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Leachate Plume Management

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LEACHATE PLUME MANAGEMENT

by

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This handbook is intended to present information on the application of technologies for the control of specific problems caused by uncontrolled waste sites. The handbook is not intended to address every conceivable waste site problem or all possible applications of these technologies. Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

FOREWORD

This is one of a series of reports being published to implement CERCLA, otherwise known as Superfund legislation. These are documents explaining the hazardous response program and, in particular, the technical requirements for compliance with the National Contingency Plan (NCP), the analytical and engineering methods and procedures to be used for compliance, and the background and documenting data related to these methods and procedures. The series may include feasiblity studies, research reports, manuals, handbooks, and other reference documents pertinent to Superfund.

This handbook provides an overview of the fundamental concepts, procedures, and technologies used in leachate plume management. Plume generation dynamics and delineation are discussed. Plume control technologies are evaluated and selection criteria for site applications are defined. Groundwater pumping, subsurface drains, low permeability barriers, and innovative technologies are the aquifer restoration technologies addressed in this handbook.

The handbook provides governmental and industrial technical personnel with the means to successfully control leachate plumes from uncontrolled hazardous waste sites. In conjunction with other publications in this series, this handbook will assist in meeting the national goal of a cleaner, safer environment.

ABSTRACT

The problem of leachate plume management has been aggravated to some extent by a lack of understanding of plume dynamics and the various remedial options available. This handbook summarizes information in the areas of leachate plume dynamics and plume management alternatives. The handbook describes factors that affect leachate plume movement, key considerations in delineating the current and future extent of the leachate plume, technologies for controlling the migration of plumes, and criteria for evaluating and selecting plume management alternatives. The handbook consists of eight chapters:

- Chapter 1 -- Introduction includes an overview discussion of leachate plume generation and a summary of the handbook's contents.
- Chapter 2 -- Plume Dynamics includes discussions on the effects of groundwater flow patterns, leachate characteristics, and plume and geologic media interactions.
- Chapter 3 -- Plume Delineation includes discussion on data sources and procedures for estimating plume boundaries, characterizing plume chemistry, and extrapolating future plume movement.
- Chapter 4 -- Plume Control Technologies includes an overview discussion of plume control technologies and how they can be evaluated and selected for site applications.
- Chapter 5 -- Groundwater Pumping includes discussions on well hydraulics and well design, installation, operation, and maintenance.
- Chapter 6 -- Subsurface Drains includes discussions on drain hydraulics and drain design, installation, and maintenance.
- Chapter 7 -- Low Permeability Barriers includes discussions on barrier materials, placement, design, installation, and maintenance.
- Chapter 8 -- Innovative Technologies includes discussions on bioreclamation, in situ chemical treatment, and emerging technologies.

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SYMBOLS

A		Area or cross-sectional area (length^2)
	A_d	Area affected by drain (length^2)
	A_n	Area normal to flow direction (length^2)
	A_o	Open area of well screen (length^2)
	A_p	Cross-sectional area of pipe
	A_s	Cross-sectional area of sample (length^2)
	A_w	Cross-sectional area of a well (length^2)
a		Geometric factor for radial flow (dimensionless) or volume fraction (dimensionless)
B		Width of area drained by a drainage pipe (length)
b		Width of trench (length)
C		Concentration of a substrate (wt/vol)
	C_f	Final concentration of substrate (wt/vol)
	C_i	Initial concentration (wt/vol)
	C_L	Concentration of oxygen in a liquid (wt/vol)
	C_m	Microbe concentration (wt/vol)
	C_o	Concentration of oxygen in a liquid (wt/vol)
	$C_{u/2}$	Concentration of substrate supporting half of the maximum growth rate (wt/vol)
	C_u	Uniformity coefficient (dimensionless)
c		Constant
D		Height of the drain above an impervious layer (length)
	D_b	Thickness of water bearing zone below drain (length)

	D_d	Distance downgradient from the drain where the water table is lowered to a desired depth (length)
	D_e	Difference in water elevation in a pumped sump (length)
	D_o	Distance from the water level in a drain to the next soil layer (length)
	D_p	Distance a leachate plume migrates from source (length)
	D_r	Aquifer thickness where radial flow occurs (length)
	D_u	Effective distance of drawdown upgradient (length)
	D_v	Aquifer thickness where vertical flow occurs (length)
	D_x	Diameter (length)
	D_1	Distance from the ground surface to water table at the drain (length)
	D_2	Distance from the ground surface to water table at distance D_d from the drain (length)
d		Equivalent depth of aquifer below drain (length)
	d_e	Effective drain depth (length)
E		Elevation (length)
	E_1	Groundwater elevation at start of aerated flow (length)
	E_2	Groundwater elevation at end of aerated flow (length)
	E_a	Activation energy
e		Base of natural logarithms
G		Geometric factor for radial flow
g		Acceleration of gravity (length/time^2)
H		Total hydraulic head (length) or Maximum height of water table above the drain midway between drains (length)
	H_k	Henry's law constant for oxygen
	H-h	Drawdown (length)
h		Height of water table after drawdown (length) or depth to bottom separation

h_d	Desired depth of water table after draining (length)
h_e	Final head (length)
h_f	Total frictional losses in well system expressed as head (length)
h_i	Initial head (length)
h_l	Total vertical lift in a well (length)
h_s	Head difference across sample (length)
h_t	Total dynamic head in a well (length)
h_v	Velocity head required to produce flow in a well (length)
h_w	Height of water in the well (length)
h_{wm}	Optimum drawdown (length)
h_1-h_2	Difference in water table levels at two points on a line parallel to flow direction (length)
I	Hydraulic gradient
i	Inflow rate (vol/time)
K	Hydraulic conductivity (length/time)
K_h	Horizontal hydraulic conductivity (length/time)
K_L	Hydraulic conductivity of lower aquifer (length/time)
K_r	Hydraulic conductivity of aquifer with radial flow (length/time)
K_v	Hydraulic conductivity of aquifer with vertical flow (length/time)
K_w	Weighted average hydraulic conductivity (length/time)
K_z	Vertical hydraulic conductivity (length/time)
K_1	Hydraulic conductivity of aquifer above drain (length/time)
K_2	Hydraulic conductivity of aquifer below drain (length/time)
K'	Hydraulic conductivity of leaky confining layer (length/time)
k	Intrinsic permeability (length^2)
k_a	Acid catalyzed hydrolysis constant
k_{app}	Apparent constant

	k_b	Base catalyzed hydrolysis constant
	k_d	Distribution coefficient
	k_m	Roughness factor
	k_N	Neutral hydrolysis constant
	k_p	Partition coefficient of substance
L		Influence of relief drain or drain spacing (length)
	L_a	Length of aerated zone (length)
	L_e	Downslope influence of drains (length)
	L_f	Length of groundwater flow path between h_1 and h_2 (length)
	L_s	Length of sample (length)
l		Length of wellscreen as a fraction of aquifer thickness (dimensionless)
m		Aquifer thickness (length)
	m_s	Saturated thickness at aquifer (length)
	m'	Thickness of leaky confining layer (length)
N		Roughness coefficient
n		Effective porosity (percent)
P		Air pressure (wt/vol)
	P_r	Average earth mass density
	ΔP	Pressure in excess of the overburden
p		Density (wt/vol)
Q		Discharge (vol/time)
	Q_g	Gravity discharge capacity (vol/time)
	Q_I	Recharge rate of injection well (vol/time)
	Q_m	Pumping discharge rate (vol/time)
	Q_p	Pumped discharge capacity (vol/time)
	Q_R	Discharge rate of pumping well (vol/time)

	Q_s	Specific capacity of fully penetrating well (vol/time)
	Q_{sp}	Specific capacity of partially penetrating well (vol/time)
	Q_y	Expected or desired yield of well (vol/time)
	Q_1	Discharge of high pressure supply water (vol/time)
	Q_2	Discharge of groundwater to supply line (vol/time)
q		Rate of water flow, specific discharge, or drainage coefficient (length/time)
R		Hydraulic radius (length)
	R_o	Radius of influence of well (length)
	R_g	Gas content (vol)
	R_i	Initial reaction rate
	R_s	Shear rate
	R_t	Temperature correct reaction rate
r		Distance from well to measured drawdown (length)
	r_d	Drain radius (length)
	r_s	Equivalent well radius (length)
	r_w	Well radius (length)
S		Coefficient of storage
	S_v	Sump storage volume (vol)
	S_y	Specific yield (percent)
s		Slope of the angle between the water table and horizontal plane
T		Transmissivity (length^2/time)
	T_a	Absolute temperature (degrees)
	T_g	Initial gel strength
	T_o	Yield stress
	T_s	Shear stress
t		Time (time)

	t_c	Cycle time of pump (time)
	t_p	Pumping time (time)
	t_r	Residence time (time)
	$t_{1/2}$	Half life (time)
U		Uniformity coefficient
	U_m	Maximum growth rate of microbes
u		Wetted perimeter and drain (length) or specific weight of fluid
V		Velocity or average linear velocity (length/time)
	V_b	Water velocity in permeable treatment bed (length/time)
	V_g	Groundwater velocity (length/time)
	V_s	Entrance velocity (length/time)
v		Viscosity
	v_a	Apparent viscosity
	v_p	Plastic viscosity
W(u)		Well function (dimensionless)
W_b		Thickness of permeable treatment bed
x		Length of drain pipe (length)
Y_d		Yield coefficient
		Volume fraction
μ_A, μ_b, τ		Well functions

CONVERSION FACTORS

Length-

1 inch (in) = 25.4 millimeters (mm).

1 foot (ft) = 0.3048 meters (m).

1 yard (yd) = 0.9144 meters (m).

1 mile (mi) = 1.6093 kilometers (km)

Area-

1 square inch (in^2) = 645.2 square millimeters (mm^2).

1 square foot (ft^2) = 0.0929 square meters (m^2).

1 square yard (yd^2) = 0.8361 square meters (m^2).

1 square mile (mi^2) = 2.59 square kilometers (km^2).

1 acre (ac) = 0.4047 hectares (ha).

Volume-

1 cubic inch (in^3) = 16.39 cubic centimeters (cm^3)

1 cubic foot (ft^3) = 0.02832 cubic meters (m^3)
= 28.32 liters (l)

1 cubic yard (yd^3) = 0.7646 cubic meters (m^3)
= 764.6 liters (l)

1 gallon = 0.0038 cubic meters (m^3)
= 3.785 liters (l)

1 acre-foot (af) = 1,233.5 cubic meters (m^3)
= 1,233,000 liters (l)

1 second foot day (sfd) = 2,451.3 cubic meters (m^3)
= 2,450,000 liters (l).

Mass-

1 ounce (oz.) = 28.35 grams (g).

1 pound (lb.) = 0.4536 kilograms (kg).

1 ton = 907.2 kilograms (kg)
= 0.907 tonne (t)

Velocity, linear-

1 foot per second (ft/sec) = 30.48 centimeters per second (cm/s)

1 mile per hour (mph) = 1.609 kilometers per hour (km/hr) = 4.47×10^{-4} meters per second (m/s).

Flow Rate (volumetric)-

1 gallon per minute (gpm) = 0.0631 liters per second (l/s) = 6.309×10^{-5} cubic meters per second (m^3/s) = 5.451 cubic meters per day (m^3/d)

1 cubic foot per second (ft^3/s) = 0.02832 cubic meters per second (m^3/s) = 28.32 liters per second (l/s).

Transmissivity-

1 square foot per day (ft^2/day) = 0.09289 square meters per day (m^2/d).

1 gallon per day per foot (gpd/ft) = 0.01242 square meters per day (m^2/d).

Viscosity-

poise = 1.45×10^{-5} pounds (weight) seconds per square inch ($lb.sec/in^2$)

Density-

1 pound per cubic foot (pcf) = 16.02 kg/m^3

Pressure-

1 pound per square inch (psi) = 70.31 grams per square centimeter (g/cm^2) = 68948 dynes per square centimeters ($dynes/cm^2$) = 6894.8 newtons per square meter (N/m^2).

Force-

1 pound (lb, weight) = 4.448 meters = 4.448×10^5 dynes = 33.36 poundals.

Temperature-

Degrees Celsius (°C) = 5/9 (°F-32)

Degrees Kelvin (°K) = degrees °C + 273.16

Work, Energy, Power, Quantity of Heat-

1 kilowatt-hour = 3.6×10^6 joules = 3,409.52 British Thermal Units (Btu).

1 horsepower = 0.746 kilowatt (kW).

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CHAPTER 1

INTRODUCTION

Cleaning up the thousands of hazardous waste sites identified across the United States is a serious environmental challenge. Of particular importance is protecting and cleaning up contaminated groundwaters that now serve or could serve as drinking water supplies. The magnitude of this problem is illustrated by the fact that nearly 70 percent of sites surveyed in 1983 and now undergoing corrective action have contaminated groundwaters (JRB Associates and Environmental Law Institute, 1983).

Contaminants can enter the groundwater system through a variety of mechanisms. Some disposal sites contain fine solids or sludges that can be transported via infiltrating solutions. The amount of solids transported depends on the size of the particles, the size and interconnectedness of void spaces, the speed of percolating solutions, and the surface area of waste in contact with the solutions. The maximum size of a particle transported is a function of pore space diameter and percolation rate. As the rate of solution movement increases, the solution's capacity to carry solids in suspension increases. When large areas of fine particle wastes are in contact with percolating solutions, a greater opportunity for transport of solids exists.

In areas where the geologic material between the waste site and the groundwater table is permeable, movement of liquid waste to the groundwater regime is possible. The amount of direct liquid waste seepage is highest when:

- Highly permeable material exist within the waste site and between the site and the groundwater table

- The distance separating wastes and groundwater is minimal, or the wastes are in direct contact with the groundwater
- The wastes consist of uncontainerized liquids that have a higher density and lower viscosity than water.

Waste organic and inorganic solutions can dissolve other waste constituents, thereby greatly increasing the mobility of some contaminants. Solvents, soaps, and emulsifying agents can also dissolve or suspend waste constituents in infiltrating solutions. Therefore, water insoluble wastes should not necessarily be considered immobile.

By far the most predominant means of contaminant movement to the groundwater system is via dissolution by infiltrating precipitation. Wastes dissolved into infiltrating solutions are carried through the waste site and underlying soil, along solution channels or seepage paths, and to the groundwater table.

Solutions resulting from the dissolution of waste constituents into percolating water are called leachate. Leachate concentration depends in part on waste constituent solubility, volume of precipitation, and length of time the infiltrating solution is in direct contact with waste. Certain constituents are more soluble in rainwater than distilled water because rainwater is chemically different from distilled water. For example, the pH of distilled water is 7.0, while rainwater in nonurban, nonindustrialized areas commonly has a pH between 5 and 6 (Freeze and Cherry, 1979). Part of this acidity is caused by the presence of dissolved CO_2 which produces up to one percent carbonic acid (Gilluly, et al., 1975). The pH of acid rain can be as low as 3 to 4 because of high sulfur oxides or nitrogen oxides in air which dissolve to produce sulfuric and nitric acids in the rainwater (Freeze and Cherry, 1979). In addition to low pH, rainfall also contains dissolved solids and can entrain microbes and other matter as it percolates through soils or wastes. Thus, wastes that do not have a high solubility in distilled water under laboratory conditions may become mobile in rainwater.

The final means by which contaminants can be leached from a waste disposal site is dissolution by groundwater. While this route is important only when wastes are in direct contact with groundwater, this route can be significant because of the large volumes of water that can become contaminated.

Once in groundwater, contaminants are not diluted and flushed from the system to the extent they would be in surface water because flow rates are slower and flow paths more tortuous. As a result, contaminants tend not to disperse but form slugs or plumes. Contaminant concentrations throughout a leachate plume can vary considerably but are generally highest in the center of the plume and closest to the contaminant source area. The boundaries of a leachate plume are usually indistinct, which makes vertical and horizontal plume delineation and control a very complicated process.

Leachate plume management has been complicated to some extent by a lack of understanding of plume dynamics and the various site remediation options available. Generally, methods for controlling the migration of a leachate plume can be placed into one of four categories--groundwater pumping, subsurface drains, low permeability barriers, and innovative technologies. Under the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA), also known as "Superfund," the Environmental Protection Agency is implementing a program to assess and disseminate information on these technologies and procedures related to plume management.

The purpose of this handbook is to serve as a summary reference on factors that affect leachate plume movement and considerations necessary to develop effective plume management plans. The handbook consists of eight chapters as follows:

- Chapter 1 -- Introduction.
- Chapter 2 -- Plume Dynamics. Includes discussions of the factors that affect a plume's movement and shape in an aquifer.
- Chapter 3 -- Plume Delineation. Includes discussions of plume delineation techniques such as the use of models, and indirect and direct methods.

- Chapter 4 -- Plume Control Technologies. Includes brief discussions about various plume control technologies including groundwater pumping, drains, barriers, and innovative technologies, and on the selection of a plume control technology to best meet site conditions.

- Chapter 5 -- Groundwater Pumping. Includes discussions of well hydraulics; well construction and installation; and system design, operation, and maintenance.

- Chapter 6 -- Subsurface Drains. Includes discussions of drain hydraulics; drain components and materials; and system design, installation, and maintenance.

- Chapter 7 -- Low Permeability Barriers. Includes discussions of barrier placement; types of barrier material; and barrier design and installation.

- Chapter 8 -- Innovative Technologies. Includes discussions of bioreclamation, in-situ chemical treatments, and emerging technologies.

Many topics related to controlling the migration of leachate plumes are not covered in this handbook. Treatment of leachate is discussed by EPA (1980), De Renzo (1978), Hammer (1975), and Liptak (1975). Complementary remedial action technologies are summarized by EPA (1982a). Procedures for safeguarding worker health and safety are addressed by Heiss (1980), Lippitt, et al. (1982), Allcott, et al. (1981), Turpin (1981), Weitzman and Cohen (1981), and NEIC (1980). The role of community relations during site remediation is discussed by Cohen, et al. (1981), Neuman and Drake (1982), and ICF (1982). Finally, case studies of leachate plume control have been described by SCS (1980), JRB and ELI (1983), and Nielson (1982).

CHAPTER 2

PLUME DYNAMICS

Leachate plumes are subject to various processes which alter their size and shape, and their direction and flow rate. These processes are related to the nature of the contaminants; the groundwater flow patterns existing in the area of concern; and the interactions between the contaminants, groundwater, and the geologic media through which the plume moves. While all the effects of these processes are not known precisely and are still being researched, general relationships have been observed or can be approximated. The purpose of this section is to discuss, in general terms, the processes affecting plume movement.

When leachate seeps into groundwater, the leachate tends not to disperse evenly. Instead, variations occur in contaminant concentration depending on position within the groundwater system. The highest concentrations of contaminants are generally found close to the source and near the center of the plume where dilution is minimal. As plume size and degree of dilution increase, the relative contaminant concentration decreases. However, the volume of groundwater affected increases, and thus the cost for treating affected water also increases.

Among other factors, plume size is determined by hydrologic factors, age of the contaminant source, and rate of contaminant release. The width and depth of a plume are directly influenced by the width and thickness of the aquifer, the aquifer's flow characteristics, and the properties of the contaminants. In most instances, plumes move in the direction of groundwater flow and are elongated in that direction. Some hydrogeologic conditions, however, cause anomalous plume migration patterns to develop. Plume shape is

a reflection of the flow patterns within the aquifer and the chemical characteristics of the leachate. In general, groundwater flow patterns tend to exert the greatest influence on plume shape and path.

Factors affecting plume migration can be grouped into three broad areas:

- Groundwater flow patterns--factors that alter groundwater flow patterns
- Leachate characteristics--the properties of the hazardous materials
- Plume and geologic media interactions--the reactions between waste and contaminant constituents and the geologic materials.

These topics are discussed in greater detail in the following sections.

2.1 Groundwater Flow Patterns

Groundwater flow patterns probably exert the greatest influence on leachate migration of all the factors affecting plume movement. This factor is probably the best understood of all the factors. Nevertheless, the site-specific influence of groundwater flow on plume movement at individual hazardous waste disposal sites is often difficult to assess because of variations in site conditions. Variations can be a result of:

- Subsurface hydrogeologic conditions
- Surface hydrologic conditions
- Human activities.

In order to understand the effects of these conditions, certain hydrogeologic concepts must be understood. These concepts are outlined in the following section.

2.1.1 Basic Hydrogeologic Concepts

Precipitation falling on land can run off into surface water bodies, directly evaporate, be taken up and used by vegetation (transpiration), or

infiltrate into the subsurface. Once in the subsurface, water may be forced back to the surface and evaporated through capillary action, may be taken up by deeper root systems and transpired, or may be transported downward under the influence of gravity. Water in the unsaturated material will move downward until it reaches the saturated zone where all available pore space (i.e., openings between particles making up unconsolidated materials, or cracks, fissures, openings in consolidated rock) are filled with water. These processes and their interactions are termed the hydrologic cycle.

Aquifers are subsurface water bearing zones capable of supplying water in usable quantities (Gary, et al., 1974). Both unconsolidated and consolidated (i.e., rock) materials can function as aquifers. Groundwater flows through voids within the aquifer, and the ratio (usually expressed as a percentage) of the volume of voids in a material to the total volume of material is termed porosity (Gary, et al., 1974). Table 2-1 lists some typical ranges of values for porosity for different geologic materials. Porosity and rate of flow are not necessarily proportional; a sample with a high porosity may not allow the passage of water unless pore spaces are interconnected. An example of this is cork, which has a high porosity yet is relatively impermeable. In addition, some highly porous, well interconnected samples (e.g., clays) still do not allow for the passage of water for other reasons. The effective porosity (n) refers to the ratio of the volume of interconnected pore spaces to total volume, and is equal to the ratio of the volume of liquid that a sample will yield under specific conditions after it is saturated (Freeze and Cherry, 1979). Groundwater flows through the interconnected pores, fractures, joints or openings that provide a flowpath.

A measure of the rate of groundwater flow is the hydraulic conductivity (K). Hydraulic conductivity is the volume of water that will move per unit time under a unit hydraulic gradient through a unit area measured at right

TABLE 2-1.

RANGE OF POROSITY VALUES FOR VARIOUS GEOLOGIC MATERIALS

Geologic Material	Freeze and Cherry (1979)	Fetter (1980)	Pettyjohn (1975)	Linsely, et al. (1975)	Davis and De Wiest (1966)
Gravel, mixed	25-40%			25%	25-38%
Gravel, well sorted		25-50%			
Gravel and sand, mixed		20-35%		20%	
Sand, mixed	25-50%		35-40%	35%	
Coarse sand			39-41%		33%
Medium sand			41-48%		36-42%
Fine sand			44-49%		45-52%
Silt	35-50%	35-50%			49-51%
Clay	40-70%	33-60%	50%	50%	48-67%
Glacial till		10-20%			
Carbonate Rocks (e.g., limestone)	0-50%	1-30%		5%	2-16%
Sandstones	5-30%	3-30%	15-20%	15%	12-29%
Shales	0-10%		0-10%	5%	
Crystalline Rocks (i.e., igneous and metamorphic rocks)					
highly fractured	5-50%				36-40%
relatively unfractured	0-10%			1%	14%

angles to the direction of flow (Freeze and Cherry, 1979). K has the dimensions of length divided by time (commonly expressed in feet/day or centimeters/second). A related term is intrinsic permeability (k). Intrinsic permeability is a property of the media through which groundwater flows, has the dimensions of length squared, and is a measure of the resistance to fluid flow through the media. Of the two, K is more useful for studying plume dynamics and delineation. Table 2-2 lists ranges of values for K for various geologic materials.

While intrinsic permeability (k) is a property of the media only, hydraulic conductivity (K) depends on both the media and the fluid. For example, a material with a hydraulic conductivity for pure water of 10 ft/day, will have a lower hydraulic conductivity for a more viscous fluid. This is one reason leachate and waste contaminants may move slower than groundwater. For most groundwater situations the influence of changes in liquid properties is usually slight, and can be ignored (Freeze and Cherry, 1979). However, this may not be the case in highly concentrated or multiphase plumes.

When the density of fluid varies significantly from that of pure water, hydraulic conductivities can be calculated to account for these density changes. Hydraulic conductivity (K) can be redefined as follows (Jorgensen, et al., 1982):

$$K = kpg/v = ku/v$$

where:

k = intrinsic permeability
p = density of fluid
g = acceleration of gravity
u = specific weight of fluid
v = viscosity of fluid

The new K value then can be used in subsequent calculations.

TABLE 2-2

RANGE OF HYDRAULIC CONDUCTIVITY VALUES
FOR VARIOUS GEOLOGIC MEDIA (After Freeze and Cherry, 1979)

Geologic Material	Hydraulic Conductivity	
	cm/sec	gal/day/sq.ft.
Gravel	10^{-1} - 10^{2}	10^{4} - 10^{7}
Sand, well sorted	10^{-4} - 1	10 - 10^{5}
Silty sand	10^{-5} - 10^{-1}	1 - 10^{4}
Silt	10^{-7} - 10^{-3}	10-2 - 10^{2}
Clay, unweathered	10^{-10} - 10^{-7}	10^{-5} - 10^{-2}
Glacial till	10^{-10} - 10^{-4}	10^{-5} - 10
Carbonate rocks	10^{-7} - 1	10^{-3} - 10^{5}
Sandstones	10^{-8} - 10^{-4}	10^{-3} - 10
Shales	10^{-11} - 10^{-7}	10^{-6} - 10^{-2}
Crystalline rocks		
Highly fractured	10^{-6} - 1	10^{-1} - 10^{5}
Relative unfractured	10^{-12} - 10^{-8}	10^{-7} - 10^{-3}

1 gal/day/sq.ft. = 1.74 x 10^{-6} ft/day

1 gal/day/sq.ft. = 4.72 x 10^{-5} cm/sec

The average linear velocity (V_g) with which groundwater flows in a system is related to aquifer properties through a set of empirical relationships named after their discoverer, Henri Darcy. Darcy's law, in its most useful form, states:

$$V_g = KIA/n.$$

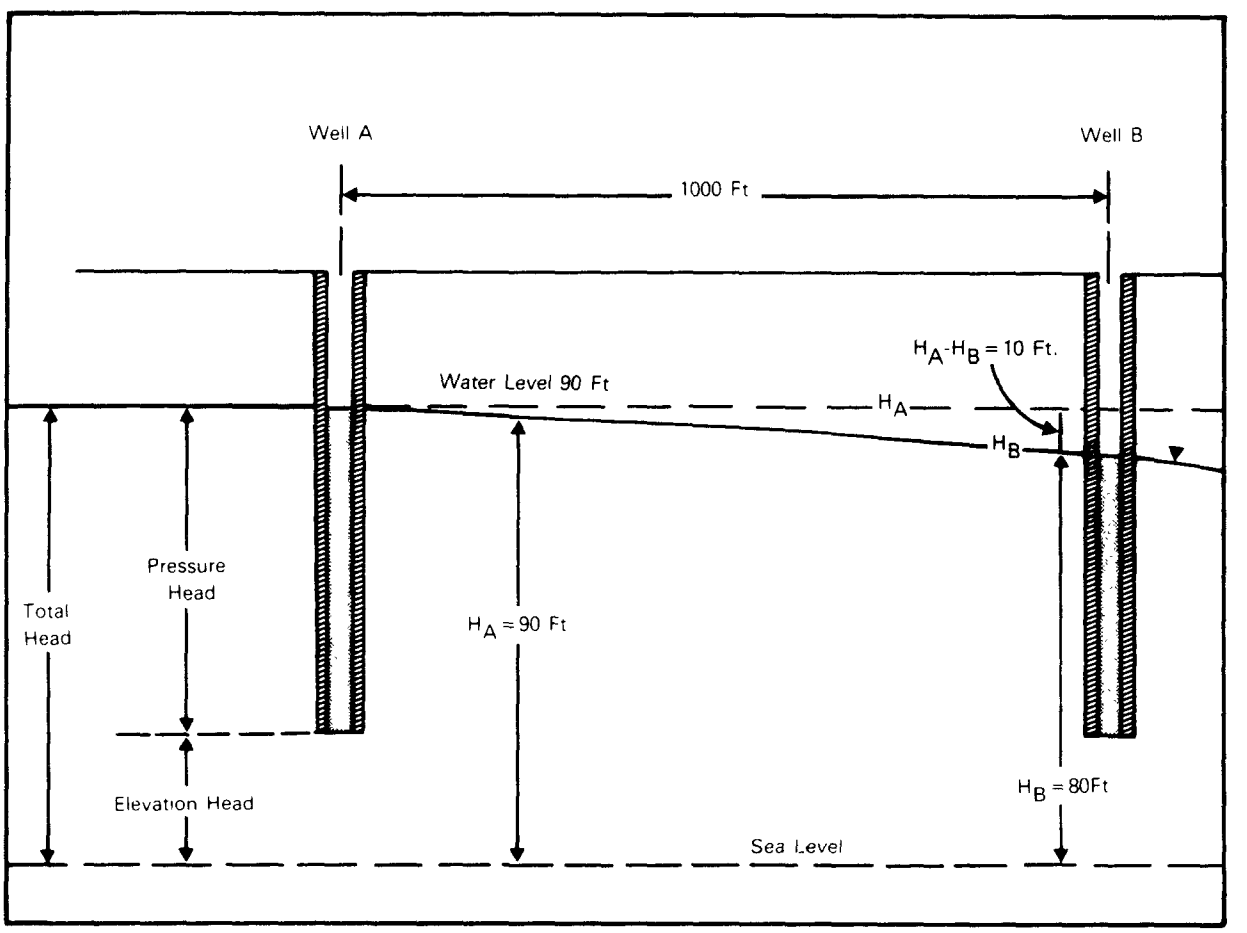
where: V_g = average linear velocity of groundwater
K = hydraulic conductivity
I = hydraulic gradient
A = cross sectional area (usually 1)
n = effective porosity.

The velocity calculated using Darcy's law is not a measure of the velocity of a water molecule as it moves within the void spaces of an aquifer; rather, it is the average rate of movement between two distant points along the same groundwater flowline in an aquifer. Darcy's law is important in delineating plumes because if V_g and the time (t) from which contaminants entered groundwater are known, then the distance that groundwater, and theoretically contaminants, will have moved over that time period (i.e., assuming non-degradation of contaminants) can be estimated.

In order to use Darcy's law in a specific situation, the hydraulic gradient (I) must be known. Hydraulic gradient is the change in head per unit distance in the direction of maximum rate of decrease in head (Gary, et al., 1974). Hydraulic head (H) is the height above a datum (usually sea level) to which a column of water can be supported by the pressure at that point. The head represents the energy level of the water at that point. The hydraulic gradient (I) is the difference in elevation of water levels in any two wells, A and B, (assuming A and B lie along the path of maximum rate of change in head), divided by the distance between A and B. The concept of head and hydraulic gradient are illustrated in Figure 2-1.

Hydraulic gradient can also be expanded to account for the effects of pressure on groundwater flow (Jorgensen, et al., 1982). The relationship

FIGURE 2-1.
EXAMPLE CALCULATION OF HORIZONTAL HYDRAULIC GRADIENT



Horizontal Gradient = (Elevation of the water table in Well A minus the elevation of the water table in Well B), divided by the horizontal distance between the two wells

$I = (H_A - H_B)/d_{AB}$

I = 10 Ft./1000 Ft.

I = 0 01%

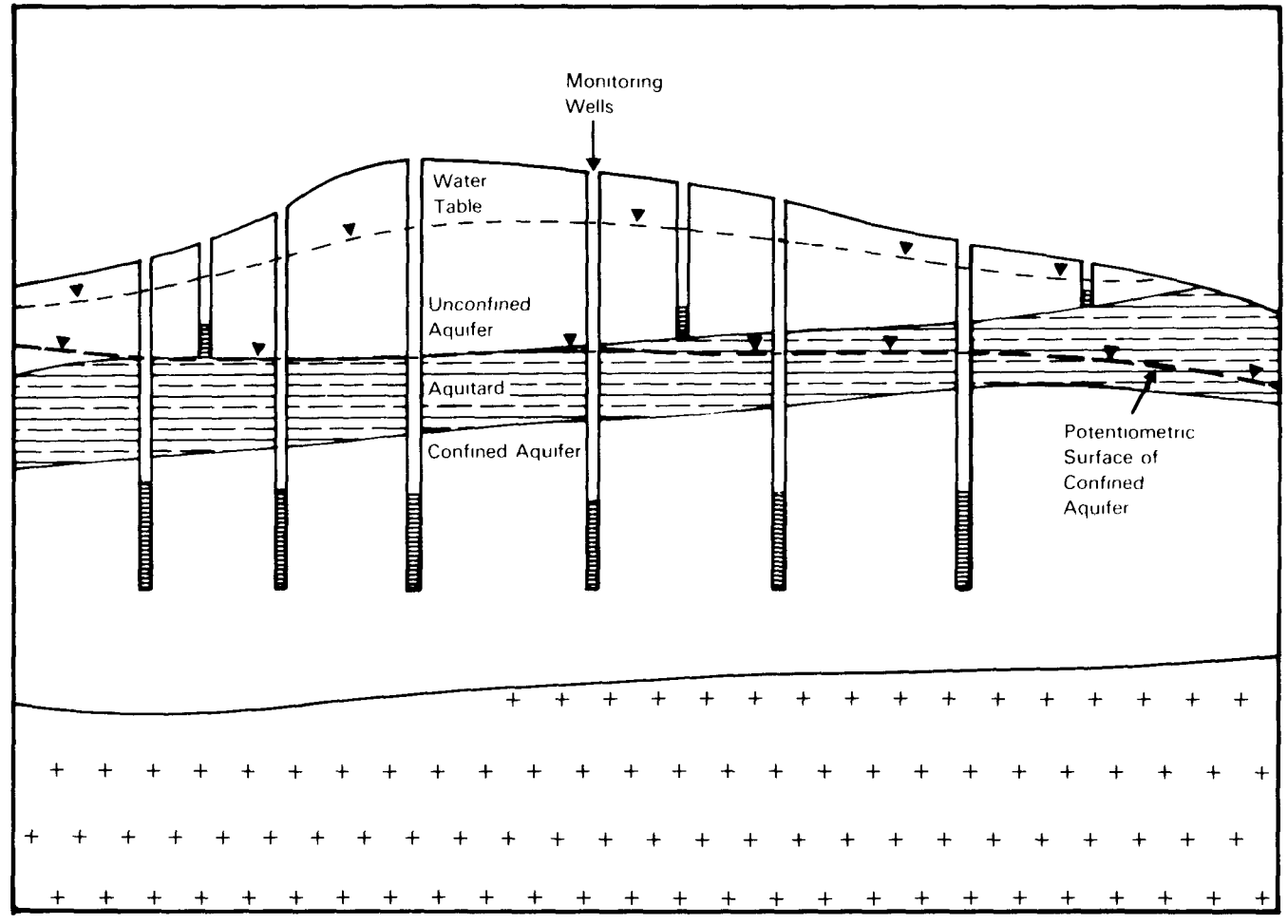
$P = \rho gH$ can be used to convert from head change equations to pressure change equations when significant density variations are present. This relationship points out that one may have zero head gradient but a definite pressure gradient because of density differences (e.g., $H_1 = H_2$, but $P_1 \neq P_2$ because $\rho_1 \neq \rho_2$).

Groundwater flows from higher energy levels to lower energy levels in the direction of decreasing head. In unconfined aquifers (aquifers that do not have a bed or layer of significantly lower K overlying the aquifer and causing the water in the aquifer to exist under pressure), the top of the saturated zone represents the energy level of the water in the aquifer and is called the water table. The water table often tends to follow land surface topography, such that the direction of flow is from areas of higher land elevation to areas of lower elevation. This is not the case in all situations. For example, in areas of flat terrain, the water table may follow steep bedrock slopes.

In situations where aquifers are saturated and separated from the land surface by zones of lower K, the energy level and direction of flow are more complicated to determine. The low K zones, known as aquitards, increase the hydrostatic pressure of groundwater in aquifers beneath them by limiting the ability of this groundwater to move upward. In these confined aquifers, the energy level of the groundwater will be greater than the saturated thickness of the aquifer. Water levels in wells penetrating a confined aquifer will rise above the level of the aquifer/aquitard interface.

An imaginary surface representing the energy level of the groundwater is known as the potentiometric surface. The potentiometric surface in an unconfined aquifer is simply the water table. Groundwater always flows from higher to lower potentiometric elevations and may be totally independent of surface topography. This concept is illustrated in Figure 2-2. Note that water levels in wells screened in the unconfined aquifer stand at different elevations than the water levels in the artesian wells.

FIGURE 2-2.
POTENTIOMETRIC SURFACES IN CONFINED AND UNCONFINED AQUIFERS



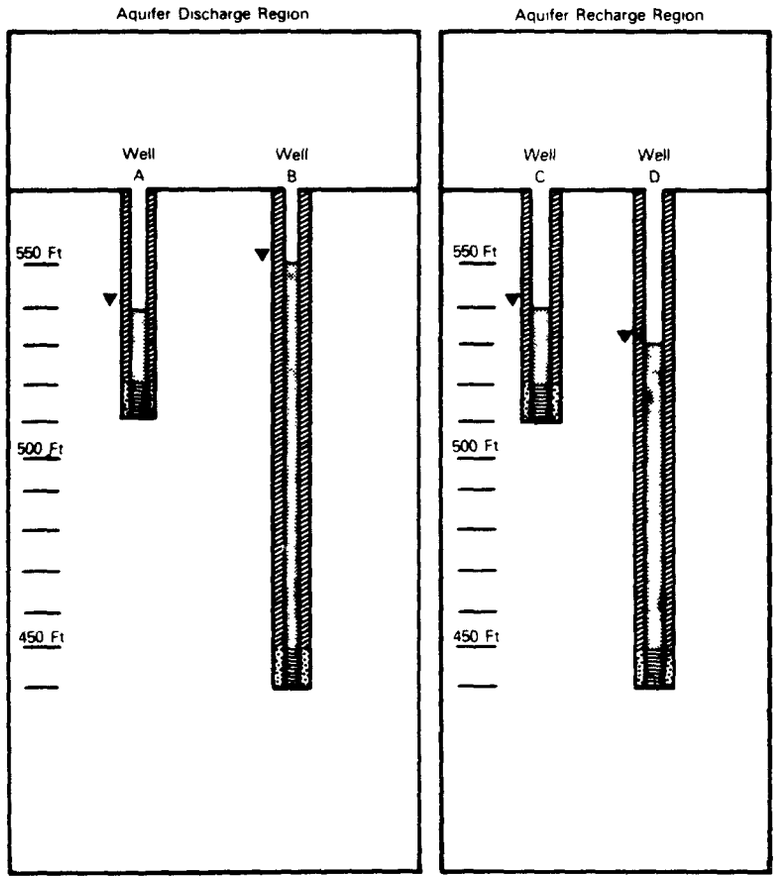
Groundwater can flow not only horizontally but also vertically. For example, two wells located close together but screened at different depths may indicate that there is a vertical hydraulic gradient, and thus a component of groundwater flow in the vertical direction, by the fact that the elevation of the potentiometric surface is different for each. This situation is illustrated in Figure 2-3. A vertical gradient may also exist between two wells screened at the same depth and having equal potentiometric surface elevations if there are sufficient differences in the fluid density in the two wells.

The fundamental problem with delineating and predicting the migration pattern of contaminated groundwater plumes is that the subsurface environment is rarely homogeneous. Zones of lower permeability and lower hydraulic conductivity can exist within aquifers, and the shape and configuration of aquifers may be highly variable. Groundwater and plume flow rates and directions will vary depending upon the characteristics of the aquifer through which they are passing. Figure 2-4 illustrates how groundwater velocity can vary with changing hydrogeologic parameters. Transmissivity (T) in Figure 2-4 is defined as the hydraulic conductivity (K) of the aquifer multiplied by the saturated thickness of the aquifer (m). Transmissivity is a measure of the quantity of groundwater flowing through an aquifer.

Variations in aquifer characteristics, primarily with regard to K, are discussed in terms of homogeneity and isotropy. If K is independent of position within an aquifer, the aquifer is homogeneous. If K is independent of the direction of measurement at a point in an aquifer (i.e., K in the vertical and horizontal directions are equal), the aquifer is isotropic. Most aquifers are heterogeneous and anisotropic, (i.e., K varies depending upon the point of measurement and the direction of measurement).

If an anisotropic aquifer has a greater vertical K than horizontal K, and all other properties (i.e., hydraulic gradient and porosity) are equal in both directions, then groundwater would flow faster vertically than horizontally. A plume in this case would tend to migrate preferentially in the vertical direction and be limited in horizontal extent. If the aquifer were also

FIGURE 2-3.
EXAMPLE CALCULATION OF VERTICAL GRADIENT



Vertical Recharge Gradient = Elevation of water table in Well A minus the elevation of the water table in Well B, divided by the vertical distance between the midpoints of the screens in the two wells.

Aquifer Discharge Region

$I = (H_A - H_B) / (\ell_A - \ell_B)$

$I = (540-550)/(515-445)$

$I = -10/70$

$I = 0.14$ (upward flow)

Aquifer Recharge Region

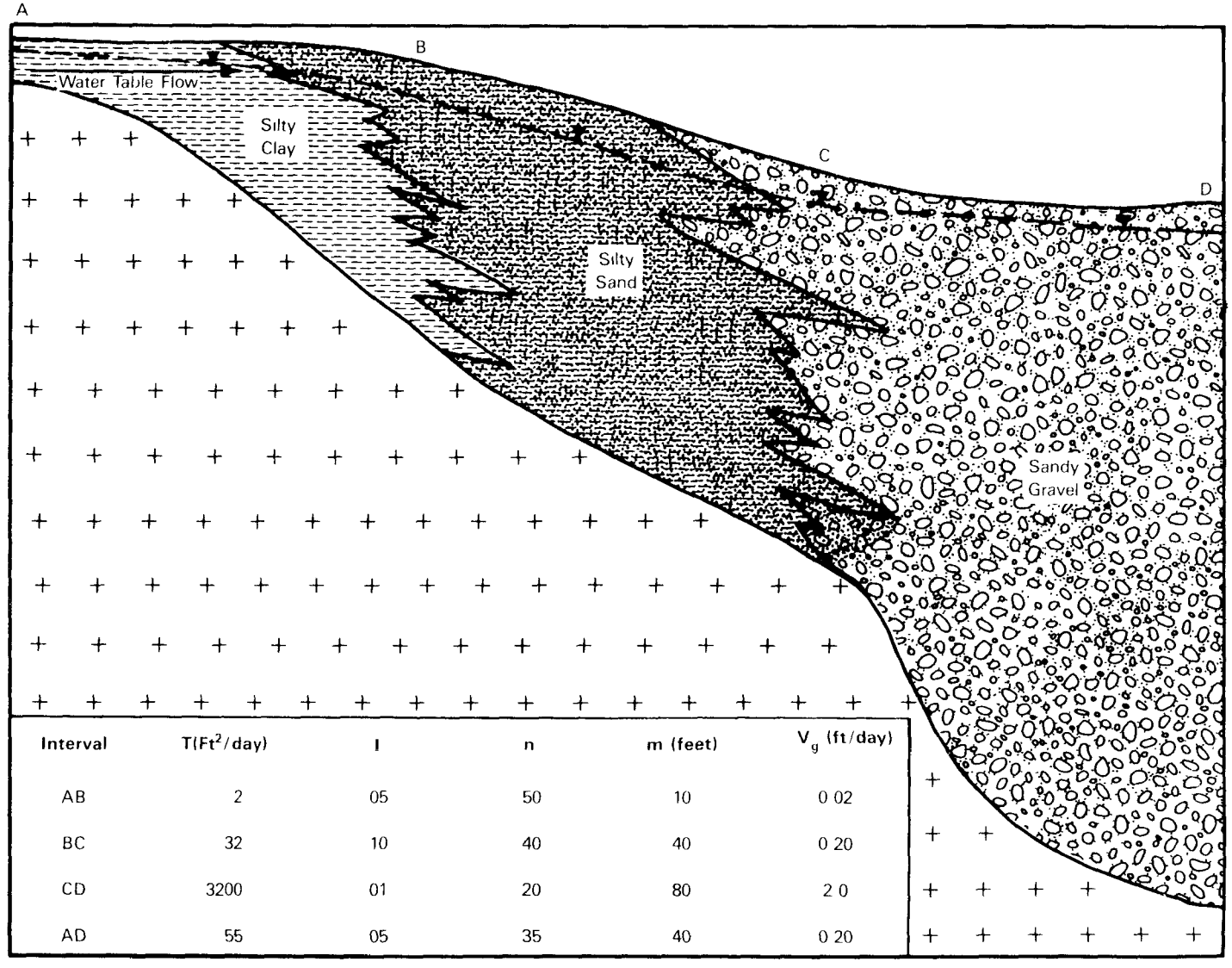
$I = (H_C - H_D) / (\ell_C - \ell_D)$

$I = (540-530)/(515-445)$

$I = 10/70$

$I = -0.14$ (downward flow)

FIGURE 2-4.
EXAMPLE OF HOW GROUNDWATER VELOCITY CAN VARY WITH DIFFERING HYDROGEOLOGIC CONDITIONS AND WITH DIFFERING SCALES OF OBSERVATION FOR KEY PARAMETERS



Interval	T(Ft^2/day)	I	n	m (feet)	V_g (ft/day)
AB	2	05	50	10	0 02
BC	32	10	40	40	0 20
CD	3200	01	20	80	2 0
AD	55	05	35	40	0 20

heterogeneous, then depending upon where the plume is located, the preferential flow paths would change. For example, the vertical K may become less than the horizontal K or the magnitude of difference between vertical K and horizontal K may change. The result of heterogeneity and anisotropy is that groundwater flow paths vary within aquifers and are not perpendicular to water level contours, thus affecting plume dynamics. For a detailed discussion of the effect of anisotropy on flow lines in aquifers the reader should refer to Fetter (1981).

In summary, groundwater flows in aquifers from higher energy levels to lower energy levels along paths of least resistance. Energy levels are represented by water levels in wells. For unconfined aquifers, groundwater flow direction in many cases mimics surface topography while direct measurements are required for confined systems. The rate of groundwater flow can be calculated using Darcy's law, if hydraulic conductivity (a measure of rate of flow in an aquifer, and indirectly a measure of resistance to flow), hydraulic gradient (a measure of the energy level driving the system), and porosity are known. Groundwater flow is complicated by the heterogeneous and anisotropic nature of the subsurface environment. This causes changes in preferential flowpath directions, and in rates of flow, depending upon position in the aquifer and direction of movement. This is a partial cause for the uneven distribution of contaminants in groundwater.

General groundwater flow theory is more complicated than explained here. For example, as described previously, hydraulic conductivity (K) and pressure (P) within the aquifer can be affected by fluid density and viscosity. These factors can impact groundwater flow directions and cause plumes to move in seemingly random directions at variable rates. However, this simplified discussion should prove adequate to understand the following sections discussing plume dynamics. Numerous introductory hydrogeology texts and research papers are available if more information on groundwater flow is desired.

2.1.2 Effects of Subsurface Conditions

The size, shape, and flow direction of a leachate plume are strongly affected by the geometry and uniformity of the aquifer through which it flows. The presence of geologic discontinuities and hydrologic barriers modify a plume's shape and flow direction. These factors are discussed below.

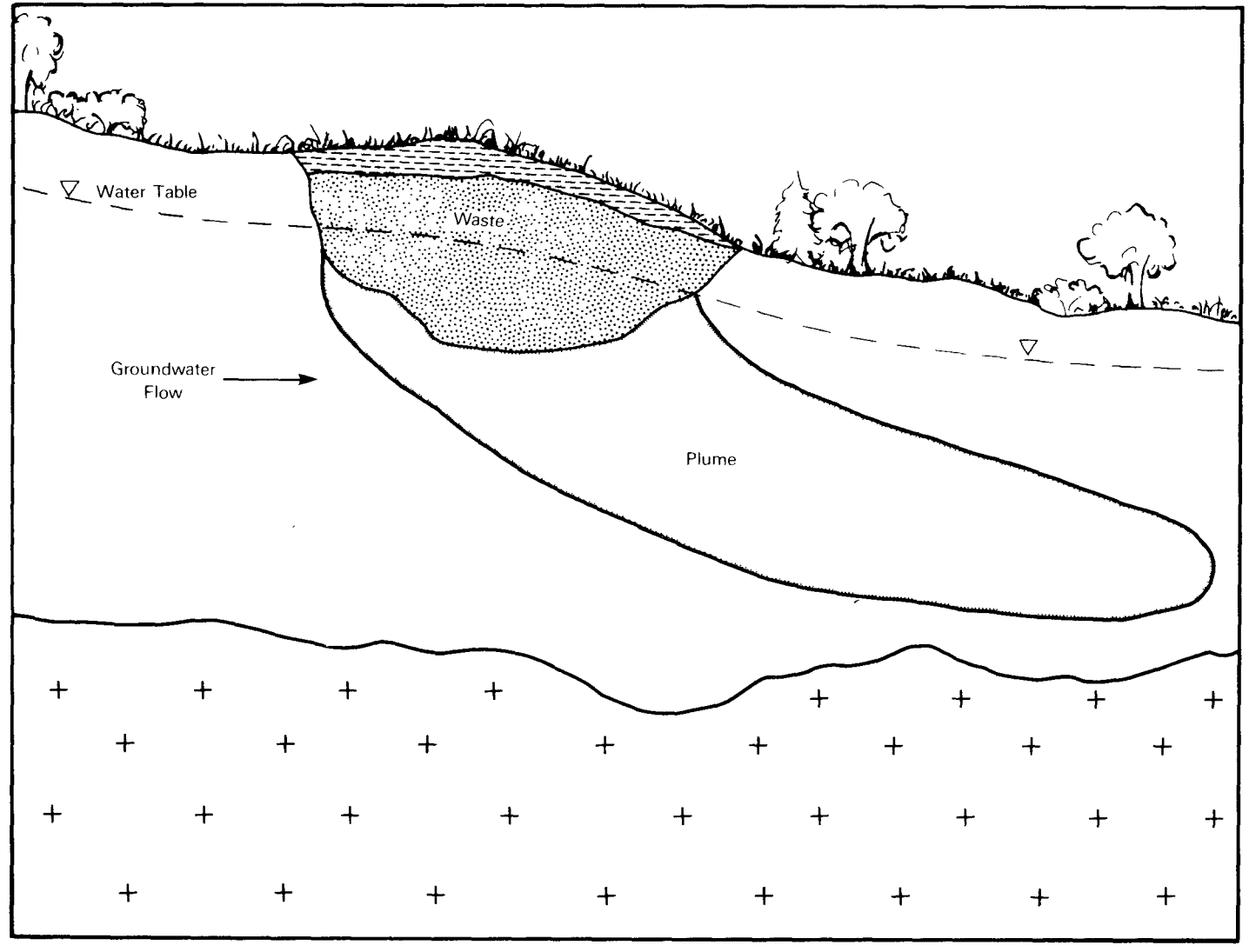
2.1.2.1 Aquifer Geometry

In large, homogeneous porous strata, the shapes of water soluble leachate plumes are relatively easy to predict. Figure 2-5 illustrates the "characteristic" shape of a water soluble contaminant migrating through a homogeneous aquifer in the direction of regional groundwater flow. In large aquifers, a low density (i.e., density less than water) contaminant can form a thin film over the upper surface of the aquifer unless the spread of the low density contaminant over the surface is inhibited for some reason. In this case, the contaminants have been observed to form a mounded layer extending above the water table (Yaniga, 1982). This situation is most likely to occur in relatively thin, unconfined aquifers.

Many highly productive aquifers do not have the classical sheet or tabular shape commonly associated with sedimentary deposits. One of the more common of these aquifer shapes is the elongate or "shoestring" aquifer, which typically consists of stream deposits. These aquifers can appear as isolated pods or lenses; as long, sinuous, and sometimes braided ribbons; as complex dendroids; or as highly variable belts (Pettyjohn, 1975). This type of deposit is generally very difficult to trace in the subsurface and can exert a major impact on plume migration.

Even more complex than shoestring deposits are aquifers having major fracture sets or solution cavities. In these aquifers, groundwater flowpaths are usually impossible to identify completely and are generally neither continuous nor uni-directional. Thus, leachate movement patterns can be highly variable and often unpredictable.

FIGURE 2-5.
"CHARACTERISTIC" PLUME OF MISCIBLE CONTAMINANTS IN A HOMOGENEOUS AQUIFER



Aquifer geometry, particularly aquifer thickness, also affects plume migration rates. In general, as groundwater velocity increases, plume migration rates also increase. Thinner aquifers having the same hydraulic gradient and discharge rate as thicker aquifers have faster flow rates because the same volume of water is forced to move through a smaller volume of aquifer. Because of this, aquifer size is not a reliable measure of plume migration rates. In situations where an aquifer decreases in thickness or "pinches out" in the direction of flow, flow rates can be expected to increase, increasing the migration rate of plumes. The opposite effect can occur in an aquifer which increases in thickness downflow.

2.1.2.2 Aquifer Hydraulic Conductivity

Most aquifers do not exhibit constant hydraulic conductivities throughout their depth, width, and length. In fact, hydraulic conductivities can vary by orders of magnitude from area to area and commonly diminish with depth into the aquifer. Changes in hydraulic conductivity may be gradual, or abrupt as in the case of multilayered aquifers. One example of the effect of nonuniform hydraulic conductivity on plume movement patterns is shown in Figure 2-6. In this example, contaminants have moved farthest from the source area in a deeper, gravel layer having a larger hydraulic conductivity than in the overlying strata. This is not an uncommon occurrence, and highlights the importance of understanding site geology prior to designing and implementing plume sampling programs.

Geologic strata which have been uplifted and tilted can alter or reverse the direction of plume movement by channelling fluid flow along zones of secondary porosity (i.e., bedding planes, fractures, faults, or solution cavities). Possible effects of various types of hydraluic conductivities caused by secondary porosity are illustrated in Figure 2-7.

