/UNSAT1D (Battelle, 1982)

CFEST (Coupled Fluid Energy and Solute Transport) predicts fluid pressure, temperature, and contaminant concentrations in saturated groundwater systems. Coupling the model with UNSAT1D allows for modeling the unsaturated zone. The model applies finite element techniques to solve equations. The flow system may be complex, multi-layered, heterogeneous and anisotropic with time-varying boundary conditions and time-varying areal sources and sinks. Sorption and contaminant degradation are presently being incorporated into the model. The model is presently being field verified for EPA at the Charles City, Iowa, site for arsenic and pharmaceutical chemical waste (organics). Model documentation is in preparation.

Pollutant Movement Simulator (Khaleel and Reddell, 1977)

The Pollutant Movement Simulator is a three-dimensional model describing the two-phase (air-water) fluid flow equations in a coupled saturated-unsaturated porous medium. Flow equations are solved by finite difference methods. A three-dimensional convective-dispersive equation was also developed to describe the movement of a conservative, noninteracting tracer in nonhomogeneous, anisotropic porous medium. Convective-dispersive equations are solved by the method of characteristics. Attenuation processes have been incorporated into the model since its original release. The model has been tested for salt (NaCl) movement in sample plots and is presently being used in coal mine contamination studies.

FEMWASTE (Yeh, 1981)

FEMWASTE, developed by G.T. Yeh at Oak Ridge National Laboratory, is a two-dimensional, finite element, mass transport model for the coupled saturated-unsaturated hydrologic zones. This model utilizes FEMWATER, also developed by Yeh, to provide the groundwater flow field allowing for a variety of boundary conditions and initial moisture conditions. Additionally, FEMWASTE incorporates the effects of convection, dispersion, chemical sorption and first order decay in the mass transport equations. FEMWASTE/FEMWATER is computer coded and available to the public. This model has been field verified and is presently being used by the Carson City office of the USGS.

Random Walk Solute Transport Model (Prickett et al., 1981)

The Random-Walk Solute Transport Model, developed by Prickett, Naymik and Lonnquist (1981), predicts the transport of chemical species (e.g., organics, metals, inorganics) in the saturated zone by the random-walk or particle-in-a-cell method. Mass transport equations include provisions for dispersion and chemical reactions (attenuation). The model also accounts for time varying pumpage, injection by wells, natural or artificial recharge, water exchange between surface water and groundwater, and flow from springs. Chemical constituent concentrations in any segment of the model can be specified. Flow equations are solved by finite difference methods. The model has been documented and made available to the public. Analytical and field (i.e., fertilizer plant, Meredosia, Illinois) verifications have been performed.

Solute Transport and Dispersion Model (Konikow and Bredehoeft, 1974)

The Solute Transport and Dispersion Model simulates the movement of conservative chemical species in a two-dimensional, coupled unsaturated-saturated hydrologic zone. Flow equations are solved using the finite difference method while mass transport equations are solved by the method of characteristics. The model allows for the incorporation of pumping or recharging wells, diffuse infiltration, and for varying the transmissivity, boundary conditions, contaminant concentrations, and saturated thickness. Analytical and numerous field verifications have been performed for the model (e.g., Hanford Reservation, Washington for radioactives; Rocky Mountain Arsenal, Colorado for pond leachate).

SWIFP (USGS, 1978)

The SWIFP model simulates the movement of nonconservative constituents through the saturated zone in three-dimensions. The model incorporates dispersion processes and also allows for deep well injection predictions. Flow and transport equations are solved by the finite difference method. This model is well documented and maintained, and has been field and analytically verified. Presently field verifications are being performed in New Jersey for sea-water intrusion, in Carson City for geothermal transport, and in Minnesota for coal tar residues. A version of SWIFP has also been developed for the Nuclear Regulatory Commission to handle radioactive materials.

Solute Transport/Groundwater Flow (Golder Associates, 1982)

This model simulates the movement of multiple conservative constituents without dispersion in the saturated zone. Flow and mass transport equations are solved using finite element techniques. The model has been analytically and field verified.