Faults are discontinuities in geologic materials that exhibit vertical or lateral displacement or both of one side of the break with respect to the other. Faults can be highly permeable or nearly impervious. If the fault has

FIGURE 2-6.
EFFECTS OF VARIABLE HYDRAULIC CONDUCTIVITY ON LEACHATE PLUME MOVEMENT IN UNCONSOLIDATED MATERIAL

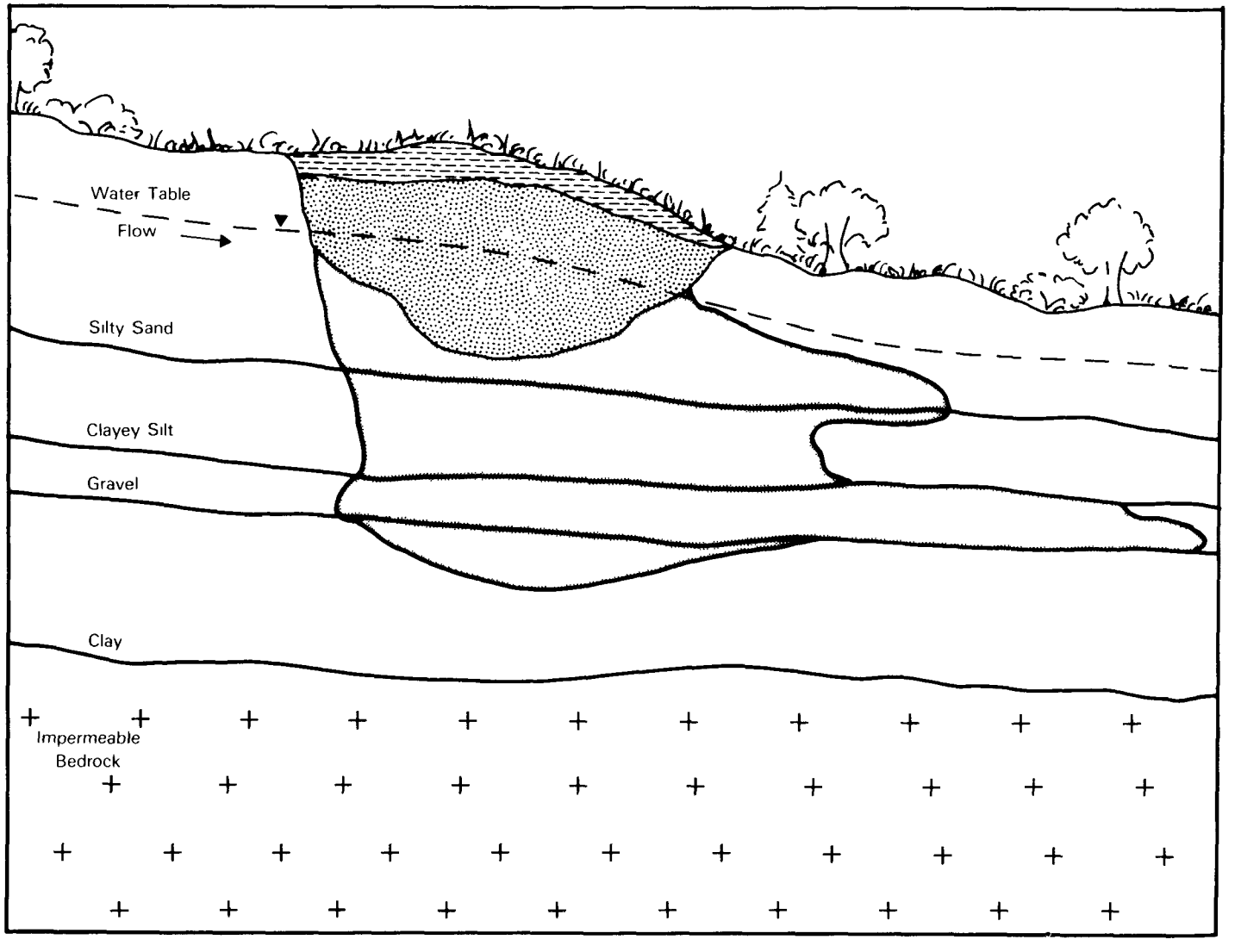
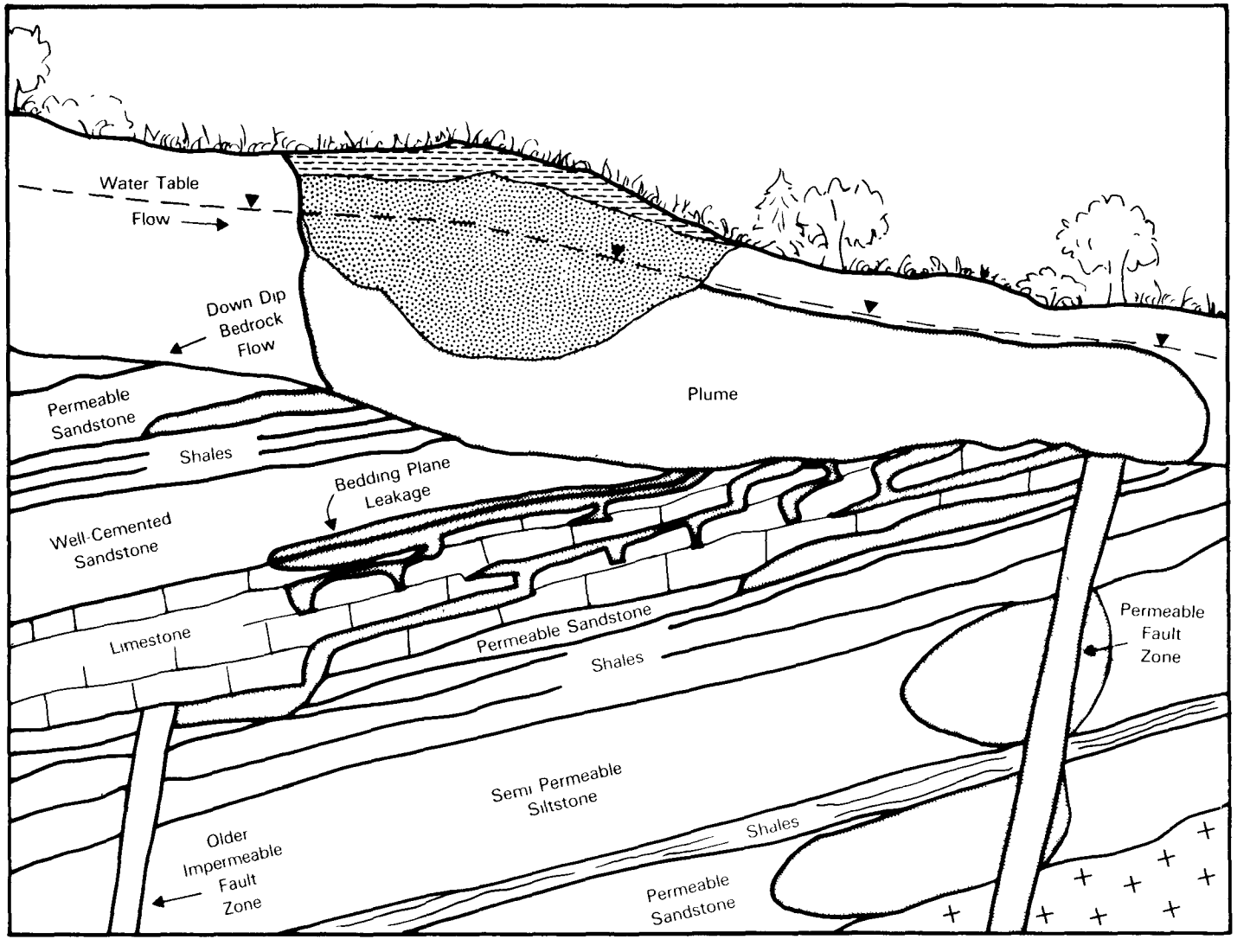


FIGURE 2-7.
EFFECTS OF SECONDARY POROSITY ON LEACHATE PLUME MOVEMENT



a low hydraulic conductivity, the fault can be a subsurface barrier to groundwater flow and deflect or redirect the regional flow pattern. If the fault has a higher hydraulic conductivity than surrounding strata, the fault can channel groundwater either upward or laterally at a higher rate than regional flow. Highly permeable faults can permit very rapid groundwater flow which promotes extensive plume migration or expansion within the groundwater system.

Some geologic units have been subjected to various types of stresses that cause fracturing of the rock. Other units, particular carbonates, can contain large, interconnected solution cavities. If a sufficient number of fractures are present, even relatively non-porous rocks can function as aquifers. Fractures and solution cavities can have pronounced effects on leachate plume flow patterns and rates, as illustrated in Figure 2-7. When these features are present within an aquifer, solution movement is controlled by the width, length, frequency, interconnectedness, angle, and surface area of the fractures or cavities. If the fractured rock dips at an angle opposite to the direction of groundwater flow in the uppermost aquifer, the direction of plume flow can be altered in the lower units. Fractures or solution cavities can also channel contaminants from an upper aquifer to a lower one.

Fractures, faults and bedding planes may also act as barriers to flow. For example, in channels that are "sealed" because of secondary deposition of clays and silicates, groundwater flow rates can be minimal. These zones can impede the spread of contaminants significantly.

Flow rates in open fractures or solution cavities can be many times higher than flow rates in non-fractured media. In unfractured water table aquifers, flow rates generally range between 3 and 330 feet/year. In unfractured confined aquifers, groundwater generally flows much more slowly, between 0.03 and 33 feet/year (Jackson, 1980). Measured rates of plume migration in unfractured deposits can range from less than 6 feet/year to more than 4410 feet/year in a very permeable sand and gravel aquifer (Apgar and Langmuir, 1977; Walker, 1973). In contrast, groundwater tracing studies conducted in solution cavities through a limestone formation in Jamaica found groundwater movement through large cavities to be occurring at rates ranging

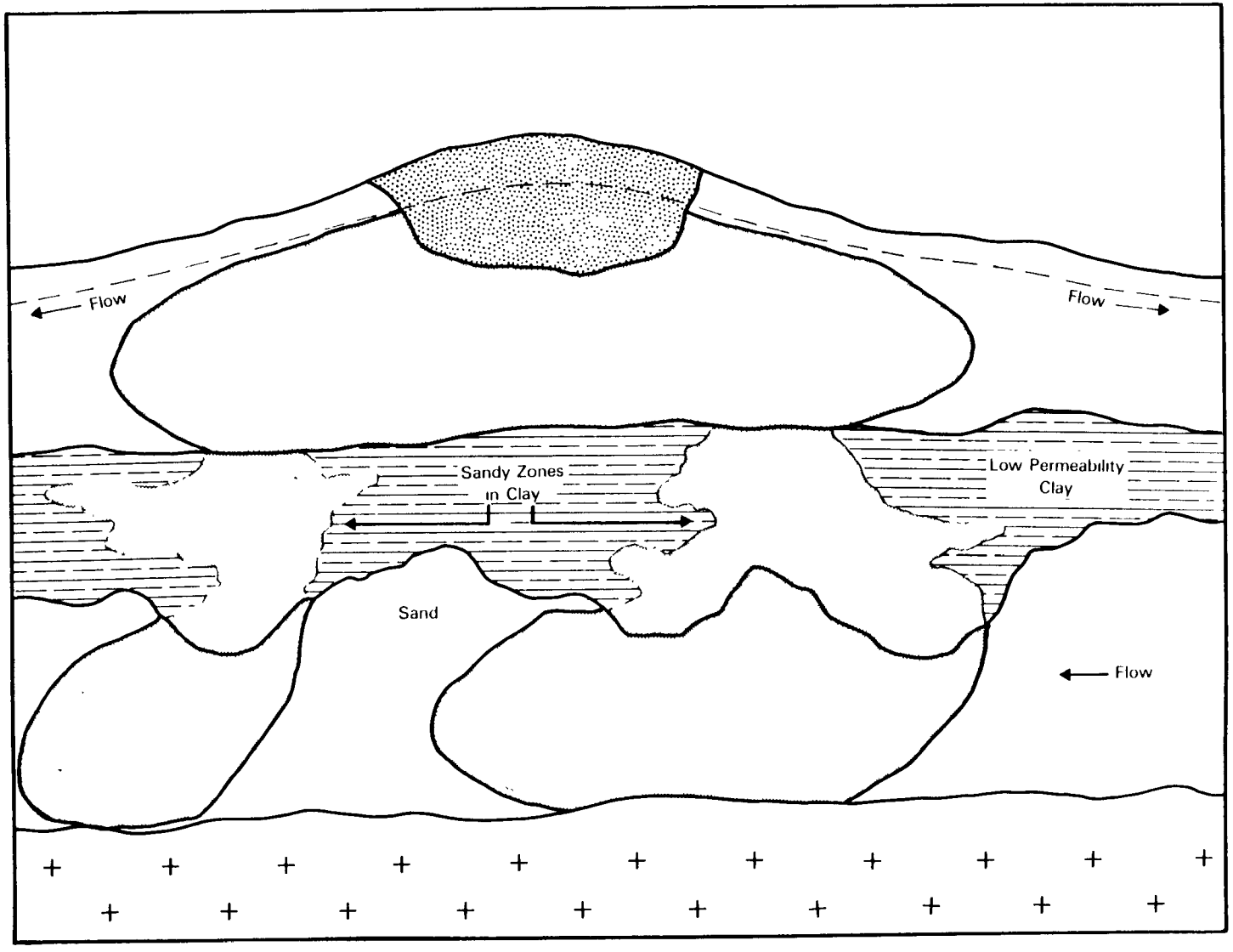
from 18,350 to 1,468,000 feet/year (Wedderburn, 1977). From this data, plume migration rates can be highly variable and certain hydrogeologic conditions at the site can permit extremely rapid flow rates.

The migration of contaminants in fractured rock can be modified by adsorption of contaminants onto materials lining fracture surfaces and by diffusion into the rock matrix. These processes are most influential when flow velocities are relatively slow and the surface area in contact with the contaminants is large. Other factors affecting attenuation include the amount of dispersed clay (for metals) and the amount of organic carbon (for organic chemicals). Matrix diffusion exerts a greater influence within porous rocks than relatively non-porous rocks. Aquifers having a high concentration of adsorptive substances coating the fracture faces are more likely to modify the characteristics of solutions flowing through the fractures than are aquifers composed primarily of relatively non-reactive substances. Types of minerals that can function as adsorptive substances include various oxides, chlorite, illite, other clay minerals, and chalk, as well as carbonates (Jackson, 1980).

The preceeding discussion focuses on faults and fractures as conduits for leachate flow to lower permeable units. Leakage to lower aquifers is not always so rapid or dramatic. Some units classified as aquitards actually contain hairline fractures, or relatively permeable areas which can allow contaminants to migrate into lower water bearing zones. Although flow rates may be extremely slow, a substantial amount of groundwater and leachate may migrate between aquifers when large areas of an aquitard leak.

Figure 2-8 illustrates one example of how contaminants can penetrate a leaky aquitard. In this example, contamination in the lower aquifer would probably not be expected because of the presence of the clay zone. However, a locally high proportion of sand or hairline fractures in the clay could permit downward leakage. Once within the lower aquifer system, contaminants could flow in entirely different patterns than expected within the upper aquifer. This figure also illustrates the differences that can exist between groundwater flow patterns within separate aquifer systems at a given locality.

FIGURE 2-8.
EFFECTS OF GROUNDWATER FLOW DIRECTION AND GEOLOGIC HETEROGENEITIES ON LEACHATE PLUME MOVEMENT



Leakage through an aquiclude does not necessarily have to be downward. If the hydraulic pressure (i.e., head) in a semiconfined aquifer were higher than the head in an overlying unconfined aquifer, groundwater would flow upward as shown in Figure 2-9. This groundwater influx could cause greater lateral spreading of the plume, faster groundwater and plume flow, and accelerated plume dispersion and dilution.

In practice, the presence of leaky aquitards tends to complicate dramatically the prediction of leachate plume migration rates, extent, directions and flow patterns.

2.1.2.3 Site Geology

Site geology can often have a significant effect on plume migration. At some sites the pattern of plume migration in the overburden (i.e., unconsolidated materials overlying bedrock) may differ from the pattern in the rock under the site. An example of this phenomena is shown in Figure 2-10. In this example, water in the overburden aquifer flows through the wastes forming leachate which is discharged at a spring or seepage area. The leachate then flows over land and contaminates a nearby lake. Some of the contaminants in the lake then enter the permeable sandstone unit underlying the lake, thus further spreading the zone of pollution. In addition to the leachate that seeps out of the hillside, a larger volume of leachate migrates downward and spreads through the permeable sandstone unit beneath the waste site. Thus, because of the geology at this site, three contaminated aquifers exist--the overburden, and two separate sandstone units. Figure 2-11 is a map view of this same example. Situations such as this would be difficult to interpret correctly unless the effects of site geology on plume migration patterns were taken into account.

This discussion of the general effects of subsurface geology and hydrology illustrate some of the types of plume migration patterns that can occur. These factors and their effects highlight the need for site-specific

FIGURE 2-9.
EFFECT OF A HIGH-HEAD, SEMI-CONFINED UNIT ON LEACHATE PLUME MOVEMENT

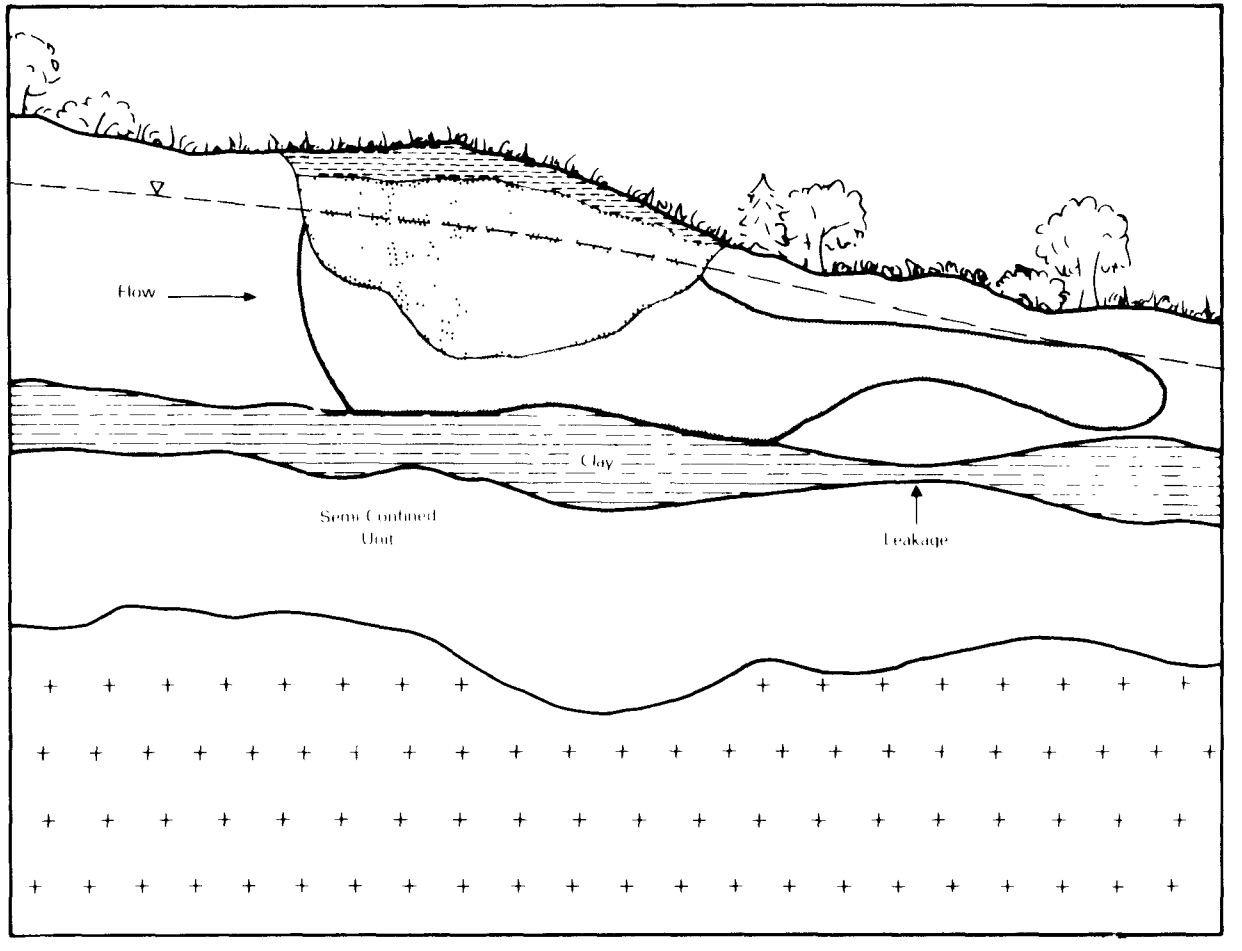


FIGURE 2-10.
EXAMPLE OF THE EFFECTS OF SITE GEOLOGY ON LEACHATE PLUME MOVEMENT
(CROSS-SECTIONAL VIEW)

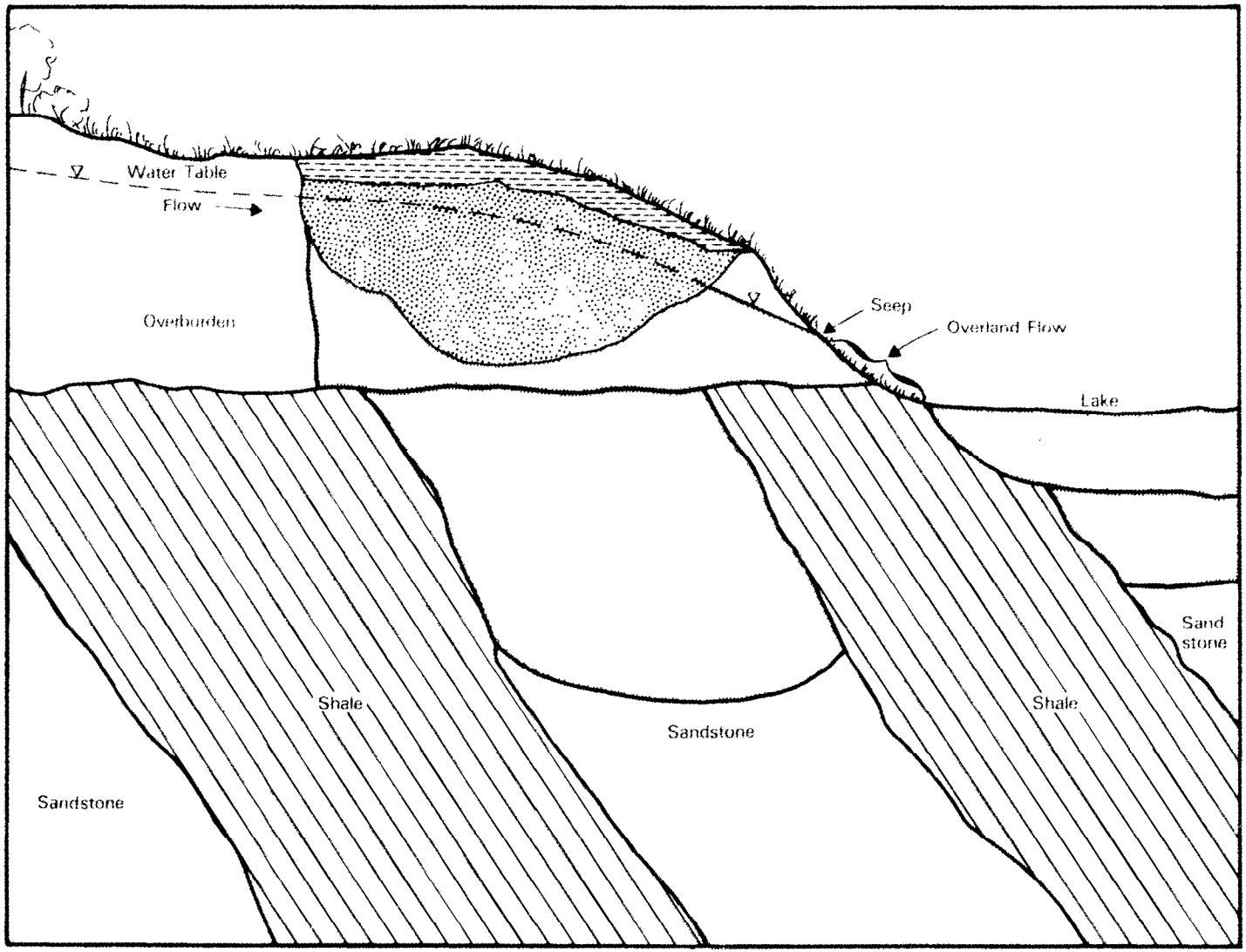
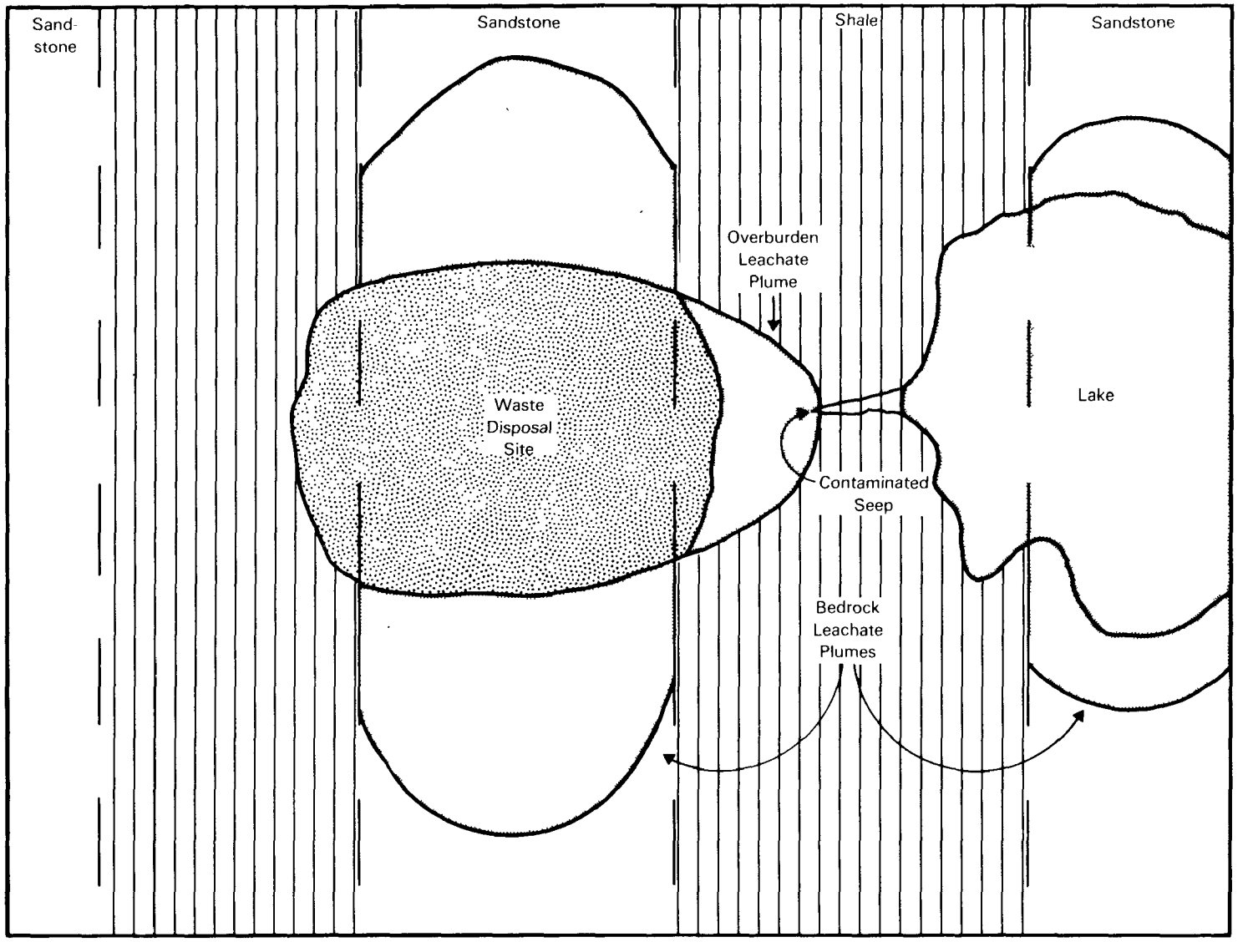


FIGURE 2-11.
EXAMPLE OF THE EFFECTS OF SITE GEOLOGY ON LEACHATE PLUME MOVEMENT
(MAP VIEW)



hydrogeologic information when attempting to predict plume movement. In addition to subsurface conditions, recharge, discharge, and other surface conditions can have a great influence on plume movement. These effects are described in the following section.

2.1.2.4 Site Hydrology

The hydrology of unconfined aquifers is in many cases controlled by surface topography. As was described in the section on basic hydrologic concepts, the water table can be a subdued expression of land surface elevations, or topography (assuming the conditions previously noted in Section 2.1.1). This generalization is true where the primary form of aquifer recharge (i.e., the addition of water to an aquifer) is by precipitation infiltration. The water table will, however, deviate from topographic patterns at hydrologic boundaries represented by discharge or recharge areas.

The location of a waste site relative to the recharge area of the nearest aquifer can affect leachate flow. Some waste sites are highly permeable and allow a considerable amount of recharge (e.g., infiltration of precipitation) to occur. This recharge can increase the volume of leachate produced, causing groundwater mounding under the waste site (Freeze and Cherry, 1979). Leachate entering groundwater in these areas tends to flow radially outward before gradually conforming to regional groundwater flow (Figure 2-8). In areas where the aquifer's vertical gradient is upward (i.e., discharge zones), opposing the downward migration of percolating leachate, mounding will occur more rapidly. In recharge zones the natural vertical gradient of the aquifer is downward, causing extensive travel of contaminants into the aquifer, but minimal mounding.

Floodwaters can also recharge groundwater. The rise in water tables as a result of flooding can cause groundwater to contact waste not normally saturated, increasing the rate of leachate generation. Also, rivers at flood stage can cause redirection of groundwater contaminant plumes because of the temporary creation of hydraulic divides by riverbank storage effects.

A third effect of recharge area location on leachate plume migration patterns is illustrated in Figure 2-12. Sites near recharge areas are likely to produce plumes that flow for long distances before reaching a natural discharge point. Conversely, plumes from sites located near discharge areas pollute a smaller volume of the aquifer before contaminants are dispersed into surface waters (Fenn, et al., 1980; Lindorff and Cartwright, 1977). At some sites, only a portion of the leachate plume enters the surface water at discharge areas, as was illustrated in Figures 2-10 and 2-11.

Recharge and discharge do not typically occur at constant rates over time. As a result, the potentiometric surfaces of aquifers can and do change, sometimes significantly. This is especially true of unconfined surface aquifers which are subject to seasonal changes in precipitation, stream flow, evaporation and transpiration, snow melt, and other conditions. These changes can occasionally be of such a magnitude that water table gradients (and hence, groundwater and pollutant flow directions) are reversed. Thus, these cyclic hydrologic changes are an important consideration in an aquifer system when assessing leachate migration patterns (refer to Pettyjohn, 1982, for more details).

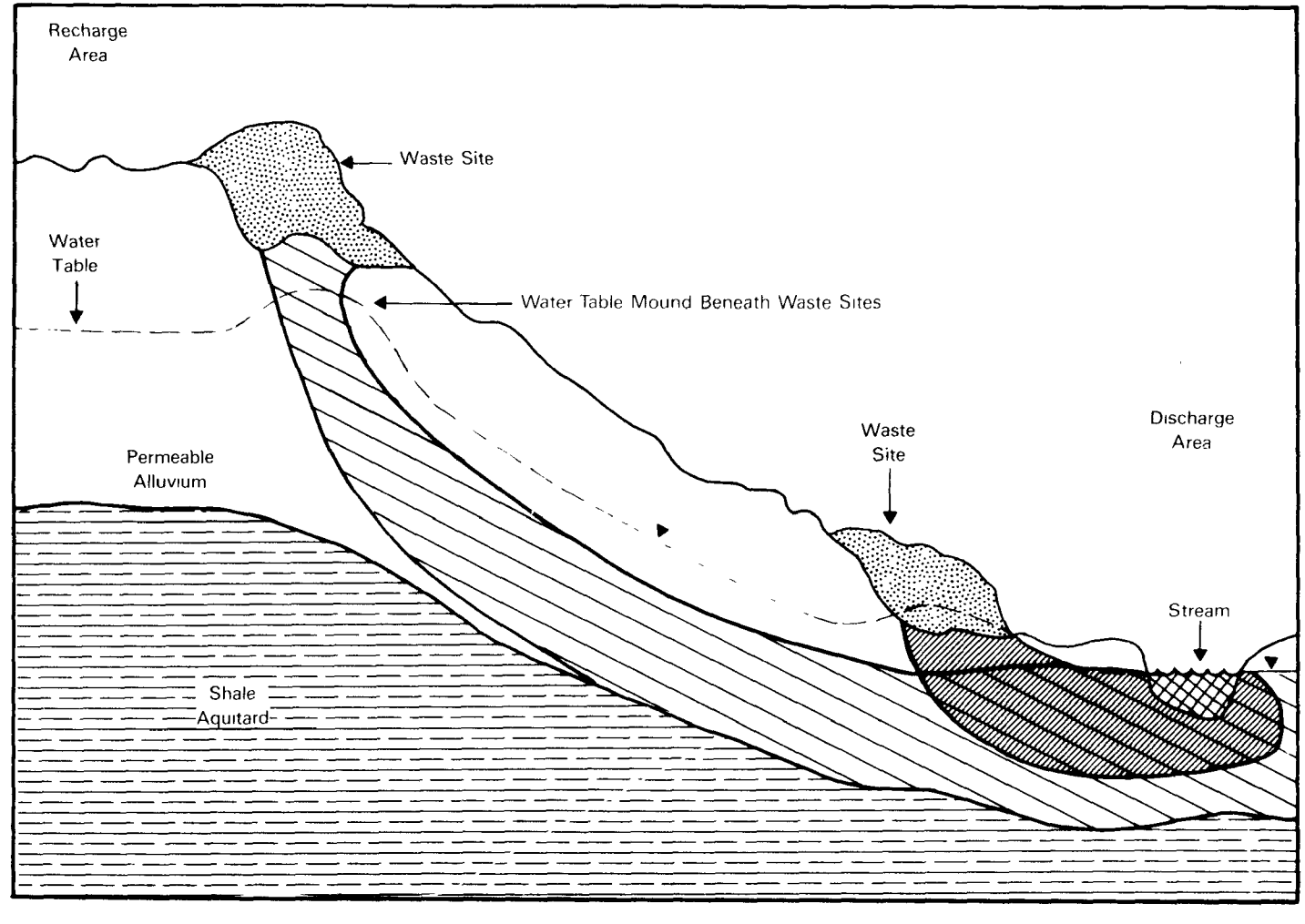
The migration of leachate plumes is controlled by a multitude of naturally occurring situations and processes. The location of recharge and discharge areas, for example, can have a large impact on plume dynamics. Plume migration can also be either slowed or promoted by various human activities. These human activities are discussed in the next section.

2.1.3 Effects of Human Activities

A number of human activities can modify the flow patterns of both groundwater and leachate plumes. The types of effects that human activities can cause include:

- Increasing or lowering the water table
- Modifying water table gradients
- Creating or obliterating recharge and discharge areas

FIGURE 2-12.
THE EFFECT OF WASTE SITE LOCATION RELATIVE TO LOCATION OF RECHARGE AND DISCHARGE AREAS ON PLUME MIGRATION



- Diverting groundwater or plume flow
- Reducing or increasing leachate production
- Containing or removing the plume from the aquifer.

The ways in which human activities accomplish these effects include:

- The use of extraction wells which can cause localized lowering of the water table, creating cones of depression surrounding the wells.
- The use of injection wells which can cause groundwater mounding, increased water table elevations, and radial flow away from the wells.
- The use of construction dewatering techniques which can redirect groundwater movement near the construction site by the installation of subsurface low permeability barriers, the construction of drainage ditches, or the use of extensive pumping from extraction wells surrounding the site.
- The presence of surface impoundments, such as lakes and lagoons, which can cause groundwater recharge.
- The presence of irrigation or drainage ditches which can cause either groundwater recharge or discharge.
- The presence of numerous other subsurface man-made features including mines, tunnels, foundations, and sewer lines.
- The use of various remedial action measures to contain or control leachate plumes.

Planned human interventions in the groundwater regime of a waste disposal site (i.e., remedial actions) can cause a significant reduction of hazards to public health and the environment. The effects of several types of remedial actions for leachate plume control are summarized in Table 2-3.

Groundwater flow patterns and leachate plume migration rates can also be inadvertently altered by human activity. Figures 2-13 and 2-14 illustrate a hypothetical example of several types of human activities and their unexpected effects on plume migration.

TABLE 2-3

EFFECTS OF SEVERAL REMEDIAL ACTION MEASURES ON LEACHATE PLUME MIGRATION (After EPA, 1982a)

Remedial Action Measure	General Effects	Effects on Leachate Plume
Sealing of site surfaces, grading, revegetating and diverting surface water from site	1. Reduces the amount of water (from rain, snow or surface run-on) that infiltrates into the site and flows through the wastes. 2. Reduces groundwater recharge and mounding beneath the site.	1. Minimizes the volume of leachate generated. 2. May result in a short-term increase in the concentration of contaminants in the plume because less dilution will occur. 3. Concentrations of water soluble constituents from above the water table should be reduced in the leachate because solubilization is less likely. 4. Radial spread of the plume beneath the landfill will be reduced.
Installing subsurface low permeability barriers upgradient from site	1. Initially diverts uncontaminated groundwater away from the site and the plume. 2. Initially reduces the water table evaluation on the down-gradient (waste site) side.	1. Initially reduces the likehood of horizontal groundwater flow through the waste site bottom. 2. Initially reduces the volume of groundwater exposed to the leachate.

TABLE 2-3 (continued)

Remedial Action Measure	General Effects	Effects on Leachate Plume
Installing of subsurface low permeability barriers upgradient from site (cont'd)	3. Initially reduces the flow rate on the downgradient side because the hydraulic gradient is reduced.	3. Temporarily lessens the potential for leachate dilution 4. May temporarily reduce the severity of conditions within the lower parts of the waste site. 5. Should temporarily reduce amount of dissolution occurring (if waste was previously in contact with groundwater). 6. Reduces the flow rate of the plume. 7. Slightly reduces the degree of plume extent. 8. May result in a short-term increase in the concentration of contaminants in the plume because less dilution will occur. 9. Slower flow rates can allow additional attention and degradation to occur.

TABLE 2-3 (continued)

Remedial Action Measure	General Effects	Effects on Leachate Plume
Installing subsurface, low permeability barriers downgradient from the site	1. Minimizes groundwater movement downgradient from barrier. 2. Causes rise in water table elevation upgradient from wall unless groundwater pumping or subsurface drains are used.	1. Prevents longitudinal migration of plume. 2. Causes lateral migration of the plume along the subsurface barrier unless groundwater pumping subsurface drains are used.
Installing subsurface, low permeability barriers completely surrounding the site	1. Isolates groundwater in the site area from regional groundwater. 2. Reduces groundwater velocity.	1. Prevents longitudinal migration of plume. 2. Minimizes dilution of leachate and leachate volume. 3. Reduces cost of treating contaminated groundwater by minimizing the amount of groundwater affected. 4. Allows dewatering of the site via groundwater pumping or subsurface drains.

TABLE 2-3 (continued)

Remedial Action Measure	General Effects	Effects on Leachate Plume
Permeable Subsurface Treatment Beds	1. Modify groundwater chemistry by introducing sorptive or chemically active substances such as activated carbon or crushed limestone.	1. May reduce concentration of certain contaminants via sorption, precipitation or neutralization.
	2. May cause local increases in flow velocity through the beds.	2. May change pH and solids content of leachate.
Groundwater Pumping	1. Upgradient pumping wells reduce the amount of groundwater flowing through the site and lower the water table elevations locally	1. Upgradient wells can help stabilize the plume if operated carefully
	2. Downgradient pumping wells reduce the amount of groundwater flowing off the site and lower the water table in localized areas downgradient of the site.	2. Downgradient wells can be used to extract the plume if pumped or divert the plume if used for injection
		3. Wells may not affect all of plume if not properly sited.
Installing Subsurface Drainage System	1. Same as for groundwater pumping.	1. Same as for groundwater pumping but with less of a change of allowing enhancements to escape from the system if properly designed.

TABLE 2-3 (continued)

Remedial Action Measure	General Effects	Effects on Leachate Plume
Bioreclamation	1. Allows microbial degradation within contaminated aquifers.	1. Promotes microbial degradation of plume constituents 2. Can alter physical and chemical properties of the leachate.

FIGURE 2-13.
EFFECTS OF HUMAN ACTIVITIES ON LEACHATE PLUME MOVEMENT
(MAP VIEW)

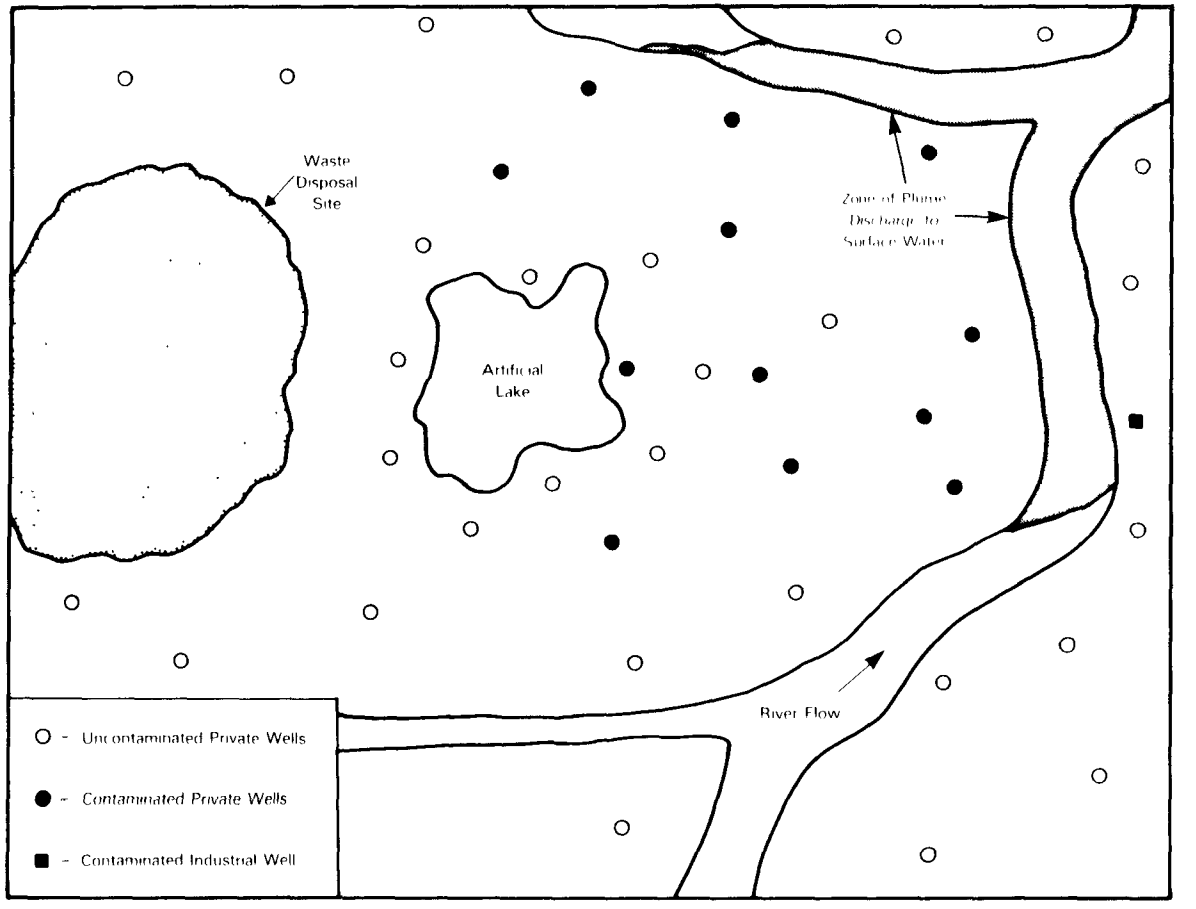
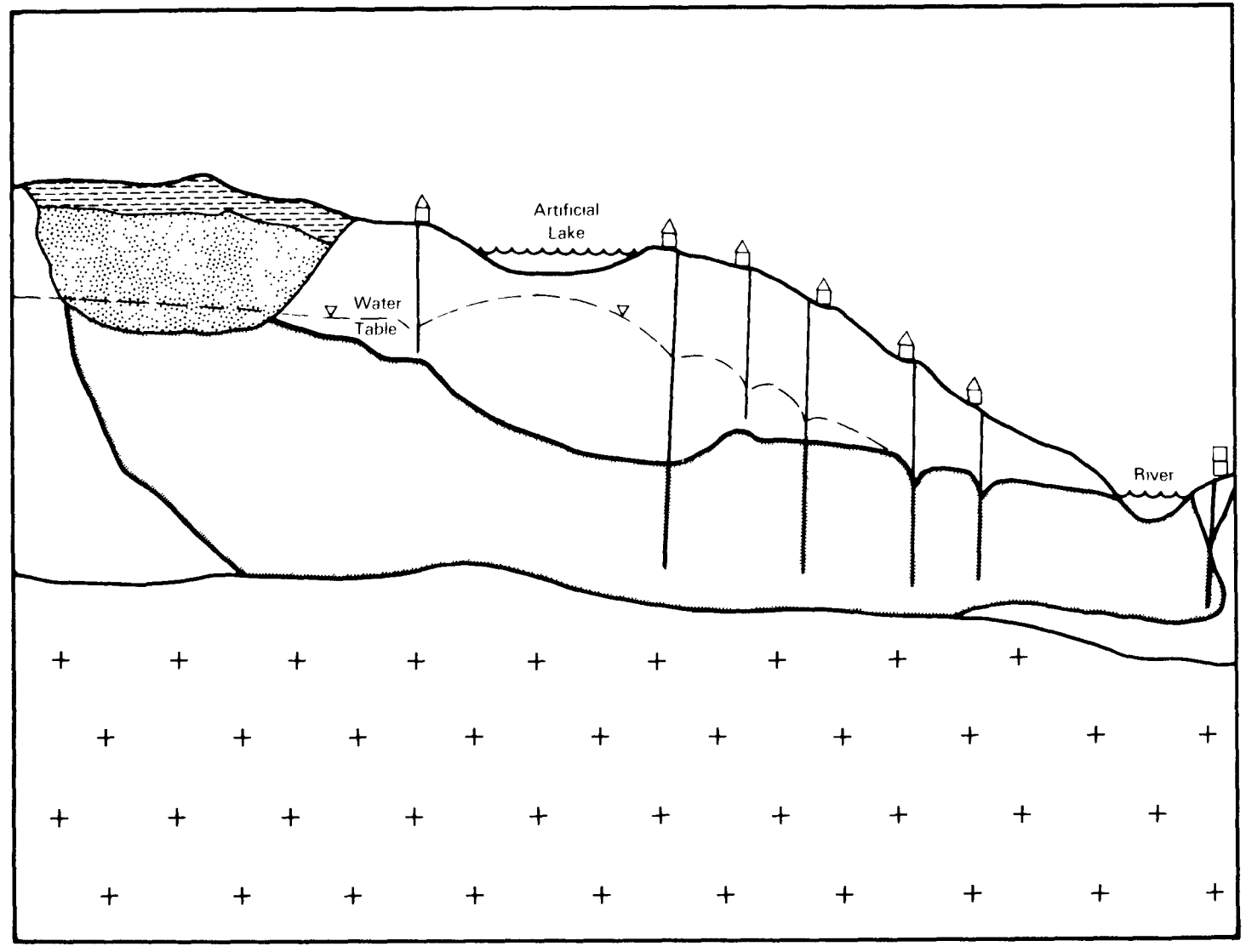


FIGURE 2-14.
EFFECTS OF HUMAN ACTIVITIES ON LEACHATE PLUME MOVEMENT
(CROSS SECTIONAL VIEW)



The waste disposal site shown in map view in Figure 2-13 is apparently releasing a leachate plume which has contaminated downgradient private wells in a somewhat anomalous pattern. A nearby lake is inexplicably free of contaminants as are wells adjacent to the lake and between the lake and the site. A river system downgradient of the lake is slightly contaminated. Some wells between the lake and the river are contaminated and some are not. Wells across the river from the disposal site are uncontaminated with one exception, a high-production industrial well. The geology of the area is fairly uniform and does not appear to be the cause of the contamination pattern.

This pattern of surface and groundwater contamination might be confusing unless changes in site hydrology from human activities are understood. To assist in understanding this situation, Figure 2-14 illustrates the site in cross section. In this figure, the lake is recharging groundwater and causing the plume to deflect downward as it flows beneath the lake. This causes shallow wells nearest the lake to remain unaffected by the site and wells extending to greater depths within the aquifer to be contaminated. Contaminated groundwater flowing under the lake discharges into the river without affecting most of the wells located on the opposite bank. Because a large volume of water is extracted from the deep industrial well a steep gradient toward this well is formed. This causes groundwater flow toward the well and pulls the leachate plume beneath the river. Private wells nearby are not contaminated because smaller volumes of water are withdrawn from these wells.

In addition to the effects of human activities illustrated in the hypothetical example given above, many other activities can channel plumes in seemingly anomalous patterns. These include constructing high permeability zones, such as tile drainage systems on low-lying farmland, domestic septic tank drainfield lines, and underground mine workings; and low permeability areas, such as foundations, retaining walls, tunnels, and areas where compacted fill materials have been placed. The effects of these features must be kept in mind when attempting to delineate the areal extent, and estimate flow rates, of leachate plumes.

2.2 Effects of Leachate Characteristics

The types and concentrations of contaminants strongly influence the physical and chemical properties of leachate. In turn, the flow patterns and chemical interactions of leachate are determined to a great extent by the distinctive set of chemical and physical properties of the constituents. Properties that influence leachate migration patterns are described below. These properties affect the types of interactions that can occur between leachate, aquifer, and groundwater constituent. Knowledge of these interactions assists in understanding the data obtained from site investigations and from studies of leachate behavior within aquifers.

The characteristics of a plume emanating from a waste site are strongly affected by the wastes present in the site. However, the waste and the leachate it produces are rarely similar in their chemical and physical properties because of variable contaminant release rates and the effects of dilution. Furthermore, the extent of mixing of different types and quantities of wastes in a disposal site will also influence leachate characteristics. For this reason, generalizations about the types and concentrations of specific contaminants found in leachates are difficult. Nevertheless, leachates from waste organic chemicals probably pose the greatest environmental concern. The approximate percentages of general contaminant types found at Superfund sites (Table 2-4) indicate that over three-quarters of the waste constituents in these sites are organics or organics mixed with metals. The remainder consist of metals and radioactive materials. A survey of leachate from solid waste disposal facilities, on the other hand, would result in much different findings. Thus, great variability in plume dynamics can be expected within any class of leachate-producing facility because of varying physical and chemical characteristics of the leachates produced.

TABLE 2-4

DISTRIBUTION OF CONTAMINANT TYPES FOR SUPERFUND SITES
(Based on EPA, 1982b)

Primary Type of Contaminant at Site	Percentage of Sites*	
	358 of 418 sites on expanded priority listing	86 of 114 sites on initial priority listing
Organics	54%	44%
Solvents, oils, fuels	(42%)	(32%)
PCBs	(7%)	(6%)
Pesticides	(5%)	(6%)
Organics and Metals, mixed	26%	36%
Metals, acids and bases, and radioactives	20%	20%

*Percentages are based on varying levels of data completeness

2.2.1 Effects of Physical Characteristics

The physical characteristics of leachate that can affect plume migration include:

- o Volume
- o Viscosity
- o Density
- o Solubility
- o Degree of dispersion
- o Temperature.

These factors, which have varying degrees of influence on migration patterns and rates, are described below. Other factors which can affect plume migration, but not discussed herein, include chemical stability, reactivity, and fluid phases present in the leachate.

2.2.1.1 Leachate Volume

The amount of leachate produced at a site affects both the volume and flow rate of the plume. Larger volumes of leachate can produce larger or more concentrated plumes or both.

The factors that affect the amount of leachate produced include:

- Duration, frequency, type, and amount of precipitation
- Mean annual temperature, temperature fluctuation, and degree of insolation
- Surface characteristics of the site including the steepness of the slopes, type and permeability of the cover material used, presence and types of vegetation present, length of the slopes, and other features which impact percolation rates
- Site subsurface characteristics including the presence of water within the wastes, volume of flow through the wastes, and rate of flow through the waste site.

Table 2-5 indicates the effects of each of these parameters on leachate production.

For some sites, estimations of leachate volume have been made using a water balance approach, whereby the interactive effects of climatic conditions such as precipitation, temperature, insolation, and evapotranspiration are calculated to produce an estimate of the yearly volume of water available for infiltration. From this value, the amount of runoff is subtracted and the remaining water is assumed to seep through the waste site to generate leachate. One of the disadvantages of the water balance method is the assumption that the waste site is not in direct contact with groundwater. Thus, the method is not directly applicable to sites where waste material has been disposed of below the level of the seasonal high water table. Additional information on using the water balance approach to predict leachate generation is provided in Fenn, et al. (1975) and Perrier and Gibson (1980).

TABLE 2-5

FACTORS AFFECTING LEACHATE VOLUME GENERATION

Factor	Effect
Precipitation	
Amount Per Year	o Greater the amount of precipation, the greater volume of leachate generated.
Highest 24-hour Rainfall	o No real effect, but the greater the number of short duration, high volume rainstorms, the less likely that precipitation will infiltrate to waste, and the more likely that most will run off as surface flow.
Mean Annual Temperature	o Higher the mean annual temperature, the greater the amount of precipitation that will evaporate and not infiltrate into waste, reducing the volume of leachate generation.
Degree of Insolation	o Increasing insolation will increase evaporation, reducing the quantity of leachate produced.
Surface Slope	o Increasing slope at a disposal site will promote surface runoff, reducing the volume of precipitation infiltrating into the waste and thus reduce the volume of leachate generated.
Cover Permeability	o Low permeability cover materials promote runoff, reducing infiltration and the volume of leachate generated.
Vegetation	o Heavy vegetative cover of grasses will increase transpiration. Heavy vegetative cover of grasses can also impede runoff thus increasing infiltration.
	o Heavy vegetative cover of trees, bushes and shrubs may increase infiltration by opening up channels of flow into waste because of deep root penetration.
Subsurface Characteristics	
Water Content of Waste	o An increased water content within waste will decrease time till leachate generation begins.
Volume of Water Flow Through Wastes	o Increased water flow through waste will increase the volume of leachate generated.
Rate of Flow Through Wastes	o Increased flow rates through waste will decrease time till start of leachate generation, but may decrease the concentration of contaminates present.

2.2.1.2 Leachate Viscosity

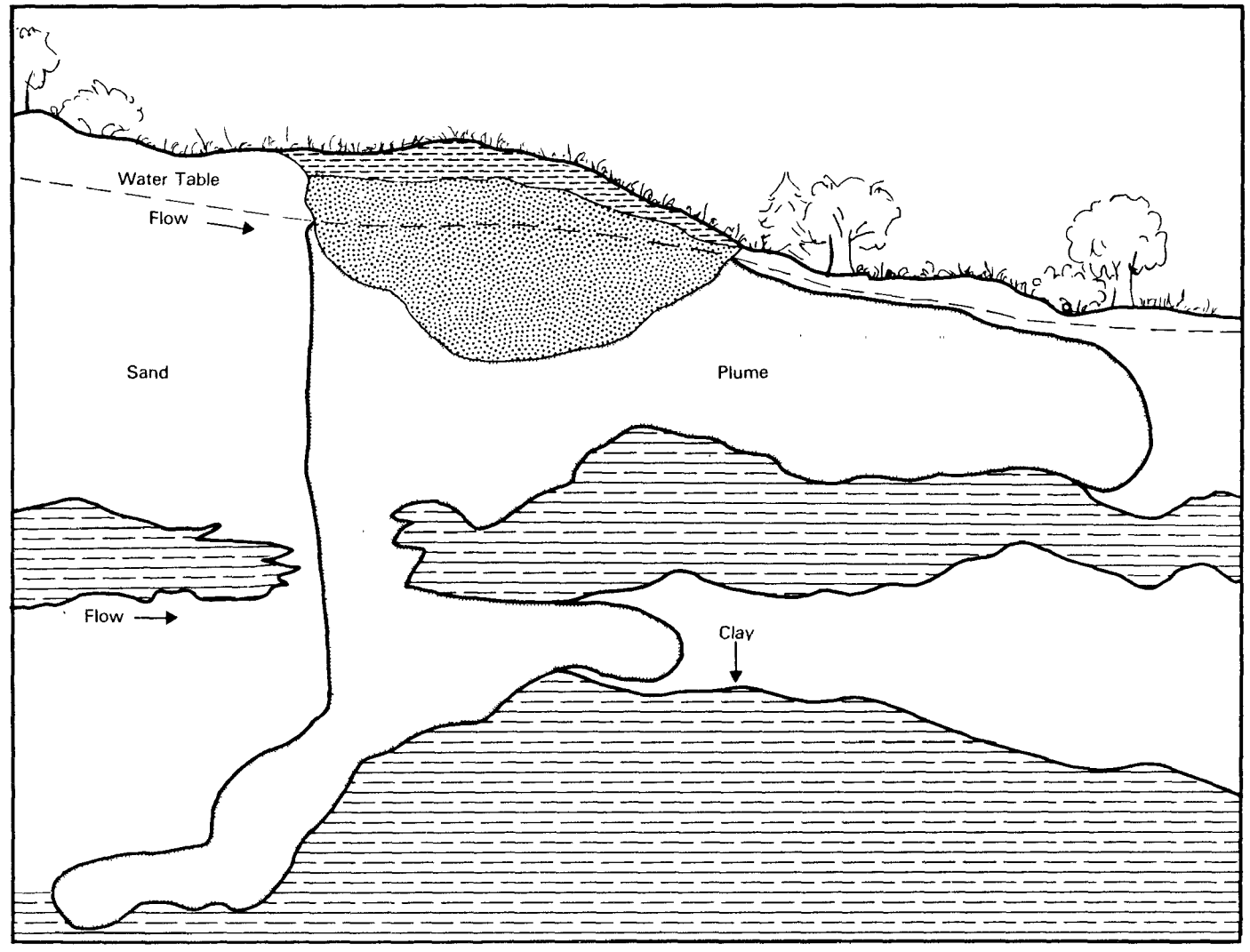
The resistance of a leachate to internal flow caused by external forces is termed its viscosity. Solutions with a higher viscosity tend to flow more slowly than less viscous ones, thus, they would be expected to disperse less, and to migrate more slowly, than solutions of lower viscosity. In addition, leachates exhibiting a viscosity substantially higher than groundwater should decrease the hydraulic conductivity of the system. Contrary to expectations based on hydraulic conductivity equations alone, field experience has indicated that the viscosity of a leachate has a minor effect on hydraulic conductivity (Jackson, 1980).

2.2.1.3 Leachate Density

If the density of a plume is significantly greater or lower than that of groundwater, plume migration patterns can be altered dramatically. Low density wastes tend to float at the top of an aquifer and spread out to form a lens of contaminants that may be several feet thick. In Mechanicsburg, Pennsylvania, for example, a gasoline spill resulted in formation of a four-foot thick gasoline layer above the water column. A fuel oil spill in Southeastern Pennsylvania produced a contaminant lens that was five feet in thickness (Lindorff and Cartwright, 1977). Low density plumes also tend to remain more concentrated than other leachates that disperse more readily in the groundwater.

High density plumes can form dense or concentrated layers near the base of aquifers. For example, chloride contaminants from oil field brines entered an aquifer in New Mexico and sank to the bottom of the aquifer because they were more dense than the native groundwater (Lindorff and Cartwright, 1977). If dense wastes contain substances that degrade or dissolve minerals, they could eventually produce a breach in an aquitard separating an upper aquifer from an underlying one. In some instances, dense plume constituents can migrate downward against the direction of regional groundwater flow. An example of this situation is shown in Figure 2-15. In this figure, the upper clay layer splits the plume into two sections as it flows. A small portion of

FIGURE 2-15.
EFFECTS OF A PERCHED ZONE ON THE FLOW OF A HIGH DENSITY PLUME



the dense plume sinks to the base of the lower clay layer and moves against the direction of groundwater flow.

Density is also important because of the effects density differences, and subsequent pressure differences, can have on plume migration. Flow equations should be formulated in terms of pressure when variable density conditions (e.g., oilfield-related brine plumes) are apparent.

Some leachates contain both low and high density constituents. This can lead to the formation of a two-plume system with widely differing chemical and physical characteristics, as illustrated in Figure 2-16. Leachates from disposal facilities accepting many waste types will commonly contain constituents with various densities. In these instances, the leachate constituents will tend to seek levels of equivalent density, and thus become spread throughout the water column with light components at the top, and heavy components lower in the water column.

2.2.1.4 Leachate Solubility in Groundwater

The ability of leachate constituents to dissolve in groundwater has distinctive effects on plume migration patterns. Water soluble (miscible) leachates tend to form larger, more dispersed plumes, whereas water insoluble (immiscible) materials tend to create small and highly concentrated plumes. In addition, less water soluble constituents tend to migrate more slowly than more soluble ones (Wilson, et al., 1980).

Solubility characteristics interact with leachate densities to produce modifications in the classical plume migration profile. Examples of plume migration patterns for various density and solubility combinations are shown in Figure 2-17. Typical waste constituents exhibiting the solubility and density combinations that are shown in Figure 2-17 are listed in Table 2-6. The characteristics of leachates containing any of these constituents will

FIGURE 2-16.
EFFECT OF DENSITY ON LEACHATE PLUME MOVEMENT

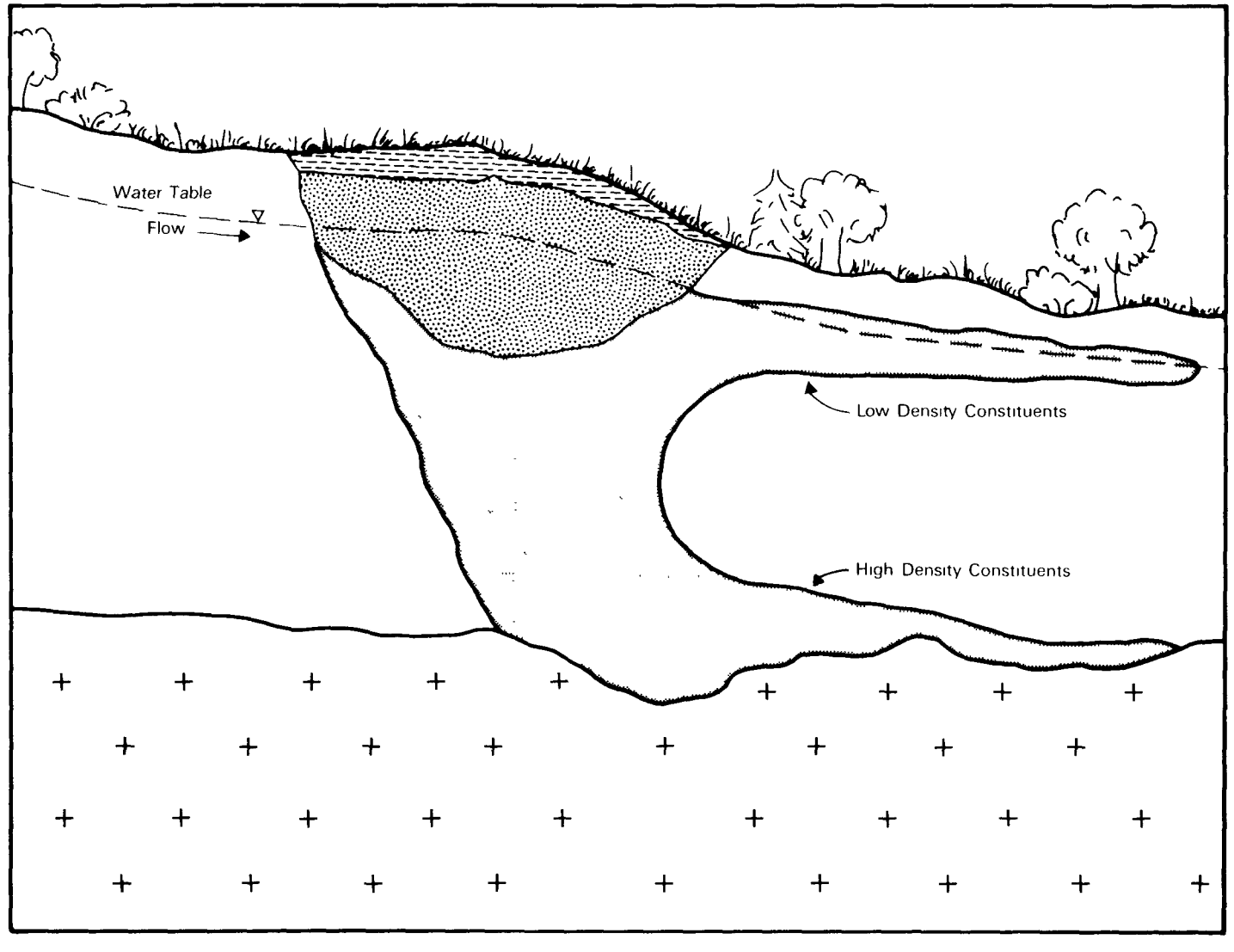


FIGURE 2-17.
CONFIGURATIONS BASED ON SOLUBILITY AND DENSITY

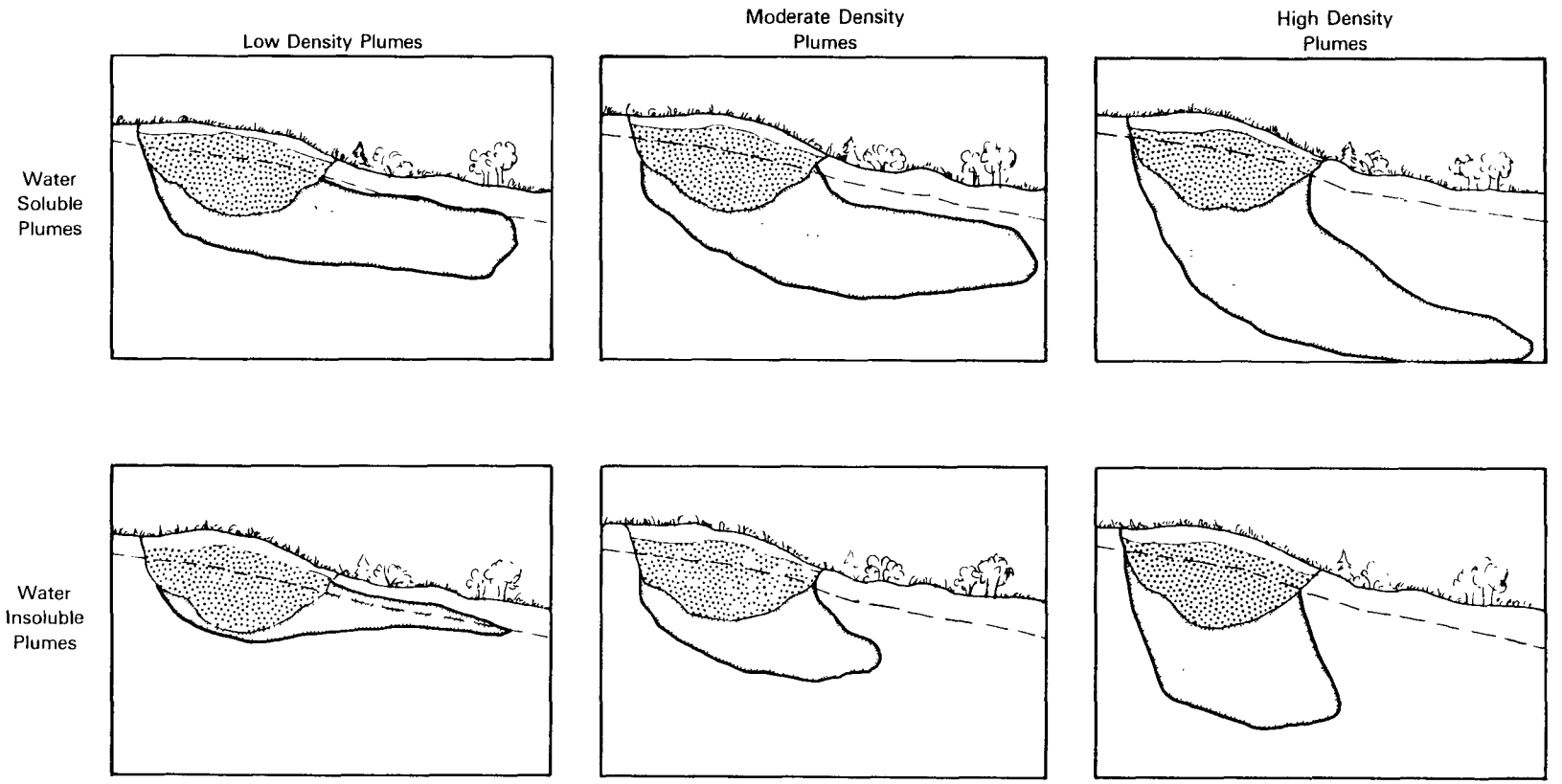


TABLE 2-6

EXAMPLES OF PLUME CONSTITUENTS BASED ON SOLUBILITY AND DENSITY

Constituents	Low Density Constituents	Moderate Density Constituents	High Density Constituents
Water Soluble	Acetone Acrylonitrile Ammonia Benzene	Acetic Acid Aniline Most metal salts Phenol	Chloroform Halogenated ethanes Halogenated phenols Nitrobenzene
Water Insoluble	Cyclohexane Ethylbenzene Gasoline and some oils Toluene Vinyl chloride Xylene	T-butyl acetate Butyl cellosolve Hexanol ester-alcohol	Carbon tetrachloride Chlorobenzene PCBs Most pesticides Most polyhalogenated benzenes and phenols Most polycyclic aromatic hydrocarbons

depend on constituent concentrations and may be substantially different than anticipated from the properties of the pure chemicals.

Although a leachate may be immiscible in groundwater, it could form an emulsion if emulsifiers are present. In these situations, the leachate would most likely behave as a moderate density solution.

2.2.1.5 Degree of Dispersion

Contaminant transport can occur via three processes in groundwater; advection, diffusion and dispersion. Contaminant transport through the motion of groundwater flow is known as advection. Solutes carried by advection only move at a rate equal to the average linear velocity of groundwater (Freeze and Cherry, 1979). However, mechanical mixing and molecular diffusion can cause contaminants to spread out from the path they would follow under advection only. Diffusion is the process whereby constituents move under the influence of their kinetic activity in the direction of their concentration gradient. Diffusion is a factor in contaminant transport at very low velocities, and is generally ignored during initial studies of contaminant transport.

Dispersion caused entirely by fluid motion is known as mechanical dispersion, and is best understood in microscopic terms. Freeze and Cherry (1979) list three mechanisms involved in dispersion:

- Variations in molecular velocities caused by drag executed on fluids in close contact with particle surfaces
- Variations in pore size which affect groundwater velocities in different pore channels
- Variations in possible flow channels caused by branching and interfingering.

Dispersion causes contaminants to "spread out" while moving by advection. The amount of plume dispersion that occurs is greatest in granular or fractured aquifers where turbulent flow predominates (Pettyjohn, et al., 1982). In

general, dispersion does not reduce the concentration of hazardous plume contaminants to acceptable levels. The processes of dilution and dispersion are too slow and ineffective to be relied upon for renovation of contaminated groundwater (Lindorff and Cartwright, 1977).

2.2.1.6 Leachate Temperature

A final physical characteristic that influences plume migration patterns is temperature. High temperatures can be generated during aerobic degradation of organic materials, or during exothermic chemical reactions that occur within the waste site. High temperature leachates are less dense than the same leachate at a lower temperature, thus they tend to rise within the aquifer. They also react more rapidly with aquifer and groundwater constituents than do cooler leachates. For example, hot (175°F) wastes from a furfural plant in South Florida were injected into a slightly-saline, confined carbonate aquifer. The wastes collected near the top of the aquifer, where they rapidly dissolved portions of the upper aquitard (McKenzie, 1976).

Although a knowledge of the physical characteristics of contaminant plumes in aquifers can assist in predicting overall migration patterns, data on a plume's chemical characteristics are also necessary to understand the overall effects of the plume on an aquifer and on groundwater, and its subsequent migration.

2.2.2 Effects of Geochemical Interactions

Data on the types, concentrations, and interactions of chemical constituents within a leachate are of extreme importance in evaluating the degree of hazard from the leachate and its effects on the aquifer and the groundwater. Information on possible leachate hazards is summarized by Mackison, et al. (1978) and Sax (1980). Leachates having a low concentration of hazardous constituents generally have a less pronounced impact on aquifer geochemistry

than highly concentrated plumes. The concentration of the plume is influenced by a number of factors, including:

- Volume and rate of infiltrate percolating through the waste site
- Solubility and density of plume constituents
- Length of contact time between infiltrate and wastes
- Initial moisture content of wastes
- Age of the waste site
- Anthropogenic factors and human influence
- Rate of groundwater flow receiving percolating leachate.

As groundwater passes through an aquifer, numerous chemical reactions occur. Although groundwater and aquifer interactions are complex, the reactions that occur as a result of leachate flow through an aquifer can be many times more intricate. Rather than attempt to enumerate the vast number of reactions possible, brief descriptions of the major processes that affect migration are given.

A number of processes interact to control the mobility of various wastes, so that not all of the waste placed in a site can leach to groundwater. Some wastes can be precipitated, biochemically degraded, or temporarily sorbed. The net result of these processes is to retard the movement of some constituents through the aquifer. This effect is termed attenuation. Important factors in contaminant attenuation include:

- Form and concentration of each contaminant
- Physical and chemical properties of the leachate mixture
- Physical and chemical properties of groundwater
- Surface area and porosity of geologic materials
- Groundwater and leachate flow rates
- Types and orders of geologic materials encountered.

Not all contaminants are attenuated to the same extent under a given set of conditions. While contaminant attenuation is a highly complex and site-specific process, certain generalizations can be made, as described below.

2.2.2.1 Attenuation of Inorganic Constituents

Many factors interact to detain various components of leachates as plumes migrate through aquifers. For cationic (positively charged ions) metals such as lead, cadmium, zinc, copper, mercury and chromium III, most of the attenuating effects are caused by sorption and precipitation. In general, the amount of cationic metal attenuation increases as the pH, cation exchange capacity, free lime content, free iron oxide content, and surface area of the aquifer's particles, increases. An increase in the ionic strength of the leachate decreases the amount of cationic sorption that can occur (Fuller, 1982).

The amount of cation exchange that takes place within an aquifer is also affected by other reactions including the weathering of clay minerals, decomposition of aquifer constituents, and occurrence of redox (reduction-oxidation) reactions. Some cations such as copper and zinc can become tightly sorbed, and then undergo secondary reactions that render them unexchangable. The types of cations in the leachate and in the original aquifer constituents also affect the amount of cation exchanges that occur. In most aquifer materials, the relative degree of affinity for specific cations is $Ca^{2+} > Mg^{2+} > K^{+} = NH_4^{+} > Na^{+}$. Other metallic cations are held less strongly than the above mentioned common cations (Kurtz and Melsted, 1973).

For anionic (negatively charged) metals, such as chromium VI, arsenic, and selenium, sorption increases as pH decreases. The ionic strength of the leachate does not seem to influence the degree of anionic metal sorption (Griffin and Shimp, 1978).

Of the anions commonly found in leachates, phosphate is the least mobile. Nitrate and chloride have approximately the same high mobility and are used as indicators of plume migration because very little attenuation of these leachate constituents occurs in most aquifers (Lindorff and Cartwright, 1977; Kurtz and Melsted, 1973; Roberts, et al., 1980).

In addition to the effects of ionic strength and pH on the attenuation of metals, the presence of certain organic constituents can substantially increase metal mobility. Under some conditions, hydrolyzable metals such as lead can become chelated by certain organic constituents even when the lead is originally undissolved. When chelated, the metal can remain in solution but still be unavailable for sorption or precipitation. For this reason, chelated metals can be highly mobile (Griffin and Shimp, 1978). Chelated metals may also be readily removed from solution if the chelating agent is itself an organic chemical (e.g., humic and fulvic acids).

Many non-ionic contaminants also exhibit considerable sorption. Sorption and attenuation can also occur because of van der Waals attractions, weak hydrogen bonding, covalent bonding, polar attraction, and other processes.

Plume migration rates can also affect attenuation. Laboratory tests of leachate attenuation in soil columns indicated that, in addition to the soil and leachate factors mentioned above, the amount of metal attenuation decreased as flow rate increased (Alessi, et al., 1980). Aquifer physical properties such as the presence of low permeability layers and the degree of compaction were also found to influence metal migration rates, probably as a result of the higher surface areas and lower flow rates associated with such conditions.

2.2.2.2 Attenuation of Organic Constituents

When organic constituents enter groundwater, one of four processes can occur. The constituent can be (Roberts, et al., 1980):

- Rapidly and completely degraded or precipitated
- Completely unaffected by groundwater conditions
- Partially sorbed or precipitated with slowly decreasing amounts of sorption occurring
- Slowly degraded, with gradually increasing amounts of degradation to a final steady level.

Attenuation of organic chemicals in the subsurface is an extremely complex problem in plume dynamics, and the subject of considerable ongoing research. Nevertheless, several generalizations can be made concerning the overall behavior of organic constituents within aquifers. These generalizations are as follows (Wilson, et al., 1980; Roberts, et al., 1980; Lindorff and Cartwright, 1977).

- Volatilization of low molecular weight organics is inhibited by aquifer particles, both by the distance volatile organic compounds must travel before they reach a free water surface, and as a result of the slow turnover of soil pore air in the unsaturated zone.
- Some organic constituents can be tightly sorbed onto aquifer particles
- Organics tend to be attenuated less than inorganics
- Most of the attenuation mechanisms are biochemical, rather than geochemical in nature
- The degradation of simpler, less toxic constituents is more pronounced than that of complex toxic substances
- Degradation is primarily anaerobic within aquifers, although some aerobic degradation has been found to occur
- The establishment of microbial degradation may occur after a phase during which the microbes become acclimated to the new organic substrate
- The presence of one organic constituent can influence the sorption and attenuation of another
- Degradation of organic leachate constituents can make tracing sources of leachates more difficult
- Low solubility or hydrophobic constituents tend to move more slowly than high solubility or hydrophilic substances
- Some sorption of dissolved organics onto organic constituents within the aquifer may occur.

The result of geochemical processes and attenuation is to reduce the rate of migration of certain constituents, remove the constituents from groundwater, or increase the mobility of certain constituents under certain conditions. A summary of these processes is presented in Table 2-7. The

TABLE 2-7

PROBABLE EFFECT OF VARIOUS PROCESSES ON THE MOBILITY OF CONSTITUENTS IN SUBSURFACE WATERS CONTAMINATED BY WASTE DISPOSAL (Jackson, 1980)

Physical Processes

Dispersion - Causes dilution of wastes and smearing of the plume front. The dispersive capacity of a porous or fractured medium is directly dependent on the groundwater velocity and the heterogeneity of the aquifer materials, and is inversely proportional to the porosity.

Filtration - Favors reduction in amounts of substances associated with colloidal or larger-sized particles (e.g., sediments and microbes). Most effective in clay-rich materials, least effective in gravels or fractured or cavernous rock.

Gas movement - Requires unsaturated conditions and high porosity if not dissolved in groundwater. Where gas movement can occur, favors aerobic breakdown of organic substances and increased rates of decomposition. Constituents mobile under oxidized conditions (e.g., chromium) will then predominate. Restriction of gas movement by impermeable, unsaturated materials or by saturated materials can produce an anaerobic state and reduced rates of organic decay. This will mobilize substances soluble under anaerobic conditions (e.g., iron, manganese)

Geochemical Processes

Complexation and ionic strength - Complexes and ion pairs most often form by combination of ions including one or more multivalent ions and increase in amount with increased amounts of ions involved. Ionic strength is a measure of the total ionic species dissolved in groundwater. Both ionic strength and complexation increase the total amounts of species in solution that would otherwise be limited by processes such as oxidation, precipitation, or sorption.

Acid-base reactions - Most constituents increase in solubility and thus in mobility with decreasing pH. In organic rich waters, the lower pH's (4 to 6) are associated with high values of carbonic acid and often also of organic acids. These will be most abundant in moisture saturated soils and rock.

TABLE 2-7 (continued)

Oxidation-reduction - Many elements can exist in more than one oxidation state. Contaminants will often be oxidized or only partially reduced in unsaturated soils and groundwater recharge areas, but will become reduced under saturated conditions when excess organic matter is present. Mobility depends on the element and pH involved: chromium is most mobile under oxidizing conditions, whereas iron and manganese are most mobile under those reduced conditions in which dissolved oxygen and hydrogen sulfide are absent.

Precipitation-dissolution - The abundance of anions such as carbonate, phosphate, silicate, hydroxide, or sulfide may lead to precipitation, especially of multivalent cations as insoluble compounds. Dilution or a change in oxygen content, where precipitation has resulted from oxidation or reduction, may return such constituents to solution.

Sorption-desorption - Ion exchange can withhold, usually temporarily, cations and to a lesser extent anions, on the surfaces of clays or other colloidal-sized materials. Amounts of sorbed metal cations will increase with increasing pH. Molecular species may be weakly retained on colloidal size materials by physical sorption. The much stronger binding forces caused by chemical action result in the formation of surface compounds involving metal ions and mineral grains. Sorbed species may return to solution when more dilute solutions come in contact with the colloidal material, depending on the nature of the sorption bond and sorption of organic chemicals by chemical interactions such as bonding and polar attraction.

Biochemical Processes

Decay and respiration - Microorganisms can break down insoluble fats, carbohydrates, and proteins, and in so doing release their constituents as solutes or particulates to subsurface waters.

Cell synthesis - N, C, K, and P, and some minor elements are required for growth or organisms, and can thus be retarded in their movement away from a waste disposal site because they are temporarily incorporated within microbial cells.

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types of processes that influence specific aquifer constituents are listed in Table 2-8 and descriptions of the overall effects of the various processes on the mobility of leachate components are given in Table 2-9. The reactions governing geochemical processes and attenuation are so complex and dependent upon site specific conditions, that pollutant attenuation is often ignored in evaluating leachate plume movement.

2.2.2.3 Estimating Leachate, Groundwater, and Aquifer Interactions

Several approaches have been used to estimate the effects of attenuation and other processes that occur within aquifers on the migration rates of inorganic and organic leachate constituents. Three of these approaches are described briefly below.

The amount of a constituent that is detained by earth materials has been estimated using laboratory studies of leachate migration through soils or aquifer materials under aquifer like conditions. One method used to measure the attenuative properties of various earth material involves saturating the pores of the sample with a pollutant laden solution. Additional amounts of the solution are allowed to flow through the sample, under anaerobic conditions, displacing the original pore volume of solution. This continues until the concentration of pollutant entering the sample (C_0) is equal to the concentration of pollutant in the solution flowing out of the sample (C). The point at which C/C_0 equals one is termed the breakthrough point (Fuller, 1982). When a contaminant concentration reaches the breakthrough point, the aquifer's ability to sorb that particular contaminant is exhausted (Roberts, et al., 1980).

A curve of C/C_0 concentrations plotted for various numbers of pore volume displacements indicates the relative amount of pollutant that will be attenuated by the soil at any particular point in the test (Fuller, 1982). Typical breakthrough curves are shown in Figure 2-18. Curve A in Figure 2-18 indicates little attenuation in occurring, while Curve E indicates that all of the pollutant entering the soil is being attenuated.

TABLE 2-8

PROCESSES WHICH MAY CONTROL AMOUNTS OF CERTAIN CONSTITUENTS IN SUBSURFACE WATERS CONTAMINATED BY WASTE DISPOSAL*
(After Jackson, 1980; Fenn, et al., 1977)

Constituent	Physical		Geochemical						Biochemical	
	Dis-persion	Filtra-tion	Complex-ation	Ionic Strength	Acid-Base	Oxid.-Red.	Precip-Solution	Adsorp.-Desorp.	Decay, Respir-ation	Cell Synthesis
Organic Solutes	X		X	(X)	X	X	(X)	(X)	X	X
Microorganisms	X	X				X			X	X
Heavy Metal Anions (Cr, V, Se, B, As)	X		X	(X)	X		X	X		
Heavy Metal Cations (Pb, Cu, Ni, Zn, Cd, Hg)	X		X	(X)	X		X	X		
NO_3	X					X			X	X
SO_4^{2}	X		(X)	(X)	(X)	X		(X)	(X)	
PO_4^{3-}	X		X	X	X		X	X	X	X
Na^+, K^+	X			(X)				X		
NH_4^+	X		X	(X)	X	X		X	X	X
Ca^{2+}, Mg^{2+}	X		(X)	X			(X)	X		
Fe^{2+}, Mn^{2+}	X		X	X	X	X	X	X		

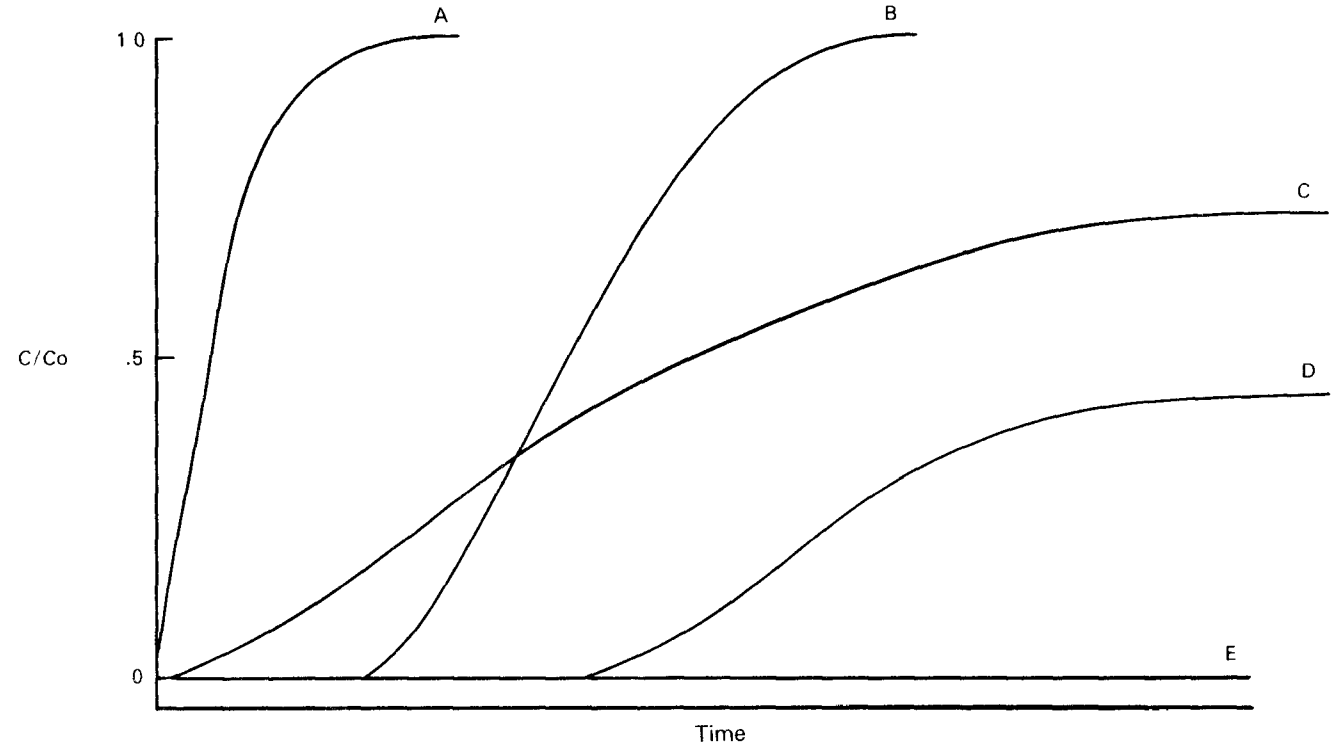
*(BRACKETS DENOTE MINOR CONTROLS)

TABLE 2-9

BEHAVIOR OF SPECIFIC CHEMICAL WASTES AT LANDFILLS
(Jackson, 1980)

Chemical Waste	Behavior
Halogenated Organic Compounds	Evaporation (because of volatility) and sorption onto other solid wastes present may reduce the release of halo-solvents to groundwater.
Phenols	Phenols are relatively soluble in groundwater, undergo biodegradation slowly, and are reversibly sorbed.
PCBs	Readily soluble in hydrocarbons, but not in water. Biodegradation inhibited by anaerobic conditions. Readily sorbed. PCBs are relatively inert within landfills and are not leached by water in high concentrations.
Mineral Oils	Sorption of oils onto solid fill material is an important attenuation process. Floats on groundwater because of low density relative to groundwater.
Organic Solvents	Some sorption on landfill materials and some biodegradation in sands and gravels have been observed.
Acids	Leaches sorbed contaminants and dissolves precipitates, thereby causing deterioration of groundwater quality. Inhibits microbiological activity. Releases harmful gases.
Cyanide (CN)	Most CN either volatilizes as HCN, or precipitates as cyanoferrate II compounds.
Heavy metals (e.g., Cr, Zn, Hg, Pb, Cd)	Heavy-metal cations (e.g., Pb^{+2}, Hg^{+2}, Cd^{+2}) controlled primarily by their insolubility as metal sulfides, carbonates, hydroxides or phosphates, and secondarily by sorption. Anionic forms (e.g., $Cr_2O_7^{-2}$) very mobile.

FIGURE 2-18.
TYPES OF BREAKTHROUGH CURVES GENERATED BY THE SOIL COLUMN TECHNIQUE
(FULLER, 1982)



Curve types vary depending on soil texture, pollutant type, and other factors. In general, clays attenuate pollutants much more effectively than sands, and inorganic cations are attenuated more effectively than are inorganic anions (Fuller, 1982). However, while dispersed clay particles in the aquifer matrix will very effectively undergo ion exchange with some contaminants, relatively impermeable clay liners and beds do not readily transmit flow and therefore provide very little attenuation. Care should be taken when attempting to relate breakthrough curve test data to actual situations, as test conditions rarely mimic site-specific situations. For example, reports of clay liner failure have shown that volatile solvents and fuels will strip the waters of hydration from clay and cause dessication cracks through which seepage occurs.

Another method used to estimate the rates of pollutant migration involves a more mathematical approach. For some specific chemicals, data have been assembled that can be used to estimate the migration distances under a specific set of conditions. One method used to express the relative degree of a chemical's mobility is to use a value termed the distribution coefficient (k_d). As k_d increases, the mobility of a substance decreases. Values of k_d for several pesticides are listed in Table 2-10. Values of k_d can be used in conjunction with mathematical models to estimate the size and shape of leachate plumes after a given period of time.

Many highly complex computer models have been developed to numerically simulate pollutant migration patterns. These models can be used to assist in a more comprehensive interpretation of the information on the site. These modeling programs, however, typically require a considerable amount of hydrogeologic data obtained over a long time period.

TABLE 2-10

PESTICIDE MOBILITY BASED ON DISTRIBUTION COEFFICIENTS
(After Jackson, 1980)

Relative Mobility	Pesticides		k_d Values	
	Name	Use	Range	Loam Soil-3% organic matter
Highly Mobile	Picloram	Herbicide	0-10	0-3
	2,4-D	Herbicide		1
	Atrazine	Herbicide		3
Somewhat Mobile	Diuron	Herbicide	10-100	10
	Ethirimol	Fungicide		15
	Lindane	Insecticide		50
Relatively Immobile	Parathion	Insecticide	100-1,000	200
No Significant Leaching	DDT	Insecticide	+1,000	1,000
	Paraquat	Herbicide		10,000+

When models have been used to predict plume behavior under field conditions, data from field studies have sometimes been found to contradict computer predictions (Perlmutter and Lieber, 1970; Roberts, et al., 1979). For this reason, many researchers recommend extreme caution when interpreting the data from mathematical modeling studies (Cole, 1982; Fenn, et al., 1980; Roberts, et al., 1979). Despite the disadvantages of various computer models, numerical simulation can be a useful tool to assist in organizing and evaluating the complex geochemical, physical, and biochemical interactions that take place within aquifers. Models useful for plume delineation are discussed further in Section 3.1.

CHAPTER 3

PLUME DELINEATION

Controlling the migration of a leachate plume requires a thorough understanding of the distribution of contaminants in the subsurface. Because contaminant movement is usually dominated by lateral groundwater movement, the direction and rate of groundwater flow must be known in order to approximate plume boundaries. Initial estimates of plume boundaries are made assuming that subsurface materials are homogeneous, isotropic, and non-reactive (i.e., degradation and attenuation of plume constituents does not occur), and that contaminant movement is controlled only by groundwater flow. Initial estimates of plume location can then be improved by making use of known contaminant characteristics to estimate how the plume will react in groundwater (e.g., denser contaminants will tend to sink in the water column) and by determining preferential flowpaths using a knowledge of subsurface geologic conditions (e.g., faults and fractures in consolidated rock or sand and gravel lenses in lower hydraulic conductivity clay matrices).

In general, the more data that are available on a site, the greater the probability that an approximation of plume boundaries will approach actual conditions. At a minimum, information on groundwater flow direction and rate, contaminant characteristics and release rates, and subsurface geologic profiles are required. Because groundwater plumes cannot be readily observed (unlike an oil spill on surface water), monitoring wells that provide direct groundwater quality data will always be required in plume delineation studies. However, because monitoring wells are fairly expensive to install and monitor, well data can be optimized by the careful application of less resource intensive data collection techniques that do not provide direct groundwater quality data. The purpose of this chapter is to provide generalized procedures for delineating leachate plumes using direct and indirect methods.

3.1 Plume Delineation Procedures

A generalized approach to identifying the boundaries of a contaminant plume is illustrated in Figure 3-1. This approach involves a number of activities which can be summarized in three major steps:

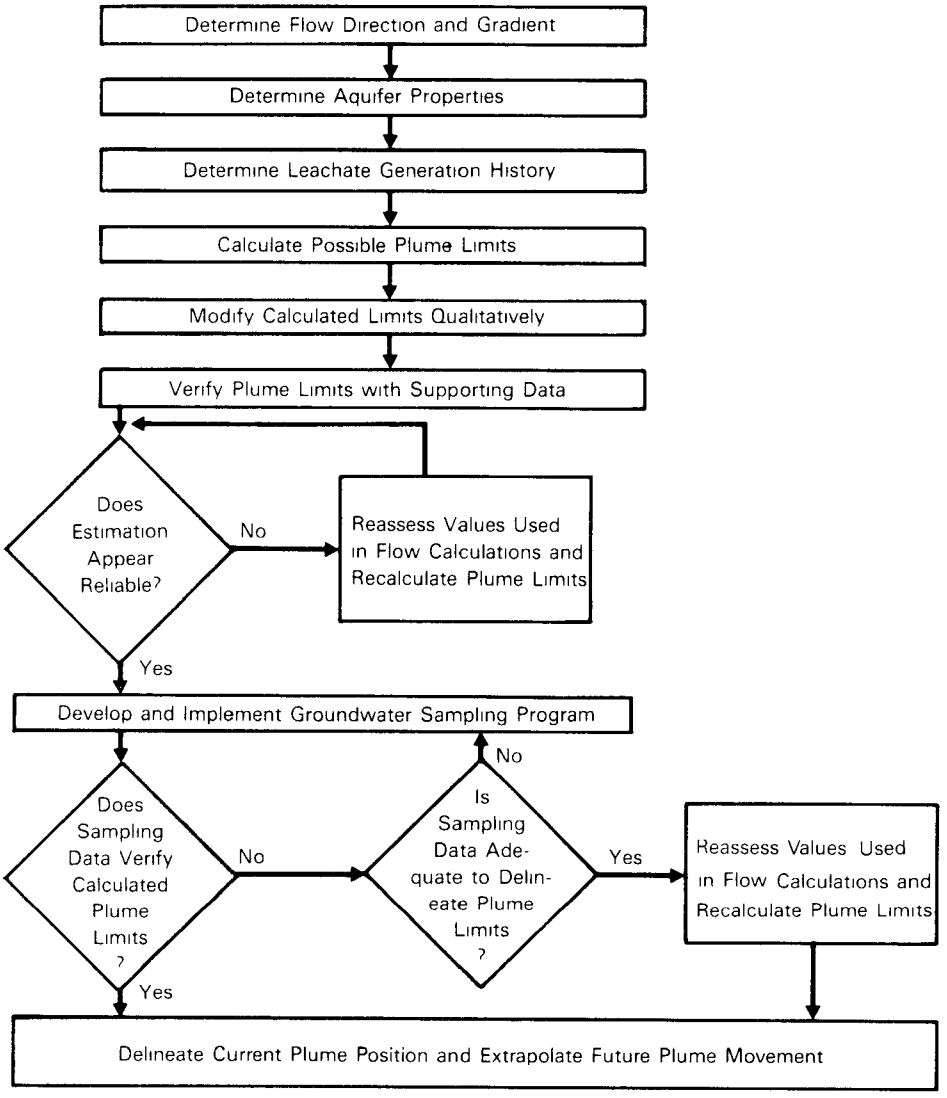
- Calculate possible plume boundaries based on site hydrogeology and contaminant release information
- Modify calculated boundaries qualitatively based on site and contaminant characteristics, and verify these boundaries, calculations and modifications using groundwater sampling data and supporting information
- Extrapolate future plume movements using the verified groundwater flow data and computer models, if needed.

The objective of this approach is not only to delineate current and future plume positions but also to ensure that information required for implementing plume control measures will be available. These steps are described in further detail below.

3.1.1 Calculate Possible Plume Boundaries

The first step in calculating possible plume boundaries is to identify the direction(s) of plume movement and what aquifer, aquifers, or portions of aquifers may be contaminated. Because information on groundwater contamination will usually be limited at this point in the process, hydrogeologic data on the site (e.g., groundwater levels) are generally used to identify water bearing zones that may lie in the path of a migrating plume. This evaluation should consider both horizontal and vertical plume movement as discussed in Chapter 2.0. If enough data are available, a potentiometric surface map of the potentially affected aquifers should be constructed.

FIGURE 3-1.
GENERALIZED APPROACH TO DELINEATING LEACHATE PLUMES



In evaluating hydraulic head data, the following points are important to remember:

- Any well pair used to calculate a hydraulic gradient should be located along the same flowpath. Wells used to establish horizontal gradients should be screened in the same stratigraphic horizon; wells used to establish vertical gradients should be in close proximity to each other.

- Water levels should be measured from an accurately surveyed, fixed reference point (e.g., top of well casing). Calculated water level elevations and potentiometric surface contours should not exceed the precision of the original survey and the water level measuring device. Calculated gradients should not exceed the precision of the elevation and distance-between-wells measurements.

- Water level data should be collected from all wells in a monitoring system at the same time to minimize temporal variations (e.g., seasonal water level fluctuations). Temporal variations should be assessed by measuring system head levels over the course of several cycles.

In the absence of reliable hydrogeologic information on the site, the usual assumptions are that (1) horizontal flow predominates over vertical flow, (2) contamination is restricted to unconfined aquifers, and (3) contamination levels will be highest in the uppermost unconfined aquifer. Subsequent field investigation activities are then planned to ascertain the validity of these assumptions. Excellent discussions of methods for determining the direction of groundwater flow are given in Fetter (1981); Pinder, et al., (1981); and Abriola and Pinder (1982).

The second step in calculating possible plume boundaries is to estimate the rate of groundwater and contaminant movement using Darcy's Law (refer to Section 2.1.1). Data required by this equation can be obtained using a variety of methods.

Initial calculations typically result in ballpark estimates based on <u>indirect data</u>. Indirect data includes generalized data not based on actual site conditions and data functionally related to the data required by Darcy's Law. Published reports, aerial imagery, and geophysical surveys are examples

of indirect data sources which are described in more detail in Section 3.2. More precise calculations typically require using direct data on site conditions. Direct data for a site is developed using hydrologic and groundwater sampling methods described in Section 3.3. Table 3-1 summarizes the availability and reliability of several information sources for Darcy's Law calculations.

Darcy's Law is an attractive means for calculating groundwater flow because it is theoretically simple, not based on many limiting assumptions, applicable to most hydrologic systems, and uses parameters that can be estimated readily. One major problem, however, is that Darcy's Law loses validity under conditions of very low and very high hydraulic gradients and hydraulic conductivities. The reason for this is that Darcy's Law is designed to predict fluid movement through saturated media under conditions of laminar flow. Under these conditions, flow paths are assumed to be layered and minimal intermolecular mixing is expected to occur. As flow rates and hydraulic gradients increase, however, mixing becomes more predominant and flow less laminar. When velocity increases sufficiently turbulent flow begins. Under these conditions, flow pathways are nonlinear and a great deal of mixing may occur. In most aquifers flow rates are normally low enough to allow for accurate determinations of velocity (Davis and DeWiest, 1966). Under certain conditions, however, turbulent flow in aquifers can occur. Conditions causing turbulent flow include aquifers with extremely porous areas such as along fracture zones, fissures, and large openings in subsurface units (Freeze and Cherry, 1979).

Calculations of contaminant movement using Darcy's Law assumes water soluble, conservative contaminants in non-dispersive and unattenuative porous media (Popkin, 1983). Fried (1975) and Freeze and Cherry (1979) present analytical methods for addressing dispersion and attenuation. In many cases, data necessary to use the dispersion and attenuation equations will not be available.

The final step in calculating possible plume boundaries is to determine the history of a site's leachate generation. Based on the amount of time (t)

TABLE 3-1
INFORMATION SOURCES FOR CALCULATING VELOCITY
USING DARCY'S LAW (V = KI/n)

Parameter	Source	Availability	Data Reliability
Hydraulic Conductivity (K)	On site aquifer testing results	Not generally available unless previous site work has been undertaken	Excellent if planned and implemented properly
	Laboratory testing results	Not generally available unless previous site work has been undertaken	Generally good, but are less reliable for unconsolidated tests of compressible material such as clay. False low values are a common problem. Sample representativeness is also a concern (see Section 3.3.1)
	Site related reports	Commonly available from a variety of sources such as feasibility studies for water supply or construction projects at neighboring sites	Generally good depending on proximity of study area and variability of hydrogeology
	Regional reports	Readily available in a variety of forms from such sources as the USGS, state geological surveys, and universities	Moderate to low unless site data was used as part of the regional database
	General guidelines	Readily available (see Table 2-2)	Generally low
Hydraulic Gradient (I)	On site surveys of water levels in wells	Not generally available unless previous site work has been undertaken	Excellent if seasonal and other temporal variations are taken into account. Generally good otherwise if the guidelines given in Section 3.1.1 are followed.

TABLE 3-1 (continued)

Parameter	Source	Availability	Data Reliability
Hydraulic Gradient (I) (continued)	Regional and site-related reports	Commonly available from the USGS, state geological surveys, universities, and other geotechnical or engineering sources	Generally moderate to low because of the highly site specific nature of hydraulic gradients
	Estimation from site topography	Topographic maps are readily available from the USGS. Larger scale topographic maps may also be available from local government agencies.	Generally low
Porosity (n)	Laboratory testing results	Not generally available unless previous site work has been undertaken	Generally moderate to good but can be highly variable because of problems associated with sample extraction and representativeness
	General guidelines	Readily available (see Table 2-1)	Generally low

that contaminants have been within the saturated zone and the estimated velocity of groundwater and contaminants (V_g) from Darcy's Law, the maximum distance (D_p) a plume could migrate from a source area is given by:

$$D_p = V_g t$$

However, determining the timing of contaminant releases from a site is commonly the most difficult and least reliable step in calculating possible plume boundaries. This is especially true of abandoned sites which lack adequate operation records.

The approach generally taken to estimate release time, t, is to assume that leachate generation is simultaneous with the initiation of site activities and then try to pinpoint the initiation of site activities. This approach is usually valid because the time between waste disposal and leachate generation is typically small relative to the age of the site. Unfortunately, determining when hazardous waste related site activities were initiated is not always possible. Some possible sources of this type of information in the absence of owner or operator records include historical aerial photographs, county tax records, chamber of commerce records, and waste generator and transporter records.

Table 3-2 provides an example to illustrate how possible plume limits can be calculated. Other methods for calculating plume limits are described by Pettyjohn, et al. (1982).

3.1.2 Modify and Verify Calculated Boundaries

Once the initial calculations of possible plume limits are completed, they should be drawn on map view and on several cross sections of the site. The calculated boundaries should then be assessed and modified as needed based on the factors affecting plume movement as described in Chapter 2. The purpose of this step is to help ensure that optimal use will be made of each

TABLE 3-2
EXAMPLE CALCULATIONS OF MAXIMUM PLUME LIMITS (D_p)

CALCULATION	METHOD
INITIAL	Given that $D_p = t(KI/n)$ where: t = 19 years based on county records showing purchase data by disposal facility operator and assumption that onsite disposal and leachate generated began immediately K = 10 ft/day based on regional geologic reports that the site is underlain by a thick deposit of silty sand (see Table 2-2) I = 0.01 based on an estimate using site topography and surface water elevations n = 35% based on published guidelines for silty sand. Maximum plume limits (assuming zero attenuation) can be estimated by: D_p = (19 years) (365 days/year) (10 feet/day) (0.01)/(0.35) D_p = 2,000 feet.
REVISED	Given that $D_p = t((T/m)\ I/n)$ where: (Note: K = T/m) t = 17.5 years based on more detailed site operation information provided by former employees and a preliminary water balance calculation for leachate generation T = 900 gpd/foot based on a pumping test m = 65 feet based on borings indicating a clay layer 90 feet deep and an average depth to water of 25 feet. I = 0.008 predominantly in the horizontal direction based on water levels taken on site n = 40% based on borings that indicate a lower proportion of sand than previously thought and published guidelines Maximum plume limits (assuming zero attenuation) can be estimated by: D_p = (17.5 years) (365 days/year) ((900 gpd/ft/7.48g/ft^3)/65 feet) (0.008)/(0.40) D_p = 236.4 ft

well and other sampling points in the site monitoring program. The key factors to consider in evaluating a calculated plume boundary are:

- Groundwater flow patterns (Section 2.1)
- Leachate or contaminant characteristics (Section 2.2.1)
- Plume and geologic media interactions (Section 2.2.2).

Of the three, groundwater flow patterns will generally be the most important and least difficult factor to evaluate in assessing the calculated plume limits. Should certain values used in the initial flow calculation appear to be inappropriate based on this step, the values should be modified and the maximum plume limits recalculated.

There are a variety of non-sampling methods for checking the validity of groundwater and plume movement calculations including preliminary site inspections, aerial imagery, and geophysical surveys. The types of information that can be obtained by these and other means are outlined later in this chapter. However, to describe succinctly how this information should be used on a site specific basis to adjust initial estimates is very difficult. Therefore, personnel well qualified to assess waste site hydrogeology should be involved in delineating the location of a site's plume.

Based on the final estimates of the plume's anticipated boundaries, a groundwater monitoring program is developed and implemented. Plume delineation by groundwater sampling is discussed in Section 3.3.2. If utilized properly, groundwater monitoring results will establish plume boundaries with the accuracy needed to develop remedial action plans. In addition, the groundwater monitoring data should be used to reassess the values used in the flow calculations to ensure their accuracy. This is particularly important if long term predictions of plume movement are to be developed.

3.1.3 Extrapolate Future Plume Movement

Most plume management strategies seek to contain, extract, or treat the plume so that it does not pose a threat to the environment. Some strategies, however, seek to control the plume's movement so that it will not affect sensitive receptors. Examples of this latter strategy include diverting the plume away from drinking water wells, counterpumping to prevent the plume from advancing in an unwanted direction, and not taking action because an immediate environmental threat is not evident. In cases where plume movement is being controlled, or the "no action alternative" is an issue, extrapolation of future plume movement trends is imperative. Predicting plume movement can be accomplished manually using calculations such as those described in Section 3.1.1 or through the use of computer models.

There are many commonly accepted models that can be used in forecasting leachate plume movements and the effects of plume management plans. These models can be divided into two major groups--release rate models and solute transport models. Typically, estimates of leachate quantity and quality released from a site are obtained from a release rate model or through a groundwater sampling program, and are used as input to a solute transport model. The theory behind some of the models is fairly complex and can be found in sources such as Bachmat, et al. (1980), Mercer and Faust (1981), Anderson (1979), Weston (1978), and Repa, et al. (1982).

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 has been released into the subsurface environment. Only after adequate determination of leachate release can a solute transport model be used. Most release rate models are based on dividing the problem of prediction into three separate components--leachate generation rates, constituent concentrations, 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. Because of the unknowns included in estimating values for these components, data are generally obtained through a groundwater monitoring program and input directly into a

solute transport model. Table 3-3 summarizes six computer models that can be used for predicting leachate release rates.

Solute transport models utilize 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 leachate plumes using the physical processes (e.g., groundwater flow) involved, while stochastic models attempt to define causes and effects using probabalistic methods.

Deterministic mathematical models can be further divided into analytical and numerical models. Analytical models simplify mathematical equations, allowing solutions to be obtained by analytical methods (i.e., functions 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 distribution, recharge and discharge points, and locations and types of boundaries
- Material Constants -- hydraulic conductivity, porosity, transmissivity, extent of hydrogeologic units
- Attenuation Mechanisms -- sorption-desorption, ion exchange, complexing, nuclear decay, ion filtration, gas generation, precipitation-dissolution, biodegradation, chemical degradation
- Hydrodynamic Dispersion -- diffusion and dispersion (transverse 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 (generally consisting of a water

TABLE 3-3
RELEASE RATE MODELS

Model	Reference	Calculation Method	Advantages	Disadvantages
Drainmod/ Drainfill	Skaggs (1982)	• Water balance method • Drain equations	• Calculates head levels within site • Predicts quantity of drainage to leachate collection system • Predicts quantity of leachate moving through underlying clay	• Data intensive • Does not consider process within cell that affects leachate quantity or quality • Untested model
HELP/ HSSWDS	Perrier and Gibson (1980)	• Water balance method	• Minimal data requirements • Estimated horizontal and vertical drainage through a maximum of 8 layers • Estimates impingement rates on collection system	• Not field tested • Does not evaluate leachate quality
LSIPE	Moore (1980)	• Linearized equation with simplified boundary conditions	• Evaluates numerous linear design types • Allows for nonlinear equation with complex boundaries • Predicts release rates	• Data intensive • Assumes landfill or surface impoundment are designed properly • Estimates through single cell only • Does not predict leachate quality
PCLTF	USEPA (1982c)	• Analytical equation series	• Addresses all three components necessary to predict release rate and quality • Assesses 7 generic sites	• Data intensive • Untested model
LTTM	Pope-Reid Associates (1982)	• Water balance method	• Calculates leachate volume, leachate head, containment time, seepage rates, travel times • Easy to use	• Does not address leachate quality • Untested model
Release Rate Computation	SCS Engineers (1982)	• Analytical equations	• Series of simple calculations • Predicts volume of leachate generated over time	• Equations may be over simplified • Does not predict concentration of constituents • Untested model

balance equation coupled with Darcy's Law) and a mass balance equation (which describes the concentrations of a chemical species in a flow pattern). 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 expressions. 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 can be performed involves changing a single parameter (within known values of occurrence) to demonstrate the effects that variations in individual parameters have on model output. This analysis helps identify those parameters that have the greatest influence on model results. A second type of sensitivity analysis involves a series of trial runs 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).

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.

Numerical models characterize groundwater contamination processes without the simplification of complex physical and chemical characteristics required

by analytical models. 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 equations) are solved by subdividing the entire problem domain into a grid system of polygons. Each polygon block has assigned hydrogeologic properties (e.g., transmissivity) associated with it that define the aquifer at that point. Accompanying each block is a node point that represents a position in the aquifer with an equation having 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 dependent 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 (DPRW) 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 DPRW 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 time consuming, expensive, and requires considerable computer storage.