Leachate Organic Migration and Attenuation Model (Sykes et al., 1982)

The Leachate Organic Migration and Attenuation Model simulates the movement of nonconservative organic solutes through the saturated-unsaturated zone. The model is generally run in one- or two-dimensions but can be modified for three-dimensional analysis. Flow and mass transport equations are solved by finite element techniques. This model is specific to sanitary landfills because it measures organics as chemical oxidation demand, and addresses biodegradation, adsorption, convection, and dispersion processes. The model is currently being revised. Field verification has been performed for the model at the Borden Landfill, Ontario, Canada for chloride and potassium, at granite sites for nuclear wastes, and for aldicarb.

5.3 MODEL LIMITATIONS

The following issues provide a context for the considering the limitations inherent in the application of models to evaluate or predict groundwater problems:

- Modeling results represent approximations of the actual movement of contaminants and groundwater; results should be used to estimate the comparative magnitude of a problem and to assign priorities.

- Models should be verified against actual field observations to determine how closely they simulate real world situations; verification should be performed in the actual hydrogeologic system to which the model is going to be applied.

- Model accuracy may vary dramatically when models are applied to situations for which they have not been verified.

- Models presently do not simulate all the processes that control contaminant movement; the equations that describe attenuation and dispersion are especially weak

- Generally the capacity of a model to simulate field situations is a function of its complexity; the more complex the model the more data are required, and the model reliabiity becomes a function fo data accuracy, i.e., "garbage in-garbage out"

- Models for which sensitivity analyses have not been conducted may generate mathematical errors when parameters are changed and assumptions modified.

Because of the complexity and limitations of models, assertions determined through the use of models should not be interpreted as actual values but only as estimates.

6.0 RECOMMENDATIONS

Zone of saturation landfills are by nature very susceptible to contaminant releases because of the large amounts of water, both groundwater inflows and leachate, that must be removed by the drainage system. If the only sites available for construction of the landfills are in the saturated zone, the following recommendations may help limit the probabilities of leachate release:

- Determine as accurately as possible the variables of inflow rates (q) and hydraulic conductivities (K) of both landfill material and soils because these values have a major impact on drain system design, especially drain spacing

- Determine the hydrogeology of the site accurately so that groundwter level variations throughout the year are known and conservative head maintenance levels can be specified (i.e., always maintaining inward hydraulic gradients)

- Design the landfill base so that it slopes toward the drains thus allowing for lower head maintenance levels and quicker leachate removal

- Specify low head maintenance levels within the fill, thus reducing the hydraulic head capable of discharging leachate (i.e., for hazardous waste sites EPA regulation require heads less than 1-foot)

- Select the proper drain equation for system design and allow for a margin of error in the results obtained (i.e., design conservatively)

- Design filters and evelopes for drain pipes that prevent silting and allow for free flow of liquids

- Remove daily fill covers from cells or ensure by some other means that the cells are hydraulically connected to the drainage system thus allowing free flow of water between cells and drains

- Incorporate drain blankets into landfill designs to reduce head maintenance levels without decreasing drain spacing and improve the overall efficiency of leachate collection

- Compact clay base and sidewall layers at optimum working moisture conditions so that high density, low permeability barriers are created without shearing or fracturing the clay

- Provide construction inspections to ensure that critical operations such as placement of leachate collection drains and filter envelopes, base gradings, and clay recompaction are performed as specified

- Design leachate collection systems so that routine maintenance and inspections can be performed to adequately maintain flows

- Provide for continuous monitoring of leachate collection volumes and head levels so that problems can be identified quickly

- Design landfill drainage systems to incorporate anticipated future operations such as the acceptance of large quantities of liquids.

Understanding the drain equations and the theory behind them and incorporating the above recommendations into the original landfill design can substantially reduce the chances of leachate release. However, unless the designs are incorporated properly during construction, the system will fail to meet its intended purpose.

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