Tables 3-4 and 3-5 list some of the available models that can be used to predict the movement of contaminants in groundwater. As with the release rate models listed in Table 3-3, each model contains elements that would make it more applicable to certain site and waste characteristics. Only eight analytical and nine numerical models are presented in these tables. Many more models exist that can be used for the prediction of chemical migration such as Weston (1978) and Thomas, et al. (1982).

3.2 Indirect Data

Indirect data includes generalized data not based on actual site conditions and data only functionally related to the information required. Field investigations involving monitoring well installation, aquifer testing, groundwater sampling and analyses, and associated activities, are typically the most critical and costly elements of groundwater quality assessment projects (Popkin, 1983). The purpose of indirect data, therefore, is to optimize the usefulness of monitoring wells by identifying the most effective locations based on anticipated plume limits. This can help minimize costs as fewer wells may be necessary. In certain instances, risks to personnel installing monitoring wells at hazardous waste sites can also be reduced (i.e., in the case where wells must be located in close proximity to buried waste).

TABLE 3-4
ANALYTICAL SOLUTE TRANSPORT MODELS (Repa, et al., 1982)

Model	Reference	Dimensions Modeled	Advantages	Disadvantages
SESOIL	Bonazountas and Wagner (1981)	1	• Ease of use • Long term fate simulations • Includes numerous degradation processes • Models organics and inorganics	• Unsaturated zone only • Not field or analytically verified • May be overly simplistic
PESTAN	Enfield, et al. (1982)	1	• Predicts pollutant velocity, length of pollutant slug, and concentrations • Screening model-rapid evaluations • Inexpensive to run • Field verified: DDT and aldicarb predictions	• Models only organic species • Unsaturated zone only • May be overly simplistic
PLUME	Wagner (1981)	2	• Two dimensional plume traces in groundwater • Incorporates degradation and sorption terms • Can be coupled with PESTAN • Field verified: chromium	• May be overly simplistic

TABLE 3-4 (continued)

Model	Reference	Dimensions Modeled	Advantages	Disadvantages
LPMP	Kent (1982)	2	• Predicts migration and mixing of contaminants in groundwater zone • Incorporates degradation terms, and dispersion and diffusion • Ease of use • Field verified: chromium	• May be overly simplistic
AT123D	Yeh (1981)	1-2-3	• Predicts migration of contaminant in saturated and unsaturated zone • Incorporates wide range of transport mechanisms • Example problems provided • Numerous source release, aquifer and waste options	• Not field or analytically verified
PATHS	Nelson and Schur (1980)	2	• Predicts plume movements in saturated zone • Inexpensive, first cut evaluations • Can be run with data generally available for site • Analytically verified	• Not field verified • Degradation and dispersion options not included • May be overly simplistic

TABLE 3-5
NUMERICAL TRANSPORT MODELS (Repa, et al., 1982)

Model	Reference	Dimensions Modeled	Advantages	Disadvantages
MMT/VTT	Battelle (1982)	1-2	o Predicts contaminant transport using discrete parcel random-walk method o Incorporated degradation terms o Provides graphic displays o Field verified: tritium	o Requires intensive data to run model accurately* o Costly to run*
CFEST/ UNSAT1D	Battelle (1982)	2-3	o Predicts contaminant transport using finite element techniques o Models complex flow systems o Analytically verified	o Degradation and sorption term presently being incorporated o In the process of field verification: arsenic and pharmaceutical chemicals o Model documentation in preparation
Pollutant Movement Simulator	Khaleel and Reddell (1977)	2-3	o Predicts contaminant transport using finite difference techniques o Incorporates convection and dispersion equations using MOC o Field tested: NaCl	o Attenuation processes not considered o In the process of being verified for mine wastes
FEMWASTE	Yeh and Ward (1981)	2	o Predicts contaminant transport using finite element methods o Incorporates convective dispersion, sorption, and degradation equations o Field verified	

*These two disadvantages apply to all numerical models

TABLE 3-5 (continued)

Model	Reference	Dimensions Modeled	Advantages	Disadvantages
Random Walk	Prickett, et al. (1981)	1-2	o Predicts contaminant transport using random walk methods o Incorporated dispersion and attenuation equations o Field verified: fertilizers o Accounts for varying discharge and recharge rates	
Solute Transport, Dispersion Model	Konikow and Bredehoeft (1974)	2	o Predicts contaminant transport using method of characteristics o Allows for varying recharges and discharges, and aquifer properties o Field verified: radioactives	o Does not incorporate attenuation of chemical species
SWIFP	INTERA	3	o Predicts contaminant transport using finite difference method o Allows for dispersion and attenuation effects o Field verified: sea-water, geothermal, radioactives, and coal tar	

TABLE 3-5 (continued)

Model	Reference	Dimensions Modeled	Advantages	Disadvantages
LOMAM	Sykes, et al. (1982)	1-2-3	o Predicts contaminant transport using finite element techniques o Incorporates attenuation and dispersion equations o Field verified: Cl, K, radioactives, and aldicarb	o Model specific to sanitary landfills o Currently under review

This section discusses several methods of obtaining indirect data useful in estimating the limits of groundwater contamination. They are:

- Previously collected data
- Aerial imagery
- Geophysical (surface and subsurface) surveys.

Indirect data will not be useful for all sites and each method has theoretical and practical limitations which affect the data obtained. The potential benefits to be gained from indirect data collection efforts (primarily geophysical surveys) must be weighed against their cost.

3.2.1 Previously Collected Data

Some data will always be available concerning sites at which CERCLA funds are being used because these sites must be rated using the Uncontrolled Hazardous Waste Site Ranking System (40 CFR 300 Appendix A), which requires specific data on hydrogeologic conditions, waste types, and site conditions. The type of information necessary to rank a site and therefore generally available in EPA files is shown as Table 3-6. While the EPA and state hazardous waste management agency files and inspection reports may provide much of the data required to plan a remedial investigation, some data items may not always be available. In these cases, supplemental information can often be obtained from published maps and reports, computer data banks, and other sources.

Published maps and reports on various topics are available from a number of sources besides EPA and state environmental agencies. Table 3-7 lists several sources in the Federal government. Many of the information items listed in Table 3-7 are also available from sister agencies at the state and local levels of government. Of these sources, probably the most useful are the U.S. Geological Survey (USGS) and the state geological surveys. Both the USGS and state surveys can provide information on studies currently being conducted in a certain geographic area and typically have on-staff experts in

TABLE 3-6
TYPE OF INFORMATION GENERALLY AVAILABLE IN
USEPA SITE INVESTIGATION REPORTS

WASTE DATA

- Type of compounds
- Physical state (solubility, viscosity, specific gravity)
- Amount
- Hazard assessments (toxicity, biodegradability, persistence, ignitable, etc.)

POPULATION AREA

- Population within 1, 2, 3, or 4 mile radii of site
- Population served by drinking water within 3 miles of site
- Population served by surface water within 3 miles of site

HYDROLOGIC DATA

- Depth to groundwater
- Depth to aquifer
- Direction of groundwater flow
- Hydraulic conductivity of aquifers
- Net precipitation and 1 year, 24-hour rainfall
- Groundwater use
- Distance to nearest drinking water well
- Groundwater sampling results
- Surface water sampling results

GEOLOGIC DATA

- Type of overburden and bedrock
- Area topography
- Location of sensitive environments
- Soil sampling results

SITE HISTORY

- Type and extent of disposal operations
- Original and subsequent owners
- Observed contamination or release incidents
- Regulatory responses

TABLE 3-7
TYPES AND SOURCES OF MAPS, REPORTS, AND RELATED INFORMATION
(ADAPTED FROM CLARK, ET AL., 1981)

Topic	Source*
Climate	NCC, NOAA, NOS, NWS
Clinometric (Slope)	GS, SCS
Coal Investigations	GS
Earthquake Hazards	GS
Floodplains	CE, FIA, GS, NOS, SCS
Geodetic Control	CE, GS, NOS
Geologic	GS
Geophysical	GS, NOAA
Groundwater	GS
Land Cover	PS, SCS
Land Use	GS, SCS
Mineral Investigations	GS
Mines	BOM
Navigable Waterways	CE, GS, TVA
River Basin/Watershed Surveys	GS, SCS, TVA
River Surveys	BR, TVA
Soils	SCS
Topography	GS
Water Resources	GS
Wildlife and Scenic Rivers	BLM, FS, PS

*Many of these types of maps and reports are also available from State and local government agencies and universities:

BLM: U.S. Bureau of Land Management
Office of Public Affairs
Washington, DC 20240

BOM: U.S. Bureau of Mines
2401 E Street, NW
Washington, DC

BR: U.S. Bureau of Reclamation
P.O. Box 25007
Denver, CO 80225

CE: U.S. Army Engineer District
Corps of Engineers, Chicago
219 South Dearborn St.
Chicago, IL 60604

(continued)

TABLE 3-7 (continued)

CE: U.S. Army Engineer District
Corps of Engineers, Nashville
Post Office Box 1070
Nashville, TN 37202

U.S. Army Engineer District
Corps of Engineers, Omaha
6014 U.S. Post Office and Courthouse Bldg.
Omaha, NE 68102

U.S. Army Engineer District
Corps of Engineers, Vicksburg
Post Office Box 60
Vicksburg, MS 39180

FIA: U.S. Federal Insurance Administration
Engineering Division
451 7th Street, SW
Washington, DC 20514

FS: U.S. Forest Service
Information Office, Rm. 3238
Post Office Box 2417
Washington, DC 20013

GS: Eastern National Cartographic Information Center
U.S. Geological Survey
Reston, VA 22092

Mid-Continent National Cartographic Information Center
U.S. Geological Survey
1400 Independence Road
Rolla, MO 65401

Rocky Mountain National Cartographic Information Center
U.S. Geological Survey
Federal Center, Building 25
Denver, CO 80225

Western National Cartographic Information Center
U.S. Geological Survey
345 Middlefield Road
Menlo Park, CA 94025

(continued)

TABLE 3-7 (continued)

NCC: U.S. National Climatic Center
Federal Building
Asheville, NC 28801

NOAA: U.S. National Oceanographic and Atmospheric Administration
Office of Public Affairs
14th Street, NW
Washington, DC

NOS: U.S. National Ocean Survey
Distribution Division (C-44)
6501 Lafayette Avenue
Riverdale, MD 20840

NWS: U.S. National Weather Service
Gramax Building
8060 13th Street
Silver Spring, MD 20910

PS: U.S. National Park Service
Office of Public Inquiries, Room 1013
Washington, DC 20240

SCS: U.S. Soil Conservation Service
Information Division
Post Office Box 2890
Washington, DC 20013

TVA: Tennessee Valley Authority
Mapping Services Branch
111 Haney Building
Chattanooga, TN 37401

local geology. State surveys will probably provide more detailed hydrogeologic information about a specific locality than would the USGS but coverage is typically more limited. Data which geologic surveys are able to provide include regional and local subsurface profiles, aquifer characteristics, and potentiometric surface maps.

The USGS also coordinates NAWDEX--the National Water Data Exchange--consisting of over 400 organizations concerned with water and water programs, which have agreed to share their water data for maximum use. While the type and quality of information varies between organizations, the service can locate local sources of water data including well locations, geologic profiles, potentiometric surfaces, water quality data, and aquifer characteristics.

NAWDEX services are readily available. Requests may be made at one of 60 NAWDEX offices or the USGS National Headquarters (703/860-6031) by phone, letter, or personal visit. The request is reviewed by NAWDEX for clarity and completeness and then is processed. NAWDEX either provides the requested data from its files or the inquiry is referred directly to organizations that hold specific data. NAWDEX does charge a fee for the service (USGS, 1980).

While the primary function of NAWDEX is not to provide data storage and retrieval services, NAWDEX does have direct access to several large data banks including:

- WATSTORE -- The USGS maintains a water data storage and retrieval system for geologic, hydrologic, and chemical data on surface water and groundwater in the U.S. Data is available from approximately 16,000 stream gaging stations, 1,000 lakes and reservoirs, 5,200 surface water quality stations, 30,000 water level observation stations and 12,500 groundwater quality wells. A groundwater site inventory file contains information for nearly 700,000 wells (Kilpatrick, 1981).
- STORET -- EPA maintains a storage and retrieval system which contains over 40 million individual observations of water quality parameters for both surface and groundwater. Information is collected from several Federal organizations as well as more than 40 state organizations.

Despite the large number of data sources, WATSTORE, STORET, and the other data banks available through NAWDEX will probably not contain information from sources close enough to a particular site to be of great value. The information they contain may provide useful regional geohydrologic data and the service should be considered as an inexpensive source of background hydrogeologic data.

In addition to the sources described above, there are also a variety of other sources of information that can be used in the plume delineation process. For example, well drillers familiar with local conditions can provide a wealth of information on subsurface conditions and aquifer characteristics. Local geotechnical contractors and testing laboratories can be a valuable source of data on porosity, hydraulic conductivity, and other subsurface characteristics. Geologists on the staff of local colleges and universities should also be familiar with regional geology and hydrology. Examples of other potential information sources are listed in Table 3-8.

A major type of previously collected data other than maps, reports, computer data banks, and local contacts is aerial imagery. Aerial imagery is the subject of the next section.

3.2.2 Aerial Imagery

Aerial imagery refers to pictorial representations produced by electromagnetic radiation that is emitted or reflected from the earth and recorded by aircraft-mounted sensors. The simplest, most common form of imagery is the photograph, which uses only the visible part of the electromagnetic spectrum. Oblique photos are taken at angles to the earth's surface and thus distort the scale of the picture (i.e., objects in the foreground are larger and objects in the background are smaller than they should be). Perpendicular, or stereophotos, which are taken from directly above the site so that there is minimal distortion, can be used in pairs to show the topography of the site in three dimensions.

TABLE 3-8
EXAMPLES OF ADDITIONAL SOURCES OF INFORMATION
FOR REMEDIAL INVESTIGATIONS

Source	Information
State and Local Government Agencies	
Health	Reports of unusual health problems related to the site, drinking water analyses, locations of contaminated wells
Planning and Zoning	Land use
Tax Assessor	Plat maps and land ownership
Engineer	Foundation and inspection reports, survey benchmark locations
Fire Department	History of fires or explosions at a facility
Law Enforcement	Complaints and violations of local ordinances during site operation
Water and Sewer	Geotechnical studies related to water supply or construction, locations of buried mains and lines, data related to quality of influent to treatment plant
Utility Companies	
Gas, Electric, Water, and Petroleum or Natural Gas Pipelines	Location of buried lines

TABLE 3-8 (Continued)

Source	Information
Contractors	
Building	Local soils, geology, and shallow water levels
Soil exploration and foundation	Local soils, geology, hydrogeology, water levels, regulations, and equipment availability
Water supply and well drillers	Local hydrogeology, aquifer properties, water quality, and locations of existing wells
Trade Associations	
National Water Well Association 500 W. Wilson Bridge Rd. Worthington, OH 43085	Local well drillers and related contractors, well design and construction information, and water supply and aquifer restoration information
Chamber of Commerce	Information on local industries including waste generators, transporters, and disposers
Universities	
Geology Engineering, Biology, and Agronomy Departments, and Medical Schools	Various types of information on local conditions

The second type of aerial imagery uses wavelengths of light outside the visible spectrum. The most common and useful type for detecting the effects of leachate is the infrared image. Infrared imagery indicates areas that are hotter or cooler than the general surroundings. This is useful as areas of dead vegetation having different radiant heating (albedo) characteristics than vegetated areas (JRB, 1980).

Indicators of leachate can be both spatial and spectral. Spatial indicators refer to physical changes to systems caused by leachate. These include gaps in vegetation or snow cover, and areas of wetness indicating an outbreak of groundwater or a leachate seep. Spectral indicators refer to color changes in either water or in vegetation, and may make use of wavelengths other than in the visible range. If historical aerial imagery is available, comparisons between new and old imagery can indicate changes in an area that may be caused by leachate movement. The gradual development of a barren area in a forest for unknown reasons may indicate the presence of hazardous wastes. In general, spatial indicators are more useful and consistent than spectral indicators (Sangrey and Philipson, 1979).

Gaps can occur in snow or vegetative cover because of leachate wetness, toxicity or heat. Gaps caused by heat would appear in close proximity to landfills because heat dissipates quickly with distance and would tend to radiate from the landfill. Gaps caused by excessive wetness or toxic substances can occur at greater distances and may radiate from a site or may occur as isolated patches. Vegetative gaps can be detected in photographs or near-infrared images. Gaps can also be distinguished in thermal infrared and microwave images because emissions from vegetation and base soil are different. Active microwave sensing by radar should detect most vegetative soil gaps not obscured from direct view. Gaps in snow are easily detected in photographs, or near-infrared, thermal infrared, and microwave images. The problem with using gaps as indicators of leachate is that gaps are not limited to leachate-affected sites. Also, gaps are not guaranteed to occur at the limits of plume extent (Sangrey and Philipson, 1979).

Damp and saturated zones or seeps at waste sites can be distinguished quite easily by remote sensing. The spectral characteristics of seeps are sufficiently distinct so that they are observable over the entire electromagnetic spectrum. Increased soil moisture decreases soil reflectance in the visible and near-infrared wavelengths, allowing for detection of near-surface water. During dry periods, these damp areas can be detected because of anomalous spectral returns caused by moisture stressed vegetation. The spectral anomaly most often used as a leachate indicator is vegetative stress. Vegetative stress can be noted by the use of color infrared scanners (Aronoff and Ross, 1982). The variations caused by vegetative stress can be a result of death or defoliation from toxic effects, reduction in foliages, or temperature changes. However, increased nutrient and moisture levels caused by leachate have been known to enhance vegetative growth. Spectral variations caused by leachate seeps, oils, or other fluid coatings on water can be detected in ultraviolet, thermal, or microwave images.

Aerial images may indicate where contamination occurs, but will not identify the boundary of the contaminated zone because gaps, wetness, and stressed vegetation are not limited to occurring only at the edges of a plume. However, these contaminated zones serve as additional data points that are useful in site investigations. For example, if calculations of flow rate indicate that the maximum extent of contamination should be X yards from a landfill, yet stressed vegetation is readily apparent at X + 300 yards, several explanations are possible. First, the input data to Darcy's Law may be incorrect: hydraulic conductivity may be greater than calculated, the time from the start of contamination may be longer than estimated, the gradient may be steeper, or porosity lower. Second, a preferential migration pathway which allows contamination to move much faster than through the majority of the geologic system may have been overlooked. Third, the constituents of the plume may be more mobile than originally estimated (i.e., dispersion in this system has a large effect on migration rates). Fourth, there may be multiple sources of contamination. Finally, the stress may not be caused by the leachate.

Geohydrologic conditions can also be determined or verified using aerial imagery. Aerial image interpretation can be used to:

- Identify rock and soil types, geomorphological features, the nature of sediments, joint and fault patterns, and outcrop areas
- Approximate stream flow, evapotranspiration, infiltration, and runoff values
- Map topography; streams, seeps, and other surface waters; and vegetation not readily apparent from ground level.

One final use of aerial imagery is in the analysis of historial images to analyze disposal site operations. Historical data can set the time of the start of disposal operations for the calculation of migration rates. The lateral extent and timing of disposal operations can also be estimated using historic air photos (Erb, et al., 1981). Finally, changes in surface topography over time can be determined. This last item was of value during the investigation at Love Canal, where historical aerial photos were used to map old swales and drainage areas since filled and thought to act as areas of preferential migration routes (Kufs, et al., 1981).

EPA has three offices that can provide information on all aspects of aerial photography and photo interpretation:

- Environmental Monitoring Systems Laboratory (EMSL)
  P.O. Box 15027
  Las Vegas, Nevada 89114
- Environmental Photographic Interpretation Center (EPIC)
  P.O. Box 1587
  Vint Hill Farm Station
  Warrenton, Virginia 22186
- National Enforcement Investigation Center (NEIC)
  Building 53, Box 25227
  Denver, Colorado 80225

Aerial imagery is available from a number of sources besides EPA. Government agencies that also perform aerial surveys include:

- Agricultural Stabilization and Conservation Service
- U.S. Geological Survey (USGS)
- U.S. Forest Service
- National Air and Space Administration (NASA)
- National Oceanic and Atmospheric Administration (NOAA)
- National Archives and Record Service
- National Weather Service
- Soil Conservation Service (SCS)
- Commodity Stabilization Service
- U.S. Air Force (USAF)
- Corps of Engineers
- Bureau of Reclamation
- U.S. Coast and Geodetic Survey
- Tennessee Valley Authority (TVA)

Government sources of aerial imagery and related data are listed by state by the USGS (1976). In general, the majority of non-military aerial photography acquired by Federal agencies can be obtained from:

- National Archives and Records Service
  Cartographic Branch
  8 Pennsylvania Avenue, N.W.
  Washington, D.C. 20408
  (for photographs taken prior to 1942)

- EROS Data Center
  U.S. Geological Survey
  Sioux Falls, South Dakota 57198

- Agricultural Stabilization and Conservation Service
  U.S. Department of Agriculture
  205 Parley's Way
  Salt Lake City, Utah 84109

- Soil Conservation Service
  U.S. Department of Agriculture
  Cartographic Section
  6505 Delcrest Road
  Hyattsville, Maryland 20782

Private sources of aerial imagery can be obtained from the American Society of Photogrammetry, 105 North Virginia Avenue, Falls Church, Virginia, 22046.

3.2.3 Geophysical Methods

Over the last 10 years, geophysical equipment, survey techniques, and data processing have been refined to the point where geophysical methods can provide valuable data during hazardous waste site characterizations (Evans, et al., 1982). Geophysical methods can be divided into two groups:

- Surface Surveys -- where instruments used in collecting data are located on the land surface
- Borehole Methods -- which require direct access to the subsurface through boreholes or wells.

Surface surveys provide data that indirectly determine the subsurface geology. Data may be obtained concerning (Fetter, 1980; Glaccum, et al., 1982; Sendlien and Yazicigil, 1981 and 1982):

- Thickness of unconsolidated strata
- Depth to water table
- Fault location and joint patterns
- Location of solution channels
- Location, thickness, and extent of subsurface bodies such as clay or gravel deposits
- Presence, distribution, and location of buried wastes
- Presence and extent of certain types of contaminants and leachate plumes.

Borehole methods provide data on (Fetter, 1980):

- Areas of high porosity and hydraulic conductivity
- Water flow rate and direction
- Subsurface stratigraphy
- Lithology of subsurface units
- Chemical and physical characteristics of water.

Surface geophysical methods appear to have more application to hazardous waste site investigations than borehole methods. Proponents of surface geophysical methods (e.g., Sendlein and Yazicigil, 1981; Pease and James, 1981; Evans, et al., 1982; Glaccum, et al., 1982) correctly point out that monitoring wells may poorly define contaminant plumes for a variety of reasons. The attractiveness of surface geophysical surveys is that they are able to provide a theoretically continuous profile of the subsurface and, therefore, are much better suited to detecting preferential migration routes. Several surface geophysical methods measure changes in the physical parameters of water that may be caused by the presence of leachate or contamination, and thus serve to outline the possible boundaries of a plume. Surface geophysical methods, when used in conjunction with conventional drilling programs, can have application to many waste site investigations (Evans, et al., 1982).

Two things should be mentioned at this point. First, surface geophysical methods are not a panacea. They are sources of valuable information but they do not have application in all situations and are not always the most effective method of obtaining information (USGS, 1977). Second, geophysical methods do not provide direct data on groundwater contamination. Those methods that outline possible plume boundaries measure changes in physical parameters (e.g., specific conductivity) that may be caused by the presence of contaminants in groundwater, but they do not provide indications of contaminant type and concentration (e.g., toluene at X ppb). Thus, groundwater samples from monitoring wells will always be necessary to provide direct data on contaminant type and concentration.

The surface geophysical methods most applicable to plume delineation include:

- Ground penetrating radar
- Electromagnetic conductivity
- Galvanic electrical resistivity
- Seismic methods.

The theory, application, data use, and limitations of each of these methods is discussed below. Borehole geophysical logging methods are also summarized. As with all investigation techniques, a careful definition of the problem and identification of the types of information required should be made prior to selecting any particular geophysical method or suite of methods.

3.2.3.1 Ground Penetrating Radar (GRP)

GPR systems generate electromagnetic radiation that reflect from various objects because of changes in the object's ability to conduct the electromagnetic wave. The reflections are recorded and interpreted to provide a continuous subsurface profile, much like a geologic cross-section, along a traverse path. GPR systems can be used to locate buried drums, detect interfaces between different geologic strata, and identify plumes of high chemical concentration (Pease and James, 1981). The depth of penetration and resolution are affected by site conditions, thus the applicability of GPR must be decided on a site specific basis.

GPR systems consist of four elements:

- Electromagnetic radiation source
- Transmitting antenna
- Receiving antenna
- Data processing and display equipment.

GPR power units and pulse transmitters feed the pulse to the transmitter antenna, which radiates the pulse into the subsurface. Reflected signals are

received, usually by the same antenna, and fed into a receiver electronically isolated from the transmitter. The reflection is amplified and processed to produce a waveform similar to the original but with a longer time base. This new signal can be processed further and displayed. The most common type of display system uses a graphic recorder. Most GPR systems mount all the necessary equipment, except for the antenna, within the survey vehicle (Horton, et al., 1981; Benson and Glaccum, 1979). Antennas may be externally mounted on the same vehicle or on an independent frame capable of being towed.

Prior to running a GPR survey across a site, several preliminary steps are usually completed. Stakes are driven into the ground to outline the survey grid and the traverse path is cleared of brush, tree limbs, tall grasses and weeds, and any other obstacles that may hinder the smooth passage of the survey vehicle. Several preliminary scans are made to calibrate the system with the specific electromagnetic conditions at the site. In addition, the velocity of the signal in the medium must be calculated, along with the dielectric constant of the soil. Once these values are calculated, the pulse repetition frequency is set, along with frequency and energy output. Different settings may be used and several surveys run to obtain maximum resolution and depth of penetration.

Surveys are performed by slowly moving the antenna over the ground surface. The survey vehicle is moved across the grid at a set speed (usually around 5 mph). Precise speed must be maintained such that the areal location of targets can be positioned within a few feet. For more detailed studies, discrete readings can be taken at selected points along a traverse (Benson and Glaccum, 1979). After the survey is run, the data is usually taken from the recorder for further processing. A graphic display can be generated on-site, and examined in the field to provide a general indication of the subsurface conditions (i.e., the general locations of drums) and the effectiveness of the survey. Fence diagrams can be developed from the subsurface profiles.

Usually GPR cannot be used to map anomalous groundwater conditions that may be indicative of contamination. However, GPR can be used very effectively, in certain conditions, to outline the water table and indicate the

direction of groundwater flow. GPR can be used to outline the boundaries of disposal sites, which is important in selecting monitoring well locations. GPR can also map the presence of many soil features including cementation, clay lenses, organic layers, sediment layering, water infiltration zones, depth to bedrock, top of bedrock topography, and fractures and cavities in bedrock.

Because most GPR systems are vehicle mounted, they are limited to sites that allow access to and over the area of concern. Sites with swamp-like conditions would probably not be good candidates for GPR surveys because of access problems and because of the presence of silt, clays, and high water content reducing the depth of penetration. Likewise, sites with large boulders, pits, or high topographic relief would not allow the required access across the site. The type of geologic material present at a site and the presence of water can dramatically influence the effectiveness of GPR. Depths up to 30 feet are commonly attained using GPR and in certain cases up to 60 feet penetration has been reported (Evans, et al., 1982). However, high clay concentrations rapidly attenuate radar waves reducing penetration to less than 3 feet (Benson and Glaccum, 1979). Lower transmission frequencies can yield greater penetration, but resolution is sacrificed (Lord, et al., 1980). The use of GPR in areas with high proportions of silt and clay size particles should be questioned.

3.2.3.2 Electromagnetic Methods (EM)

EM methods are available which measure the electrical conductivity of subsurface materials. The conductivity (reciprocal of resistivity) of a substance is a measure of the ease with which a current passes through it. Conductivity methods have been used to (McNeill, 1980):

- Determine rock lithology and bedrock depth
- Locate and map aggregate and clay deposits
- Map groundwater extent and salinity
- Detect pollutant plumes
- Locate geothermal areas.

The theory behind using EM fields to induce currents in conductive media is well established. A magnetic current is created by passing an alternating current through a wire loop. When the wire loop is in close proximity to conductors (i.e., earth materials), a current is induced in the conductor. The strength of the current flowing through the conductor is dependent upon its conductivity. Since any current generates a magnetic field, the induced current produces a secondary magnetic field with the same frequency as the primary field, but of reduced intensity. The secondary field induces a current in another wire loop, the receiver. By measuring the reduction in voltage between the primary and secondary fields, the conductivity (resistivity) of the material can be determined.

The ability of soils and rocks to conduct electrical currents depends primarily on the amount, salinity, and distribution of water (USGS, 1977). Saturated soil and rock can be considered insulator particles immersed in a conductive fluid. The total resistivity of the system depends on the conductive fluid and the impedence placed on current flow by the insulators (McNeill, 1980). Minerals in sand and silt fractions of soils are generally excellent insulators. Dry clay is also an excellent insulator, but moisture allows certain ions adhering to the surface of clay particles to transmit electrical current decreasing the resistivity. The effect of organic matter on resistivity has not yet been determined. Gases dissolved in water tend to inhibit electrical conductance and thus increase resistivity. Water is by far the most important influence on conductivity (McNeill, 1980).

There are two types of EM survey techniques:

- Profiling -- where lateral changes in conductivity along a set depth from the surface are measured
- Sounding -- where vertical changes in conductivity are measured.

In profiling, depth of exploration is set by separating the primary (generator) and secondary (receiver) coils. A traverse is surveyed and conductivity at the set depth of exploration can be recorded continuously or can be measured at discrete points. Contours of equal conductivity are

plotted. The location and shape of subsurface features with anomalous conductivity values can be described, but only at the depth of exploration. Since profile surveys can be run almost as fast as a survey team can walk, data from several surveys along the same traverse but with varying depths of exploration can be evaluated to provide a three dimensional profile of conductivity anomalies.

In sounding surveys, vertical changes in conductivity are measured by adjusting electrode spacing and measuring changes in vertical conductivity with depth. Using available EM equipment for sounding is inconvenient because most EM setups have set electrode spacings. Sounding surveys are generally performed using galvanic electrical resistivity equipment.

In order to map contaminant plumes, a contrast in conductivity must exist between the contaminant plume and local geohydrologic background values. Leachates with high total dissolved solids usually have high conductivity, but an uncontaminated saturated clay lens may exhibit the same response. In mapping depth to water table and groundwater flow direction, the great change in conductivity between saturated and unsaturated materials is usually sufficient to allow the water table to be mapped.

Limitations of EM conductivity include (Pease and James, 1981):

- Ability to detect non-conductive pollutants is limited
- Ability to detect plumes is limited if there is not a sharp contrast between plume and natural groundwater
- Lateral variations in stratigraphy complicate interpretation
- Buried conductive objects may result in anomalous readings.

Horizontal profiling surveys using EM conductivity and performed under suitable conditions can rapidly trace the outlines of contaminant plumes up to 200 feet deep. Vertical sounding can be performed using EM conductivity, but is usually performed using Galvanic electrical resistivity.

3.2.3.3 Galvanic Electrical Resistivity

Galvanic electrical resistivity methods are among the most widely used methods in groundwater studies (Yazicigil and Sendlein, 1982; Fetter, 1980). Resistivity surveys involve applying a direct electrical current into the subsurface and measuring voltage passing between electrodes. In applying the current, four electrodes are used--two to generate the current (a positive and negative electrode) and two potential electrodes to measure the resultant current flow through the earth materials. By knowing the current flowing through the ground and the voltage between electrodes, the apparent resistivity of earth materials between the electrodes can be calculated (difference between current applied and current measured). The same properties affecting conductivity also affect resistivity surveys and they may be used in the same manner to detect contaminant plumes and provide data on subsurface geology.

There are three electrode arrays commonly used in direct electrical resistivity surveys (Fetter, 1980; Lord, et al., 1981; Raghunath, 1982):

- Wenner Array -- where the positive and negative electrodes and the potential electrodes are spaced evenly apart
- Schlumberger Array -- where the potential measuring electrodes are spaced close together and the current electrodes are spaced far apart
- Dipole-dipole Array -- where the potential and current electrode pairs are spaced a mutual distance (x) apart and the pairs are an integral multiple of x from each other. This array is usually limited to very deep surveys.

Both sounding and profiling can be performed using electrical resistivity equipment. In sounding, the spacing between the potential and current electrodes is increased. This causes the current to move deeper into the subsurface and the apparent resistivity measured incorporates the resistivity of the deeper material. Either the Wenner or Schlumberger Array may be used; but the Schlumberger Array is usually much more convenient because the two inner electrodes are spread only occasionally (Fetter, 1980). Sounding begins with electrodes positioned close to each other. After a reading is made, the

electrodes are positioned farther apart and new readings made. A plot of calculated apparent resistivities taken at different electrode spacings, but centered on a single location, is termed a sounding curve. This curve reflects the changes in electrical characteristics with depth. The curve is generally interpreted as a series of horizontal subsurface layers each with a specific resistivity (Urish, 1983).

In horizontal profiling, electrode spacing remains constant, but the electrodes themselves are moved in a grid pattern over the land surface. The Wenner Array is used most often in horizontal profiling. If subsurface conditions are uniform, apparent resistivity should not change across the study area. Any changes can be interpreted as being caused by subsurface heterogeneity (e.g., clay lenses in sands) or by contaminated groundwater. In setting up the grid pattern, the spacing between stations can conveniently be equal to, or an integral multiple of, the spacing between electrodes. This facilitates movement of the array. A decision must be made as to the effort involved in increasing the number of stations and the risk of bypassing contaminant plumes if stations are farther apart.

Direct electrical resistivity methods, while not as mobile as EM methods, allow any depth (up to limits of the equipment) to be explored. However, there are limits to use including (Pease and James, 1981; Urish, 1983):

- Ability to detect non-conductive pollutantsis limited
- Ability to detect plumes is limited if sharp resistivity contrasts do not exist
- Lateral variations in stratigraphy complicate interpretation
- Equipment range (dependent upon electrode spacing, power output and subsurface resistivity) is site specific
- Rocks, trees, buildings, and other obstacles may restrict grid patterns
- Electrical interferences, power lines and buried cables, in general, must be kept at least one electrode spacing from the grid.

Nevertheless, direct electrical resistivity profiling is a relatively fast and inexpensive method of collecting subsurface data and may be used to outline plume boundaries prior to monitoring well installation. The advantage of direct electrical resistivity over EM methods is in the ability to more precisely set exploration depth.

3.2.3.4 Seismic Refraction Surveys

In a seismic refraction survey, shock waves generated by small explosive charges or by heavy blows on a metal plate travel downward through the earth and are refracted back to the surface from the interfaces between different layers. The refracted waves are picked up at various points on the ground surface by geophones and recorded. By knowing the arrival times of different waves at set distances from the energy source, the velocity of propagation of the waves through each rock layer can be calculated. Particular rocks under specific conditions have characteristic velocities, thus subsurface profiles can be developed. Depths to several hundred feet can be probed using seismic refraction techniques (Ragnuhath, 1982).

The most common use of seismic methods is in mapping bedrock surface. Buried valley aquifers, for instance, have been mapped in this way. Water table depth can usually be determined using seismic methods, but only if the water table lies in unconsolidated deposits. Furthermore, thin layers of saturated sediments lying above bedrock may not be detected nor will layers underlying a higher velocity layer. In certain cases, lateral lithologic changes in aquifers can be mapped. However, large amounts of seismic data and correlation with borehole data is usually required. Vertical stratigraphic profiles are more easily developed than horizontal profiles using seismic methods.

While seismic refraction cannot delineate plume boundaries, determining the depth to groundwater (in alluvium) and fault locations are two major uses in groundwater studies. The great depth of penetration of this method makes seismic refraction useful in areas where other geophysical methods are not applicable.

3.2.3.5 Geophysical Well Logging

Geophysical well logs are an important tool in defining the subsurface environment, and often provide the only practical measurement of undisturbed subsurface sediments. Geophysical logs are useful in determining (USGS, 1977):

- Lithology
- Geometry
- Resistivity
- Bulk density
- Porosity
- Hydraulic conductivity
- Moisture content
- Specific yield of water-bearing rocks
- Chemical and physical characteristics of water.

Borehole logging complements drillers and geologists logs, and soil and rock samples. Cased wells can often be logged through the casing. Borehole logs correlated with surface geophysical methods usually increase the value of data obtained from both sources (USGS, 1977). There are many logging methods in use today, several of which are described below. Geophysical logs are usually run as pen and ink strip charts. In general, borehole systems consist of a probe which is lowered into the borehole on a cable containing powerlines to the probe, transmitter cables to the recorder, and a support cable. The probe contains the required electronics, energy or nuclear sources, and detectors. Types of borehole logs are summarized below.

Caliper Logs -- Caliper logs measure the diameter of uncased boreholes in bedrock units. In plume delineation studies, they can be used to indicate solution cavities, bedding planes, faults and joints, and other preferential routes of water movement.

Temperature Logs -- A temperature log is a continuous vertical record of fluid temperature in the borehole. Temperature logs are usually the first log

run on a borehole to minimize mixing effects. The assumption implicit in temperature logging is that fluid is in equilibrium with the geologic materials. Temperature logs are useful in locating points of groundwater entrance or loss from the borehole. Temperature logs are also important in correcting resistivity and spontaneous potential logs in deeper wells (USGS, 1977).

Single-Point Resistance -- Single-point resistance is one of several methods for measuring electrical resistance. A single electrode is lowered into the borehole on an insulated cable while the second electrode remains on the surface. As the electrode is lowered into the borehole, the resistance of the material from land surface is measured (Fetter, 1980). The single-point resistance method has a very limited depth of investigation and does not measure the resistance of specific formations.

Resistivity -- Resistivity can be measured by spaced electrodes of various configurations. The short-normal configuration measures resistivity of the zone closest to the borehole. The long-normal method measures resistivity farther from the borehole. Lateral configurations can be spaced to measure set distances from the borehole. However, the wider the spacing the less likely that thin beds of different material may be found. Resistivity is an important indication of stratigraphy, lithology and water quality.

Spontaneous Potential -- Spontaneous potential (SP) logs measure the natural electric potential that develops from contact between the formation and borehole fluids. SP logs are used chiefly for correlating geologic units, determining bed thickness, calculating formation water quality, and separating porous and non-porous rocks such as shale-sandstone and shale-carbonate sequences (USGS, 1977). SP logs consist of a surface electrode and a borehole electrode connected to a voltmeter to measure potential.

Nuclear Logging -- Nuclear logging can be done in cased or uncased holes. Measurements can be made of either natural radiation or of the ability of subsurface materials to attenuate induced radiation. Natural-gamma logs are

records of natural gamma radiation emitted by all rocks. They are used primarily to identify lithology and correlate stratigraphy, especially in detrital sediments. Neutron logging utilizes a probe with a radioactive source of neutrons and a detector. Neutron logs respond to water. Below the water table they measure porosity, while above they measure moisture content. Gamma radiation logs measure the absorption or scattering of cobalt 60. Absorption is proportional to bulk density of earth materials and is an indirect reading of porosity.

Table 3-9 summarizes the applications of various borehole logging techniques.

3.3 Direct Data

Direct data includes any information which can be applied directly and precisely to delineating a leachate plume. Direct data can be qualitative (e.g., observing a leachate seep is direct qualitative evidence of groundwater contamination at a point) or quantitative (e.g., chemical analysis results). The three primary means of collecting direct data include:

- On-site and near-site inspections
- Hydrologic testing
- Sampling.

These activities should be undertaken at all sites prior to beginning aquifer restoration design and implementation activities.

On-site inspections serve three purposes. First, the accuracy of the background data collected prior to the site visit must be verified. Second, observations that only a trained investigator can make at a site must be compiled. Third, a decision must be made on what follow-up work should be performed at the site and how and when this work should be accomplished.

TABLE 3-9
SUMMARY OF BOREHOLE LOG APPLICATIONS (KEYS AND MacCARY, 1971)

Information Required	Applicable Logging Techniques
Lithologic and stratigraphic correlation of aquifers and associated rocks	Electric, sonic, or caliper logs made in open holes. Nuclear logs made in open or cased holes.
Total porosity and bulk density	Calibrated sonic logs in open holes; calibrated neutron or gamma-gamma logs in open or cased holes.
Effective porosity and true resistivity	Calibrated long-normal resistivity logs.
Clay and shale content	Gamma logs and resistivity log.
Secondary porosity--fractures, solution openings	Caliper, sonic, or borehole television logs.
Specific yield of unconfined aquifers	Calibrated neutron logs.
Location of water levels and saturated zones	Electric, temperature or fluid conductivity in open hole or inside casing. Neutron or gamma-gamma logs in open hole or outside casing.
Moisture content	Calibrated neutron logs.
Infiltration	Time-interval nuetron logs under special circumstances or radioactive tracers.
Dispersion, dilution, and movement of waste	Fluid conductivity and temperature logs and gamma logs for some radioactive wastes.
Source and movement of water in a well	Injectivity profile. Flowmeter or tracer logging during pumping or injection. Temperature logs.
Chemical and physical characteristics of water including salinity, temperature, density, and viscosity	Calibrated fluid conductivity and temperature in the well. Neutron chloride logging outside casing. Multielectrode resistivity.

TABLE 3-9 (continued)

Information Required	Applicable Logging Techniques
Construction of existing wells; diameter and position of casing, perforations, and screens	Gamma-gamma, caliper, collar, perforation locator, and borehole television.
Guide to screen setting	All logs providing data on the lithology, water-bearing characteristics, and correlation and thickness of aquifers.
Cementing	Caliper, temperature, or gamma-gamma. Acoustic for cement bond.
Casing corrosion	Under some conditions caliper or collar locator.
Casing leaks or plugged screen	Tracer and flowmeter.

Table 3-10 highlights the usefulness of selected site inspection observations to plume delineation. General site inspection procedures have been described in several sources including Sisk (1981), JRB (1980) and numerous shorter references which will not be detailed here. The following two sections describe the roles of hydrologic testing and sampling in plume delineation.

3.3.1 Hydrologic Testing

Improving the precision of plume boundary calculations generally requires hydrologic tests of aquifer properties. However, use of aquifer testing results is secondary to designing plume migration control measures such as pumping systems. The most important aquifer properties to determine for plume control technology design are:

- Hydraulic conductivity (K) -- a coefficient of proportionality that describes the rate at which water, of a prevailing density and viscosity, can move through a porous medium

TABLE 3-10
SIGNIFICANCE OF SELECTED ON-SITE OBSERVATIONS TO PLUME DELINEATION

Site Feature	Observation	Significance to Plume Delineation
Facility Information	Locations of waste transfer, treatment, storage, or disposal activities	Possible sources of contamination
	Facility design, construction, and operation	Indicative of facility's ability to control releases
	Locations of water supply or monitoring wells	Possible future sampling points
	Settling cracks in foundations or basement seepage	Possible high water table; possible preferential routes of leachate migration (backfill for foundations)
	Waste types, volumes, forms, and modes of disposal	Indicative of the possibility, mobility, size, and hazardousness of the leachate plume
Evidence of Contamination	Spills	Possible source of contamination
	Dead, sparse, or stunted vegetation, and barren areas	Possible effects of near-surface contaminated groundwater
	Discolored, disturbed, or odorous soil	Possible source or result of contaminant release
	Leachate seep	Presence of contaminated groundwater
Groundwater Flow	Springs	Intersection of water table with land surface; flow rate related to water table gradient and hydraulic conductivity

(Continued)

TABLE 3-10 (Continued)

Site Feature	Observation	Significance to Plume Delineation
Groundwater Flow (continued)	Depth to water in wells	Elevation and gradients of the water table
	Land surface topography	Possible directions of flow in unconfined aquifer
	Variations in stream discharge through site	Possible indication of whether streams recharge groundwater (termed losing or influent streams) or whether groundwater recharges streams (termed gaining or effluent steams)
	Locations, discharges, and schedules of pumping wells	Possible effects of groundwater withdrawal on plume movement; possible future sampling points
Surface Water Flow	Surface water elevations	Can indicate directions of groundwater and plume flow
	Locations	Possible locations of seeps; possible sampling points; relation of site to discharge and recharge areas in drainage basin
	Chemical quality and evidence of contamination	Possible indication of subsurface leachate discharge
	Drainage patterns	Possibly indicative of bedrock structure and controls on groundwater and contaminant flow (Howard, 1967)

TABLE 3-10 (Continued)

Site Feature	Observation	Significance to Plume Delineation
Surface Water Flow (continued)	Drainage density	Possibly indicative of infiltration capacity of soil and leachate generation potential
Geomorphology	Slopes	Possibly indicative of infiltration capacity and leachate generation potential; possibly indicative of recharge and discharge areas
	Sinkholes/karst topography	Usually indicative of solution channel control of groundwater and contaminant flow
	Floodplains, meander scars, and other stream features	Possibly indicative of buried stream channels and other preferential routes of leachate migration
	Dunes	Indicative of unconsolidated sand deposits of high permeability
	Wetlands	Near-surface water table; many seeps likely
	Tidal flats	Near-surface water table; possible changes in groundwater flow directions because of the influences of tides
	Glacial deposits	Possibly indicative of highly stratified materials of varying permeability; clay deposits may be fractured
Vegetation	Type of plants (e.g., phreatophytes such as willow, ash, and cottonwood)	Possibly indicative of general availability of water, depth to water, moisture retention, and other hydrologic properties

TABLE 3-10 (continued)

Site Feature	Observation	Significance to Plume Delineation
Vegetation (continued)	Distribution of plant types	Possibly indicative of soil or bedrock types and structures, and site hydrogeology
	Stressed, dead, or absent vegetation	Possibly indicative of near-surface contamination
Surface material	Texture (grain size mix), sorting, angularity, organic content and other properties	Indicative of the hydraulic conductivity and porosity of the material, and possibly the rate of groundwater and leachate movement
	Munsell color	Possibly indicative of moisture conditions and leachate generation potential
	Soil structure	Indicative of infiltration capacity and leachate generation potential
	Classification and distribution	Possibly indicative of bedrock conditions and groundwater movement routes
	Bedrock outcrops (i.e., attitude of bedding, joints, faults and other discontinuities; rock types)	Possibly indicative of rates of groundwater and contaminant movement

- Transmissivity (T) -- the rate at which water is transmitted through a unit saturated thickness (m) of an aquifer or confining bed under a unit hydraulic gradient

- Storativity (S) -- the volume of water an aquifer releases from or takes into storage per unit surface area of the aquifer per unit change in head.

Storativity must be determined in the field. Hydraulic conductivity can either be determined in the laboratory or be derived in the field. Estimates of T can be made in the field or using the relationship:

$$T = Km$$

where m is the saturated thickness of the aquifer. The following sections describe some of the different types of aquifer tests for determining K, T, and S.

3.3.1.1 Laboratory Tests

Many methods are available to determine hydraulic conductivity in the laboratory. In all cases a sample of the earth material must be collected in the field and returned to the laboratory for testing. The typical method for collecting samples of unconsolidated material is with a tube sampler (e.g., Shelby tubes, split spoons) and for collecting rock samples is core drilling. Numerous references are available from the American Society for Testing and Materials which describe the techniques and tools available for obtaining samples so that disturbance to the sample is minimized. Samples collected in this manner are commonly labeled "undisturbed samples." In reality, "undisturbed samples" is a misnomer because certain sample properties (e.g., porosity, sorting) will change to some extent, regardless of the care taken in sample extraction. Furthermore, gross aquifer characteristics (e.g., fractures, bedding planes, cavities) are seldom represented intact and in the proper proportion to the rest of the sample. As a result, laboratory and field studies of the same geologic unit can, under certain conditions, produce significantly different estimates for aquifer properties.

Two basic methods are available for determining hydraulic conductivities in the laboratory--the constant head method and the falling head method. For the constant head method, inflow fluid level is maintained at a constant head and outflow rate is measured as a function of time. Hydraulic conductivity is then calculated using:

$$K = QL_s/h_sA_s$$

where:

K = hydraulic conductivity (ft/day)
Q = outflow rate (ft^3/day)
L_s = length of sample (ft)
h_s = fluid head difference across sample (ft)
A_s = cross-sectional area of sample (ft^2)

For the falling head method, the head of the inflowing fluid is allowed to decrease over time, while the fluid head at the outflow is held constant. Assuming a large standpipe, hydraulic conductivity for this method can be calculated using:

$$K = (2.3A_pL_s/A_st) \log (h_i/h_e)$$

where:

K = hydraulic conductivity (ft/day)
A_p = cross-sectional area of standpipe (ft^2)
L_s = length of sample (ft)
A_s = cross-sectional area of sample (ft^2)
t = time for head level decline (day)
h_i = initial head level (ft)
h_e = final head level (ft)

Several other methods are available for determining hydraulic conductivities but all are generally based in part on these two simple methods.

Table 3-11 summarizes the more common methods utilized to obtain hydraulic conductivities and their applications. In practice, the test performed on a sample sent to a laboratory will probably be dependent upon the instrumentation available, not on the applicability of the test to the soil type. Roberts and Nichols (1981) provide additional information on methods for determining hydraulic conductivity.

When a laboratory hydraulic conductivity test is performed, there are numerous potential sources for error in the estimates. Table 3-12 lists some of these potential sources and the effect they have on the calculated hydraulic conductivity. If a laboratory measured hydraulic conductivity does not appear to be consistent with field observations, a second test should be performed keeping in mind the areas where potential error sources can be introduced. Laboratory measurements of hydraulic conductivity can be many orders of magnitude lower than field tests of a sample for a variety of reasons. However, most comparisons are within an order of magnitude (Olson and Daniel, 1979).

3.3.1.2 Field Tests

The determination of hydraulic conductivity by a well test is highly dependent on the design, construction, and development of the well to be tested. Tests performed on improperly constructed wells will reflect conditions caused by well construction rather than those of the earth material. In some cases, the well to be tested will already be in place. For these instances, a well log and as-built drawing should be obtained for the well so that elements that may affect the test can be identified. Newly drilled wells should be designed, installed, and developed properly to ensure accurate test results (refer to Chapter 5).

Methods available for determining an aquifer's properties based on an in-situ field test can be divided into single well tests and multiple well tests (i.e., a test well with observation wells). All of these tests either require the removal of water from the well or the injection of water into the well.

TABLE 3-11
LABORATORY METHODS FOR DETERMINING HYDRAULIC CONDUCTIVITIES

Method	Application	Mathematics
Constant head	• Best for samples with high conductivities (i.e., coarse grained) • Can be used with fine grained samples but test times may be prohibitively long	$K = QL_s/h_sA_s$
Falling head	• Any soil type • Best suited to materials having a low permeability	$K = (2.3\ A_pL_s/A_st)\ \log\ (h_i/h_e)$
Modified constant head	• Any soil type • Best suited for fine-grained soils	$K = QL_s/h_sA_s$
Triaxial cell	• Any soil type • Especially suited for fine-grained, compacted cohesive soils in which full fluid saturation is difficult to achieve	$K = QL_s/h_sA_s$
Pressure-chamber	• Any soil type • Remolded samples	$K = (2.3\ A_pL_s/A_st)\ \log\ (h_i/h_e)$

TABLE 3-12
EFFECTS OF VARIOUS TYPES OF ERRORS ON LABORATORY MEASURED VALUES OF HYDRAULIC CONDUCTIVITY (EPA, 1982d)

Source of Error	Measured K
Voids formed in sample preparation	High
Smear zone formed during trimming	Low
Use of distilled water as a permeant	Low
Air in sample	Low
Growth of microorganisms	Low
Use of excessive hydraulic gradient	Low or High
Use of temperatures other than the test temperature	Varies
Ignoring volume change caused by stress change (confining pressure not used)	High
Performing laboratory rather than in situ tests	Usually low
Impedance caused by the test apparatus, including the resistance of the screen or porous stone used to support the sample	Low

Single well tests can be utilized to measure hydraulic conductivities when only one well is present at a site or when other wells are located too far away to make them usable for observation purposes. Most of these methods, termed slug tests, require the instantaneous removal or addition of water to the well. These instantaneous changes in head levels can be caused by pumping or bailing water out of the well; by adding water into the casing very rapidly; or by lowering a plug with known water displacement volume into the well, waiting for the water level to stabilize, and then removing the plug quickly. In all slug test methods, the recovery of head level to original levels is measured over time. A single well pump test, on the other hand,

does not require the instantaneous change in head within the well which permits a slower extraction of water. Pumping, drawdown, and recovery rates must be measured during the pump test.

Table 3-13 presents some of the commonly accepted single well tests for determining hydraulic conductivity. This table also presents the applicability of the test to various aquifer conditions (i.e., confined vs. unconfined aquifers, expected hydraulic conductivity) and possible sources of error when the test is performed. Detailed descriptions of the methods used in performing the tests or calculating the results are available in the references cited.

Multiple well tests require either the constant withdrawal or injection of water into a test well and the measurement of head level changes in nearby observation wells. Multiple well tests measure aquifer properties over a larger test section than either laboratory tests or single well tests, and are considered to be far more accurate in characterizing aquifer properties. These tests can also be used to determine the presence of hydraulic barriers and boundaries with the proper placement of observation wells.

Performing these tests can be expensive because special well drilling may have to be performed (i.e., test well with observation wells), and data acquisition and interpretation is more intensive. Furthermore, large volumes of contaminated groundwater may be pumped during the test requiring appropriate procedures for treatment and disposal. However, the data generated from such tests can save time and money because of the better understanding one can gain of possible plume paths. This understanding can maximize resource use in terms of proper planning of monitoring and remedial actions.

Table 3-14 summarizes applicability, procedures, and calculation method for major classes of multiple well tests. The Jacob method is generally much easier to use and obtain results from than the Theis method but requires meeting certain additional test conditions. USGS (1977) provides a listing of test methods that have been developed for specific site conditions and subject emphases, and can be used as a reference guide to aid in selecting an appro-

TABLE 3-13
SINGLE WELL TESTS

Test Method	Applicability	Procedure	Calculation	Error Sources
Conventional Slug Test: Confined Aquifer (Lohman, 1972)	• Moderately permeable formations • Confined aquifers • Entire aquifer open to well • Aquifer uniform in all directions	• Slug removal of water • Measure recovery rate to 85% initial head	• Plot of h/h_0 vs t on semilog paper • Superimpose on type curve • Calculation of transmissivity and storativity	• Entire thickness of aquifer not open to well • Flow to well not radial • Measurement errors in recovery rates and times • Improper type curve matching
Pressurized Slug Test (Bredehoeft and Papadopulos, 1980)	• Low to extremely low hydraulic conductivities (silts, clays, shales) • Confined aquifers	• Sudden pressurization of packed-off area within well with water • Pressure decay rate monitored	• Plot of pressure decay vs time • Superimpose plots on type curve • Calculation of transmissivity	• Hydraulic properties throughout aquifer are same as packed-off section • Measurement errors in recovery rates and times
Conventional Slug Test: Unconfined Aquifer (Bower and Rice, 1976)	• Moderately permeable formations • Unconfined aquifers • Fully or partially penetrating well • Measured value principally in horizontal direction	• Sudden change in head level by removal of submersed weight • Measurement of water level recovery rates	• Analytical equation based on measured parameters • Calculation of hydraulic conductivity and effective radius of well	• Measurement errors in recovery rates and times • Significant flow of water from above the test zone
Piezometer Test (Hvorslev, 1951)	• Any formation • Unconfined aquifers • Fully or partially penetrating wells	• Sudden change in head levels caused by removal or injection of water • Measurement of recovery rate over time	• Plot of head level change vs time • Analytical equation based on collected data • Calculation of hydraulic conductivity	• Measurement errors in recovery rates and times
Pumping Test	• Any formation • Confined and unconfined aquifers	• Removal of water over time by pumping • Measurement of drawdown, pumping, and recovery rates	• Semilog plot of drawdown vs time • Analytical equation based on data collected • Calculations of transmissivity and storage	• Measurement errors in drawdown rates, recovery rates, and times • Recovery storage not equal to discharge storage

TABLE 3-14
MULTIPLE-WELL PUMP TESTS

Test Method	Applicability	Procedure	Calculation Method
Theis Method (Theis, 1935)	• Any formation type; although low hydraulic conductivity formations are difficult to pump at constant rates • Confined and unconfined aquifers • Well fully penetrates aquifer • Requires the use of one or more observation wells	• Pump well at constant rate • Measure drawdown verses time in observation wells • Measure discharge rate	• Log-log plot of drawdown versus distance to observation well divided by time • Match plot with type curve
Jacob Method (Jacob, 1950)	• Any formation (same as Theis) • Confined and unconfined aquifer • Fully penetrating well • Requires the use of one or more observation wells	• Pump well at constant rate • Measure drawdown versus time in observation wells or recovery rates versus time in observation wells • Measure discharge rate	• Semilog plots of: - Drawdown versus time - Drawdown versus observation well distances from test wells - Recovery versus time • Calculate transmissivity and storativity with appropriate equations

priate test when site conditions are known. Other discussions of the design and interpretation of multiple well pump tests are provided in Powers (1981), Lohman (1972), and Stallman (1971).

3.3.2 Sampling

Direct sampling of groundwater (and possibly other media) will always be required during studies concerning delineation and management of leachate plumes. Sampling provides data on constituent type and concentration in groundwater. Also, sampling is required to verify or identify inaccurate calculations or assumptions made concerning plume extent, to identify unexpected problems or factors affecting plume dynamics, and to develop and monitor remedial actions. Procedures involved in completing a sampling program include:

- Planning -- Identification of data needs and development of proper system design and necessary activities to collect the required data
- Implementation -- Completion of activities necessary to collect the required data, including well installation and development, purging, sample collection and handling, and laboratory analysis
- Data Evaluation -- Reduction of raw data to assess its reliability and integration into the overall program to satisfy sampling program objectives. Typically, this may include the calculation of concentration gradients and the development of isoconcentration maps.

The following sections discuss each phase involved in a sampling program, along with factors affecting sample validity. Procedures related to groundwater monitoring are discussed where appropriate. Publications providing more details concerning sampling programs include Sisk (1981), Scalf, et al. (1981), USGS (1977), and EPA (1982c).

3.3.2.1 Sampling Program Plan

A sampling program plan should include descriptions of:

- Objectives -- describing the questions to be answered by the sampling program and the data needed to address the questions
- Program Design -- including sample analysis parameters, frequency, QA/QC, and integration of data with other media sampling programs
- System Design -- detailing sample site locations, numbers, depths, and well configuration
- Component Design -- describing well construction materials, screen type and setting, and security measures
- Implementation Procedures -- specifying well installation and development procedures and other field activities relating to sample collection and analysis activities
- Data Evaluation Procedures -- including how data will be integrated with other data to attain sample program objectives.

Clearly defined program objectives should be developed prior to other planning activities. Identification of program objectives is important so that the reason for monitoring is clearly understood. The program can be planned to meet objectives with a minimum amount of time and resources, when all activities are focused. Monitoring program objectives should reflect site conditions and available data, and should be modified as needed as new data is compiled. If objectives are modified during a study, the monitoring program should be reevaluated to keep activities focused on meeting the new objectives. A monitoring program designed to meet one set of objectives may be totally inadequate to accomplish subsequent revisions.

All data necessary to attain objectives should be identified once the objectives are clearly defined. For example, if sampling objectives include determination of the extent of contamination plus migration potential, then data needs would include constituent types and concentrations over time, as well as vertical and horizontal groundwater gradients, vertical and horizontal

concentration gradients, and aquifer characteristics. Data needs should be evaluated as to completeness and used to develop a data acquisition strategy.

The acquisition strategy includes general procedures or components that must be accomplished to collect the necessary data. For example if surface water and groundwater relationships appear to be important, a sampling program addressing only groundwater is inadequate. The applicability of the data acquisition strategy to the situation can be addressed by listing data resulting from each step or component of the system and comparing this to the required data list developed previously. The acquisition strategy, in addition to data requirements and program objectives, will set standards for program, system, and component (well) design.

Program design refers to how the monitoring system will be implemented to meet the stated objectives. Items to be specified include:

- Media to be sampled (e.g., groundwater, surface water)
- Sample types and frequency (e.g., groundwater grab samples at three month intervals)
- Parameters to be measured (e.g., priority pollutants)
- Integration with other media sampling programs
- Activities scheduled (e.g., well installation, development, purging, sample collection and handling, laboratory analyses, and data evaluation)
- QA/QC plans to ensure sample representativeness and validity.

One of the more critical elements of program design is Quality Assurance/ Quality Control (QA/QC). QA/QC procedures are essential to produce data that meet user requirements in terms of completeness, accuracy, representativeness, and comparability. QA/QC enters into each study phase and specific procedures should be specified in the program plan. For example, QA/QC plans for ground-

water sampling and analysis would incorporate the use of prepared samples such as:

- Field Blanks -- All procedures normally followed during sampling are performed but deionized or distilled water is added to the sample bottles instead of groundwater or surface water. This procedure checks for contamination resulting from sampling apparatus that come into contact with the sample.

- Spiked Samples -- Known amounts of a particular constituent are added to blank samples to determine the accuracy of laboratory procedures. Spiked samples can be prepared in the field or at the laboratory.

- QA Samples -- Duplicate or triplicate samples (taking samples from the same source and placing portions into two or more containers) are collected using normal sampling procedures. Samples are identified independently and sent to the same or different laboratories to check for procedure precision.

- Trip Blanks -- Blank or spiked samples are prepared in the laboratory and sealed. These accompany the sample containers from the laboratory, through field sampling and back to the laboratory. These provide a check for interferences and contamination from external sources not related to sampling.

These QA/QC checks provide indications of sample method accuracy and precision, as well as possible cross-contamination resulting from sample collection, handling, or analysis.

QA/QC plans should be a part of any sampling program plan. Several items of concern for groundwater sampling program components include:

- Sampling Program Design -- Does the design result in all necessary data? Are schedules planned to take advantage of common activities?

- Well Installation -- Will the installation procedures avoid cross contamination between aquifers in multiaquifer systems and between wells?

- Sample Collection -- Will the planned procedure prevent cross contamination between samples and from sampling equipment? Are there adequate procedures for purging and handling purge water, calibrating field measurement equipment, and decontaminating equipment?

- Sample Handling -- Will there be proper procedures for cleaning and preparing bottles, and for sample preservation and shipment?
- Analysis -- Is there a complete and adequate laboratory QA/QC plan?

All these factors should be considered when developing or evaluating a QA/QC plan.

System design refers to the relationships between the individual components of the program. For example, in groundwater monitoring studies system design would specify:

- Number of wells
- Well locations
- Screen settings (depth)
- Well configurations.

Well configuration refers to the relation of a well to adjacent wells and to individual aquifers. Figure 3-2 illustrates five types of well configurations that have been used for groundwater monitoring. Table 3-15 lists some of the advantages and disadvantages of each configuration.

Component design refers to the proposed construction of individual wells in the monitoring systems. Typical well design specifications are listed in Table 3-16. Although monitoring well design differs significantly from the design of wells for plume extraction, the procedures used for well installation and development are similar. These procedures are described in Chapter 5.

After the program and individual components are designed, three activities can be developed to carry out the program. These implementation procedures should be detailed and in a "cookbook" fashion such that sample

FIGURE 3-2.
WELL CONFIGURATIONS USED FOR GROUNDWATER MONITORING
(AFTER MASLANSKY, 1982)

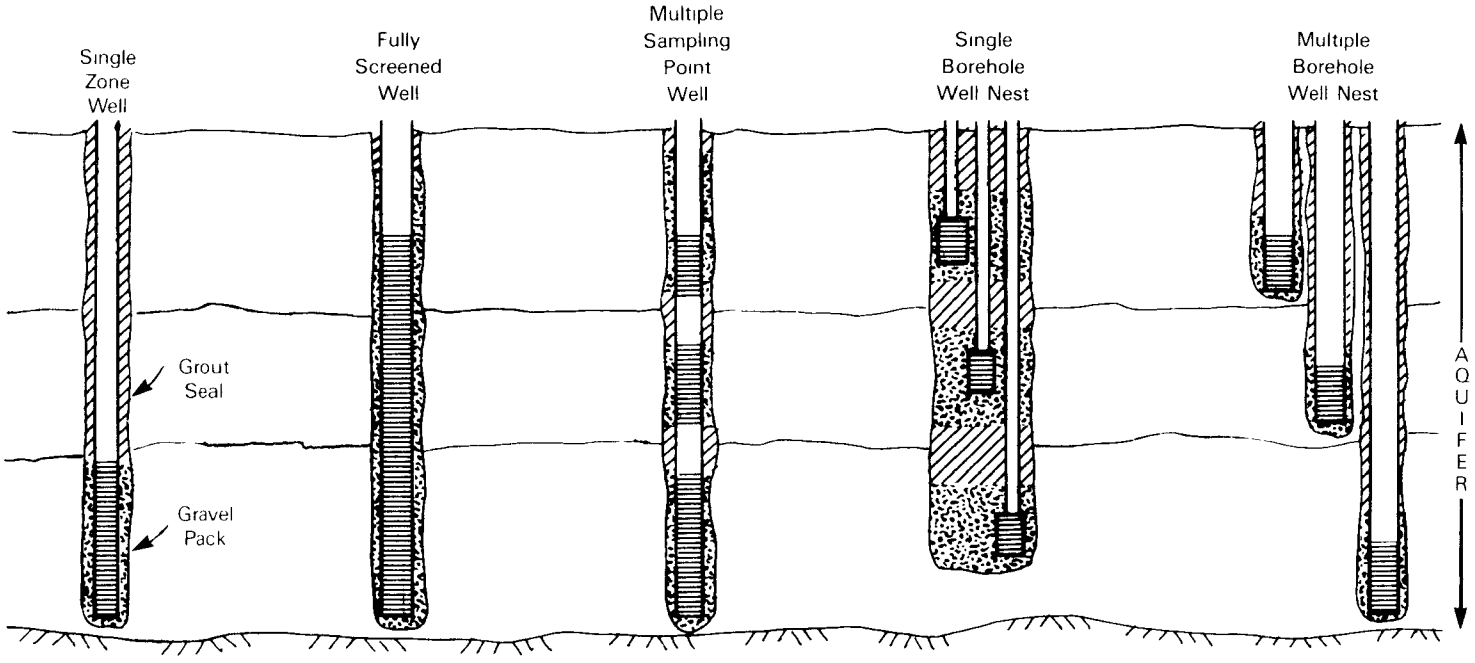


TABLE 3-15
ADVANTAGES AND DISADVANTAGES OF VARIOUS TYPES OF MONITORING WELL CONFIGURATIONS

Well Configuration	Advantages	Disadvantages
Single Zone Well	• Relatively simple to install • Can be installed by a variety of methods • Can provide discrete samples from a precise interval thus aiding data interpretation • Easy to prevent interaquifer contamination if designed and installed properly	• Vertical distribution of contaminants or hydraulic gradients cannot be determined • Many wells are needed to delineate plume increasing costs and the time required to install and sample the system
Fully Screened Well	• Relatively simple to install • Can be installed by a variety of methods • Can provide composite samples of large intervals thus reducing the number of samples • Produces relatively higher yields and thus are more amenable to pump testing	• Highly contaminated waters may be diluted by less contaminated waters during sampling biasing results • Vertical distributions of contaminants and hydaulic gradients cannot be determined • Vertical migration of contaminants may occur over screened interval spreading contaminants to clean zones • Impossible to prevent interaquifer mixing if screened over more than one aquifer
Multiple Sampling Point Well	• Can provide information on the vertical distribution of contaminants and hydraulic gradients • Installation is rapid and simple, although construction takes longer then for wells with a single screen • Can be used to obtain composite samples • Fewer wells are needed in a monitoring system thus reducing costs	• Preventing interaquifer contamination is difficult if not impossible • Sampling is complicated, time consuming, and requires specialized equipment • Cost per well is fairly high

TABLE 3-15 (continued)

Well Configuration	Advantages	Disadvantages
Single-Borehole Well Nest	• Provides information on the vertical distribution of contaminants and hydraulic gradients • Preventing interaquifer contamination is generally not difficult • Sampling is not difficult but may require specialized equipment depending on well diameters	• Number of suitable installation methods is restricted • Improper construction can reduce effectiveness and cause vertical movement of contaminants • Installation is fairly time consuming but not difficult • Cost per nest is fairly high although cost per well is low
Multiple-Borehole Well Nest	• Provides information on the vertical distribution of contaminants and hydraulic gradients • Installation simple by a variety of methods • Preventing aquifer cross contamination is not difficult • Sampling is simple and usually does not require specialized equipment	• Installation is fairly time consuming but not difficult • Cost per nest is very high

TABLE 3-16
DESIGN SPECIFICATIONS FOR MONITORING WELLS

Design Item	Example Specification	Significance
Borehole diameter	6-inch outside diameter	Must be large enough to accommodate well and sufficiently thick sand pack
Borehole depth	25 feet below land surface (BLS)	Must accommodate well and sump (if specified)
Well diameter	4-inch	Based on required yields and sampling equipment to be used
Casing length	25 feet	Equal to the total borehole depth plus stickup minus the screen length
Screen length	10-foot (two 5-foot sections, threaded)	Based on required yields and aquifer to be sampled
Screen setting	25 to 20 feet BLS	Based on aquifer to be sampled
Screen openings	10 slot/inch; micromesh	Based on aquifer particle sizes, gradations of sand pack designs, and aquifer to be sampled
Casing type	PVC or high carbon steel	Based on type of contamination expected and costs
Screen type	PVC or stainless steel (match with casing type)	Based on type of contamination expected and costs
Sump	2-foot	Used as a collection basin for fine particles passing through screen
Joints	Threaded couplings, welded or glued (depending on casing type)	Based on materials and installation procedures to be used

(Continued)

TABLE 3-16 (continued)

Design Item	Example Specification	Significance
Sand pack depth	At least 3 feet over top of screen	Must allow for settlement and an adequate thickness for the seal
Sand pack type	Silica sand; Q-rok #1	Based on particle sizes and gradations of aquifer to be sampled
Bentonite setting and thickness	3 feet over sand pack	Used to separate sand pack from grout
Bentonite type	Sodium bentonite pellets	Less likely to mix with sand pack than fluid grouts
Grout thickness	5-foot minimum	Based on total well depth
Grout type	Portland cement and bentonite mix at a ratio of 20:1	Based on groundwater quantity and quality
Well finish	3-foot stickup	Based on traffic loads and needs for easy location
Protective casing	12-inch oversize casing sunk 3 feet BLS	Used to protect well head from most types of damage
Security	Locking cap and traffic guards	Used to protect well from accidental damage or intentional tampering
Identification	Numbering system with color coding	Used to identify specific well locations especially in well nests

team members can use the plan to complete their activities. The sampling plan should also include a data evaluation section specifying:

- Data resulting from program activities
- Evaluation procedures (including references) for reducing the raw data into a usable form
- Integration of the usable data to achieve the specified objectives.

3.3.2.2 Sample Program Implementation

Implementation of groundwater sampling programs generally involves four major elements--well installation and development, sample collection and direct measurement, sample handling, and sample analysis. Well installation typically consists of the following steps:

- Opening a borehole to the desired depth
- Installing a well constructed in accordance with the component design
- Filling the annulus (i.e., the space between the well and the borehole wall) with a sand pack
- Sealing the sand pack from surface water infiltration by installing a bentonite seal
- Grouting the well in place
- Installing security measures to prevent tampering.

Drilling procedures and well finishing are discussed in Chapter 5.

One very important concern in drilling at hazardous waste sites is to prevent contaminant migration into an uncontaminated aquifer or zone within an aquifer. This is especially important in areas consisting of multiaquifer systems. A drilling program to install wells into an aquifer separated from contaminated groundwater must incorporate methods to seal off the contaminated

zone from the uncontaminated zones. One method for accomplishing this objective consists of:

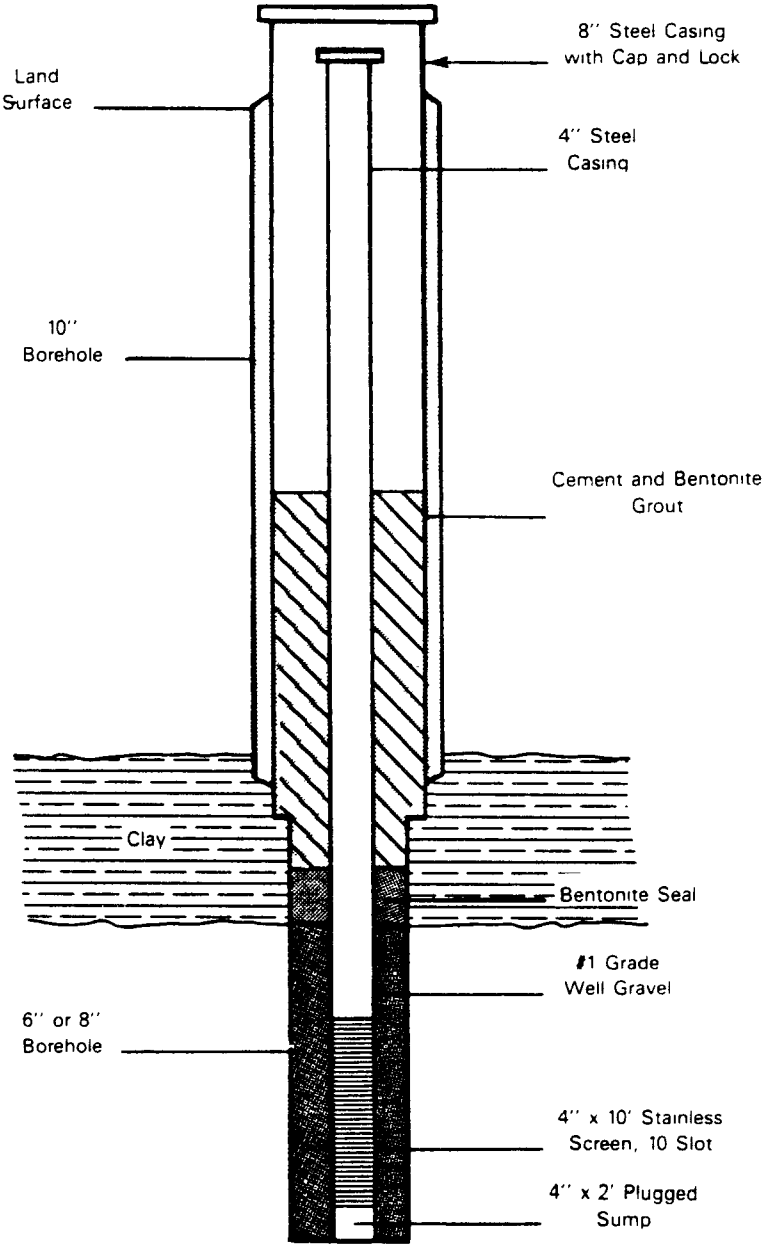
- Boring through the contaminated aquifer and into, but not through, the separating less permeable layer
- Installing an outer casing to seal off the contaminated aquifer
- Drilling through the casing plug and separating layer into the lower aquifer
- Installing the monitoring well as described previously.

A double based well installed using this procedure is illustrated in Figure 3-3. The use of the outer casing and seal prevents contaminants from migrating between aquifers. Drilling equipment should be throughly decontaminated before drilling into the lower uncontaminated zone.

Wells should be designed and constructed to avoid possible interference with sample analysis. In general, well materials made of steel are more durable, subject to less chemical degradation than plastics (e.g., PVC), and cost more. If PVC or other plastic material is to be used, couplings should be threaded and sealed with Teflon tape and not glued. PVC glues contain organic solvents and other constituents which could affect sample results. There is some concern at present as to the effect of PVC well material on groundwater samples in areas of high concentrations of organic contaminants. Studies are currently underway to determine the leaching of contaminants from PVC in these situations. If metals are of concern, high carbon steel or stainless steel wells may adversely affect sampling results for chromium, nickel, iron, and zinc.

Screens should be installed that are compatible with the chemical contaminants anticipated and the aquifer grain size distribution. If information is not available concerning aquifer grain size, 10 slot screens (i.e., screens having slot widths of 0.01 inch) and appropriate sand-pack materials can usually be used with good results (Sisk, 1981).

FIGURE 3-3.
WELL DESIGN FOR PREVENTING INTERAQUIFER CONTAMINATION
(WICKLINE, et al., 1983)



Sample collection includes the following activities:

- Purging -- which involves removing a predetermined volume of water from a well prior to sampling to ensure that "stagnant" water is removed
- Field Measurements -- which involves obtaining values for certain physical parameters at the sample site using direct reading equipment
- Sample Withdrawal -- which involves methods to remove a specific volume of water from a well and introducing this water into specified sample containers.

Groundwater sampling should only be performed after wells have been purged and the groundwater system has returned to equilibrium. The amount of water to be purged is typically specified in terms of multiples of the casing volume (i.e., the volume of standing water in a well). The number of casing volumes to purge is a topic of discussion. EPA recommends three to five casing volumes, while the U.S. Army (USATHAMA) recommends five casing volumes (where "casing" includes the sand pack). The merits of purging between 10 and 20 casing volumes, and in some cases, up to 100 casing volumes have also been presented (Schmidt, 1982). The USGS recommends pumping until certain water quality parameters (e.g., pH, temperature and specific conductance) stabilize. However, the focus of most USGS studies is basin wide as opposed to site specific. Pumping large quantities (i.e., more than 10 to 20 casing volumes) of contaminated groundwater can distort plume boundaries and movement patterns, dilute the actual sample with groundwater from some distance away from the well, and cause significant purge water disposal problems. Reports have noted that organic chemical concentrations changed continuously over several hours of pumping despite stabilization of the USGS indicator parameters within only a few minutes of pumping (Keely and Wolf, 1983). Therefore, unless there is a specific reason for pumping large volumes of water (e.g., time series sampling; Keely, 1982) this practice should be avoided. The EPA recommendation of three to five casing volumes is probably acceptable for most situations.

There is a special case where purging less than three casing volumes is considered adequate. Low-yield monitoring wells, usually of shallow depths,

can be pumped dry with the proper equipment. In cases where recovery is adequate, the well is usually pumped three times, allowing partial recovery between pumping. Where recharge is slow, the well is emptied twice.

The selection of equipment for purging depends on the well to be sampled. Limitations include well diameter (e.g., a 4-inch submersible pump will not fit into a 3-inch well), depth (e.g., centrifugal pumps cannot be used if the water table is more than 25 feet deep), and the volume of water to be purged (e.g., a 1 gpm pump is not appropriate if 200 gallons must be purged).

Equipment decontamination is an important step in preventing cross contamination (i.e., transfer of contaminants from one well to another) and as a health and safety consideration. Pump decontamination is usually easier to accomplish as components are pulled from a well. Distilled water (or other approved water) and nonresidue detergent should be used for decontamination.

Direct measurements provide additional data points for subsequent evaluation. They can also indicate if there are changes in water type sampled. These measurements are taken both before and after well purging, and before and after sampling. Usually, a volume of water is emptied into a Nalgene (or other suitable material) bucket for measurement, although flow-through systems have recently been developed. Precise values, both before and after sampling, usually indicate that external influences have not affected the sample. Direct measurements are obtainable in the field for a variety of parameters; however, the four most often measured are:

- Specific Conductance -- a measure of the water's ability to carry ar electrical current under specific conditions. Ionized salts are measured, giving an indication of the concentration of dissolved solids in the sample and an indication if a different source of water is being sampled.

- Temperature -- important in groundwater sampling because temperature may indicate that different water sources are being sampled. Digital electric probes or two calibrated thermometers can be used to collect data.

- pH -- a measure of the effective hydrogen-ion concentration which can indicate the solution of certain metals and the presence of

contaminants or other water sources. pH can be measured using pH paper but more precisely with a calibrated pH meter.

- Water Level Measurements -- used in subsequent analyses to derive the direction of groundwater flow and calculate groundwater gradients. Water level measurements are typically obtained using either a tape measure with some water level marking device (i.e. wetted tape method), or an electrical probe where an electrical circuit is completed by contact with the water and is registered on a meter. There are a variety of marking methods which can be used with tape measures including marking the tape with chalk or a water soluble marker and noting the depth at which the mark is washed off by water in the well. By holding the tape at a reference point on the well (i.e., marked point on the well casing that has been surveyed), the depth to water is easily determined. Because of the possibility that samples may be affected by the tape marking, the electrical probe method is generally recommended. Water level measurements have limited usefulness unless there is a surveyed reference point from which to measure. Water levels should be taken a minimum of two times or until close agreement with measurements is obtained.

There are a variety of sample withdrawal methods currently available. Some of these methods are listed in Table 3-17 along with their advantages and disadvantages. In most instances, using a Teflon bailer is preferable because of ease of use and decontamination, thus minimizing the potential for cross contamination.

Sample preservation, packaging and storage is a very important step in maintaining sample validity. Preservatives serve to maintain the physical, chemical, and biological or bacteriological integrity of samples from collection to analysis. Preservative methods usually include pH adjustment, chemical addition, and refrigeration. Table 3-18 lists recommendations for sampling containers, preservatives, and holding times.

In monitoring wells, siltation may cause minute quantities of suspended solids to be collected in samples. Introduction of suspended solids into containers with acid preservatives may cause the dissolution of metal ions from the solids because of the pH change. This would increase the concentration of dissolved metals and would indicate that groundwater contains higher than actual concentrations. Therefore, EPA recommends filtration of samples for dissolved solids. Filtration in the field is performed using a vacuum

TABLE 3-17
SELECTED GROUNDWATER SAMPLE WITHDRAWAL METHODS

Method	Advantages	Disadvantages
Bailers (all types)	• Quick and easy to use • Minimizes cross contamination • Useful in almost all situations • Inexpensive	• May become time consuming for very deep wells • Sample exposed to air during withdrawal
Suction Lift Pumps	• Easy to use • Fairly rapid to use	• Maximizes potential outgassing of samples • Can only be used to depths of 30 feet • Check valve must be used to prevent cross contamination
Submersible Pumps	• May be used at any depth • Fairly rapid to use	• Outside power source required • Maximizes potential outgassing • Difficult to prevent cross contamination • May not work in cold weather or silted wells • Limited to wells 2-inches in diameter or over
Gas Lift	• May be used at any depth	• Outside source of compressed gas required • Complex equipment required • Difficult to prevent cross contamination • Maximizes potential outgassing of samples
Positive Displacement Pumps	• Can sample from various depths	• Difficult to prevent cross contamination • Outside air or gas source required

TABLE 3-18
CONTAINERS, PRESERVATION, AND HOLDING TIMES (U.S. ARMY, 1982)

Measurement	Container[a]	Preservative[b]	Maximum Holding Time[c]
Acidity	P	Cool to 4°C	14 days
Alkalinity	P	Cool to 4°C	14 days
Ammonia	P	Cool to 4°C H_2SO_4 to pH<2	28 days
Biochemical oxygen demand	P	Cool to 4°C	48 hours
Biochemical oxygen demand, carbonaceous	P	Cool to 4°C	48 hours
Bromide	P	None required	28 days
Chemical oxygen demand	P	Cool to 4°C H_2SO_4 to pH<2	28 days
Chloride	P	None required	28 days
Chlorine, total residual	P	Determine on site	2 hours
Color	P	Cool to 4°C	48 hours
Cyanide, total and amenable to chlorination	P	Cool to 4°C NaOH to pH>12,[f] 0.008% $Na_2S_2O_3$	14 days
Dissolved oxygen			
Probe	G bottle & top	Determine on site	1 hour
Winkler	G bottle & top	Fix on site	8 hours
Fluoride	P	None required	28 days
Hardness	P	HNO_3 to pH<2	6 months
Hydrogen ion (pH)	P	Determine on site	2 hours
Kjeldahl and organic nitrogen	P	Cool to 4°C, H_2SO_4 to pH<2	28 days
Metals[d]			
Chromium VI	P	Cool to 4°C	48 hours
Mercury	P	HNO_3 to pH<2, 0.05% $K_2Cr_2O_7$	28 days
Metals except above	P	HNO_3 to pH<2	6 months

(continued)

TABLE 3-18 (continued)

Measurement	Container[a]	Preservative[b]	Maximum Holding Time[c]
Nitrate	P	Cool to 4°C	48 hours
Nitrate and nitrite	P	Cool to 4°C, H_2SO_4 to pH<2	28 days
Nitrite	P	Cool to 4°C	48 hours
Oil and grease	G	Cool to 4°C, H_2SO_4 to pH<2	28 days
Organic carbon	G	Cool to 4°C, H_2SO_4 to pH<2	28 days
Organic Compounds[e]			
Extractables (including phthalates, nitrosamines, organochlorine pesticides, PCBs, nitroaromatics, isophorone, polynuclear aromatic hydrocarbons, haloethers, chlorinated hydrocarbons and TCDD)	G, teflon lined cap	Cool to 4°C 0.008% $Na_2S_2O_3$[f]	7 days (until extraction) 30 days (after extraction)
Extractables (phenols)	G, teflon lined cap	Cool to 4°C 0.008% $Na_2S_2O_3$[f]	7 days (until extraction) 30 days (after extraction)
Purgeables (halocarbons, aromatics, acrolein, and acrylonitrile)	G, teflon lined septum	Cool to 4°C, 0.008% $Na_2S_2O_3$[f]	14 days
Orthophosphate	P	Filter on site, Cool to 4°C	48 hours
Pesticides	G, teflon lined cap	Cool to 4°C 0.008% $Na_2S_2O_3$[f]	7 days (until extraction) 30 days (after extraction)
Phenols	G	Cool to 4°C, H_2SO_4 to pH<2	28 days
Phosphorous, elemental	G	Cool to 4°C	48 hours
Phosphorous, total	P,G	Cool to 4°C, H_2SO_4 to pH<2	28 hours

(continued)

TABLE 3-18 (continued)

Measurement	Container[a]	Preservative[b]	Maximum Holding Time[c]
Residue, total	P	Cool to 4°C	14 days
Residue, filterable	P	Cool to 4°C	14 days
Residue, nonfilterable	P	Cool to 4°C	7 days
Residue, settleable	P	Cool to 4°C	7 days
Residue, volatile	P	Cool to 4°C	7 days
Silica	P	Cool to 4°C	28 days
Specific conductance	P	Cool to 4°C	28 days
Sulfate	P	Cool to 4°C	28 days
Sulfide	P	Cool to 4°C, Zinc Acetate	28 days
Sulfite	P	Cool to 4°C	48 hours
Surfactants	P	Cool to 4°C	48 hours
Temperature	P	Determine on site	Immediately
Turbidity	P	Cool to 4°C	48 hours

a - Polyethylene (P) or Glass (G).
b - Sample preservation should be performed immediately upon sample collection. For composite samples each aliquot should be preserved at the time of collection. When use of an automatic sampler makes preservation of each aliquot impossible, samples may be preserved by maintaining at 4°C until compositing and sample splitting is completed.
c - Samples should be analyzed as soon as possible after collection. The times listed are the maximum times that samples may be held. Samples may be held for longer periods only if the laboratory has data on file to show that the specific types of samples under study are stable for the longer time. Some samples may not be stable for the maximum time period given in the table. A laboratory is obligated to hold the sample for a shorter time if knowledge exists to show this is necessary to maintain sample integrity.
d - Samples should be filtered immediately on site before adding preservative for dissolved metals.
e - Guidance applies to samples to be analyzed by GC, LC, or GC/MS for specific organic compounds.
f - This should only be used in the presence of residual chlorine.

(Compounds not found on table should be preserved at 4°C; storage: 1 week).

pump, appropriate glassware, and filter paper. Approved EPA filtration holders and media are described by EPA (1982c).

Confusion has arisen over filtered versus unfiltered samples because os the increased use of "Priority Pollutant Scans" in hazardous waste investigations. The confusion is a result of several items, the first being that the priority pollutant values for metals specifies total metals and EPA procedures do not call for field filtration. Priority pollutant scans therefore do not require filtration which may result in elevated metal concentrations being detected for groundwater. To avoid this, filtration can be conducted for one set of samples for dissolved metals analysis, while another unfiltered sample is collected for total metals. Thus, values for total, dissolved, and suspended metals are obtained.

Organic chemical concentrations may be affected by filtering. Phthalates or other binders may be washed out of filters, thus indicating the false presence of these compounds in groundwater. Nonfiltering may allow sorption of organics to suspended particles and sediments. One compromise is to decant the clearest portion of a sample while discarding the cloudy portion of the sample containing the highest concentration of sediments and suspended solids.

All samples must be shipped according to DOT standards. Groundwater samples are usually not considered hazardous materials and may be shipped counter to counter via priority air freight to the identified laboratory. Chain of custody (COC) procedures should be followed during transportation. The purpose of chain of custody procedures is to document the identity of the sample and sample handling from the point of collection until laboratory analysis is complete and laboratory QA/QC procedures confirm accuracy. Documentation supports this by providing a written record of procedures followed and data collected during sampling.

Sample analyses should follow approved EPA procedures wherever possible. EPA has recently published the second edition of "Test Methods for Evaluation of Solid Waste" (EPA, 1982c) which contains procedures that may be used to determine chemical composition of wastes or the presence of hazardous wastes

in other matrices. However, many of these methods have not yet been fully tested.

QA/QC procedures followed in the laboratory should be in accordance with those outlined in EPA (1979), EPA (1980), and EPA (1982c). Laboratories participating in EPA's performance evaluation and quality assurance review program should have these procedures in place.

Appropriate QA/QC data must accompany sample results so that sample validity can be determined. The following QA/QC checks are typically provided by laboratories:

- Surrogate Recovery Data -- Surrogates are chemical compounds similar in chemistry to the compound under analyses that are added to samples to monitor method efficiency. Surrogate recovery results outside of established limits may indicate poor quality data but may also reflect interferences with similar chemical compounds (Gurka, et al., 1982).
- Duplicate Analyses Results -- Duplicates are reanalyses of individual samples. Close agreement between sample and duplicate results is a good indication of method precision.
- Field and Trip Blanks, and QA and Spiked Samples Results -- These samples have been discussed previously.

Laboratory QA/QC performance should be evaluated prior to laboratory selection and before any samples are taken. Performance evaluation can be performed by sending split samples to several laboratories for comparison.

3.3.2.3 Evaluation

The first step in the evaluation process is to assess the validity of the data. Because leachate constituent concentrations are typically in the parts per million (ppm) or part per billion (ppb) range, even a seemingly innocuous event can have a major impact on the resulting data. Table 3-19 summarizes some of the factors that can affect the validity of groundwater samples. This assessment helps to place the results and subsequent evaluations and conclusions in the proper context.

Sample results are typically reported as concentrations of specific chemicals in mg/l (ppm) or ug/l (ppb). These data can be used to plot isoconcentration maps and cross sections in which areas having equal concentration of selected contaminants are connected by contour lines. These maps and cross sections are drawn in much the same manner as potentiometric surface maps and cross sections, but use concentration results instead of water levels. The problem inherent in interpreting these isoconcentration figures is that a three dimensional phenomenon (i.e., a leachate plume) is being presented in two dimensions. Furthermore:

- The data typically represents only one point in time and may become obsolete quickly depending on the rate of plume movement
- Samples taken from wells designated as points on a map or cross section may actually represent large three dimensional surfaces within the aquifer
- Analytical results must be extrapolated between wells to produce contours
- There is a certain degree of natural and analytical variability that cannot be expressed easily in a figure.

These concerns should be kept in mind when interpreting sampling results.

Initial calculations of concentration gradients generally assume conservative and nonreactive leachate constitutents, and use linear interpolation. For example, if sampling results indicate that well M-1 contains 500 ppb of some constituent and well M-2 contains 100 ppb and these wells are screened in the same aquifer 200 feet apart along the same groundwater flowpath, then the linear concentration gradient can be calculated by:

$$(500 \text{ ppb} - 100 \text{ ppb})/200 \text{ feet} = 2 \text{ ppb/foot}$$

This gradient can then be used to estimate the location of the plume boundary. In the example described above, the boundary would be 50 feet beyond well M-2 along the same groundwater flowpath.

TABLE 3-19
POTENTIAL SOURCES OF FACTORS AFFECTING GROUNDWATER SAMPLE VALIDITY (After Fetter, 1983; and Nacht, 1983)

Source or Activity	Effect
Drilling Equipment and Techniques	• Equipment can be a source of organic solvents, diesel fuel and gasoline, oils and greases, and cross contaminants from other drill sites
	• Drilling fluids can add contaminants or affect certain physical parameters (e.g., COD, BOD, pH)
Well Construction and Materials	• PVC casing and elements used to join PVC casing can leach organic chemicals and may sorb certain heavy metal compounds
	• Stainless steel or high carbon steel may release metals to water and sorb certain organic compounds
	• Bentonite used in drilling fluids or in annulus seals may sorb heavy metals and organics
Sampling Equipment and Techniques	• Inadequate purging can introduce stagnant water into sample reducing concentrations of volatile organics
	• Excessive purging can dilute contaminants below detectable limits by mixing major concentrations with large amounts of uncontaminated formation water
	• Pumps can sorb metals, organics, or induce outgassing of volatile organics through turbulence
	• Gas lift methods can induce outgassing and oxidation
	• Field filtering of samples may add dissolved solids if dissolution of organic solvents occurs or cause oxidation and outgassing if contact with air is allowed
	• Improper cleaning may introduce contaminants from one well to another

(continued)

TABLE 3-19 (continued)

Source or Activity	Effect
Sampling Equipment and Techniques (cont.)	• Excess turbulence caused while placing the sample in containers can result in outgassing and oxidation
Sample Preservation and Analysis	• Samples that are not preserved properly or that are held too long prior to analysis can undergo a variety of chemical changes
	• Bottles cleaned with organic solvents (e.g., methylene chloride) can introduce a variety of contaminants into the sample depending on the purity (i.e., grade) of the solvent and the degree to which the solvent was volatilized from the bottle prior to sealing
	• Certain organic compounds (phthalates, methylene chloride, trichlorofluoromethane) are common contaminants in laboratories and may affect sample results

Both concentration and hydraulic gradients are not always linear, but frequently exhibit logarithmic changes. Rather than calculate logarithmic gradients (such as radioactive decay rates), gradients are generally easier to approximately locate if isoconcentration contours or plume boundaries are plotted by using log paper as a reference in the same manner that a ruler is used in linear interpolation.

Nonlinear concentration gradients can also be a result of pulsed releases of contaminants. In general, differences of a few 10's of parts per billion are most often meaningless. Orders of magnitude changes in concentration are usually more reliable as plume indicators, but may not be suitable to easy interpretation in complex hydrogeologic situations.

Plume migration rates are calculated simply by dividing the maximum distance the plume has moved from the source by the time since leachate generation began. This calculation assumes that the duration of leachate generation is known (Section 3.1.1), the boundaries of the plume have been correctly delineated, and the rates of leachate generation and movement are constant over time.

Another use of sample results involves comparing this direct data on plume boundaries and migration rates to hydrologic calculations made with previously collected data. If results differ widely, some input to the calculations must be wrong or some natural factor is affecting plume movement. If sample results indicate the plume has migrated farther than had been calculated initially, then one or more of the following is true:

- There are preferential routes of migration that are not accounted for in the calculations
- The transmissivity (or hydraulic conductivity), hydraulic gradient, porosity, or time from the beginning of leachate generation has been underestimated
- The average saturated thickness of the aquifer has been over estimated.

If sampling results indicate the plume has not migrated as far as had been calculated initially, then one or more of the following is true:

- Pollutant attenuation is significant and the assumption cannot be made that leachate constituents are conservative and nonreactive
- Natural or artificial factors have at some time or times reversed or changed flow directions or patterns
- Leachate generation volumes and rates have been over estimated, or leachate generation has not been constant over time
- Unidentified natural barriers to contaminant migration are present
- Plume migration patterns are more complex than previously supposed because of either subsurface hydrogeologic conditions or the physical and chemical properties of the leachate

- The transmissivity, hydraulic gradient, porosity, or time from the beginning of leachate generation has been over estimated
- The average saturated thickness has been under estimated.

All calculations should be checked to try to identify the source of the discrepancy so that modifications can be made to the hydrogeology data base prior to planning plume control measures. If the sampling information verifies the initial calculations, then aquifer restoration planning can begin.

CHAPTER 4

PLUME CONTROL TECHNOLOGIES

Methods for controlling the migration of a leachate plume can be placed into one of four categories -- groundwater pumping, subsurface drains, low permeability barriers, and innovative technologies.

Groundwater pumping technologies involve the extraction or injection of groundwater, through wells, to alter the direction of leachate plume movement. In pumping a groundwater extraction well, a cone of depression is created which causes groundwater to flow toward the well. A cone of impression or mound is created around a groundwater injection well which causes groundwater to flow away from the well. Groundwater injection and extraction wells can be used separately, or in combination, to change the flow of groundwater in order to contain or remove a leachate plume.

Subsurface drains are continuous, permeable barriers designed to intercept groundwater flow. As such, they can be used much like pumping systems. Groundwater collected in subsurface drains flows to a sump where collected water can be pumped to a treatment system.

Subsurface barriers consist of a vertical wall of low permeability material constructed underground for the purpose of redirecting groundwater flow. Impermeable barriers can be used to contain contaminant plumes associated with waste sites by completely surrounding a waste site or plume, or they can be used to lower a groundwater table to prevent contact with wastes.

In addition to these more established techniques, innovative technologies are being developed to control leachate plume movements. These include

in situ biological treatment (i.e., bioreclamation), in situ chemical treatment, permeable treatment beds, block displacement, and others. In situ treatment techniques have already been used on a limited basis for aquifer restoration.

The purpose of this chapter is to present an overview of available techniques for controlling the migration of leachate plumes and to present methods for evaluating and selecting a technique. These techniques are described in more detail in the following chapters.

4.1 Control Techniques

4.1.1 Groundwater Pumping

Well systems can be designed to perform several functions with or without the assistance of other technologies (e.g., barrier walls). The main applications in plume management are groundwater level adjustment, plume containment, and plume removal.

Well systems to adjust groundwater levels can be designed using extraction wells to lower water levels or using injection wells to create groundwater mounds. By adjusting groundwater levels, plume migration can be stopped at the source or the speed and direction of the plume can be altered. In either case, contaminated water is not extracted from the groundwater system as is the case with containment and removal techniques.

Well systems used to contain a plume may incorporate extraction wells or extraction and injection wells in combination. Containment differs from removal in that the source of contamination generally is not stopped, so that contamination is an ongoing process. Because containment requires removing contaminated groundwater, a treatment or disposal method must be developed to handle the system discharge.

Plume removal implies a complete purging of the groundwater system of contaminants. Removal techniques are suitable when contaminant sources have

been stopped (e.g., by waste removal or site capping) or contained (e.g., by barrier walls) and aquifer restoration is desired. Extraction or extraction and injection well systems can be used in plume removal. Numerous arrays and patterns are available for injection and extraction wells depending on site-specific suitability. Extraction and injection techniques can also be used with flushing compounds to accelerate contaminant removal. As with containment strategies, treatment of pumped water is necessary.

Previous utilization of pumping technologies to manage plumes has shown that these methods are most effective at sites where underlying aquifers have high hydraulic conductivities (e.g., coarse grained sands) and where the contaminants move readily with groundwater (e.g., benzene). Pumping methods have also be utilized with some effectiveness at sites where pollutant movement is occurring along fractured or jointed bedrock. However, the fracture patterns must be traced in detail to ensure proper well placement.

The shape and size of a well's cone of depression is dependent upon the pumping rate and cycle, slope of the original water table, location of hydraulic barriers, aquifer characteristics, and location of recharge zones. Two aquifer characteristics important in determining the cone's configuration are the coefficients of transmissivity (T) and storage (S). Both of these parameters are inversely related to drawdown and the deepening of the cone. The extent of the cone under equilibrium pumping conditions is termed the radius of influence. The radius of influence can be used in determining well spacing, pumping rates, pumping cycles, and screen lengths when flow rate is low. However, when moderate flow rates exist at a site, distortion occurs in the flow lines such that a plume may pass between two adjacent wells even though their cones of depression overlap. For these cases velocity plots must be developed to properly design a well system.

Selecting and designing an appropriate well system requires that adequate information be available on the site's hydrogeologic conditions and the plume's characteristics. Four basic types of wells are used in plume management: well points, suction wells, ejector wells, and deep wells. Using the site data, selection of an appropriate well type can be made. Table 4-1 lists selection criteria for these well types.

TABLE 4-1. CRITERIA FOR WELL SELECTION
(Powers, 1981)

Parameters	Wellpoints	Suction Wells	Ejector Wells	Deep Wells
Hydrology				
• Low hydraulic conductivity (e.g., silty or clayey sands)	Good	Poor	Good	Fair to Poor
• High hydraulic conductivity (e.g., clean sands and gravel)	Good	Good	Poor	Good
• Heterogeneous materials (e.g., stratified soils)	Good	Poor	Good	Fair to Poor
• Proximate recharge	Good	Poor	Good to Fair	Poor
• Remote recharge	Good	Good	Good	Good
Depth of well	Shallow < 20 ft	Shallow < 20 ft	Deep > 20 ft	Deep > 20 ft
Normal spacing	5-10 ft	20-40 ft	10-20 ft	> 50 ft
Normal range of capacity (per unit)	0.1-25.0 gpm	50-400 gpm	0.1-40.0 gpm	25-3000 gpm
Efficiency	Good	Good	Poor	Fair

The installation of wells varies with geologic materials, well type, and diameter, but most procedures include opening the borehole, installing casing, completing the well, and developing the well. The first of these steps consists of dislodging and removing the earth materials located in the path of the well. Screens and casings may be installed simultaneously with the opening of the hole or after the hole has been completed. The sequence of events depends upon the technique used to open the hole and the geologic characteristics in the immediate vicinity of the hole. Installation of screens, filters, pumps, and grout completes well construction. Well development is the last step and consists of removing material that has built up on the well screen and walls of the borehole during the previous steps. Removal of these particles maximizes well yield and prevents pump damage.

4.1.2 Subsurface Drains

Subsurface drains include any type of buried conduit used to collect liquid discharges (e.g., contaminated groundwater) by gravity flow. The major components of a subsurface drainage system include: drain pipes, envelope or filter, backfill, manholes or wetwells, and pumping stations. Subsurface drains function similarly to an infinite line of extraction wells. That is, they create a continuous zone of depression which runs the length of the drainage trench.

Although subsurface drains perform many of the same functions as pumping systems, drains may be more cost effective in certain circumstances. For example, they may be particularly well suited to sites with relatively low hydraulic conductivities where the cost of pumping may be prohibitively high because of the need to locate wells very close together. However, there are a number of limitations to the use of subsurface drains as a remedial technique. They are not well suited to areas of high hydraulic conductivity and high flow rate. Also, contamination at great depth may cause construction costs to be prohibitive, particularly if a substantial amount of hard rock must be excavated. Subsurface drains are also not suitable when the plume is viscous or reactive because this type of leachate may clog the drain system.

Functionally, there are two basic types of drains--relief drains and interceptor drains. Relief drains are installed in areas where the hydraulic gradient is relatively flat. They are generally used to lower the water table beneath a site or to prevent contamination from reaching a deeper, underlying aquifer. Relief drains are installed in parallel on either side of the site such that their areas of influence overlap and contaminated groundwater does not flow between the drain lines (i.e., same principle of flow apply to drains as for wells). They can also be installed completely around the perimeter of the site. Interceptor drains, on the other hand, are used to collect groundwater from an upgradient source in order to prevent leachate from reaching wells or surface water located hydraulically downgradient from the site. They are installed perpendicular to groundwater flow. A single interceptor located at the toe of the landfill, or two or more parallel interceptors may be needed depending upon the circumstances.

Drainage system are classified by function (i.e., as either relief or interceptor drains), by the type of drain pipe, and by their configuration. Perforated pipe, available in chemically resistant concrete, vitrified clay, and various plastics, is the most widely used drain pipe material for remedial action work, particularly if the drains are to be placed deep in the substratum. Other types of drainage conduits include jointed clay and concrete tiles, and flexible corrugated plastic pipe. The configuration of the drainage system depends on the site configuration and size, and the groundwater flow rate. The system may be singular, consisting only of lateral pipes discharging to a collection sump, or composite where laterals discharge to larger collector pipes which may in turn discharge into a main before reaching a collection sump.

In a subsurface drainage system containing parallel relief drains, depth and spacing are interdependent design variables. In theory, the deeper the drains are the greater the spacing that can be used to obtain the same zone of depression. This relationship between depth and spacing is critical to the design of effective parallel drainage systems. In designing a system for hazardous waste sites, the distance between drains located on either side of the site must be determined so that their drawdowns intersect and thereby

capture the entire plume. This spacing, as with wells, will be dependent on flow rates. The higher the flow rates the closer the drains will have to be placed (holding other factors constant). Velocity plots for determining capture zones are probably the best method available to adequately calculate the spacing on drains. Also, drain depth and spacing may need to be manipulated in order to capture the entire plume. However, in designing parallel drains for hazardous waste sites, a minimum spacing is often imposed by the boundaries of the waste since excavation through the waste material can be extremely hazardous. A maximum depth may be imposed by the prohibitive cost of trench excavation.

The shape of the drawdown curve upgradient of the site is independent of hydraulic conductivity but is a function of head or hydraulic gradient. The upgradient influence or drawdown extends for a distance which is inversely proportional to the water table gradient. The distance to which the water table is lowered downgradient of the interceptor drain is directly proportional to the depth of the drain. Theoretically, a true interceptor drain lowers the water table downgradient to a depth equal to the depth of the drain. The distance downgradient to which the drain is effective in lowering the water table is infinite provided there is not recharge. This however is never the case since infiltration from precipitation recharges the groundwater. The quantity of flow is also proportional to the depth of the drain. If an interceptor drain is placed at the midpoint between the water table and an impervious layer, a little less than 50 percent of the flow will be intercepted. The upgradient and downgradient influence of an interceptor can be determined theoretically or in the field.

In addition to designing the subsurface drainage system for appropriate depth and spacing, the hydraulic design must also be developed. The drain's pipe size and gradient must be adequate to cause the water to flow after it enters the pipe. The gradient of the drain pipes should be great enough to result in a flow velocity that prevents siltation, yet will not cause turbulence. The Soil Conservation Service (1973) has published data on minimum recommended grades for various pipe diameters and maximum velocities recommended for various soil types. The diameter of the drain for a given

capacity is dependent on flow, hydraulic gradient and the roughness coefficient, which in turn is a function of the hydraulic resistance of the drain material. The formula for hydraulic design is based on the Manning formula for rough pipes. SCS (1973) and Cavelaars (1974) have developed monographs for estimating pipe diameter based on the Manning formula when hydraulic gradient, flow rate, and drainage area are known. Roughness coefficients have been published for all drain types.

The other criterion for successful hydraulic design is to ensure that the pipe will accept the drainage water when it arrives at the drainline. To meet this criterion, the relationship between the hydraulic conductivity of the gravel envelope or filter material, the perforations in the drain pipe, and the hydraulic conductivity of the base soil material must be evaluated. 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 is more permeable than the surrounding soil. Although filters and envelopes have distinctly different functions, requirements of both a filter and an envelope can be met. SCS (1973) has developed distinct design criteria for gravel filters and envelopes, whereas the Bureau of Reclamation (1978) has developed one set of standards for a well graded envelope which meets the requirements of both a filter and an envelope. For tile drains, the criteria are based on a comparison of the grain sizes of the envelope and base soil material. Where perforated pipe drains are being used, the minimum size of the envelope material is based on the size of the perforations. Geotextile fabrics offer an alternative to the more conventional sand and gravel filters provided the fabrics are compatible with the waste components in the leachate so that the drains do not clog. Design of the drainage sump and pumping plant are also considered part of the total hydraulic design.

Construction and installation of subsurface drains can be divided into two major phases--trench excavation and drain installation. Trench excavation is often the most complex and costly aspect of construction and installation. The ease or difficulty of excavation can have a dramatic effect on the cost of the total installation. Difficult excavation may result in exclusion of subsurface drainage as a viable technique because of prohibitive costs. Once

trench excavation is completed the components of the subsurface drain can be installed. This process consists of installing drain pipe bedding, drainage pipe, a gravel or soil envelope, filter fabric, backfill, and auxiliary components including manholes and pumping stations.

4.1.3 Low Permeability Barriers

Low permeability barriers can be used to divert groundwater flow away from a waste disposal site or to contain contaminated groundwater leaking from a waste site. There are three major types of low permeability barriers that are applicable to leachate plume management--slurry walls, diaphragm walls, and grout curtains.

A slurry wall is formed by excavating a trench using a bentonite and water slurry to support the sides. The trench is backfilled with materials having lower permeability than the surrounding soils. The slurry backfill trench or slurry wall reduces or redirects the flow of groundwater.

A diaphragm wall is designed for structural strength and integrity in addition to low permeability. Diaphragm walls can be made of cast in place concrete or precast panels with cast in place joints.

A grout curtain is formed by pressure injecting one of a variety of special grouts to seal and strengthen a rock or soil body. Once in place, these grouts set or gel in the rock or soil voids. This greatly reduces the permeability of and imparts increased mechanical strength to the grouted mass, and results in a grout wall or curtain. Because a grout curtain can be three times as costly as a slurry wall, grout curtains are rarely used when groundwater has to be controlled in soil or loose overburden. Grout is used primarily to seal voids in porous or fractured rock when other methods of controlling groundwater are impractical.

Barrier walls are classified by the materials of which they are composed and the position in which they are placed with respect to the pollution

source. There are three major types of barrier walls categorized according to material used to backfill the trench:

- Soil-bentonite
- Cement-bentonite
- Diaphragms.

Soil-bentonite walls are composed of soil materials (often the trench spoils) mixed with small amounts of bentonite slurry from the trench. Cement-bentonite walls are composed of a slurry of portland cement and bentonite. Diaphragm walls are composed of precast or cast in place reinforced concrete panels (diaphragms) installed by excavating a short, slurry supported trench section, using a clamshell bucket or other suitable piece of equipment. Upon completion of excavation, the trench section is filled with a precast, reinforced concrete panel or tremied concrete around a reinforcement cage. Alternate or primary panels are installed first, followed by the secondary panels. Joints between panels are formed using stop end tubes that are concreted after adjacent panels are completed. Another method involves using a cement-bentonite slurry as the exavation fluid which forms the joint between panels when set.

In general, soil-bentonite walls can be expected to have the lowest permeability, the widest range of waste compatibilities, and the lowest cost. They also offer the least structural strength (highest elasticity), usually require the largest work area, and are restricted to a relatively flat topography unless the site can be terraced.

Cement-bentonite walls can be installed at sites where there is insufficient work area to mix and place soil-bentonite backfill. By allowing wall sections to harden and then continuing the wall at a higher or lower elevation, they can be installed in a more extreme topography. Although cement-bentonite walls are stronger than soil-bentonite walls, they are at least an order of magnitude more permeable, resistant to fewer chemicals, and more costly.

Diaphragm walls are the strongest of the three types, and are the most costly. Provided the joints between panels are installed correctly, diaphragm walls have approximately the same permeability as cement-bentonite walls and because of a similarity of materials, about the same chemical compatibilities. Diaphragm walls are most typically used in situations requiring structural strength and relatively low permeability.

Configuration refers to the vertical and horizontal positioning of a barrier wall with respect to the location of the pollution source and groundwater flow. There are two types of configuration--vertical and horizontal. The wall configuration, combined with any other remedial measures (e.g., pumping, capping), determine barrier wall effectiveness in controlling leachate plume migration.

Vertical configuration refers to the depth of the wall with respect to both geologic formations and the water table. Based on vertical positioning, walls are either keyed into a low permeability formation below the aquifer or placed to intercept only the upper portion of the aquifer. The latter type, known as a hanging barrier wall, can be used to control contaminants such as petroleum products which do not mix with the groundwater but float on top of it. In these situations, the barrier need only extend to a depth in the water table sufficient to intercept the contaminants. Keyed barrier walls are excavated through the water table to a confining layer to contain contaminants that mix with or sink to the bottom of the aquifer. The connection between the wall and the confining layer is very important to the overall effectiveness of the barrier.

Horizontal configuration refers to the positioning of the barrier relative to the location of the pollution to be controlled and the direction of groundwater flow (i.e., the gradient). Based on horizontal configuration, barrier walls may completely surround the pollution source or be placed upgradient or downgradient from it.

Circumferential placement refers to placing a barrier wall completely around the site. Although this requires a greater wall length and higher cost

than either upgradient or downgradient placement alone, some advantages are offered. A circumferential barrier wall, when used with a surface infiltration barrier such as a cap, can greatly reduce the amount of leachate a site will release to the environment. If a leachate pumping or drainage system is used, as often is the case, a waste site can be virtually dewatered. In addition to vastly reducing the amount of leachate to be treated, dewatering can help increase the longevity of the wall.

Upgradient wall placement refers to the positioning of a wall on the groundwater source side of a waste site. This type of placement theoretically can be used where there is a relatively steep gradient across the site in order to divert uncontaminated groundwater around the wastes. In such cases, clean groundwater is prevented from becoming contaminated thereby reducing leachate treatment requirements.

Placement of a barrier wall on the side opposite the groundwater source is referred to as downgradient placement. This placement configuration does nothing to limit the amount of leachate being generated and so is practical only in situations where there is a limited amount of groundwater or contaminant flow, such as near drainage divides. This type of barrier theoretically can contain leachate so the leachate can be recovered for treatment. Although this wall configuration may be keyed into a confining layer for miscible or sinking contaminants, most often the wall is hanging to contain and recover floating contaminants.

Grout curtains are the most practical and efficient method for sealing fissures, solution channels, and other voids in rock. There are four basic techniques for installing a grout curtain. These are:

- Stage-up method
- Stage-down method
- Grout-port method
- Vibrating beam method.

The first three methods are injection methods in which the grout is injected from either the bottom of a borehole to the top (stage-up), or the

top of a borehole to the bottom (stage-down), or through a slotted injection pipe that has been sealed into the borehole with a brittle portland cement and clay mortar jacket (grout-port). The grout port method utilizes rubber sleeves which cover the outside of each slit (or port) permitting grout to flow only out of the pipe. The vibrating beam method is not a true injection technique, rather, the method is a way of placing the grout in which an I-beam is vibrated into the soil to the desired depth and then raised at a controlled rate. As the beam is raised, grout is pumped through a set of nozzles mounted on the base of the beam, thus filling the newly formed cavity. When the cavity is completely filled, the beam is moved in the direction of the wall, leaving a suitable overlap to ensure continuity. The process is continued until a wall of the desired length is constructed (Harr, et al., unpublished).

Grouting materials fall into three basic groups--cement, bituminous, and chemical. Some specific grout mixtures include portland cement, sand and cement, clay and cement, clay and bentonite, bituminous emulsions, sodium silicate, and acrylamide. The applicability of each material is based on the size of the openings in the soil or rock formation and the anticipated area of grout penetration. The major grouts in use are cement and clay which make up approximately 95 percent of all grouts used.

Cement grouts utilize materials that set, harden, and do not disintegrate in water. Because of their large particle size, cement grouts are more suitable for rock than for soil applications. Materials may be added to cement grouts to improve their applicability. Sand may be added to portland cement to create a grout suitable for coarse materials, while bentonite may be added to improve the penetration of cement in alluvial soils.

Clays have been used widely as grouts either alone or in formulations. In general, coarse sands and gravels are initially grouted using clay or clay and cement because they are inexpensive. Bentonite is an excellent clay grouting material because of its swelling and gel formation properties.

As with slurry walls, placing a grout curtain upgradient from a waste site can redirect flow so that groundwater does not contact the wastes that

are creating the leachate plume. However, placement of a grout curtain downgradient from a hazardous waste site may not be successful because of grout and leachate interactions. For example, in many instances grout setting time is difficult to control, increasing the difficulty of emplacing a curtain of reliable integrity. Additional problems can occur in attempting to grout a horizontal curtain or layer beneath a waste site. Injection holes must be drilled either directionally from the site perimeter or directly through the wastes. The first case may not always be feasible and may be quite costly, and the second might be quite hazardous to construction crews.

4.1.4 Innovative Technologies

Groundwater pumping, subsurface drains, and low permeability barriers are well established technologies that were originally developed for nonhazardous applications and have been adapted for use in controlling leachate plume migration. A variety of other techniques for plume management are under development, many of which were designed specifically for application at hazardous waste release sites. Two technologies in particular, bioreclamation and in situ chemical treatment, appear especially promising and have been demonstrated on a limited basis for site cleanup. These two techniques are described briefly below; other innovative technologies are described in Chapter 8.

4.1.4.1 Bioreclamation

Bioreclamation is an in situ groundwater treatment technique based on the concept of utilizing microorganisms, combined with aeration and the addition of nutrients, to accelerate the biodegradation rate of groundwater contaminants.

Many species of bacteria, actinomycetes, and fungi have been found to degrade hydrocarbons associated with petroleum. Bacteria are the prime microorganisms involved with biodegradation of organics in groundwater. Naturally occurring species of the genera Pseudomonas, Arthrobacter, Nocardia, Achromobacterium, and Flavobacterium have been found to attack petroleum

hydrocarbons and other organic chemicals (Raymond, et al., 1976). These bacteria can be stimulated by adding nutrients and oxygen to develop a population that is adapted to readily degrade organic chemicals present in groundwater.

An alternative to developing adapted populations from naturally occurring bacteria is to inoculate the subsurface with microorganisms developed in the laboratory to degrade specific organic chemicals or chemical groups. The particular advantages of this alternative are that overall biodegradation rates of specific organics may be increased and the time required for adaptation of a naturally occurring population is eliminated. In this method, the parent microorganisms are collected from a naturally occurring source containing the specific chemical groupings, such as oil refinery treatment plant sludge for phenol and cyanide treatment. Mutant strains are developed from irradiation and these mutant bacteria are cultured in large amounts and packaged for use. The mutant bacteria can be cultured in even larger amounts in holding ponds at the site and introduced into the subsurface by spray irrigation, surface flooding, or subsurface injection.

Cell nutrients required for proper cell growth and respiration include nitrogen, phosphorus, and trace elements (e.g., potassium, sulfur, sodium, calcium, magnesium, iron, copper). Groundwater usually only contains sufficient amounts of nitrogen and phosphorous for degradation of 10 to 20 milligrams per liter (mg/l) of organic material. More nutrients must be added for biodegradation of higher contaminant levels. Oxygen is also required for the aerobic decomposition of organics by bacteria. Roughly 3 or 4 mg/l of oxygen are required for every mg/l of organic constituent degraded. A constant supply of dissolved oxygen must be supplied to the contaminated groundwater in order to maintain biological activity. Subsurface aeration has been the method most commonly used during previous bioreclamation projects of groundwaters contaminated with gasoline. However, since this method can only provide a maximum of about 10 mg/l of dissolve oxygen, biodegradation of high levels of organics in the subsurface can be limited. Alternative oxygenation techniques include the use of pure oxygen systems, ozone, and low concentrations of hydrogen peroxide.

The selection of bioreclamation as a plume management technique depends on the biodegradability of the components in the contaminant plume. Biodegradabilities of various organic substances can be estimated using the ratio of Biochemical Oxygen Demand (BOD) to Chemical Oxygen Demand (COD). Compounds with a BOD/COD ratio of less than 0.01 are considered relatively undegradable, compounds with a ratio of 0.01 to 0.1 are considered moderately degradable, and componds with a ratio of 0.1 or greater are considered degradable (Lyman, et al., 1974).

Implementation of the bioreclamation process involves the placement of extraction wells to control migration of the contaminant plume by pumping. Groundwater pumped to the surface is mixed with nutrients and reinjected upgradient of the extraction wells. Specialized bacteria may also be added along with the nutrients. The groundwater may be oxygenated with air, oxygen, or hydrogen peroxide. The bioreclamation technique has been used successfully in a number of cases to treat contaminated groundwater plumes from underground gasoline and hydrocarbon leaks.

4.1.4.2 In Situ Chemical Treatment

In situ chemical treatment techniques involve the injection of a chemical into a leachate plume to neutralize, detoxify, precipitate, or otherwise affect the contaminant materials. These techniques are highly dependent on the contaminant and have in the past been used only for spills of specific chemicals. Dilute solutions of acids or bases, such as nitric acid or sodium hydroxide, could theoretically be used to neutralize acidic or basic groundwater contaminants. A system of extraction and injection wells could be used to disperse the neutralizing agent, and contain and cycle groundwater until the appropriate pH was attained. Similarly, chemical agents could be used in this manner to detoxify plume contaminants. For instance, sodium hypochlorite, has been used to oxidize cyanide contaminated groundwaters. Other oxidizing chemicals such as hydrogen peroxide or ozone may find potential application in this type of remedial approach. Solutions of sodium sulfide have also been proposed to precipitate toxic metals from groundwater, thereby resulting in their immobilization. Recently, an underground spill of acrylate

monomer was treated by the injection of a catalyst which caused the plume of acrylate monomer to polymerize and solidify (Williams, 1982).

4.2 Technology Evaluation and Selection

Identifying the most appropriate technique for managing a leachate plume can be an extremely complex process. The selection process involves the acquisition, evaluation, and application of data that vary in reliability, applicability and depth of detail. Although data development is relatively straightforward, the evaluation and use of this data can be difficult because of the many interrelated elements involved. Furthermore, developing and evaluating site remediation plans requires exercising a great deal of technical judgement.

Under the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA or Superfund), EPA developed the National Contingency Plan (NCP) for evaluating site remediation alternatives and their costs. The NCP (47 FR 31180, July 16, 1982) specifies a three step procedure for selecting long term site remediation alternative consisting of:

- Develop alternatives for resolving site problems
- Screen the alternatives based on site and waste characteristics
- Conduct a detailed analysis of the alternatives based on expected implementation and performance problems.

EPA, et al., (1983) provides general guidance for using this procedure to evaluate and select site remediation alternatives. This chapter describes how the procedure can be applied to selecting leachate plume management alternatives. Other sources of information on procedures for selecting among remedial action alternatives include EPA (1982a) and RADIAN (1983).

4.2.1 Development of Alternatives

The first step in developing a plume management plan involves identifying and prioritizing the problems at a site, setting response objectives, and

developing remedial action alternatives. These alternatives may consist of variations of the same basic technology (e.g., keyed, circumferential, cement-bentonite walls; hanging, upgradient, soil-bentonite walls) or combinations of different technologies (e.g., a low permeability barrier combined with a groundwater pumping system).

Typically, hazardous waste release sites pose many types of environmental threats which must be prioritized and translated into a series of corrective actions. For example, a site may pose fire and air contamination hazards from ignitable and volatile wastes present at the surface in addition to groundwater contamination. The first action at such a site might be to mitigate the most immediate hazard by removing surface waste to safeguard the local populace. Once the threats of fire and noxious emissions are alleviated, a thorough investigation can be implemented to plan long term corrective measures for groundwater contamination. If the investigation were to identify drinking water contamination, providing alternative supplies to local residents would take priority over implementation of plume management plans. The need for identifying, prioritizing, and addressing site hazards often continues throughout the response process as new data is developed and evaluated.

Based on the assessment of a site's problems and the priorities set for mitigating these problems, specific response goals can be established. Response objectives can be stated in specific terms, like "Reduce the concentration of substance X in the groundwater at point Y to less than 3 times background levels within one year." The objectives can also be presented in a more general manner, such as by stating the degree of plume control that should be achieved. Establishing the goals of a remediation effort prior to the technology screening and detailed analysis steps is extremely important to focus control efforts on the most critical problems of the site.

Response objectives can be used in a number of ways. First, they can be used to schedule plume migration control efforts in order of priority. One example of such a ranking could be (1) Protect nearby municipal wells, (2) Reduce or eliminate plume movement, and (3) Remove the plume. Second, response objectives can be used during design preparation and review to

compare the projected efficiency of a remedial action alternative to the actual efficiency required to control the plume. For example, if the response objective was to collect 90 percent of a plume and a groundwater pumping system was projected to collect only 75 percent of the plume, then that alternative would be considered to be 83 percent effective. Third, response objectives can be used to set the standards by which the efficiencies of the various technologies are judged after implementation. If, for example, a pumping system were designed to collect at least 60 percent of a plume but only collected 40 percent of the plume during operation, the pumping system would be considered to be 66 percent effective. In developing response objectives for evaluating site remediation alternatives, a number of criteria should be addressed including:

- Priorities -- Do the response objectives focus on the most critical problems at the site?
- Scope -- Do the response objectives address all site problems?
- Effectiveness -- To what extent can a given response objective be fulfilled? To what extent can all response objectives be fulfilled?
- Feasibility -- How difficult will fulfilling a given response objective be? How difficult will fulfilling all response objectives be?
- Timing -- How long before beneficial changes are apparent and how long will the changes last?
- Requirements -- What resources will be needed to fulfill a given set of response objectives?
- Benefits -- If fulfilled, will the response objectives produce environmental changes that are significantly preferable to not taking action?

Developing plume management alternatives based on the established response objectives requires exercising a considerable amount of technical judgement. While there are basically four plume management technologies available (i.e., well systems, subsurface drainage systems, low permeability barriers, and in situ treatment techniques), variations and combinations of these technologies can lead to a large number of possible plume management alternatives to be considered.

To further complicate matters, many plume management technologies cannot be used by themselves to effectively control the contamination caused by leachate plume migration. One example of this situation is the use of extraction wells to remove contaminated water from an aquifer. Once contaminated water has been removed, the extracted water must be stored in a secure area and treated to reduce contaminant concentrations to acceptable levels. Residues from the treatment process must also be disposed of properly and the treated water must be either released to surface waters, placed in seepage basins to recharge the aquifer, or reinjected into the aquifer. Thus, the use of extraction well systems also requires the use of contaminated water storage, treatment, and disposal facilities.

Once the technologies and their required auxiliary measures have been identified and developed into a site restoration alternative, a preliminary evaluation of each alternative can be initiated. This screening step involves assessing the suitability of each alternative relative to specific site conditions.

4.2.2 Screening of Alternatives

Screening involves making a preliminary comparison of the plume management alternatives to identify those alternatives that are precluded by site and waste conditions, thereby, reducing the number of alternatives for detailed analysis to a manageable few. There are three fundamental criteria to consider in screening remedial action alternatives--technical feasibility, environmental and public health impacts, and costs.

Technical feasibility screening consists of a review of all pertinent site and waste characteristics to identify any conditions that would preclude or otherwise effect the use of a certain alternative. Tables 4-2 through 4-5 highlight the effects that selected site and waste characteristics can have on the applicability and performance of the four basic plume management technologies. Additional information on technology applications is detailed in Chapters 5 through 8. In conducting a technical feasibility screening of alternatives, a great deal of technical judgement is required to extrapolate

TABLE 4-2
THE INFLUENCE OF SITE GEOLOGY ON THE SELECTION AND PERFORMANCE OF LEACHATE MIGRATION CONTROL TECHNOLOGIES

Geologic Characteristic	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Soil or overburden thickness or depth to bedrock	Little or no effect	Drains are generally restricted to applications in unconsolidated materials less than 100 feet deep	Slurry walls are generally restricted to applications in unconsolidated materials less than 100 feet deep; grouts can be used at any depth if injected	Little or no effect depending on the delivery system
Ease of excavation	Little or no effect	Difficult excavation may preclude the use of drains	Difficult excavation may preclude the use of barrier walls	Little or no effect depending on the delivery system
Soil texture and lithology	Will affect the design of well screens and gravel packs	Will affect the design of the envelope and filter	May be a factor in slurry loss during installation	Will affect the design of the delivery system
Stratigraphy, horizonation, structure	Will affect well design but not necessarily performance	Will affect drain design but not necessarily performance	No effect as long as key-in is effective and excavation is easy	No effect as long as delivery system functions and biochemical conditions are favorable
Mineralogy; geochemistry; soil chemistry	No effect as long as groundwater chemistry is favorable (i.e., well materials will not be corroded or encrusted)	No effect as long as groundwater chemistry is favorable (i.e., drains are not clogged by mineral deposits)	No effect as long as groundwater chemistry is favorable (i.e., wall materials are not degraded)	Can have a major affect on treatment reaction rates and extents

TABLE 4-3
THE INFLUENCE OF SITE HYDROLOGY ON THE SELECTION AND PERFORMANCE OF LEACHATE MIGRATION CONTROL TECHNOLOGIES

Hydrologic Characteristic	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Aquifer type	Can be used in both confined and unconfined aquifers	Generally restricted to unconfined aquifers	Generally restricted to unconfined aquifers	Depends on the treatment technology and delivery system used
Aquifer depth and thickness	Little or no adverse effect	Drain depth is generally limited to less than 100 feet	Wall depth is generally limited to less than 100 feet	Depends on the delivery system used
Aquifer extent and configuration	May affect design but generally will not influence selection or performance	May affect design but generally will not influence selection or performance	Little or no effect	Aquifer configuration may affect the design of the delivery system
Aquifer homogeneity and isotropy	May affect performance if system is not designed appropriately	May affect performance if system is not designed appropriately	Little or no effect	May effect performance of delivery systems
Aquifer properties (e.g., transmissivity, hydraulic conductivity, porosity)	Will affect design but generally not performance, although well systems may not be the most efficient alternative for low transmissivity aquifers	Generally restricted to aquifers having a low to moderate transmissivity	Excess slurry loss may occur in highly permeable aquifers; grouting is generally not effective in low transmissivity aquifers	May affect the selection of a delivery system

(continued)

TABLE 4-3 (continued)

Hydrologic Characteristic	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Hydraulic gradient	Will affect design but not selection or performance	Will affect the selection of intercepter drains (high gradients) versus relief drains (low gradients)	Generally no effect although very high hydraulic gradients across a barrier may induce failure	May affect delivery system
Hydraulic barriers (i.e., recharge and discharge boundaries)	Will affect design and performance if barriers are not identified adequately	Will affect design and performance if barriers are not identified adequately	Little or no effect especially if site is capped effectively	Will affect design and performance of the delivery system
Aquiclude effectiveness and adjacent aquifers	May affect performance if system is not designed appropriately	May affect performance if system is not designed appropriately	Keyed systems will be ineffective if aquiclude leaks	May affect design and performance of the delivery system
Groundwater chemistry	Can be a factor in corrosion and encrustation of well screens and pumps	Can be a factor in envelope and drain clogging	Can be a factor in wall degradation	Can affect the rate and extent of some treatment systems as well as the delivery system

TABLE 4-4
THE INFLUENCE OF PLUME CHARACTERISTICS ON THE SELECTION AND PERFORMANCE OF LEACHATE MIGRATION CONTROL TECHNOLOGIES

Plume Characteristic	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Flow direction	Little or no effect because the system will readjust flow directions	Little or no effect because the system will readjust flow directions	Very important for placing noncircumferential walls especially if flow directions change seasonally	Little or no effect
Flow rate	Important in system design and operation but generally does not affect selection or performance	High flow rates generally preclude the use of subsurface drains	Can be very important for noncircumferential walls especially downgradient walls	Can be very important depending on the treatment system used
Volume and extent	Little or no effect	Generally not practical for very large plumes	Generally not practical for isolating very large plumes	Generally not practical for very large plumes
Contaminant types	Can affect the selection of system materials and components; some contaminants can clog systems	Can affect the selection of system materials and components; some contaminants can clog systems	Can affect wall placement and performance if contaminants degrade wall materials	Generally the primary consideration in selecting a treatment system
Concentrations	Little or no effect if the system is designed and operated properly	High concentrations of some contaminants will clog drainage systems	High concentrations of some contaminants can degrade wall materials	High contaminant concentrations cannot be treated effectively by some treatment systems
Density	Will influence the placement of screens in individual wells	Can reduce system effectiveness if drain is not situated properly within the aquifer	Very important for hanging walls	May be a factor in the design of the delivery system

(continued)

TABLE 4-4 (continued)

Plume Characteristic	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Viscosity	Highly viscous leachate may clog screens, pipes, and pumps thus reducing well efficiency	Highly viscous leachate may clog envelope materials resulting in drain failure	Little or no effect	Effect depends on the delivery system used
Solubility in groundwater	Pumping systems can be designed for both soluble and insoluble contaminants	Drainage systems can be designed for both soluble and insoluble contaminants, although drains tend to function more effectively when contaminants are soluble in groundwater	Little or no effect	Effect depends on treatment technology used
Reactiveness, explosiveness, corrosiveness, volatility	Care must be taken during installation and the operation of pumps and treatment systems; some contaminants may degrade well materials	Care must be taken during installation and the operation of pumps and treatment systems; some contaminants may degrade drain materials	Care must be taken during installation; some contaminants may degrade wall materials	Care must be taken in implementing in situ treatment activities. The ability of contaminants to enter into chemical and biochemical reactions, however, is a requirement of in situ treatment technologies.
Toxicity	Little or no effect except for treatment system	Little or no effect except for treatment system	Little or no effect	Effect depends on the treatment technique used. Some highly toxic contaminants can not be bioreclaimed.

TABLE 4-5
THE INFLUENCE OF SURFACE CONDITIONS ON THE SELECTION AND PERFORMANCE OF LEACHATE MIGRATION CONTROL TECHNOLOGIES

Surface Condition	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Climate	Generally no effect	Excess precipitation may cause a reduction in system effectiveness	Freeze-thaw cycles and drought may cause wall failure	Some treatment reactions are inhibited at low temperature
Topography, slopes	Generally no effect	Installation may be difficult or impossible in rough terrain	Installation may be difficult or impossible in rough terrain	Generally no effect depending on the delivery system
Vegetation	Generally no effect except to restrict access or require clearing	Generally no effect except to restrict access or require clearing	Generally no effect except to restrict access or require clearing	Generally no effect except to restrict access or require clearing
Existing well pumpage	May reduce system effectiveness if not accounted for in design	May reduce system effectiveness if not accounted for in design	Generally no effect	May effect delivery system performance
Locations of facilities (e.g., buildings, pipelines, utilities, roads)	Generally little or no effect	May require extra care in installation and special design considerations	May require extra care in installation, special procedures to prevent slurry loss, and special design considerations	Little or no effect except as it influences the delivery system

(continued)

TABLE 4-5 (continued)

Surface Condition	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Receptors	Site security required	Site security usually required	Site security usually advisable even if site is capped	Site security required
Potential for future land usefulness	Good if waste source and plume are removed	Good if waste source and plume are removed	Not advisable	Good if waste source is removed and treatment process was effective

general guidelines on technology applications (such as those listed in Tables 4-2 through 4-5) to site specific plume management alternatives. Furthermore, certain site and waste conditions may not preclude a given alternative when considered singly but would if considered together. Therefore, reconsidering some decisions made during the screening process may be necessary in subsequent detailed analyses.

Screening alternatives for environmental and public health impacts involves two major issues. First, the anticipated effectiveness of the alternative in reducing risks related to the site must be determined. This evaluation generally uses both the response objectives described previously and the no action alternative as points of reference for the effectiveness of the alternatives. Second, all potential effects of implementing the alternative which are adverse to the environment or the public health must be identified. For example, excavation of a low permeability barrier or subsurface drain may create dust or waste volatilization problems. The results of this screening step is a summary of the benefits of implementing a given remedial action alternative.

Cost screening is based on a review of projected capital and operation and maintenance (O&M) costs. Capital costs of a remedial action alternative can include (EPA, et al., 1983):

- Construction costs -- includes equipment, labor (including fringe benefits and workman's compensation), and materials required to install a remedial action.
- Equipment costs -- includes remedial action and service equipment.
- Land and site development costs -- includes land related expenses associated with purchase of land and development of existing property.
- Buildings and service costs -- includes process and nonprocess buildings and utility hook-ups.
- Relocation expenses -- includes costs for temporary or permanent accommodations for affected residents.
- Engineering expenses -- includes administration, design, construction supervision, drafting, and testing of remedial action alternatives.

- Legal fees, license and permit costs -- includes administrative and technical costs necessary to obtain licenses and permits for facility installation and operation.

- Start up and shake down costs -- includes costs incurred during remedial action start up.

- Contingency allowances -- Contingency allowances should correspond to the reliability of estimated costs and experience with the remedial action technology.

O&M costs can include (EPA, et al., 1983):

- Operating labor costs -- includes all wages, salaries, training, overhead, and fringe benefits associated with the labor needed for post construction operations.

- Maintenance materials and labor costs -- includes the cost for labor, parts, and other materials required to perform routine maintenance of facilities and equipment for the remedial alternative.

- Auxiliary materials and energy -- includes such items as chemicals and electricity needed for treatment plant operations, water and sewer service, and fuel costs.

- Purchased services -- includes such items as sampling costs, laboratory fees and professional services for which the need can be predicted.

- Disposal costs -- includes transportation and disposal of any waste materials, such as treatment plant residues generated during remedial operations.

- Administrative costs -- includes all costs associated with administration of remedial action operation and maintenance not included under other categories such as labor overhead.

- Insurance, taxes, and licensing costs -- includes such items as liability and sudden and accidental insurance, real estate taxes on purchased land or right-of-way licensing fees for certain technologies, and permit renewal and reporting costs.

- Maintenance reserve and contingency funds -- represents annual payments into escrow funds to cover anticipated replacement or rebuilding of equipment and any large unanticipated O&M costs, respectively.

- Other costs -- includes all other items which do not fit into any of the above categories.

These costs should be updated to current values using standard costs indices. If the alternatives have significantly different time periods for implementation, the costs should be adjusted by conducting a present worth analysis (EPA, et al., 1983).

The cost screening evaluation involves comparing the costs of competing remedial action alternatives to establish the relative costs of alternatives producing similar environmental, public health, and public welfare benefits. Competing alternatives should be eliminated if they are deemed expensive (i.e., an order of magnitude or more) and offer similar or lesser environmental and public health benefits. Alternatives that are more expensive but offer greater benefits should not be eliminated.

Alternatives that have passed this final stage of preliminary screening will undergo a more detailed evaluation using the parameters outlined in Section 4.2.3.

4.2.3 Detailed Analysis of Alternatives

The detailed analysis of the remaining alternatives is similar to screening in the criteria assessed but is different in that technical judgement plays a less significant role. Instead, additional data are collected bearing on the technical feasibility, environmental and public health impacts, and costs of each alternative. These data are analyzed and used as a basis for eliminating inappropriate alternatives and selecting the best site remediation alternative.

Assessing an alternative's technical feasibility involves examining its anticipated reliability and implementability. The reliability of a remedial action alternative depends on the alternative's:

- Performance history -- The degree of success experienced with the same or a similar remedial action alternative under similar site or waste conditions or both (CERCLA funds can be used only if the technologies included in an alternative have been demonstrated effective for waste site restoration).

- Effectiveness -- The extent to which the alternative is expected to meet or exceed each response objective.

- Durability -- The projected period of effectiveness and the amount of maintenance required.

- Flexibility -- The ability of the alternative to be adapted to changes in site and waste conditions without causing environmental damage.

- Sensitivity -- The ability of the alternative to accommodate transient upset conditions without a major reduction in effectiveness.

- Reliance on companion measures -- The extent to which the primary plume control technology depends on associated remedial measures to achieve its maximum effectiveness.

able 4-6 summarizes the general reliability of the four basic leachate migraion control technologies.

The implementability of a remedial action alternative depends on the following alternatives:

- Ease of installation -- The difficulty anticipated in installing and making an alternative operational because of site conditions (e.g., topography, geology, hydrology, soils, climate).

- Ease of construction -- The difficulties anticipated because of the alternative's special technical requirements (e.g., access, availability of equipment and materials, special technical skills required, QA)

- Time to implementation -- The anticipated time necessary to collected additional data required, design the system, install the system, make the system operational, and note an improvement in environmental conditions.

- Ease of operation -- The difficulty anticipated in operating the system over its design life to fulfill the response objectives and in monitoring the system's performance.

- Ease of repair -- The difficulty anticipated in repairing, replacing, or rehabilitating key components of the system.

- Safety requirements -- The equipment and procedures needed to protect the health and safety of site workers and local inhabitants.

Table 4-7 summarizes the general implementability of the four basic leachate migration control technologies.

TABLE 4-6
FACTORS AFFECTING THE RELATIVE RELIABILITY OF LEACHATE MIGRATION CONTROL TECHNOLOGIES

Reliability Criterion	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Performance history	Used successfully at many hazardous waste sites and for related dewatering applications	Used successfully at many sites but not as commonly as well systems; also used frequently for related dewatering applications	Used successfully at selected hazardous waste sites; use of slurry trenching techniques is increasing; grouting applications are relatively minor	Certain treatment systems have been demonstrated at hazardous waste sites including bioreclamation, flushing, and polymerization; used primarily for single-chemical spills
Effectiveness	Generally very effective if designed, operated, and maintained properly. Poor operation and maintenance are the chief reasons for poor effectiveness.	Can be very effective and more efficient than pumping systems if designed and maintained properly. Drain clogging and pipe failure are the chief reasons for poor effectiveness.	Very effective if installed properly although some leakage will generally occur	Variable effectiveness depending on site and leachate characteristics and the treatment technology used
Durability	Generally very durable and easy to maintain. Wells can generally last the full period of operation of the system although pumps require periodic replacement.	Not as durable as pumping systems because drain envelopes tend to clog. Maintenance is not as simple as for well systems.	Durability will be highly variable depending on the hydrologic and chemical stresses on the wall. Maintenance is generally minimal.	Not applicable if the leachate source is removed

(continued)

TABLE 4-6 (continued)

Reliability Criterion	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Flexibility	Very flexible. System can be adapted to changes simply by adding wells, revising pumping rates and schedules, or changing treatment operations.	Not very flexible because drain influence cannot be controlled without changing system design	Not flexible once installed, although this is typically not restrictive	Somewhat flexible depending on the delivery system and treatment technology used, although this is typically not restrictive
Sensitivity	Generally can accommodate minor upset conditions such as high water levels resulting from heavy precipitation. Pump failure can render portions of the system ineffective, however.	Generally can accommodate upset conditions except for major system failure (e.g., clogged drainage main)	Generally can accommodate upset conditions except for major wall failures	Variable depending on the type of upset condition and the delivery system and treatment technology used
Reliance on companion measures	Generally requires leachate storage and treatment facilities; may require source removal or isolation	Generally requires leachate storage and treatment facilities; may require source removal or isolation	Generally requires site capping for circumferential walls, and wells or drains for downgradient and floating walls; may require source or leachate removal or treatment	Requires a delivery and recovery system such as wells or drains; generally requires source removal and treatment or disposal

TABLE 4-7
FACTORS AFFECTING THE RELATIVE IMPLEMENTABILITY OF LEACHATE MIGRATION CONTROL TECHNOLOGIES

Implementability Criterion	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Ease of installation	Generally very easy to install but can be somewhat difficult to fine tune operations	Can be very difficult to install if hard rock excavation and shoring and dewatering is needed, if the design drain depth is large, and if the topography is very uneven; these problems are not insurmountable	Can be difficult or impossible to install in areas of extreme topography or climate	Variable depending on the delivery system and treatment technology used. Climate, hydrology, and soil geochemistry are the most significant factors.
Ease of construction	Most systems can be constructed by qualified local drillers and pump installers	Some systems may require the use of specialized equipment and materials; QA is an important consideration; local contractors may not be qualified	Some systems may require the use of specialized equipment and materials; QA is an important consideration; local contractors may not be qualified	May require specialized equipment and materials. Most in situ treatment is undertaken by specialty firms
Time to implementation	Fairly rapid to install and affect changes in plume migration patterns	Moderately time consuming to install but will affect changes in plume migration patterns fairly rapidly	Moderately time consuming to install and may not affect any improvement in site conditions for some time depending on system placement	Highly variable depending on site size and treatment effectiveness

(continued)

TABLE 4-7 (continued)

Implementability Criterion	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Ease of operation	Can be fairly complex to fine tune system and operate at maximum efficiency; treatment may also be fairly complex	Fairly simple since groundwater collection is a function of gravity flow; treatment may be fairly complex	Implementation limited to monitoring and operating associated remedial action technologies	Can be very complex because of the variables involved in system operation
Ease of repair	Generally, fairly easy to rehabilitate or replace wells, pumps, and surface piping; treatment system repair may be more complex	Pipe and envelope clogging requires excavation and reinstallation; pump and treatment system repair are the same as for well systems	Slurry trench sections that have failed generally cannot be repaired but instead are replaced with new wall sections; grout failure usually cannot be repaired effectively	Generally, fairly easy to rehabilitate or replace system components
Safety requirements	Generally the least stringent of the technologies because subsurface disturbances are minimal; can be significant if explosive wastes are present or if wells must be installed through waste piles	More stringent than well systems because of the greater extent of subsurface excavation and the potential for waste volatilization and dust generation; trench cave-in is a primary safety concern	Slurry trench requirements are similar to those for drains; grouting requirements are similar to those for wells	Variable, but can be as stringent or more stringent than for drains depending on the delivery system and the treatment agents used

Conducting a detailed environmental analysis of each alternative involves predicting all changes (both adverse and beneficial) from existing conditions. This applies to both sensitive biotic environments (e.g., endangered and protected species, wildlife habitats and breeding grounds, vulnerable ecosystems) and human resource use patterns (e.g., recreational areas, fishing and hunting grounds, historic sites, traffic patterns, water supplies). These changes can be evaluated by comparing them to the response objectives, the no action alternative, or published environmental criteria such as those given in:

- "Quality Criteria for Water" (Federal Register November 28, 1980) -- Criteria documents are available for each of 64 toxic pollutants.
- "Quality Criteria for Water" (Red Book, July 1976) -- Contains some additional criteria not preempted by above (e.g., temperature, aesthetics, dissolved oxygen).
- Safe Drinking Water Act (40 C.F.R. 141) -- Maximum levels set for six contaminants. Suggested No Adverse Response Levels (SNARLS) are given for others.
- Pesticide Residue Amendment to the Food, Drug and Cosmetic Act (40 C.F.R. 180) -- Safe levels of 301 pesticides in soil based on crop tolerance levels.

The degree of actual or potential harm caused by the plume must also be addressed. Contamination of some aquifers, notably saline ones, may be considered a relatively low priority problem, whereas the presence of a hazardous waste plume in an aquifer that is the sole drinking water source for numerous people is a high priority issue.

Anticipating and preventing adverse impacts from site remediation activities is a key step in the detailed environmental assessment. Areas that are disturbed during excavation of subsurface drainage trenches, construction access roads, sumps, slurry walls, recharge basins, slurry mixing areas, and leachate storage facilities are extremely susceptible to erosion unless erosion prevention measures are used. The soil carried off of the site as a result of erosion becomes deposited in surface water bodies and can clog water ways, interfere with wildlife, and contribute to flooding downstream. If the eroded sediments contain chemical contaminants, the degradation of water

quality is considerably more serious. The quality of the air surrounding the site can also become degraded when disturbance of the waste site results in fires or explosions, volatilization of waste constituents, or production of toxic gases, mists, or dusts.

When technologies that remove the plume from the aquifer are used, the extracted plume must be treated to reduce its contaminant concentrations to acceptable levels. In some cases, the treated water will still contain higher than background levels of contaminants. If this water is discharged to surface waters, the treated water may result in a reduction in surface water quality. Furthermore, removal of uncontaminated groundwater upgradient of the site without recharging this water downgradient of the site can lower the water table downgradient and result in modifications in both groundwater and surface water flow and use, and vegetation patterns. These conditions would be most pronounced if the extracted groundwater was diverted to a different watershed. Hydrophillic (water loving) plants in areas that formerly had a high water table may become replaced by less water tolerant species. This could then cause a modification in wildlife populations or distribution in the affected areas.

Also important in the detailed environmental assessment is an evaluation of the hazards that will be posed until a site restoration alternative can be implemented and the risks and hazards associated with the failure of an alternative to fulfill the response objectives. In addition to the technical feasibility and environmental impact aspects of each alternative, institutional and health and safety requirements should be evaluated.

Local, state and federal laws require provisions for regulating certain activities. Because of these laws, special permits, bonds or licenses may be required to conduct certain activities at some sites. For example, removal of groundwater from some aquifers, particularly in arid and semiarid areas, may be prohibited without special permission, and transfer of water from one watershed to another is illegal in some areas. Table 4-8 highlights regulations that may be applicable to site remediation activities. Other important institutional issues to be addressed include community relations and the need

TABLE 4-8
SELECTED REGULATORY REQUIREMENTS FOR
AQUIFER RESTORATION ACTIVITIES

Source of Requirement	Requirement or Prohibition
FEDERAL PERMITS AND REGULATIONS	
Resource Conservation and Recovery Act	RCRA permit or interim status authorization may be needed if hazardous wastes are treated, stored, or disposed
	RCRA manifest needed if wastes are to be transported from the site
Federal Water Pollution Control Act (also called the Clean Water Act)	National pollutant discharge elimination system (NPDES) permit needed if wastewater is to be discharged into navigable waters
	Dredge or fill permit needed if these materials are to be discharged into navigable waters
Safe Drinking Water Act	Underground injection control permit may be needed if treated groundwater is reinjected into the subsurface
	Sole source aquifer permit may be required if reinjection involves a sole source aquifer
Wild and Scenic Rivers Act	Prohibits any action that will adversely affect the wild, scenic, or recreation status of a river on the National Inventory
Endangered Species Act	Prohibits jeopardizing any endangered or threatened species or its habitat
Fish and Wildlife Coordination Act	Prohibits modifying a body of water without state approval if it will affect wildlife resources
Coastal Zone Management Act	Prohibits actions inconsistent with Federally approved state coastal zone management programs

(continued)

TABLE 4-8 (continued)

Source of Requirement	Requirement or Prohibition
Archaeological and Historic Preservation Act	Prohibits actions resulting in the loss or destruction of significant scientific, historical, or archaeological data
Atomic Energy Act; Low Level Radioactive Waste Policy Act	Prohibits the transport and disposal of radioactive wastes in certain areas under certain conditions
Flood Disaster Protection Act; National Flood Insurance Act; Executive Order 11988	Requires Federal agencies to evaluate potential effects of actions planned for floodplains
	Requires flood insurances in some cases
National Environmental Protection Act	May require the preparation of environmental impact statements and environmental assessments
Executive Order 11990	Requires that activities in wetlands include all practical means of minimizing harm to the environment
STATE AND LOCAL PERMITS AND REGULATIONS	
Federal delegation of authority or state and local requirements analogous to Federal requirements	Requires permits, rulings, or approvals for the treatment, storage, transport, and disposal of hazardous wastes; for the development, operation, and closure of waste management facilities; for wastewater discharges to surface waters (NPDES); for activities likely to result in hazardous air emissions; for activities involving underground injection; for dredge or fill, wetlands, and sensitive resources; and for activities that may require state or local funding or support

(continued)

TABLE 4-8 (continued)

Source of Requirement	Requirement or Prohibition
Zoning regulations, building codes (fire, water, electrical, sewer), water use regulations	Requires permits or the preparation of approved plans for construction or demolition activities, erosion and dust control, right-of-way, easements, well installation, and water use and discharge
Public safety regulations	Generally requires advanced planning and coordination with local police, fire departments, hospitals, and rescue squads

to coordinate activities with site owners and governmental agencies (e.g., EPA, USGS, COE, and state environmental agencies).

Safeguarding the health and safety of both the personnel installing the plume management technologies and the people living near the site is a primary factor in evaluating alternatives. In addition to the safety hazards associated with any major construction activity, construction at hazardous waste sites involves other hazards, including potential fires and explosions, and possible release of chemicals from buried drums or storage tanks. Personnel working at hazardous waste sites and local residents can also be exposed to numerous health hazards. These hazards can reach the worker through direct dermal exposure, inhalation of toxic constituents, or indirect ingestion of dusts from the site. Direct dermal exposure can be prevented by requiring workers to wear personal protective gear when working in hot zones at the site. This gear can also assist in preventing inhalation of toxic constituents, if respirators or self contained breathing apparatus (SCBA) are used. Indirect ingestion of toxins can be prevented by using strict site security to prevent public access, by prohibiting eating, drinking or smoking in hazardous areas of the site, and by requiring the workers to wash prior to eating or smoking.

Indirect ingestion can be a particular problem at sites where contaminated dusts are present. If dusts are inhaled, the majority will be captured in the upper respiratory tract so very little will reach the lungs. However, the dust that enters the upper respiratory tract will accumulate there until the individual coughs to dislodge it. Most individuals will then swallow, and the contaminated dust will enter the digestive tract. Site workers can be protected from this hazard by using SCBA or respirators with properly selected filters. Local inhabitants can be protected by controlling dust generation and if necessary, by evacuation.

In evaluating the health and safety aspects of a site restoration alternative, precautions for safeguarding site workers and the public can increase dramatically the time and resources required to implement the alternative. Therefore, assessing both alternatives that minimize health and safety risks and those that attempt to control risks is important.

The cost assessment portion of the detailed analysis of alternatives is intended to provide measures of both the total costs and cost effectiveness over time associated with each alternative under consideration. The most reliable method for comparing the costs of proposed options is to prepare preliminary designs for each of the alternatives that are being considered for a site. These designs can then be used to compile site specific cost estimates. These cost estimates generally have an accuracy of +50 to -30 percent (EPA, et al., 1983). Key cost items and the relative costs of the four basic leachate migration control technologies are summarized in Table 4-9.

The procedure recommended for conducting a cost assessment of remedial action alternatives consists of four steps (EPA, et al., 1983):

- Estimate costs -- Determine capital and annual operating costs for remedial alternatives.
- Standardize costs -- Calculate stream of payments and present worth for each remedial action alternative using estimated costs.
- Assess cost sensitivity -- Evaluate risks and uncertainties in cost estimates.

TABLE 4-9
RELATIVE COSTS AND KEY COST ITEMS OF LEACHATE MIGRATION CONTROL TECHNOLOGIES

Cost Criterion	Leachate Migration Control Technologies			
	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
CAPITAL COSTS				
Relative cost	Generally low although groundwater treatment, storage, and disposal can have a major impact	Fairly high depending on site size, depth and ease of excavation, and system design	Moderate to high depending on site size, depth and ease of excavation, and system design	Highly variable depending on site size, treatment method used, and associated technologies required
Key items	• Well installation and materials • Pumps • Surface piping • Effluent storage and treatment facilities	• Drain excavation and installation • Drain materials • Pumps and piping • Effluent storage and treatment facilities	• Wall excavation and installation • Wall materials	• Treatment agents • Delivery systems • Associated technologies
O&M COSTS				
Relative cost	High to very high depending on treatment requirements	High to very high depending on treatment requirements	Fairly low depending on associated technologies used	Highly variable depending on associated technologies used
Key items	• Effluent treatment • Pump operation • System maintenance • Monitoring	• Effluent treatment • System maintenance and operations • Monitoring	• Monitoring • System maintenance	• Monitoring • System maintenance • System operation

- Assess cost effectiveness -- Judge the importance of the poential benefits for each alternative relative to its prospected costs.

Detailed descriptions of this procedure and a matrix based methodology for assessing remedial action cost effectiveness are given in Radian (1983) and EPA, et al., (1983). Based on the cost effectiveness analysis, the best site remediation alternative should be selected and developed in detail. Table 4-10 presents a general summary of the advantages and limitations of the four basic leachate migration control technologies.

4.3 Design and Implementation of Alternatives

Once the best (or best few) remedial action alternative(s) has been selected, a conceptual design should be prepared. The conceptual design should include:

- Engineering drawings showing present topography and proposed topographic changes; site boundaries and property lines; locations of buildings, waterways, and utility lines, and other surface features; locations of sewer lines, pipelines, and other subsurface features; and locations of remediation systems (such as wells) to be installed
- Estimated dimensions of subsurface drains or barriers, number of wells or other specific size related data
- Lists of required auxiliary measures
- Estimations of equipment and materials needs
- Estimated implementation periods and milestones
- Other factors such as requirements for worker safety and public protection, waste removal, transport and disposal (if necessary) and potential for adverse environmental impacts
- Budget cost estimates.

During preparation of these designs, additional factors or considerations may be identified. These should also be incorporated into the conceptual design document.

TABLE 4-10
SUMMARY OF THE FOUR BASIC LEACHATE MIGRATION CONTROL TECHNOLOGIES

Criteria	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Performance history	Applied successfully at both hazardous and nonhazardous sites; poor system operation is the chief cause of performance problems	Applied successfully at both hazardous and nonhazardous sites; system clogging is the chief cause of performance problems	Applied successfully at both hazardous and nonhazardous sites; wall degradation caused by chemical and hydrologic conditions is the chief cause of performance problems	Limited number of applications at hazardous sites and no applications at nonhazardous sites; effectiveness of the delivery system is the chief cause of performance problems
Effectiveness	Generally good	Generally good	Generally good	Variable depending on site conditions
Durability	Excellent with occasional maintenance	Dependent on leachate and groundwater chemistry	Dependent on leachate and groundwater chemistry	Not applicable
Flexibility	Very good	Very good	Limited	Usually very limited
Sensitivity	Good	Good	Very good	Limited
Reliance on companions measures	Usually requires leachate treatment	Usually requires leachate treatment	Usually requires site capping	Usually requires source removal

(continued)

TABLE 4-10 (continued)

Criteria	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Ease of installation	Very good	Variable depending on shoring, dewatering, and excavation requirements	Variable depending on topography and formation permeability	Variable depending on treatment scheme and the delivery system
Ease of construction	Very good	Good; some specialized equipment may be required	Generally good; some specialized equipment may be required	Requires contractor with specialized experience, equipment, and materials
Time to implementation	Relatively short	Moderate to short	Moderate to short	Fairly long to system operation but short to completion of treatment
Ease of operation	Fairly complex	Relatively simple	Very simple	Complex
Ease of repair	Fairly simple	Fairly complex	Fairly complex	Complex to simple
Safety requirements	Minimal relative to other technologies	Very stringent because of the need to control dust, leachate, and groundwater during construction	Fairly stringent because of the need to control dust and leachate during construction	Very stringent depending on the treatment scheme and the delivery system

(continued)

TABLE 4-10 (continued)

Criteria	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
Capital costs	Relatively low; high if on site treatment is needed	Relatively high; very high if on site treatment is needed	Relatively high	Variable depending on the treatment scheme and the delivery system
O&M costs	Relatively high; very high if leachate treatment is needed	Relatively low to very high if leachate treatment is needed	Relatively low	High to low depending on system selected
General comments	• Can be designed for nearly any hydrogeologic condition and leachate type, although pumping systems tend to be less efficient in low transmissivity aquifers and with viscous leachates	• Especially useful in low transmissivity aquifers and for collecting groundwater mounds beneath sites • Use may be restricted by high flow rates, difficult excavation, and leachates that can clog drain components	• Slurry trenches can be used in both high and low transmissivity aquifers although slurry loss can be a problem in some cases; grouts can generally only be used in high permeability formations	• Treatment system selection and implementation is greatly influenced by groundwater and leachate chemistry. The success of the system will depend largely on these factors: the effectiveness of the delivery systems, and the ability of the operator to adjust to changes in site conditions.

(continued)

TABLE 4-10 (continued)

Criteria	Groundwater Pumping	Subsurface Drains	Low Permeability Barriers	In Situ Treatment
General comments (continued)	• Pumping systems can be installed rapidly and inexpensively by qualified local well drillers, pump installers, and general contractors • System design can be modified easily to accommodate changes in site conditions • Pumping systems are relatively simple to repair but difficult and expensive to operate, especially if on site leachate treatment is needed	• Operation is generally simpler than for pumping systems, although maintenance and repair is more difficult • O&M costs can be very high if on site leachate treatment is needed	• Circumferential walls, the most commonly used type of barrier; require site capping and effective aquiclude key-in to be successful • The most significant limitation of barriers is the ability of wall materials to withstand chemical attack by reactive leachates • Operation and maintenance is generally simple and relatively inexpensive	• Implementation generally requires using specialty firms as opposed to local contractors • No long term site presence required if leachate source is removed and treatment is effective

Engineering specifications generally provided in the design of a remediation alternative include detailed descriptions of material requirements and local availability, work practices, and product performance. Other items described in these specifications include quality control and quality assurance procedures and equipment requirements. Schedules should also be provided which allow for segmenting the operations so that convenient stopping-points are identified in advance. If the operation is set up in this manner, the difficulties associated with work stoppages, because of supply shortages, strikes, funding problems or adverse weather conditions can be minimized. Phasing plans that allow maximum coordination between contractors working at the site also can reduce costs and speed construction.

Once an acceptable leachate migration control plan is developed, site remediation efforts can begin. Details related to designing and implementing plume management systems are described in Chapters 5 through 8.

CHAPTER 5

GROUNDWATER PUMPING

5.1 Introduction

This section describes the theory, applications, design considerations and costs associated with groundwater pumping methods to contain and remove plumes originating from hazardous waste disposal facilities. Plume containment is defined here as arresting the further spread of groundwater contamination by altering hydraulic gradients. Containment methods do not necessitate stopping contamination at the source. Plume removal is defined as removing the contaminants from the groundwater system so that the aquifer is purged of harmful constituents. Both approaches emphasize the active diversion and removal of groundwater rather than the passive approach of installing hydraulic barriers.

An associated problem arises from utilizing groundwater pumping methods (or subsurface drains) to control plumes, that is, the treatment or disposal of the removed contaminated groundwater. Numerous treatment processes (e.g., carbon absorption) are available to remove contaminants from the water. However, these techniques are beyond the scope of this document and will not be discussed. Injection of treated water back into the groundwater system will however be discussed as it relates to altering groundwater gradients.

Previous utilization of pumping technologies to manage plumes has shown that these methods are most effective at sites where underlying aquifers have high intergranular hydraulic conductivities (e.g., coarse grained sands) and with contaminants that move readily with the groundwater flow (e.g., benzene). Pumping methods have also been utilized with some effectiveness at sites where pollutant movement is occurring along fractured or jointed bedrock. However, the fracture patterns must be traced in detail to ensure proper well placement.

5.2 Well Theory

The economic and effective withdrawal of contaminated water from a groundwater system is highly dependent on the proper design and construction of a well. In order to make proper design decisions, an understanding of the principles of well hydraulics is needed. Applications and extensions of these theories permit solutions of problems related to flow towards wells under a variety of situations. This section provides some of the basic well hydraulic equations describing the relationships among the various hydraulic characteristics of aquifers and the response of aquifer systems to pumping wells.

5.2.1 Darcy's Law

Darcy's Law describes flow through a porous medium and thus provides the foundation for all groundwater flow theory. Common forms of Darcy's law are:

$$Q = KIA = VAn$$

where:

$$V = KI/n$$

$$I = (h_1 - h_2)/L$$

and the variables are defined as:

Q = flow (ft^3/day)

K = hydraulic conductivity of the porous medium (ft/day)

I = hydraulic gradient (dimensionless)

A = area normal to flow direction (ft^2)

V = velocity of groundwater flow (ft/day)

n = effective porosity (dimensionless)

$h_1 - h_2$ = water levels at two points along the same groundwater flow path (ft)

L = length of the flow path between h_1 and h_2 (ft)

Darcy's Law is only valid when conditions of laminar flow exist.

Based on Darcy's Law, the hydraulic gradient varies directly with flow velocity and indirectly with area. This relationship can be used to explain the formation of a cone of depression around a pumping well (Figure 5-1). When a well is pumped, the water level in the vicinity of the well is lowered and the greatest depth of lowering occurs at the well. Depth of lowering or drawdown is less at greater distances from the well and at some point in the aquifer lowering is nonexistent. Because the water level is lowest at the well, water moves from the aquifer into the well to replace the withdrawn water. Gravity and water pressure (head) drive the water towards the well in the direction of decreasing head. As the flow converges towards the well under an increased hydraulic gradient, the velocity increases according to Darcy's Law. This process results in the typical cone of depression associated with pumped wells (Figure 5-2).

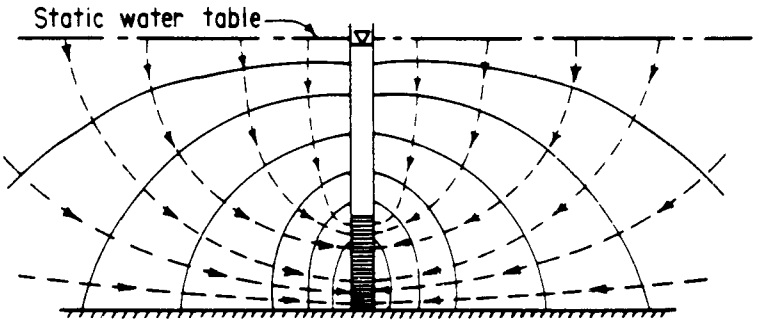
The shape and size of the cone is dependent upon the pumping rate, pumping period (i.e., cycles), slope of the original water table, hydraulic barriers, aquifer characteristics, and recharge zones. Two aquifer characteristics that are important in determining the cone's configuration are of transmissivity (T) and coefficients of storage (S). Transmissivity indicates the amount of water that can move through an aquifer. The coefficient of storage indicates the amount of water which can be removed by pumping. Transmissivity (T) is related to the hydraulic conductivity (K) of an aquifer by:

$$T = Km$$

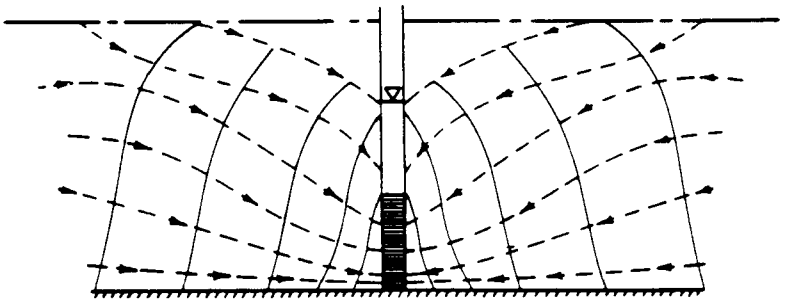
where m is the saturated thickness of the aquifer. The effects of transmissivity and storage on the shape of the cone of depression is illustrated in Figure 5-3. This illustration shows that drawdown is inversely related to the coefficients of storage and transmissivity.

Values of transmissivity and storage are generally determined for an aquifer by drilling wells and conducting pumping tests. Once these values

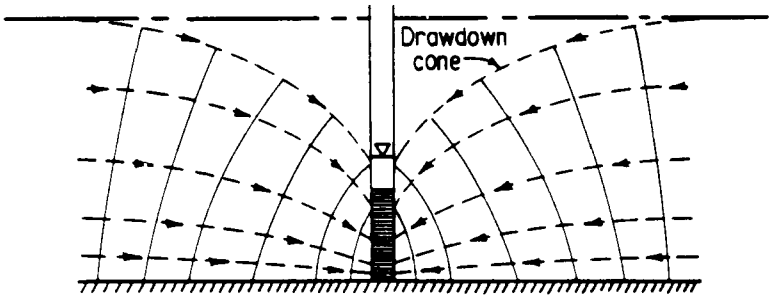
FIGURE 5-1.
DEVELOPMENT OF FLOW DISTRIBUTION ABOUT A DISCHARGING WELL IN A FREE AQUIFER – A FULLY PENETRATING AND 33-PERCENT OPEN HOLE (BUREAU OF RECLAMATION, 1977)



5-1a. Initial stage in pumping a free aquifer Most water follows a path with a high vertical component from the water table to the screen



5-1b. Intermediate stage in pumping a free aquifer Radial component of flow becomes more pronounced but contribution from drawdown cone in immediate vicinity of well is still important



5-1c. Approximate steady state stage in pumping a free aquifer. Profile of cone of depression is established Nearly all water originating near outer edge of area of influence and stable primarily radial flow pattern established.

Flow Lines

Equipotential Lines

FIGURE 5-2.
FORMATION OF CONE OF DEPRESSION
FOR A PUMPED WELL

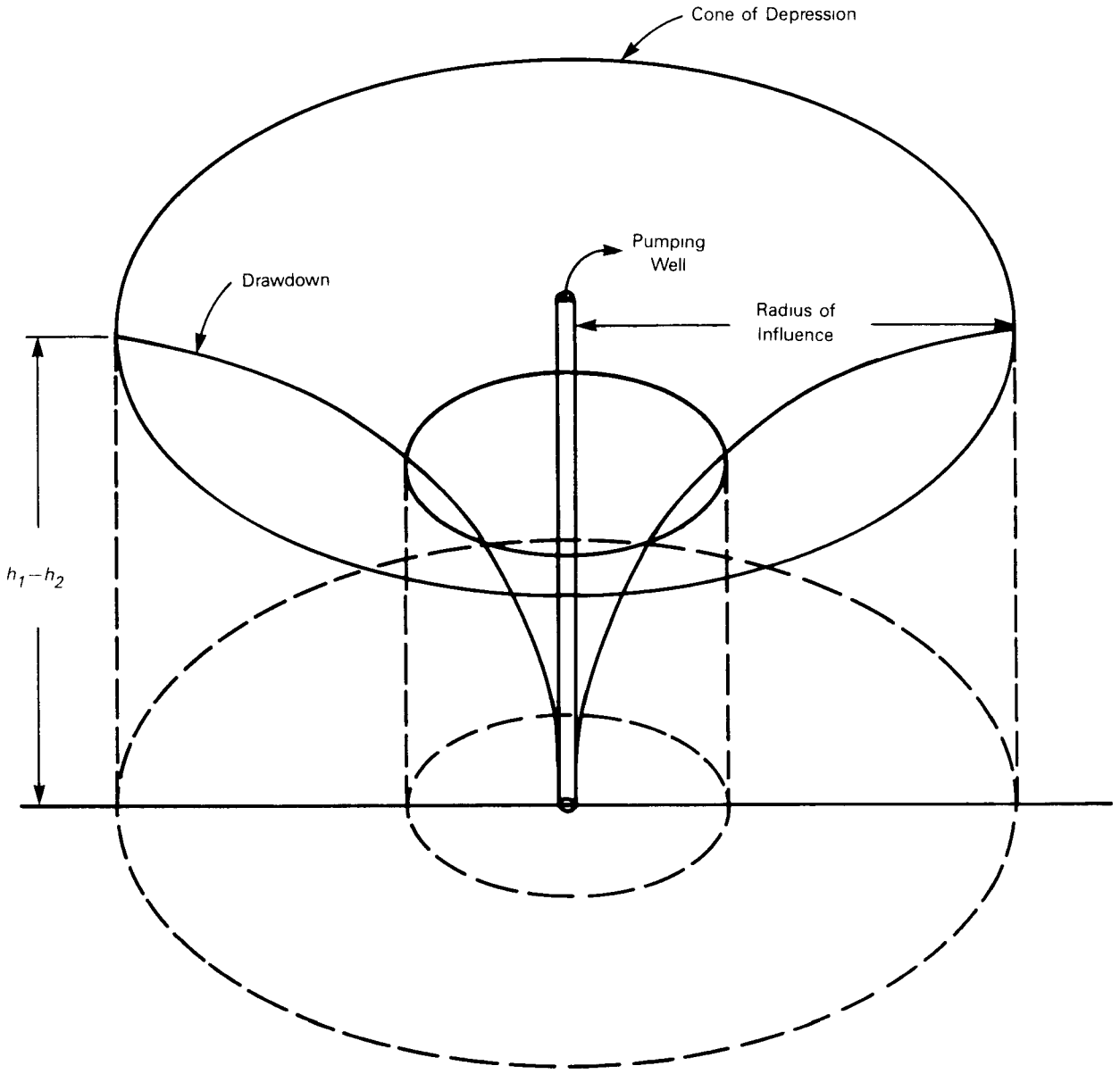


FIGURE 5-3.
EFFECT OF STORAGE AND TRANSMISSIVITY ON THE SHAPE OF THE CONE OF DEPRESSION (BUREAU OF RECLAMATION, 1977)

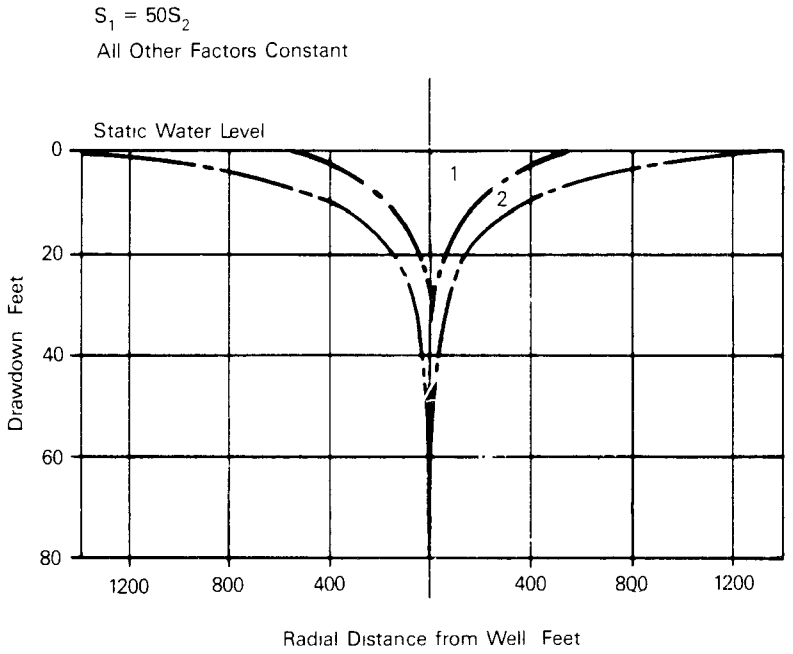


Figure 5-3a. Influence of storativity on drawdown in a well.

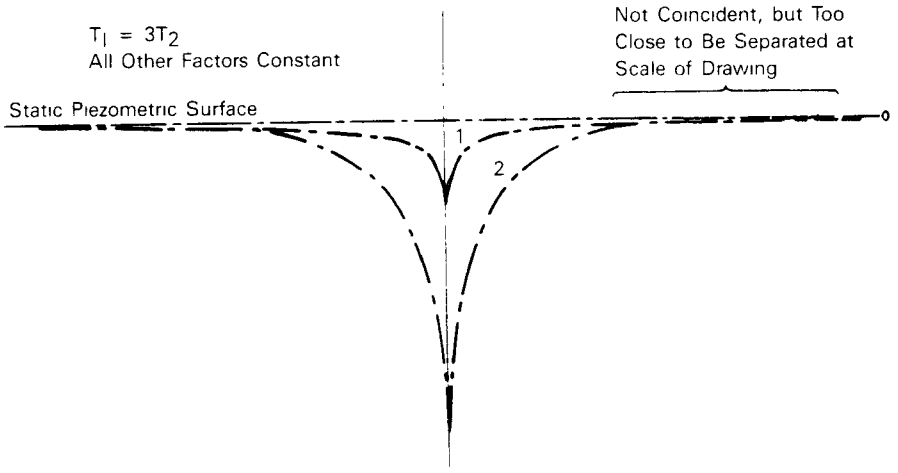


Figure 5-3b. Influence of transmissivity on drawdown in a well.

have been obtained, predictions can be made which are critical in designing and constructing effective pumping systems for plume management. Some of the predictions which can be made are (Johnson Division, UOP Inc., 1975):

- Specific capacity of wells with differing diameters
- Drawdown of an aquifer at varying distances from the well
- Drawdowns with varying pumping rates and times.

In plume management the radius of influence (i.e., the extent of the cone of depression) and the resulting change to the groundwater volocity will be the most important unknowns. By controlling the radius of influence, the flow of contaminants in the groundwater system can be controlled.

5.2.2 Equilibrium Well Formula

C. Theim and P. Forchheimer independently derived equations for equilibrium (steady state) radial flow to wells under confined and unconfined aquifer conditions. Both equations assume recharge at the edge of the cone so that the cones dimensions remain constant as long as pumping rates remain constant.

The basic formula for an <u>unconfined</u> aquifer (Figure 5-4) is:

$$H^2 - h_w^2 = (Q/\pi K) \ln (R_o/r_w)$$

where:

H = saturated thickness of the aquifer (ft)

h_w = height of water in the well (ft)

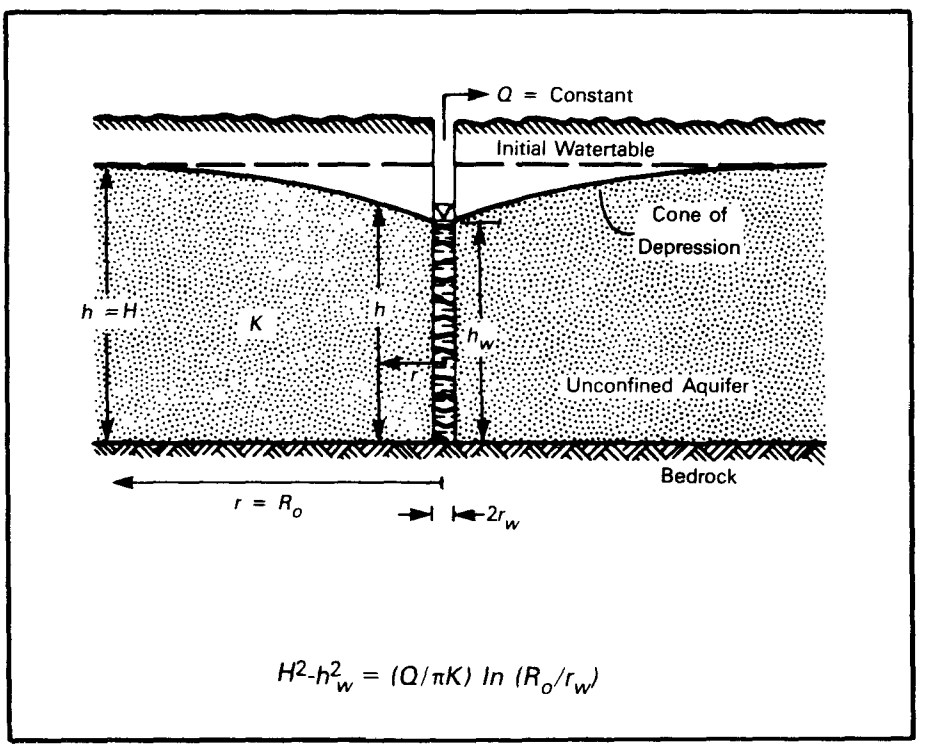
Q = pumping rate (ft^3/day)

K = hydraulic conductivity (ft/day)

R_o = radius of influence of the cone of depression (ft)

r_w = radius of the well (ft).

FIGURE 5-4.
UNCONFINED AQUIFER FLOW
(DAVIS AND DeWIEST, 1966)



The height (h) of the water table at any distance r from the well where r is greater than 1.5H, can be estimated by:

$$h = (H^2-(Q/\pi K)\ln (R_0/r_w))^{0.5}$$

For solutions to h where r is less than 1.5H, the empirical relationships developed by Boreli can be applied. Drawdown (H-h) close to a water table well can be estimated for ratios of r/H between 0.3 and 1.5 by (Powers, 1981):

$$H-h = Q(0.13 \ln (R_0/r))(\ln 10(R_0/H))/\pi KH$$

and for ratios less than 0.3 by:

$$H-h = Q(0.13 \ln(R_0/r) - 0.0123 \ln^2(R_0/10r))(\ln (10R_0/H))/\pi KH$$

Equations for drawndown resulting from a well pumping in a confined aquifer are simpler than those for an unconfined aquifer. Drawdown (H-h) at any distance r from the well in a confined aquifer (Figure 5-5) is given by:

$$H-h = (Q/2\pi T) \ln (R_0/r_w)$$

Plots of H-h versus r on semilogarithmic paper can be developed for both the unconfined and confined conditions, so that predictions of R_0 and the wetted screen length can be made. Figure 5-6 shows two such plots.

FIGURE 5-5.
CONFINED AQUIFER FLOW
(DAVIS AND DeWIEST, 1966)

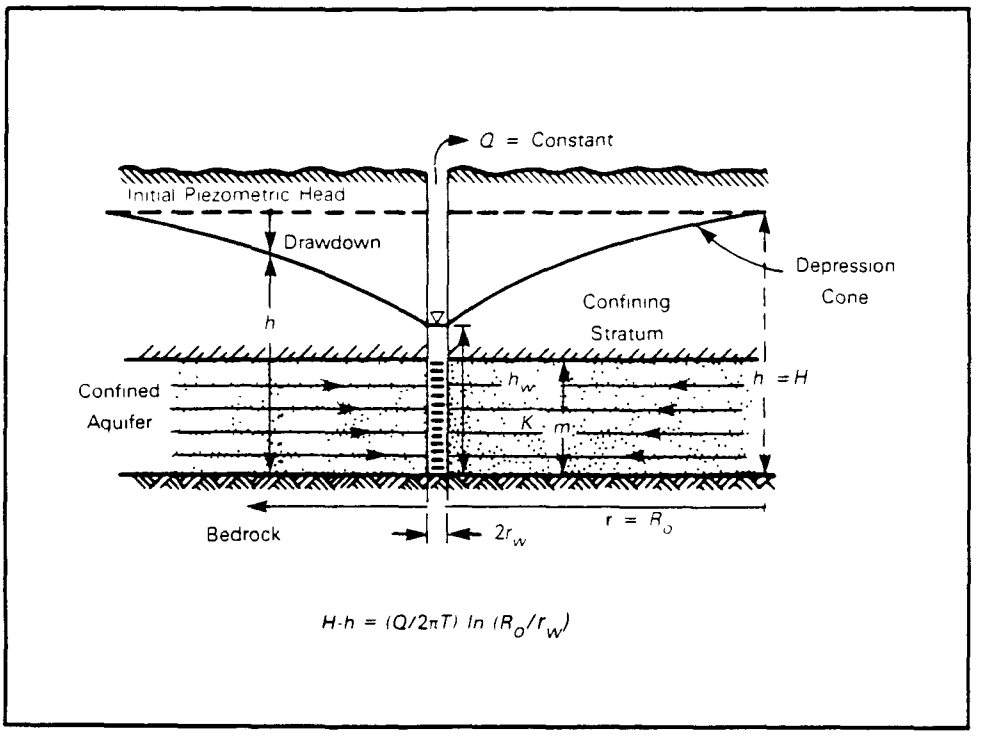


FIGURE 5-6.
PLOTS OF H-h VERSUS r FOR UNCONFINED AND CONFINED AQUIFERS (POWERS, 1981)

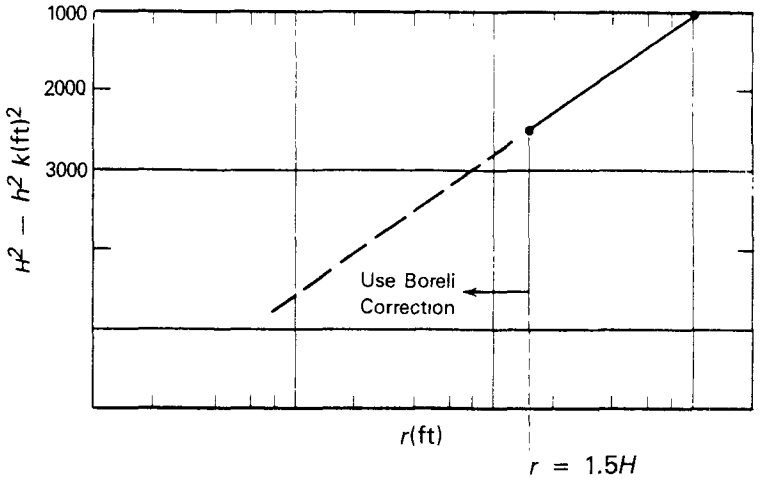


Figure 5-6a. Equilibrium plot for a water table aquifer. Q = 500 gpm. R_o = 1000 ft. K = 300 gpd/ft.^{2} H = 100 ft. r_w = 0.5 ft.

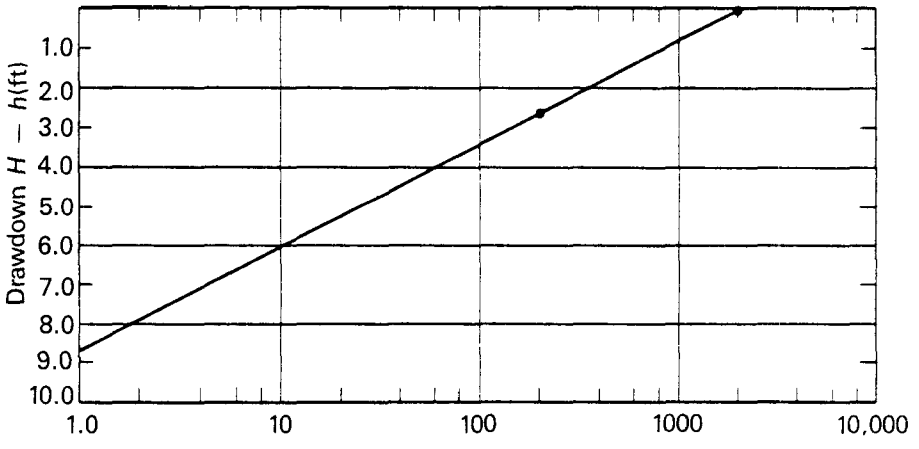


Figure 5-6b. Equilibrium plot for a confined aquifer. Q = 500 gpm. K = 300 gpd/ft.^{2} m = 100 ft. R_o = 2000 ft. r_w = 0.5 ft.

The equilibrium equations presented here are based on the simplifying assumptions presented in Table 5-1. Utilization of these equations appears to be severely limited by the simplifying assumptions, but in practice, the assumptions customarily do not limit the equation's use.

5.2.3 Non-Equilibrium Well Formula

Non-equilibrium well formulas take into account the effects of varying rates of pumping on well yield. Using these equations, drawdown at a specified distance from a well can be predicted for any time after pumping has started assuming the coefficients of storage and transmissivity for a confined aquifer and specific yield for an unconfined aquifer are known. These values are usually determined by pumping tests.

The Theis Equation describes non-equilibrium flow in <u>confined</u> aquifers. This formula is:

$$H-h = (Q/4\pi T)W(\mu)$$

where is given as:

$$\mu = r^2S/4Tt$$

and:

$H-h$ = drawdown (ft)

Q = pumping rate (ft^3/day)

T = coefficient of transmissivity (ft^2/day)

$W(\mu)$ = well function of μ (dimensionless)

r = distance from well center to point where drawdown is measured (ft)

S = coefficient of storage (dimensionless)

t = time elapsed since pumping started (day).

TABLE 5-1. SIMPLIFYING ASSUMPTIONS FOR STEADY STATE EQUATIONS
(Johnson Division, UOP Inc., 1975)

Simplifying Assumption	Practice
The water-bearing materials are of uniform hyraulic conductivity within the radius of influence of the well.	Uniform hydraulic conductivity is seldom found in a real aquifer, but the average hydraulic conductivity as determined from aquifer pumping tests has proved to be reliable for predicting well performance.
The aquifer is not stratified.	For artesian wells where most of the aquifer thickness is penetrated and screened, the assumption of no stratification is not an important limitation. For water-table aquifers, where drawdown reduces the saturated thickness considerably, the situation can be handled when the stratification is known and taken into account in applying the formula.
For a water-table aquifer, the saturated thickness is constant before pumping starts; for an artesian aquifer, the aquifer thickness is constant.	The assumption of constant thickness is not a serious limitation because variations in aquifer thickness within the cone of depression in most real situations is relatively small. Where changes in thickness are important, they can be taken into account.
The pumping well is 100 per cent efficient.	The assumption that the well is 100 per cent efficient can cause the calculated well yield to be seriously in error where the real well may be inefficient as the result of improper design or construction.
Neither the water table nor piezometric surface has any slope; both are horizontal surfaces.	The assumption that the water table or the piezometric surface is horizontal before pumping is never fulfilled. The slope or hydraulic gradient, however, is usually very flat and the effect on calculations of well yield is negligible in most cases. The slope of the water table or the piezometric surface does cause distortion of the cone of depression, making it more elliptical than circular.

(Continued)

TABLE 5-1. (Continued)

Simplifying Assumption	Practice
Laminar flow exists throughout the aquifer and within the radius of influence of the well.	Flow at all places in the aquifer is considered to be laminar. Some investigators have theorized that turbulent flow near a well could result in relatively high head losses. Actual laboratory and field tests, however, shows that some departure from laminar flow near a well causes additional head loss of only minor proportions.
The pumping well penetrates to the bottom of the aquifer.	Other equations given later describe partially penetrating wells.
The cone of depression has reached equilibrium so that both drawdown and the radius of influence of the well do not change with continued time of pumping at a given rate.	Other equations given later describe non-steady state pumping.

For a calculated value of μ, a value of $W(\mu)$ can be obtained from Table 5-2. As $1/\mu$ becomes large, the value of $W(\mu)$ becomes asymptotic and nearly constant. Simplifying assumptions for the Theis formula are essentially the same as for equilibrium formula except the dimensions of the cone of depression need not have reached equilibrium.

A well pumping an unconfined aquifer under non-equilibrium conditions does not follow the Theis equation because water flowing to the well is derived from storage release and specific yield. A solution developed by Numen for determining drawdown in unconfined aquifers that takes into account the dual release mechanism is:

$$H-h = (Q/4\pi T)W(\mu_A,\mu_B,\tau)$$

where $W(\mu_A,\mu_B,\tau)$ are defined as:

$$\mu_A = r^2S/4Tt \quad \text{(for early drawdown)}$$

TABLE 5-2. VALUES OF $W(\mu)$ FOR VARIOUS VALUES OF μ (Ferris et al. as cited by Lohman, 1972)

μ	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
$\times 1$	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
$\times 10^{-1}$	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
$\times 10^{-2}$	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
$\times 10^{-3}$	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
$\times 10^{-4}$	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
$\times 10^{-5}$	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
$\times 10^{-6}$	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
$\times 10^{-7}$	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
$\times 10^{-8}$	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
$\times 10^{-9}$	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
$\times 10^{-10}$	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
$\times 10^{-11}$	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
$\times 10^{-12}$	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
$\times 10^{-13}$	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
$\times 10^{-14}$	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
$\times 10^{-15}$	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

$$\mu_B = r^2 S_y / 4Tt \text{ (for later drawdown)}$$

$$\tau = r^2 K_z / H^2 K_h$$

and the variables not previously defined are:

S_y = specific yield

K_h = horizontal hydraulic conductivity

K_z = vertical hydraulic conductivity.

Once μ_A or μ_B and have been calculated, the function $W(\mu_A, \tau)$ or $W(\mu_B, \tau)$ can be derived from Tables 5-3 and 5-4. Type μ_A values are used for early drawdown times when waters are derived from storage and type μ_B values are used for later drawdown when gravity drainage (specific yield) is predominant. Simplifying assumptions for the formulas are similar to those for the Theis equations.

5.2.4 Semiconfined Aquifers

Generally confined aquifers are not truly confined. They receive vertical recharge through the semipervious layers above or below (when the hydraulic gradient is favorable). Two types of semipervious layers exist; leaky confining layers without storage and leaky confining layers with storage. Figure 5-7 depicts a semiconfined aquifer.

The Hantush-Jacob formula can be applied to leaky confining layers without storage. The basic equation is given by:

$$H-h = (2.3Q/4\pi T)W(\mu, r/B)$$

where:

$$\mu = r^2 S/4Tt$$

$$r/B = r(Tm'/K')^{0.5}$$

and the new variables are defined as:

m' = thickness of the leaky confining layer

K' = hydraulic conductivity of the leaky confining layer.

TABLE 5-3. VALUES OF THE FUNCTION $W(\mu_A,\tau)$ FOR WATER-TABLE AQUIFERS (Fetter, 1980)

$1/\mu_A$	τ = 0.001	τ = 0.01	τ = 0.06	τ = 0.2	τ = 0.6	τ =1.0	τ =2.0	τ =4.0	τ =6.0
4.0×10^{-1}	2.48×10^{-2}	2.41×10^{-2}	2.30×10^{-2}	2.14×10^{-2}	1.88×10^{-2}	1.70×10^{-2}	1.38×10^{-2}	9.33×10^{-3}	6.39×10^{-3}
8.0×10^{-1}	1.45×10^{-1}	1.40×10^{-1}	1.31×10^{-1}	1.19×10^{-1}	9.88×10^{-2}	8.49×10^{-2}	6.03×10^{-2}	3.17×10^{-2}	1.74×10^{-2}
1.4×10^{0}	3.58×10^{-1}	3.45×10^{-1}	3.18×10^{-1}	2.79×10^{-1}	2.17×10^{-1}	1.75×10^{-1}	1.07×10^{-1}	4.45×10^{-2}	2.10×10^{-2}
2.4×10^{0}	6.62×10^{-1}	6.33×10^{-1}	5.70×10^{-1}	4.83×10^{-1}	3.43×10^{-1}	2.56×10^{-1}	1.33×10^{-1}	4.76×10^{-2}	2.14×10^{-2}
4.0×10^{0}	1.02×10^{0}	9.63×10^{-1}	8.49×10^{-1}	6.88×10^{-1}	4.38×10^{-1}	3.00×10^{-1}	1.40×10^{-1}	4.78×10^{-2}	2.15×10^{-2}
8.0×10^{0}	1.57×10^{0}	1.46×10^{0}	1.23×10^{0}	9.18×10^{-1}	4.97×10^{-1}	3.17×10^{-1}	1.41×10^{-1}		
1.4×10^{1}	2.05×10^{0}	1.88×10^{0}	1.51×10^{0}	1.03×10^{0}	5.07×10^{-1}				
2.4×10^{1}	2.52×10^{0}	2.27×10^{0}	1.73×10^{0}	1.07×10^{0}					
4.0×10^{1}	2.97×10^{0}	2.61×10^{0}	1.85×10^{0}	1.08×10^{0}					
8.0×10^{1}	3.56×10^{0}	3.00×10^{0}	1.92×10^{0}						
1.4×10^{2}	4.01×10^{0}	3.23×10^{0}	1.93×10^{0}						
2.4×10^{2}	4.42×10^{0}	3.37×10^{0}	1.94×10^{0}						
4.0×10^{2}	4.77×10^{0}	3.43×10^{0}							
8.0×10^{2}	5.16×10^{0}	3.45×10^{0}							
1.4×10^{3}	5.40×10^{0}	3.46×10^{0}							
2.4×10^{3}	5.54×10^{0}								
4.0×10^{3}	5.59×10^{0}								
8.0×10^{3}	5.62×10^{0}								
1.4×10^{4}	5.62×10^{0}	3.46×10^{0}	1.94×10^{0}	1.08×10^{0}	5.07×10^{-1}	3.17×10^{-1}	1.41×10^{-1}	4.78×10^{-2}	2.15×10^{-2}

TABLE 5-4. VALUES OF THE FUNCTION $W(\mu_B,\tau)$ FOR WATER-TABLE AQUIFERS

$1/\mu_B$	τ = 0.001	τ = 0.001	τ = 0.06	τ =0.2	τ = 0.6	τ = 1.0	τ = 2.0	τ = 4.0	τ = 6.0
4.0×10^{-4}	5.62×10^{0}	3.46×10^{0}	1.94×10^{0}	1.09×10^{0}	5.08×10^{-1}	3.18×10^{-1}	1.42×10^{-1}	4.79×10^{-2}	2.15×10^{-2}
8.0×10^{-4}								4.80×10^{-2}	2.16×10^{-2}
1.4×10^{-3}								4.81×10^{-2}	2.17×10^{-2}
2.4×10^{-3}								4.84×10^{-2}	2.19×10^{-2}
4.0×10^{-3}					5.08×10^{-1}	3.18×10^{-1}	1.42×10^{-1}	4.78×10^{-2}	2.21×10^{-2}
8.0×10^{-3}					5.09×10^{-1}	3.19×10^{-1}	1.43×10^{-1}	4.96×10^{-2}	2.28×10^{-2}
1.4×10^{-2}					5.10×10^{-1}	3.21×10^{-1}	1.45×10^{-1}	5.09×10^{-2}	2.39×10^{-2}
2.4×10^{-2}					5.12×10^{-1}	3.23×10^{-1}	1.47×10^{-1}	5.32×10^{-2}	2.57×10^{-2}
4.0×10^{-2}					5.16×10^{-1}	3.27×10^{-1}	1.52×10^{-1}	5.68×10^{-2}	2.86×10^{-2}
8.0×10^{-2}				1.09×10^{0}	5.24×10^{-1}	3.37×10^{-1}	1.62×10^{-1}	6.61×10^{-2}	3.62×10^{-2}
1.4×10^{-1}			1.94×10^{0}	1.10×10^{0}	5.37×10^{-1}	3.50×10^{-1}	1.78×10^{-1}	8.06×10^{-2}	4.86×10^{-2}
2.4×10^{-1}			1.95×10^{0}	1.11×10^{0}	5.57×10^{-1}	3.74×10^{-1}	2.05×10^{-1}	1.06×10^{-1}	7.14×10^{-2}
4.0×10^{-1}			1.96×10^{0}	1.13×10^{0}	5.89×10^{-1}	4.12×10^{-1}	2.48×10^{-1}	1.49×10^{-1}	1.13×10^{-1}
8.0×10^{-1}	5.62×10^{0}	3.46×10^{0}	1.98×10^{0}	1.18×10^{0}	6.67×10^{-1}	5.06×10^{-1}	3.57×10^{-1}	2.66×10^{-1}	2.31×10^{-1}
1.4×10^{0}	5.63×10^{0}	3.47×10^{0}	2.01×10^{0}	1.24×10^{0}	7.80×10^{-1}	6.42×10^{-1}	5.17×10^{-1}	4.45×10^{-1}	4.19×10^{-1}
2.4×10^{0}	5.63×10^{0}	3.49×10^{0}	2.06×10^{0}	1.35×10^{0}	9.54×10^{-1}	8.50×10^{-1}	7.63×10^{-1}	7.18×10^{-1}	7.03×10^{-1}
4.0×10^{0}	5.63×10^{0}	3.51×10^{0}	2.13×10^{0}	1.50×10^{0}	1.20×10^{0}	1.13×10^{0}	1.08×10^{0}	1.06×10^{0}	1.05×10^{0}
8.0×10^{0}	5.64×10^{0}	3.56×10^{0}	2.31×10^{0}	1.85×10^{0}	1.68×10^{0}	1.65×10^{0}	1.63×10^{0}	1.63×10^{0}	1.63×10^{0}
1.4×10^{1}	5.65×10^{0}	3.63×10^{0}	2.55×10^{0}	2.23×10^{0}	2.15×10^{0}	2.14×10^{0}	2.14×10^{0}	2.14×10^{0}	2.14×10^{0}
2.4×10^{1}	5.67×10^{0}	3.74×10^{0}	2.86×10^{0}	2.68×10^{0}	2.65×10^{0}	2.65×10^{0}	2.64×10^{0}	2.64×10^{0}	2.64×10^{0}
4.0×10^{1}	5.70×10^{0}	3.90×10^{0}	3.24×10^{0}	3.15×10^{0}	3.14×10^{0}	3.14×10^{0}	3.14×10^{0}	3.14×10^{0}	3.14×10^{0}
8.0×10^{1}	5.76×10^{0}	4.22×10^{0}	3.85×10^{0}	3.82×10^{0}	3.82×10^{0}	3.82×10^{0}	3.82×10^{0}	3.82×10^{0}	3.82×10^{0}
1.4×10^{2}	5.85×10^{0}	4.58×10^{0}	4.38×10^{0}	4.37×10^{0}	4.37×10^{0}	4.37×10^{0}	4.37×10^{0}	4.37×10^{0}	4.37×10^{0}

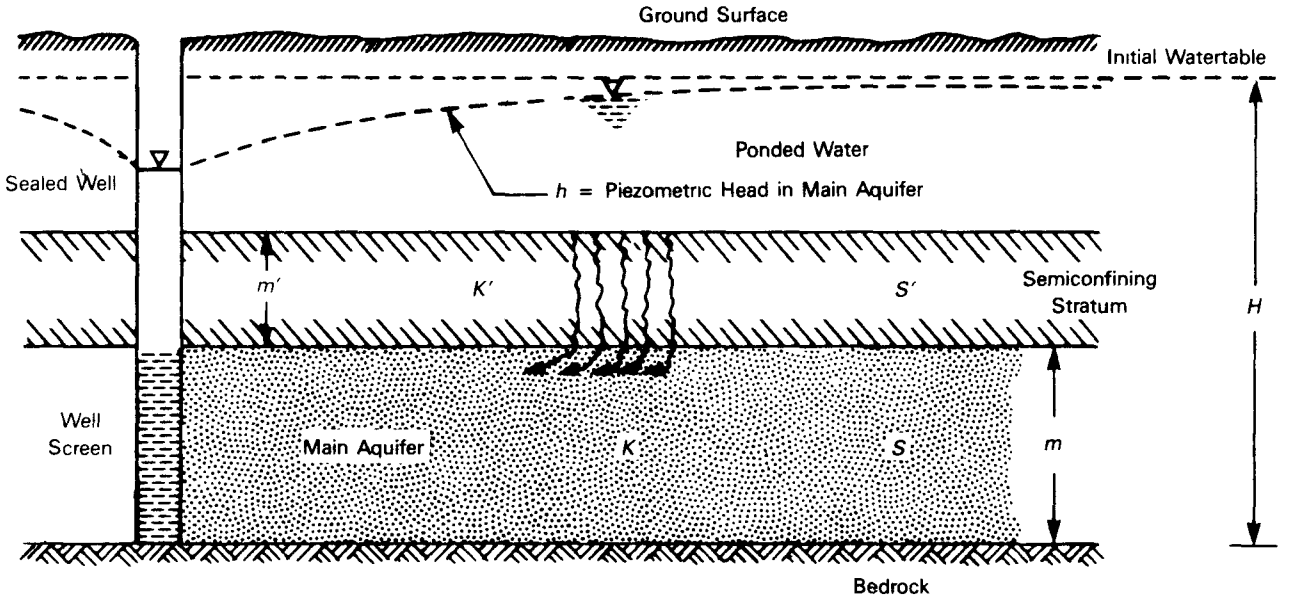
(Continued)

TABLE 5-4. (Continued)

$1/\mu_B$	τ = 0.001	τ = 0.001	τ = 0.06	τ = 0.2	τ = 0.6	τ = 1.0	τ = 2.0	τ = 4.0	τ = 6.0
2.4×10^2	5.99×10^0	5.00×10^0	4.91×10^0	4.91×10^0	4.91×10^0	4.91×10^0	4.91×10^0	4.91×10^0	4.91×10^0
4.0×10^2	6.16×10^0	5.46×10^0	5.42×10^0	5.42×10^0	5.42×10^0	5.42×10^0	5.42×10^0	5.42×10^0	5.42×10^0
8.0×10^2	6.47×10^0	6.11×10^0	6.11×10^0	6.11×10^0	6.11×10^0	6.11×10^0	6.11×10^0	6.11×10^0	6.11×10^0
1.4×10^3	6.67×10^0	6.67×10^0	6.67×10^0	6.67×10^0	6.67×10^0	6.67×10^0	6.67×10^0	6.67×10^0	6.67×10^0
2.4×10^3	7.21×10^0	7.21×10^0	7.21×10^0	7.21×10^0	7.21×10^0	7.21×10^0	7.21×10^0	7.21×10^0	7.21×10^0
4.0×10^3	7.72×10^0	7.72×10^0	7.72×10^0	7.72×10^0	7.72×10^0	7.72×10^0	7.72×10^0	7.72×10^0	7.72×10^0
8.0×10^3	8.41×10^0	8.41×10^0	8.41×10^0	8.41×10^0	8.41×10^0	8.41×10^0	8.41×10^0	8.41×10^0	8.41×10^0
1.4×10^4	8.97×10^0	8.97×10^0	8.97×10^0	8.97×10^0	8.97×10^0	8.97×10^0	8.97×10^0	8.97×10^0	8.97×10^0
2.4×10^4	9.51×10^0	9.51×10^0	9.51×10^0	9.51×10^0	9.51×10^0	9.51×10^0	9.51×10^0	9.51×10^0	9.51×10^0
4.0×10^4	1.94×10^1	1.94×10^1	1.94×10^1	1.94×10^1	1.94×10^1	1.94×10^1	1.94×10^1	1.94×10^1	1.94×10^1

SOURCE: Adapted from S. P. Neuman, Water Resources Research, 11(1975):329-42 as cited by Fetter, 1980.

FIGURE 5-7.
SEMICONFINED AQUIFER
(DAVIS AND DeWIEST, 1966)



Leaky Confining Bed Without Storage

$H\text{-}h = (2.3Q/4\pi T)\ W(\mu, r/B)$

Leaky Confining Bed with Storage

$H\text{-}h = (Q/4\pi T)\ H(\mu, B)$

Values of $W(\mu, r/B)$ for calculated values of μ and r/B are given in Table 5-5. Underlying assumptions for these equations are similar to the Theis formula with the addition of (Fetter, 1980):

- confining bed leakage is vertical and proportional to drawdown
- hydraulic head in the leaking layer is constant
- storage in the confining bed is negligible

TABLE 5-5. VALUES OF FUNCTIONS $W(\mu, r/B)$ AND $W(\mu_A, r/B)$ FOR VARIOUS VALUES OF μ OR μ_A

	r/B																		
μ or μ_A	0.01	0.015	0.03	0.05	0.075	0.10	0.15	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.5	2.0	2.5
0.000001	9.4425	8.6319	7.2471	6.2285	5.4228	4.8541	4.0601	3.5054	2.7449	2.2291	1.8488	1.5550	1.3210	1.1307	0.9735	0.8420	0.4276	0.2278	0.1247
0.000005	9.4413	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
0.00001	9.4176	8.6313	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
0.00005	8.8827	8.4533	7.2450	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
0.0001	8.3983	8.1414	7.2122	6.2282	"	"	"	"	"	"	"	"	"	"	"	"	"	"	"
0.0005	6.9750	6.9152	6.6219	6.0821	5.4062	4.8530	"	"	"	"	"	"	"	"	"	"	"	"	"
0.001	6.3069	6.2765	6.1202	5.7965	5.3078	4.8292	4.0595						"	"	"	"	"	"	"
0.005	4.7212	4.7152	4.6829	4.6084	4.4713	4.2960	3.8821	3.4567	2.7428	2.2290	"	"	"	"	"	"	"	"	"
0.01	4.0356	4.0326	4.0167	3.9795	3.9091	3.8150	3.5725	3.2875	2.7104	2.2253	1.8486	"	"	"	"	"	"	"	"
0.05	2.4675	2.4670	2.4642	2.4576	2.4448	2.4271	2.3776	2.3110	1.9283	1.7075	1.4927	1.2995	1.2995	1.1210	0.9700	0.8409	"	"	"
0.1	1.8227	1.8225	1.8213	1.8184	1.8128	1.8050	1.7829	1.7527	1.6704	1.5644	1.4422	1.3115	1.1791	1.0505	0.9297	0.8190	0.4271	"	"
0.5	0.5598	0.5597	0.5596	0.5594	0.5588	0.5581	0.5561	0.5532	0.5453	0.5344	0.5206	0.5044	0.4860	0.4658	0.4440	0.4210	0.3007	0.1944	0.1174
1.0	0.2194	0.2194	0.2193	0.2193	0.2191	0.2190	0.2186	0.2179	0.2161	0.2135	0.2103	0.2065	0.2020	0.1970	0.1914	0.1855	0.1509	0.1139	0.0803
5.0	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0011	0.0010	0.0010	0.0009

SOURCE: After M. S. Hantush, "Analysis of Data from Pumping Tests in Leaky Aquifers," Transactions, American Geophysical Union, 37 (1956): 702-14 as cited by Fetter, 1980.

- $t > (30r_w^2 S/T)(1-(10r_w/m)^2)$
- $r_w/B < 0.1$.

If significant storage occurs in the confining layer, then flow is derived from storage in the layer during initial time periods. As pumping continues drawdown will occur in the unpumped aquifer. If the confining layer has finite storage, drawdown does not occur in the unpumped layer, and time intervals are short (S'm'/10K'), then the drawndown equation for leaky aquifers with storage becomes:

$$H-h = (Q/4\pi T)H(\mu,\beta)$$

where:

$$\mu = r^2S/4Tt$$

$$\beta = (r/4B)(S'/S)^{0.5}$$

$$B = (Tm'/K')^{0.5}$$

Values at $H(\mu,\beta)$ can be obtained from Table 5-6 after μ and β are calculated.

5.2.5 Partially Penetrating Wells

The equations presented previously assume that the well penetrates the entire thickness of the aquifer. In many cases, wells do not penetrate the total thickness such that true radial flow to the well does not exist. Figure 5-8 shows two examples of flow to partially penetrating wells in confined aquifers. These figures show that water moving from the lower part of an aquifer must move along curved lines to reach the well. The result is increased flow path lengths and increased head loss at the well. Therefore, for a given yield drawdown will be greater in a partially penetrating well than in a fully penetrating well in the same aquifer.

Formulas for partial penetration are difficult to solve under simple conditions (i.e., homogeneous isotropic aquifers) and extremely difficult to solve for complex conditions (i.e., stratified aquifers). Kozeny developed an

TABLE 5-6. VALUES OF THE FUNCTION $H(\mu,\beta)$

	β											
	0.001	0.005	0.01	0.05	0.10	0.20	0.50	1.0	2.0	5.0	10.0	20.0
0.000001	11.9842	10.5908	9.9259	8.3395	7.6497	6.9590	6.0463	5.3575	4.6721	3.7756	3.1110	2.4671
0.000005	10.8958	9.7174	9.0866	7.5284	6.8427	6.1548	5.2459	4.5617	3.8836	3.0055	2.3661	1.7633
0.00001	10.3739	9.3203	8.7142	7.1771	6.4944	5.8085	4.9024	4.2212	3.5481	2.6822	2.0590	1.4816
0.00005	9.0422	8.3171	7.8031	6.3523	5.6821	5.0045	4.1090	3.4394	2.7848	1.9622	1.3943	0.8994
0.0001	8.4258	7.8386	7.3803	5.9906	5.3297	4.6581	3.7700	3.1082	2.4658	1.6704	1.1359	0.6878
0.0005	6.9273	6.6024	6.2934	5.1223	4.4996	3.8527	2.9933	2.3601	1.7604	1.0564	0.6252	0.3089
0.001	6.2624	6.0193	5.7727	4.7290	4.1337	3.5045	2.6650	2.0506	1.4776	0.8271	0.4513	0.1976
0.005	4.6951	4.5786	4.4474	3.7415	3.2483	2.6891	1.9250	1.3767	0.8915	0.4001	0.1677	0.0493
0.01	4.0163	3.9334	3.8374	3.2752	2.8443	2.3325	1.6193	1.1122	0.6775	0.2670	0.0955	0.0221
0.05	2.4590	2.4243	2.3826	2.1007	1.8401	1.4872	0.9540	0.5812	0.2923	0.0755	0.0160	0.00164
0.1	1.8172	1.7949	1.7677	1.5768	1.3893	1.1207	0.6947	0.3970	0.1789	0.0359	0.00552	0.00034
0.5	0.5584	0.5530	0.5463	0.4969	0.4436	0.3591	0.2083	0.1006	0.0325	0.00288	0.00015	
1.0	0.2189	0.2169	0.2144	0.1961	0.1758	0.1427	0.0812	0.0365	0.00993	0.00055	0.00002	
5.0	0.00115	0.00114	0.00112	0.00104	0.00093	0.00076	0.00042	0.00017	0.00003			

SOURCE: Condensed from M. S. Hanfush, "Modification of the Theory of Leaky Aquifers,: Journal of Geophysical Research, 65 (1960): 3713-25 as cited by Fetter, 1980.

FIGURE 5-8.
FLOW TO PARTIALLY PENETRATING WELLS
(BUREAU OF RECLAMATION, 1977)

Figure 5-8a. Just Penetrating Top of Aquifer

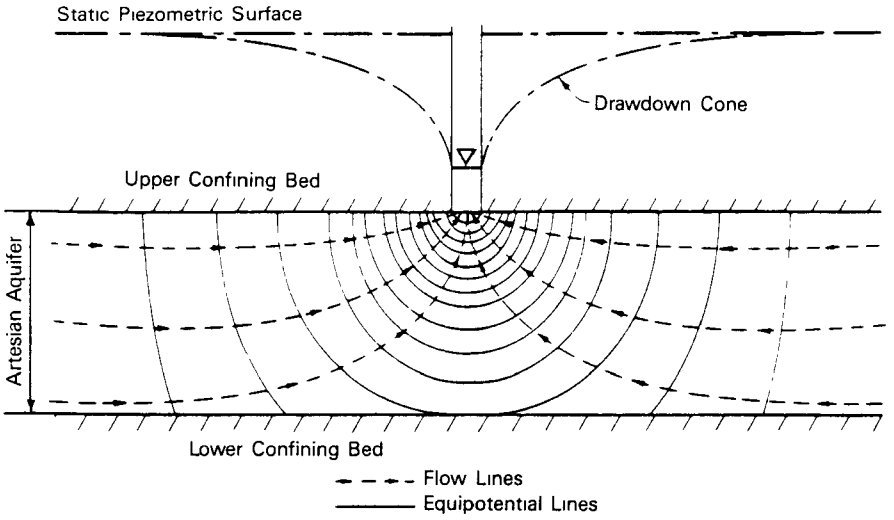
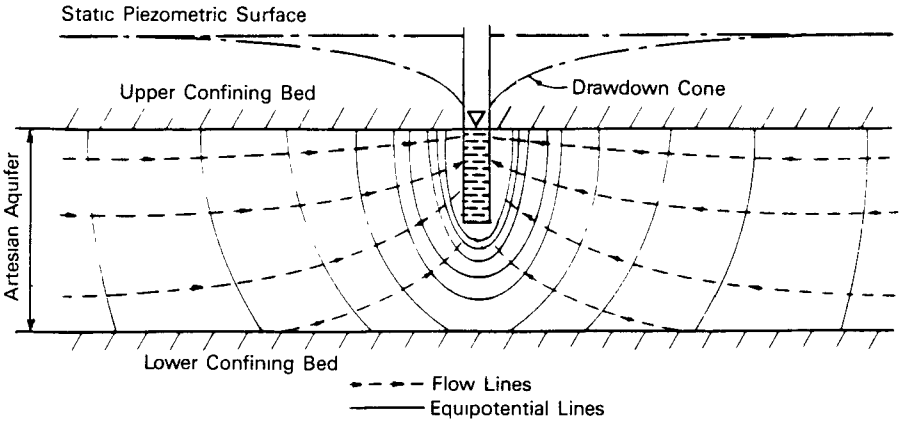


Figure 5-8b. Penetrating 50% of Aquifer



equation for partially penetrating wells in confined aquifers that are nearly homogeneous. His equation can be written as (Johnson Division, UOP Inc., 1975):

$$(Q/s_p)/(Q/s) = l\ (1+7(r_w/2ml)^{0.5}\cos(\pi l/2)$$

where:

Q/s_p = specific capacity of a partially penetrating well (ft/day)

Q/s = specific capacity of a fully penetrating well (ft/day)

r_w = well radius (ft)

m = aquifer thickness (ft)

l = well screen length as a fraction of aquifer thickness (dimensionless)

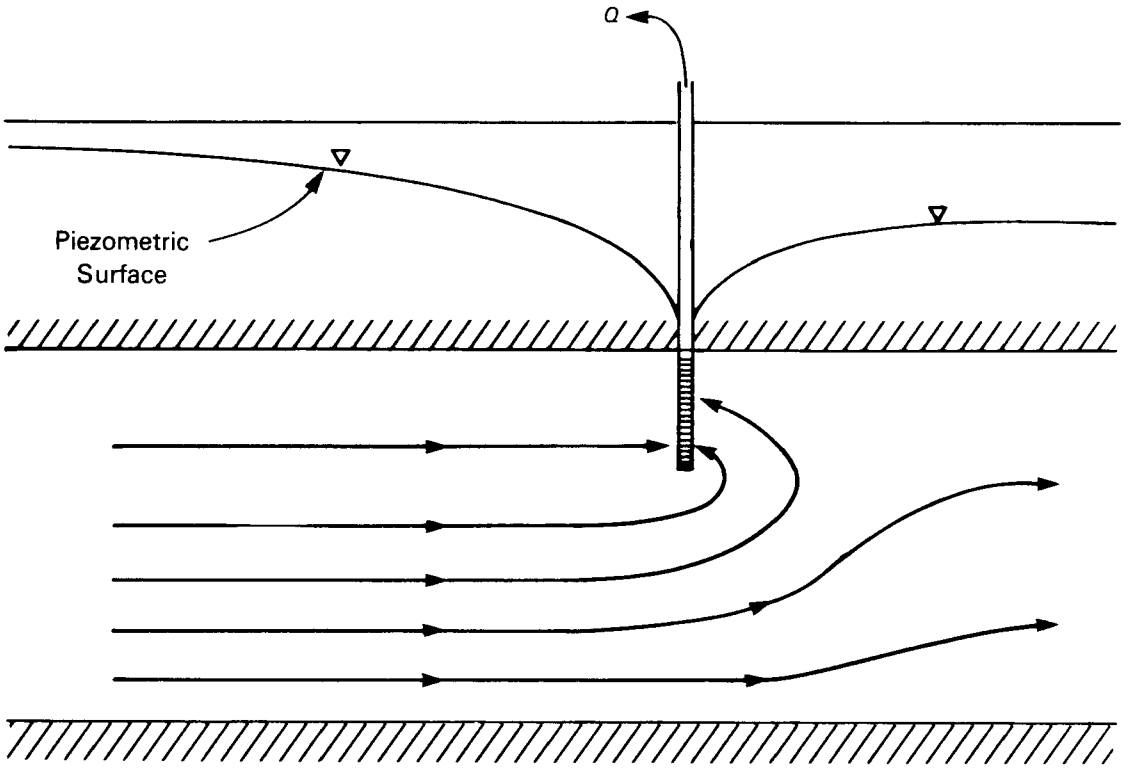
Kozney's formula is valid when the aquifer thickness is large, the percent penetration is small, and the well radius is small.

A major problem associated with the use of partially penetrating wells for the containment of plumes is that if the well does not fully penetrate the aquifer and pumping is not sufficient, contaminants can flow under the well (Figure 5-9). Partially penetrating wells may be most useful in collecting floating contaminants.

5.2.6 Cumulative Drawdown

The formulas presented thus far have been for single wells pumping an aquifer. When more than one well is pumping the same aquifer, their cones of depression will possibly intersect. For containing and removing plumes from an aquifer, this principle is important because it allows for the determination of well spacing intervals, pumping rates, and pumping times. Theoretically, if the cones of depression (radii of influence) intersect each other, flow will not occur between the wells and any contaminants in the groundwater system will be extracted. This theory is probably valid for

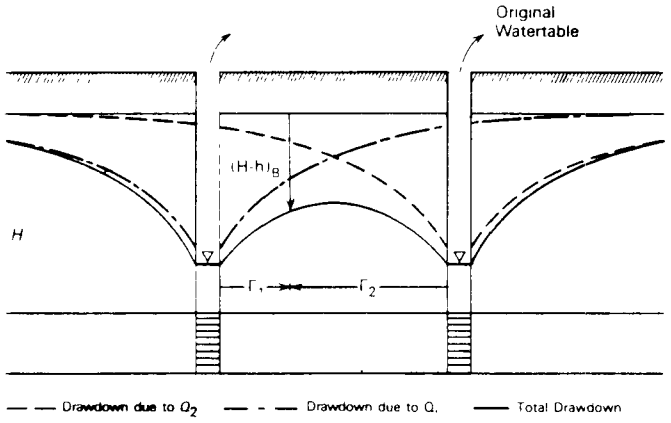
FIGURE 5-9.
FLOW IN CONFINED AQUIFER WITH PARTIALLY PENETRATING WELL



aquifers with low natural flow velocities, however, for aquifers with high natural flow velocities overlapping cones may not be sufficient to capture the plume (Keely and Tsang, 1983).

For confined aquifers with several wells tapping the same system, the composite pumping cone can be calculated by summing the individual drawdowns caused by each well (Figure 5-10). This can be accomplished by graphical superposition (i.e., adding the drawdown curves, H-h, for each well) or by

FIGURE 5-10.
COMPOSITE DRAWDOWN IN A CONFINED AQUIFER
(FREEZE AND CHERRY, 1979 AS CITED BY EPA, 1982a)



calculation. For a system of n wells pumping at any rate, the drawdown at a radius r from each well is given for equilbrium conditions by:

$$H-h = (Q_1 \ln(R_o/r_1)/2\pi T)+(Q_2 \ln(R_o/r_2)/2\pi T).....(Q_n \ln(R_o/r_n)/2\pi T)$$

and for non-equilibrium conditions by:

$$H-h = (Q_1 W(\mu_1)/4\pi T)+(Q_2 W(\mu_2)/4\pi T)....(Q_n W(\mu_n)/4\pi T)$$

where:

$$\mu_i = r_i^2 S/4Tt_i \qquad \text{for } i = 1,2,...n$$

Linear superposition is only valid for confined aquifers because the value of transmissivity does not change with drawdown.

For unconfined aquifers, linear superposition of individual well drawdowns results in predicted composite drawdowns that are less than they will actually be because a decrease in saturated thickness reduces the transmissivity. Composite drawdowns for unconfined aquifers under equilibrium conditions can be calculated from;

$$H^2-h^2 = (Q_1 \ln(R_o/r_1)/\pi K)+(Q_2 \ln(R_o/r_2)/\pi K)....(Q_n \ln(R_o/r_n)/\pi K)$$

Alternatively, composite drawdowns can be derived by graphical superposition using the calculated H^2-h^2 for each well (Figure 5-11). In this case the H^2-h^2 are added and the drawdowns are found by taking square roots.

FIGURE 5-11.
COMPOSITE DRAWDOWN IN AN UNCONFINED AQUIFER
(DAVIS AND DeWIEST, 1966)

Figure 5-11a. Plane View

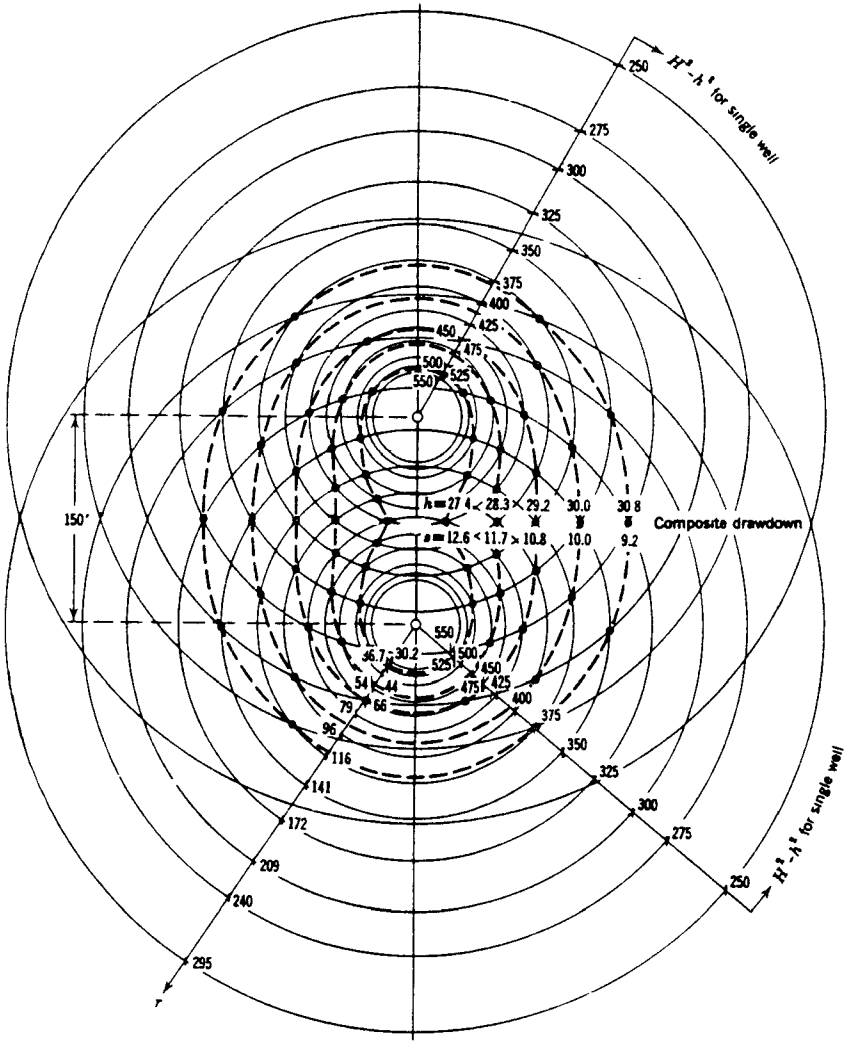
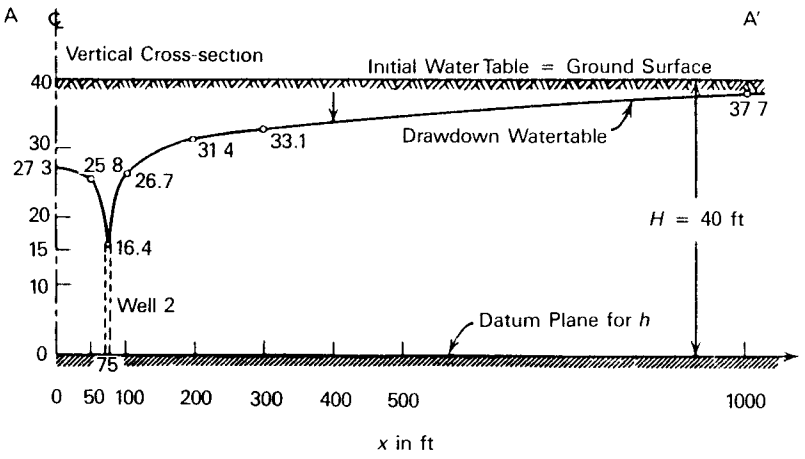


Figure 5-11b. Cross-sectional View



Determining well interference or intersecting cones of depression is a necessary exercise in designing well-field layouts for plume management. For aquifers that have high flow velocities, a velocity plot may also need to be developed to determine well field layout. Based on these analyses the following parameters can be determined:

- Well spacing to contain contaminants
- Length of pipe needed to carry discharge from the well to the treatment plant
- Well pump characteristics (e.g., type, horsepower)
- Time interval for pumping, if not equilibrium pumping.

5.2.7 Hydrogeologic Boundary Effects

Typically, wells are not located in aquifers that have infinite areal extent and therefore drawdown cones extend until they intercept a recharge boundary or a barrier boundary. Recharge boundaries (e.g., streams) are areas where aquifers are replenished with water and barrier boundaries (e.g., impermeable zones) are where aquifers terminate. The effect of the boundary conditions on drawdowns is shown in Figure 5-12. Total drawdown and the rate of drawdown are less than theoretical predictions when recharge boundaries are present and greater when barrier boundaries are present.

Calculating the effect of barriers on wells is usually performed using the method of images. Where recharge boundaries are present, rechargee image wells are constructed and when barrier boundaries are present discharge image wells are utilized. Figure 5-13 shows an example for each type of boundary. In both cases, image wells are drawn about the axis where the boundary occurs. For recharging images, cones of depression are added to obtain the resultant cone and for discharging images, cones of depression are subtracted.

FIGURE 5-12.
EFFECT OF RECHARGE AND BARRIER BOUNDARIES ON DRAWDOWN (FETTER, 1980)

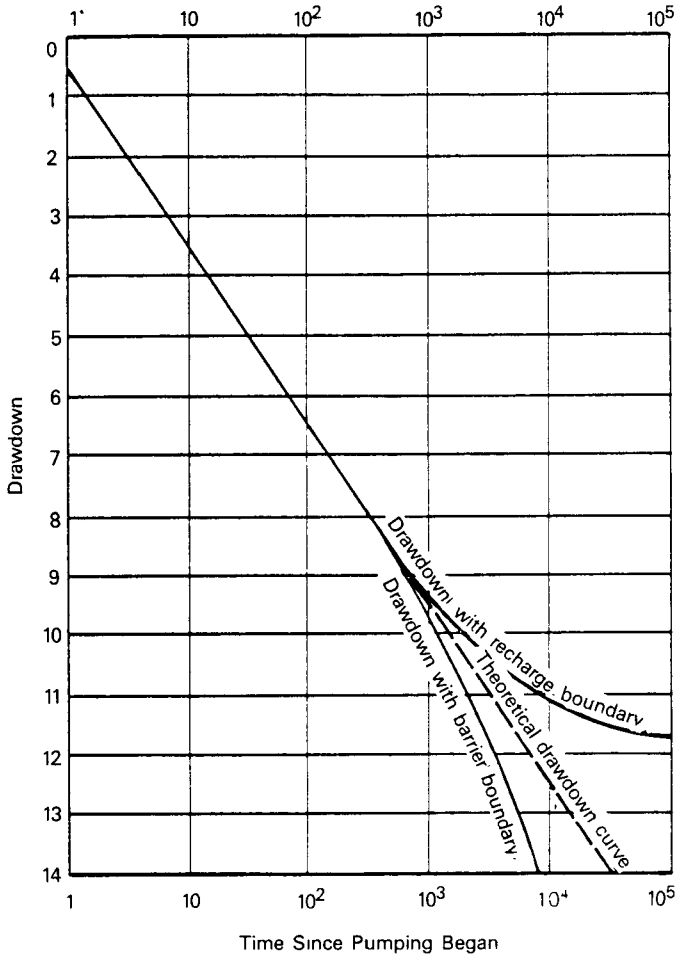
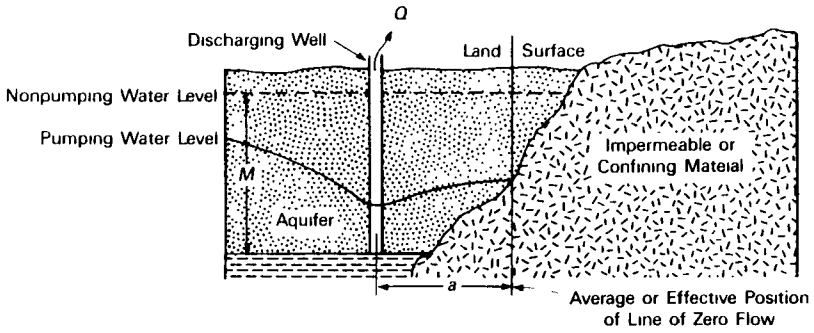
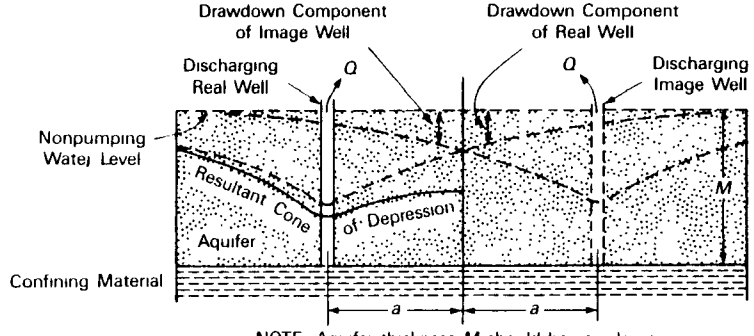


FIGURE 5-13. METHOD OF IMAGES FOR DETERMINING RESULTANT CONE OF DEPRESSION (FERRIS et al., 1962, AS CITED BY LOHMAN, 1972)

Figure 5-13a. Hydraulic Barrier



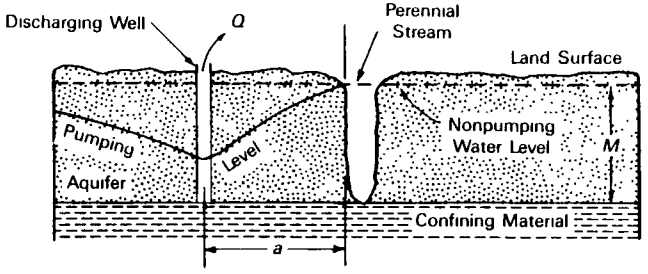
REAL SYSTEM



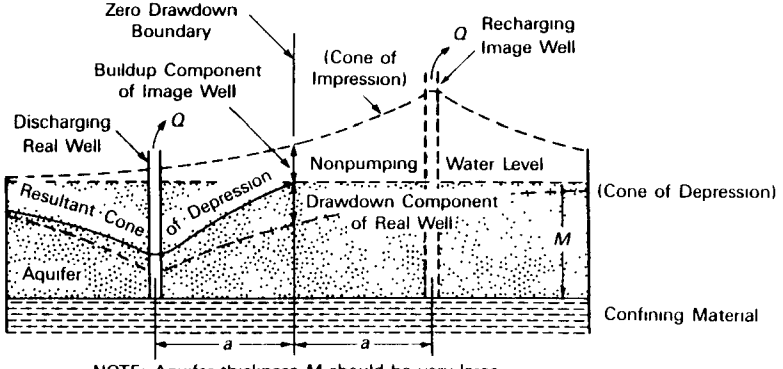
NOTE Aquifer thickness M should be very large compared to resultant drawdown near real well

HYDRAULIC COUNTERPART OF REAL SYSTEM

Figure 5-13b. Recharge Barrier



REAL SYSTEM



NOTE: Aquifer thickness M should be very large compared to resultant drawdown near real well

HYDRAULIC COUNTERPART OF REAL SYSTEM

5.2.8 Flow Between Discharge and Recharge Wells

Injection well theory is identical to the discharge well theory previously described except that cones of depression and drawdowns are inverted to above the water table or piezometric surface and become cones of impression and additional head. The use of recharge (injection) wells in combination with discharging wells allows contaminants to be flushed more rapidly or diverted (i.e., against the normal flow path) to recovery wells. Injection wells can also be utilized to create hydraulic barriers to stop the migration of contaminants, or to change the hydraulic head so that contaminants can be diverted or made to flow against the original flow path.

An example of an extraction and recharge well operating in the same confined aquifer is given in Figure 5-14. In the example, the recharge and discharge wells are pumping at the same rate ($Q_R = Q_I$). To estimate the shape of the piezometric surface formed by pumping these wells, the cone of depression and the cone of impression are plotted and then the two are added. Because the aquifer is confined, the cones can be added directly (refer to the section on cumulative drawdown). This technique cannot be applied directly to unconfined aquifers because the saturated thickness changes resulting in transmissivity changes. Plotting the H^2-h^2 for each well and then superimposing them will result in correct answers for unconfined aquifers.

5.2.9 Radius of Influence

Determining the radius of influence for a well in a given aquifer is critical in remedial action design because it can be used for determining well spacing, pumping rates, pumping cycles, and screen lengths. As can be seen from the equations presented previously, the radius of influence of a well increases as pumping continues until equilibrium conditions are reached (i.e., when aquifer recharge equals the pumping rate or the discharge rate). Recharge can occur through one or more of the following situations (Johnson Division, UOP Inc., 1975):

- Storage release from the aquifer

FIGURE 5-14.
RECHARGE AND DISCHARGE WELLS IN A CONFINED AQUIFER
(DAVIS AND DeWIEST, 1966)

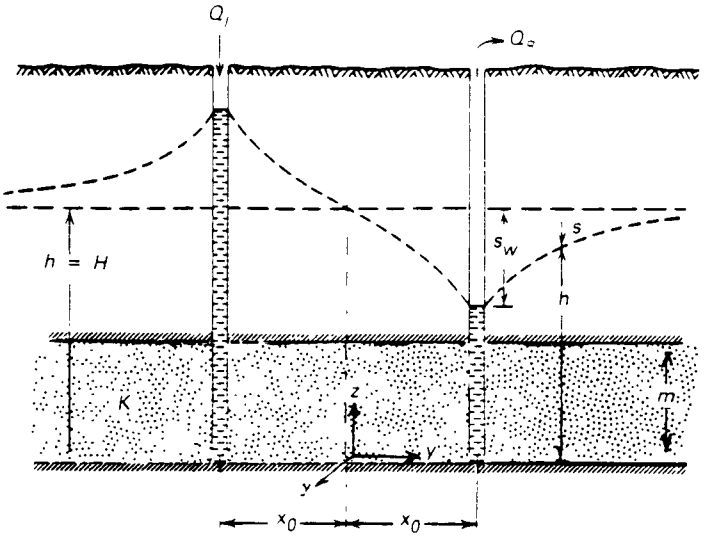


Figure 5-14a. Cross-sectional View

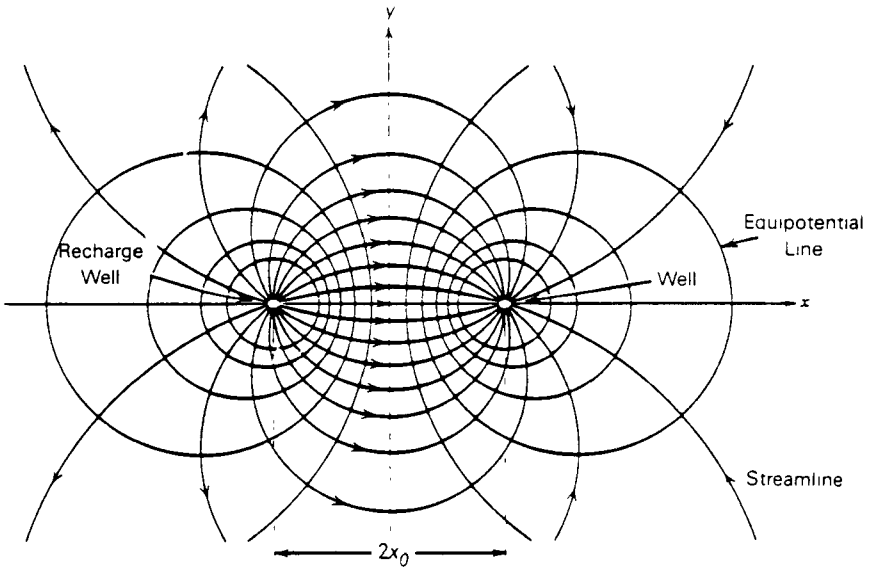


Figure 5-14b. Plane View

- Surface water entering the aquifer
- Recharge from ugradient flow and from precipitation within the radius of influence
- Leakage through overlying or underlying aquifers.

Once equilibrium conditions have been reached, the shape of the cone does not change with increased pumping time (as long as the other factors remain constant). The time required to reach equilibrium conditions after pumping is started can be very short, e.g., within 2 hours, or may take as long as years. Typically, the radius of influence for a confined aquifer is larger than that for an unconfined aquifer and increases as hydraulic conductivity increases.

Estimates of the radius of influence can be made by using several techniques. Generalized values for the radius of influence for various porous media under unconfined conditions are given in Table 5-7. These values are estimates and should be treated as such.

TABLE 5-7. RADIUS OF INFLUENCE FOR VARIOUS UNCONSOLIDATED MATERIALS (i.e., BASED ON TYPICAL HYDRAULIC CONDUCTIVITY RANGES)

Porous Medium	Radius of Influence (R_o) (ft)
Coarse Gravel	5,000
Fine Gravel	1,300-1,650
Coarse Sand	650-1,300
Medium Sand	300-650
Fine Sand	150-300
Very Fine Sand	50-150
Silty Fine Sand	<50

Rough approximations of R_o without recharge can be made by adapting the Jacob formula to:

$$R_o = r_s + (Tt/SC)^{0.5}$$

where:

r_s = well radius or equivalent radius
T = coefficient of transmissivity
t = time
S = coefficient of storage
C = constant (based on the units used).

The above equation is valid only for confined aquifers and the value obtained should be adjusted downward on the basis of possible recharge. Using the above relationship for unconfined aquifers provides reasonable results as long as the drawdown ($H-h_w$) is not a large percentage of the saturated thickness (H).

Sichart and Kyrieleis (1930) developed an empirical relationship for calculating the radius of influence (R_o) as a function of drawdown ($H-h_w$) and hydraulic conductivity (K):

$$R_o = 3\ (H-h_w)(0.47K)^{0.5}$$

where $H-h_w$ is in feet, K is in gpd/ft^2 and R_o is in feet. Theoretically, the radius of influence does not depend on drawdown, but rather, depends on the pumping rate which does not appear in this formula.

For exact solutions, distance-drawdown diagrams can be constructed using the equations given previously. Distance-drawdown diagrams are plotted on semilogarithmic paper with the distance being plotted on the logarithmic scale. Two such diagrams are shown in Figure 5-15. The radius of influence can be obtained from the intercept of the plots with the distance axis at zero drawdown. For cases where pumping rates are variable but all other parameters

FIGURE 5-15.
DISTANCE DRAWDOWN DIAGRAMS FOR A) VARYING PUMPING RATING AND B) VARYING PUMPING TIMES (JOHNSON DIVISION, UOP INC., 1975)

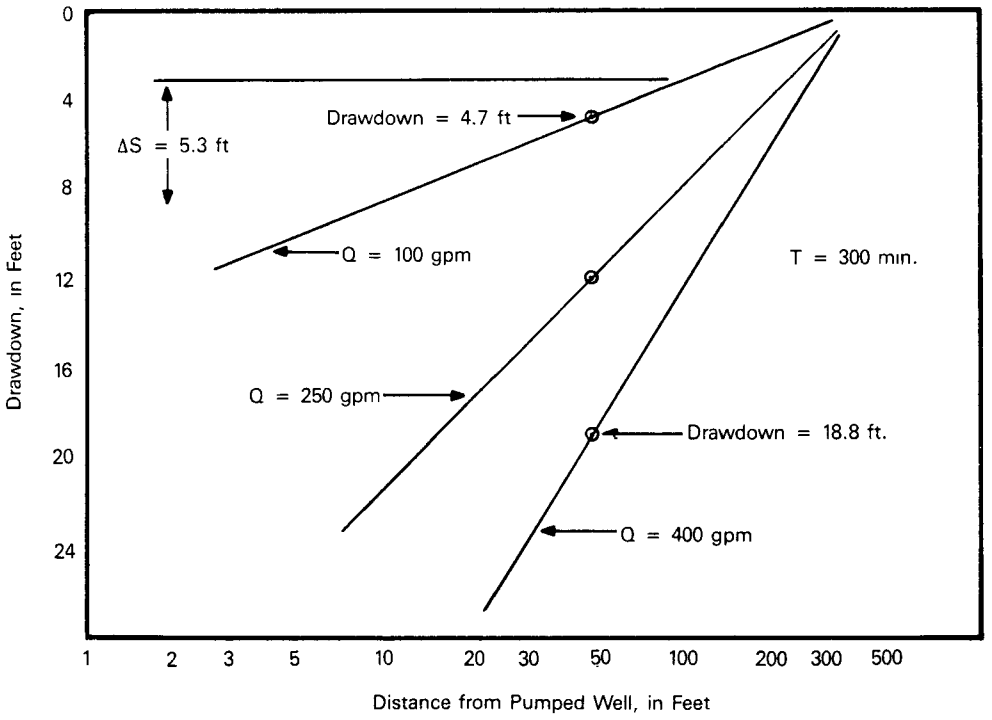


Figure 5-15a. Pumping Rates

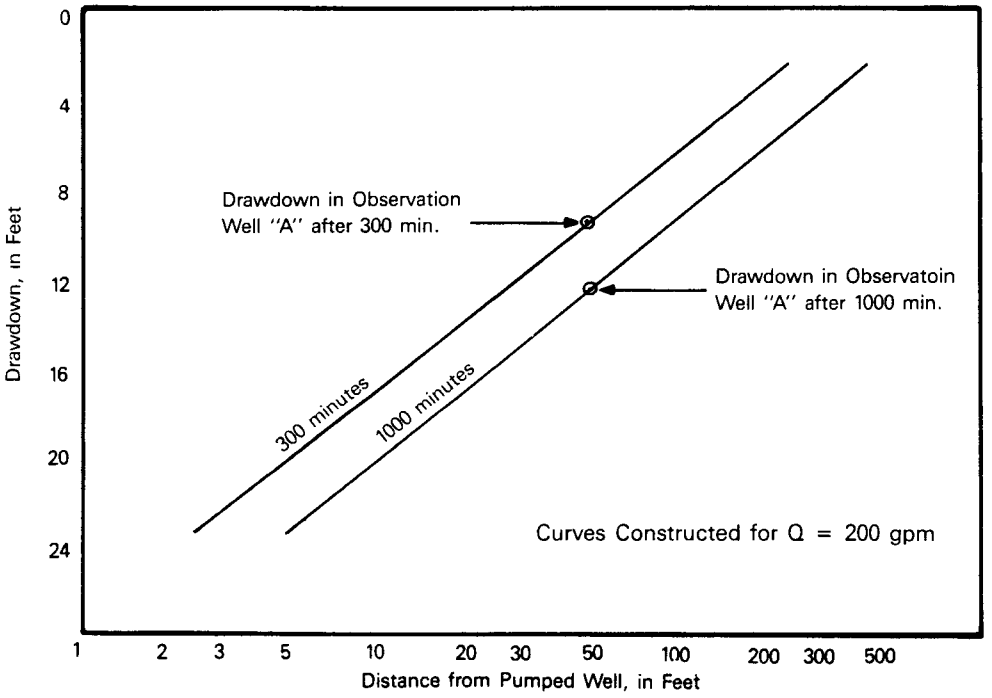


Figure 5-15b. Pumping Times

pumping times are varied, however, the radius of influence is changed such that as pumping time increases, the radius of influence increases. This relationship holds true until equilibrium is reached after which the radius of influence does not increase with continued pumping.

Two useful relationships for understanding the effects of drawdown on the pumping volume and the radius of influence are:

$$Q/H-h = 2\pi T/\ln(R_0/r) = \text{Constant} \quad \text{(for confined aquifers)}$$

and

$$Q/H^2-h^2 = \pi K/\ln(R_0/r) = \text{Constant} \quad \text{(for unconfined aquifers)}$$

Thus, the radius of influence will not change with the pumping rate under equilibrium conditions. The factor that affects the radius of influence greatest under non-equilibrium conditions is the pumping time.

Table 5-8 summarizes the equations that can be used to determine the radius of influence, R_0. An example R_0 calculation for an unconfined aquifer being pumped at steady state conditions is given below.

An unconfined aquifer is pumped at a rate of 500 gpm and the system has reached equilibrium. The aquifer's hydraulic conductivity is 300 gpd/ft^2, drawdown in the well is 35 feet and the saturated thickness is 100 feet. The radius of the pumping well is 0.5 feet. The radius of influence of the well is calculated by:

$$\begin{aligned} \ln R_0 &= (K(H^2-h_w{}^2)/458Q) + \ln r_w \\ &= (300(100^2-65^2) / 458\ (500)) + \ln 0.5 \\ &= 6.872 \\ R_0 &= e^{6.872} \\ &= 965 \text{ feet.} \end{aligned}$$

TABLE 5-8. METHODS FOR CALCULATING THE RADIUS OF INFLUENCE (R_o)

Pumping Condition	Water Table Aquifer (Unconfined)	Confined Aquifer
Equilibrium		
- Exact	$\ln R_o = (K(H^2 - h_w^2)/(458Q)) + \ln r_w$	$\ln R_o = (T(H-h_w)/229Q)) + \ln r_w$
Non-equilibrium		
- Exact	Drawdown vs. log distance plots or Theis Method	
Approximate	$R_o = r_w + (Tt/4790S)^{0.5}$	
	$R_o = 3(H-h_w)(0.47K)^{0.5}$	

R_o = Radius of influence (ft)
K = Hydraulic conductivity (gpd/ft^2)
H = Total head (ft)
h_w = Head in well (f)
Q = Pumping rate (gpm)
r_w = Well radius (ft)
T = Transmissivity (gpd/ft)
t = Time (min)
S = Storage coefficient (dimensionless)

5.3 Applications

This section briefly describes the applications of well systems for plume management. The general types of wells that can be used are wellpoints, suction wells, ejector wells, and deep wells, which can fully penetrate or partially penetrate the aquifer. Well placement and use (i.e., extraction or injection) is highly variable and depends on the hydrogeologic characteristics of the site and the plume's characteristics. Well systems can be designed to perform almost any function from plume containment to plume removal with or without the assistance of other technologies (e.g., slurry walls). The applications for which well systems are typically designed are:

- Groundwater level adjustments
- Containment of plumes to prevent their migration
- Removal of plumes from the groundwater system.

5.3.1 Groundwater Level Adjustment

Well systems for adjusting groundwater levels can be designed using extraction wells to lower water levels or using injection wells to create groundwater mounds (i.e., barriers). By adjusting groundwater levels, plume development can be stopped at the source (e.g., lower water table under a site) or the speed and direction of the plume can be altered. In either case, contaminated water is not extracted from the groundwater system as in the case with containment and removal techniques described in following sections. Groundwater level adjustments are not usually practical, however, because of problems associated with operation and design so as not to allow contaminants to escape or be misdirected. Also, contaminated water may inadvertently be extracted if care is not taken during adjustments.

Plume development can be controlled at sites where the water table intercepts disposed wastes by lowering the water table with extraction wells. In order for the pumping technique to be effective, infiltration into the waste pile must be eliminated and liquid wastes must be completely removed. If these conditions are not met, the potential exists for development of a

contamination plume. The major drawback to using well systems for lowering water tables is the continued costs associated with the maintenance of the system after site closure.

Groundwater barriers can be created using injection wells to change both the direction of the plume and the speed of plume migration. Figure 5-16 shows an example of plume diversion using a line of injection wells to protect domestic water sources. By creating an area with a higher hydraulic head, the plume can be forced to change direction. This technique may be desirable when short term diversions are needed or when the plume will have sufficient time to naturally degrade so that containment and removal is not required.

5.3.2 Plume Containment

Well systems can be used to contain a plume's migration (i.e., prevent further migration and contamination) using extraction wells or extraction and injection wells in combination. Containment differs from removal in that the source of contamination is not stopped and groundwater contamination is an ongoing process. Because containment requires removing contaminated groundwater, a treatment or disposal method must also be developed.

Figure 5-17 depicts the use of a line of extraction wells to contain a plume and protect a domestic water supply. The distance between wells is determined such that the plume is captured, i.e., their radii of influence overlapped for low natural flow velocities or velocity plots developed for high natural flow velocity, thus preventing contaminated groundwater from flowing between adjacent wells. Wells do not necessarily have to be drilled in front of the plume's leading edge but can be drilled behind it as long as the edge is contained within the stagnation zone developed by the pumping well.

Extraction and injection well systems can also be designed to contain a plume. In this type of system, injection wells are used to direct contaminants to extraction wells. Injection well water can be derived from treated extraction well water thus allowing for a convenient disposal method. This

FIGURE 5-16.
PLUME DIVERSION USING INJECTION WELLS

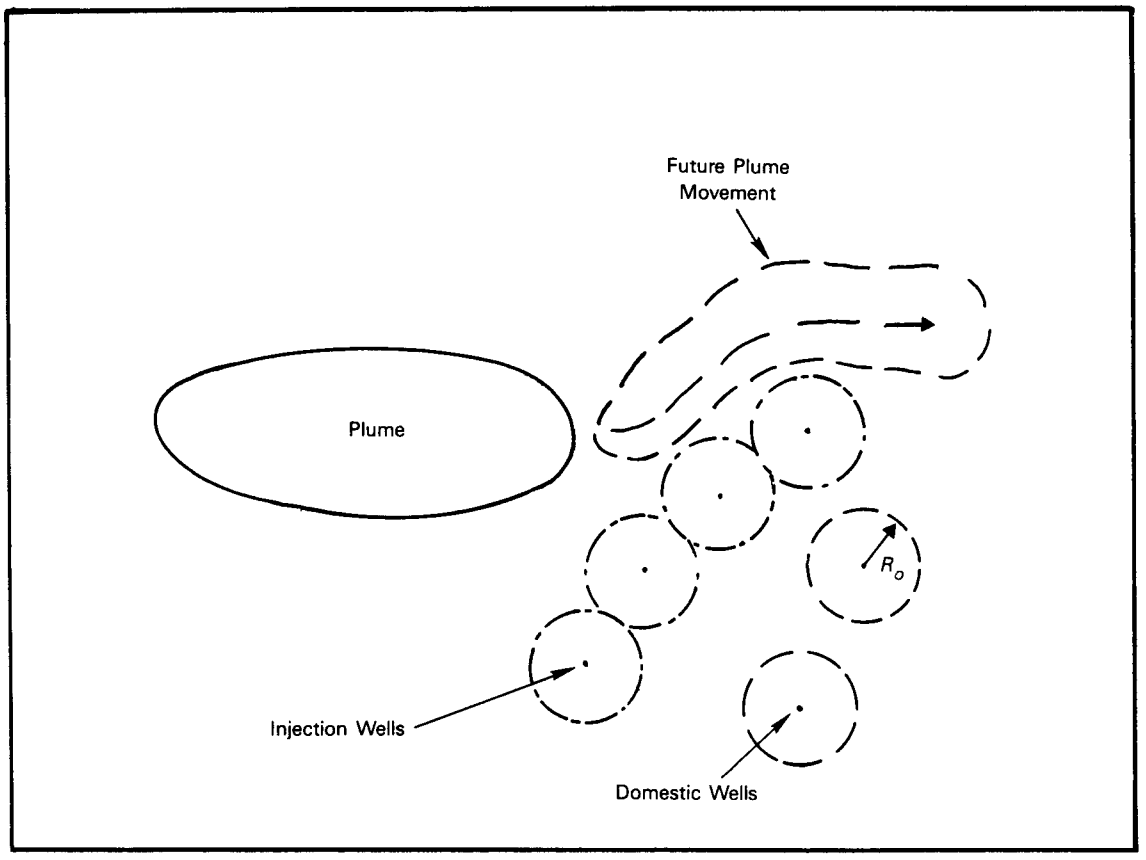


FIGURE 5-17.
CONTAINMENT USING EXTRACTION WELLS

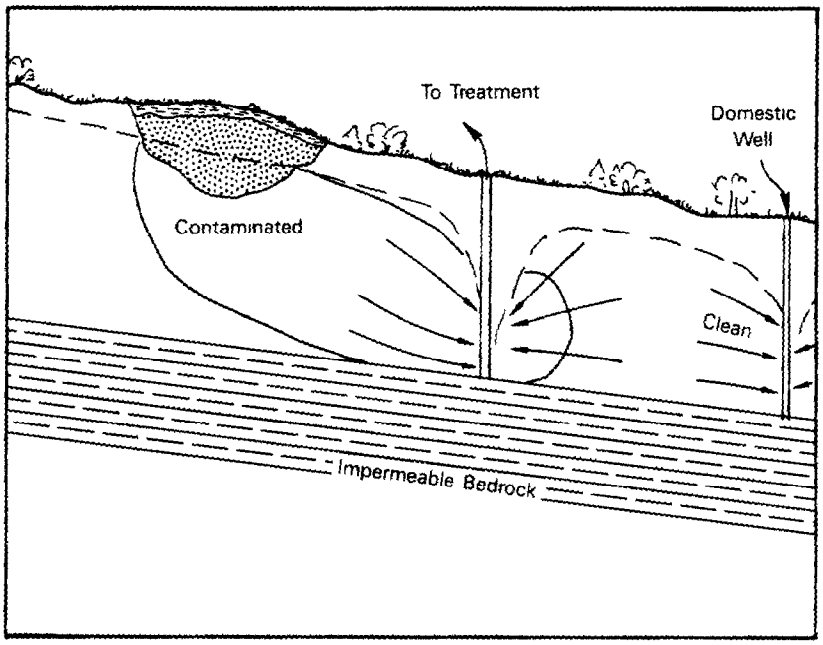


Figure 5-17a. Cross-sectional View

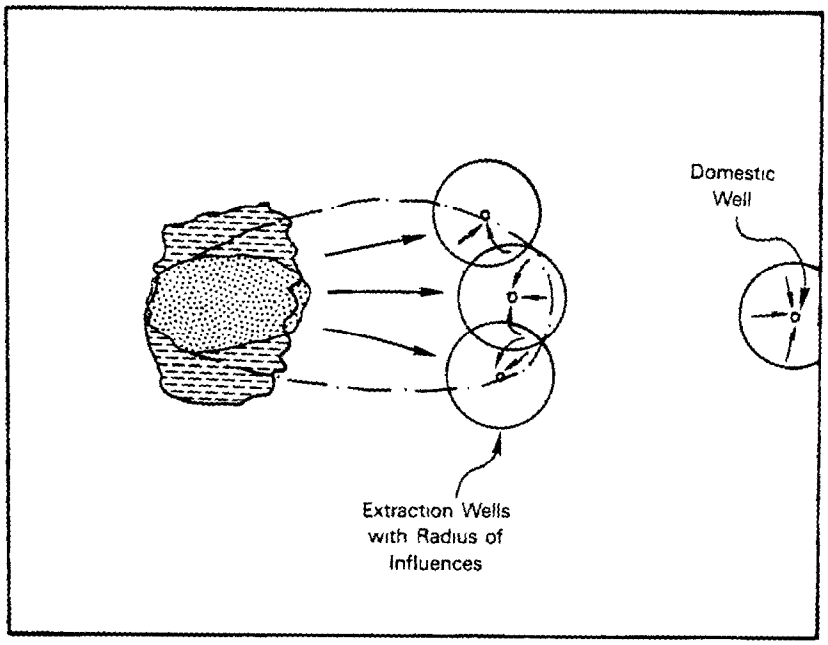


Figure 5-17b. Plane View

type of system may be most advantageous in situations where complete dewatering of a local groundwater system is not desired.

Well systems can also be used in conjunction with other plume management technologies. The use of wells with barrier walls, for instance, is potentially the most useful technology combination. However, the barrier wall used would have to be designed to be compatible with the waste so that wall degradation would not occur.

5.3.3 Plume Removal

Plume removal implies a complete purging of the groundwater system to remove contaminants. Plume removal techniques are utilized after contaminant sources have been fixed, encapsulated, or removed, and aquifer restoration is desired. Plumes of this type may originate from spills, pipeline breaks (e.g., gas line), or other accidental discharges. Extraction, and extraction and injection well systems can be used in plume removal. As with containment, plume removal typically requires treatment of the extracted groundwater before disposal.

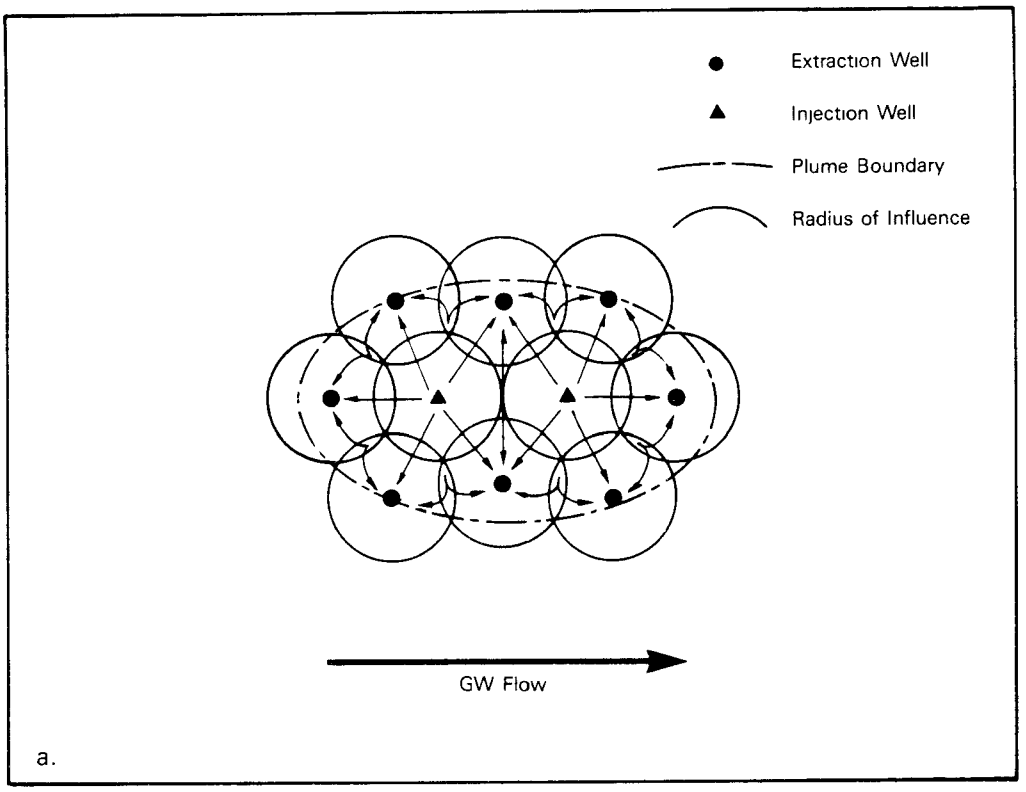
Plume removal through the use of extraction wells is most appropriate where:

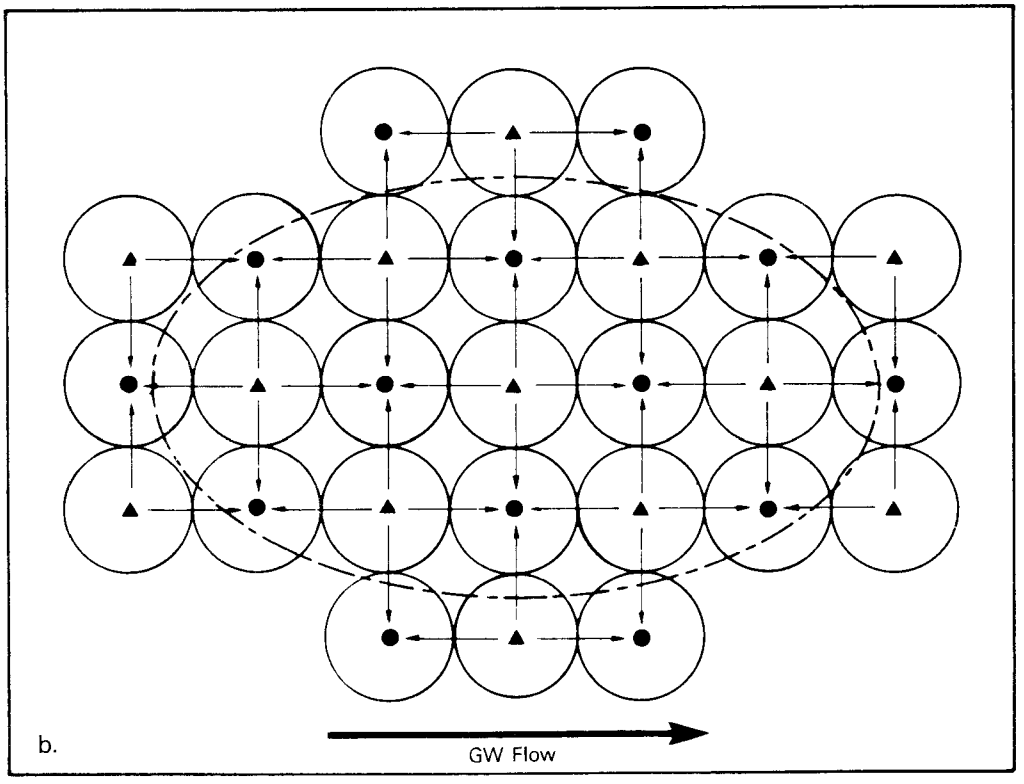
- Hydraulic gradients are steep (i.e., >0.2 ft/ft)
- Contaminants are flowing with velocities equal to or greater than the groundwater
- Removal times can be long (i.e., quick removal is not necessary).

As with containment, wells are placed downgradient of the plume near the plume's leading edge with their radii of influence or capture zones overlapping so as not to allow contaminant escape.

Extraction and injection well systems may be designed to remove contaminants from the groundwater as shown in Figure 5-18. The major difference

FIGURE 5-18.
EXTRACTION AND INJECTION WELLS PATTERNS
FOR PLUME REMOVAL





between the two removal plans is the placement of wells in relationship to each other. In Figure 5-18a, extraction wells surround the injection wells in a staggered array. In Figure 5-18b, extraction wells alternate with injection wells in straight lines. Both methods are effective in plume removal and the choice of pattern would depend on the size of the plume to be removed and required number of wells. However, dead spots (i.e., areas where water movement is very slow or nonexistent) can occur when these configurations are used. The size of a dead spot is directly related to the amount of overlap between adjacent radii of influence (i.e., the greater the overlaps the smaller the dead spots). The system's efficiency should be monitored in these dead spots. Another problem with extraction and injection systems is operational problems associated with injection wells. These wells can suffer from many operation problems including air locks, and frequent maintenance and well rehabilitation.

Extraction and injection systems are most appropriate where:

- Hydraulic gradients are low (i.e., <0.2 ft/ft)
- Contaminants are flowing at velocities less than that of the groundwater
- Required removal time is short.

A special case where these techniques are especially applicable is with contaminants that move slowly or cling to geologic materials. Extraction and injection well systems can also be designed to allow for flushing (e.g., using surfactants) and recovering contaminants that could not be recovered using normal pumping methods.

5.4 Design and Construction

5.4.1 Well Design

Designing the most appropriate type of well and well system requires having adequate information on the site's hydrogeology and the plume's

characteristics. The types of data that are typically required are presented in Table 5-9. As with any design program, the more accurate the data the more likely the system designed will perform its intended function.

There are four basic well types that can be used in plume management; wellpoints, suction wells, ejector wells, and deep wells. Using the site data, selection of an appropriate well type can be made. Table 5-10 lists the conditions under which these well types would be most applicable to a particular site.

Wellpoint systems are the most versatile method, being effective in almost any hydraulic situation. They are best suited for shallow aquifers where extraction is not needed below more than 15 to 20 feet. Beyond this depth suction lifting, the standard pumping technique for wellpoints, is ineffective. Suction wells operate in a similar fashion and are also depth limited. The only advantage of suction wells over wellpoints is that they have higher capacities. For extraction depths greater than 20 feet, deep wells and ejector wells are typically utilized. Deep well systems are better suited to homogeneous aquifers with high hydraulic conductivities and where large volumes of water may be pumped. Ejector wells perform better than deep wells in heterogeneous aquifers with low hydraulic conductivities. A problem with ejector systems is that they are inefficient and are sensitive to constituents in the groundwater which may cause chemical precipitates and well clogging (Powers, 1981).

The following section gives descriptions of the basic components of wells and the design factors in developing a system. Wellpoints and suction wells have been combined because of the similarities between the two.

5.4.1.1 Deep Wells

The typical components of a deep well are a screen, casing, filter pack and seal, and turbine pump. Motors for pumps can be submersible and attached to the pump, or at ground level driving the pump with a shaft. Components of a typical deep well are shown in Figure 5-19 and described in Table 5-11. Not

TABLE 5-9. DATA REQUIREMENTS FOR WELL SYSTEM DESIGN

Geology (consolidated and unconsolidated materials)

- Structure
- Stratigraphy
- Lithology

Aquifer

- Types (i.e., confined, unconfined, perched)
- Thicknesses, depths, and formational designations
- Boundaries
- Hydraulic conductivities, storativities, transmissivities
- Discharge and recharge zones
- Groundwater and surface water relationships
- Locations of existing wells
- Pump test data (e.g., drawdowns, pumping rates, times)

Plume

- Areal extent and depth
- Types of contaminants
- Location in aquifer

TABLE 5-10. CRITERIA FOR WELL SELECTION (Powers, 1981)

Parameters	Wellpoints	Suction Wells	Ejector Wells	Deep Wells
Hydrology				
• Low hydraulic conductivities (e.g., silty or clayey sands)	Good	Poor	Good	Fair to Poor
• High hydraulic conductivities (e.g., clean sands and gravel)	Good	Good	Poor	Good
• Heterogeneous materials (e.g., stratified soils)	Good	Poor	Good	Fair to Poor
• Proximate recharge	Good	Poor	Good to Fair	Poor
• Remote recharge	Good	Good	Good	Good
Depth of Well	Shallow <20 ft	Shallow <20 ft	Deep >20 ft	Deep >20 ft
Normal Spacing	5 - 10 ft	20 - 40 ft	10 - 20 ft	>50 ft
Normal Range of Capacity (per unit)	0.1 - 25 gpm	50 -400 gpm	0.1 -40 gpm	25 - 3000 gpm
Efficiency	Good	Good	Poor	Fair

FIGURE 5-19.
COMPONENTS OF TYPICAL DEEP WELL
(POWERS, 1981)

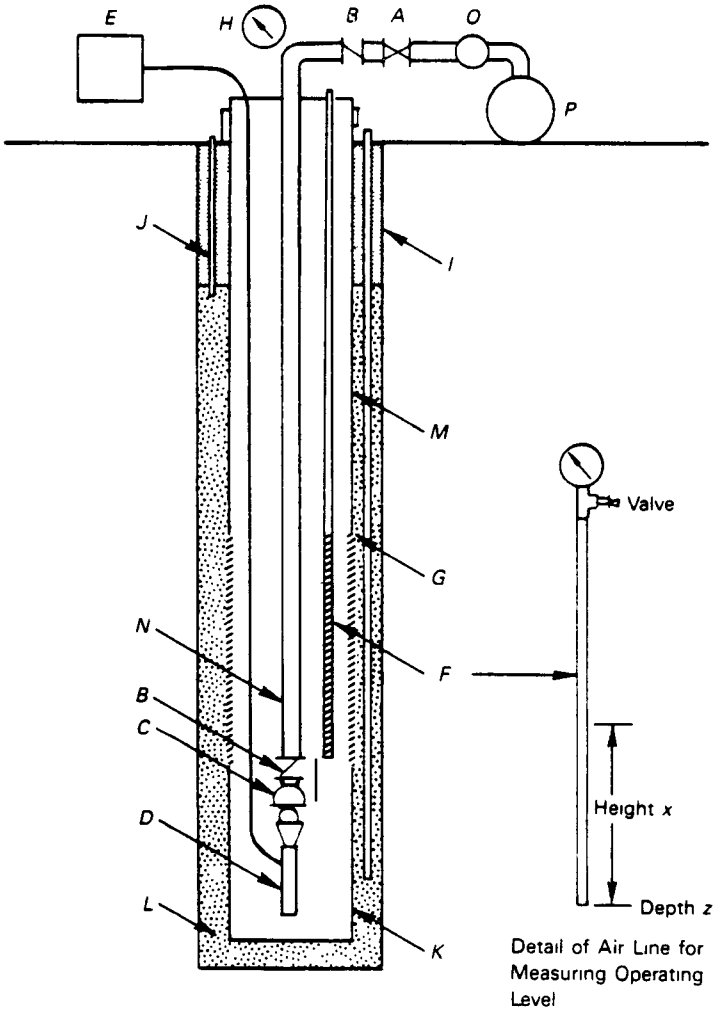


TABLE 5-11. DEEP WELL COMPONENTS AND SELECTION CRITERIA (Powers, 1981)

Item Description (See Figure 3-22)		Selection Criteria
A	Throttle Valve	Prevents damage caused by pumping surges when the pump is operating at less then the design capacity (typically after initial drawdown), also ensures maintenance of a constant discharge
B	Check Valve	Prevents backflow (recharge) from occurring if the pump fails in systems where a number of wells are connected to a common manifold
C	Turbine Pump	Sized to suit aquifer conditions, may need to be corrosion resistant
D	Electric Motor	Sized to operate within its service range under any pump load that may be encountered
E	Control Panel	May need to be in a tamper proof enclosure
F	Air Line	Used to measure the operating level of the well; monitors performance of the pump and the well
G	Filter Piezometer	Used to measure screen loss during development and fractional increases caused by encrustation
H	Pressure Gauge	Used to monitor pump wear
I	Seal	Prevents surface inflow of water into well, typically cement grout or bentonite
J	Gravel Pipe	Used to introduce new gravel in sealed wells during subsequent redevelopment
K	Well Screen	Size in accordance to needs, may need to be corrosion resistant
L	Filter Material	
M	Casing	

TABLE 5-11. (Continued)

Item Description (See Figure 3-22)		Selection Criteria
N	Discharge Column	Sized for maximum discharge with acceptable fraction; may need to be corrosion resistant, must also carry weight and hydraulic thrust of motor
O	Meter Device	Used for monitoring well discharge
P	Manifold	Used to collect water pumped from wells and carry pumped water to treatment

all components listed will be utilized or are necessary for individual wells, but incorporation should be based upon such factors as the life expectancy of the well, the well's design, the treatment facilities available, and the corrosiveness of the contaminated groundwater.

5.4.1.1.1 Pumps

Two types of pumps are commonly utilized in deep well applications, the turbine submersible pump and the vertical line shaft pump. Turbine submersible pumps were originally designed for water supplies but have been used extensively in construction for dewatering. Submersible pumps have the advantage of being relatively slender for their capacity allowing their use in small diameter wells. Commercially available pumps can be obtained with capacities greater than 100 gallons per minute and with motors of several hundred horsepower.

Vertical lineshaft pumps are similar to submersible pumps except that the motor is located on the surface rather than being attached directly to the pump. This type of pump is used for extracting high volumes of groundwater and is available in sizes ranging from a few horsepower to over a thousand horsepower. The cost of a lineshaft pump is typically greater than for a submersible pump of similar horsepower sizes.

Pump impellers and diffusers (components that drive water) are available in a variety of materials (e.g., plastic, cast iron, bronze) and can be made of special metals for wastes containing corrosive chemicals. Abrasive matter such as sand can cause premature wearing of these parts and lose of well efficiency. To avoid this, wells should be developed (i.e., pumped free of fines as discussed in a later section) and cleaned to remove particles after drilling. Also, the pump should be placed above the bottom of the well (i.e., >2 ft) to minimize pump plugging caused by fines.

Numerous problems can arise when using deep well pumps for plume management. These include the following:

- Corrosion of pump parts by contaminated waters
- Electrical malfunction (submersible pumps)
- Lightning damage
- Excessive part wear (e.g., impeller wear caused by intake of fines)

These problems should be factored into the design specification to minimize operational problems.

Before selecting a pump for installation, accurate information on the required capacity of the well and the total head is necessary. Required capacity can be determined from the equations presented previously for the radius of influence (R_0) and drawdown (H-h). Factors such as pumping cycles, peak loads, dynamic head, and future needs should also be considered in design capacity. Total dynamic head (h_t) can be calculated using the formula:

$$h_t = h_e + h_f + h_v$$

where:

h_e = total vertical lift, from the pumping level in the well to the water discharge point (ft)

h_f = total frictional losses expressed as head (ft)

h_v = velocity head required to produce the desired flow (ft).

Total vertical lift (h_e) is the sum of the vertical distance from the pumping level in the well to the outlet and the hydrostatic pressure head at the discharge outlet. Velocity head (h_v) is determined using the formula, $h_v = 0.155V^2$. Usually h_v is very small and only taken into account at installations where:

- Very high discharges are expected at very low lift and discharge head
- Velocity of flow is very high.

Frictional losses in pipes and fittings are calculated using tabulated values. Tables 5-12 and 5-13 give values for smooth pipe and fittings.

Once the total dynamic head is obtained and the capacity is determined, a pump and motor can be chosen using performance curves. These curves typically plot capacity versus total head and percent efficiency of the system. An example of performance curves for 325 gallon per minute pumps is given in Figure 5-20. The most efficient operating range for these pumps is between 250 gallons per minute and 380 gallons per minute. If, for example, the total calculated head is 200 feet and the desired capacity is 300 gallons per minute, a 25 horsepower, 5 stage pump would be selected for use (i.e., a pump that exceeds the expected use).

Another criteria in selecting a pump that should be considered is the type of drawdown anticipated and the pumping cycle. If the pump is to be utilized under varying head pressures and cyclic pumping conditions, a steep head pump should be chosen (i.e., steep drawdowns). If the pump will be used for steady pumping, resulting in a flat drawdown, a flat head pump should be chosen. Proper selection of either a flat or steep head pump will reduce O&M cost and improve system efficiency.

5.4.1.1.2 Well Casing and Depth

Casing size (diameter) is chosen to satisfy two requirements. The casing must be large enough to (Johnson Division, UOP Inc., 1975):

- Accommodate the pump with enough clearance for effective operation
- Ensure proper hydraulic efficiency of the well at the intake section.

The controlling factor is usually the size of the pump selected for the desired capacity and head. A good rule of thumb is that the casing size should be two standard pipe sizes larger than the nominal diameter of the pump or pump bowl, and not less than one size larger. Recommended well casing diameters are given in Table 5-14. For example, a 300 gallon per minute pump

TABLE 5-12. LOSS OF HEAD DUE TO FRICTION IN SMOOTH PIPE*
(approximate head loss in ft. per 1,000 ft. of pipe)
(Johnson Division, UOP Inc, 1975)

Flow Rate in gpm	Nominal Pipe Size, in inches										
	1-1/4	1-1/2	2	2-1/2	3	4	5	6	8	10	12
10	20	9	2								
15	44	20	6								
20	79	35	10	4	1						
25	123	55	16	6	2						
30	178	79	22	9	3						
40		142	40	16	5						
50		222	64	25	8	2					
60			90	37	11	3					
75			140	57	18	4	1				
100			258	102	30	8	2				
125				159	46	12	3				
150				228	68	17	5	2			
175					90	23	7	2			
200					122	30	9	3	1		
250						47	14	5			
300						70	20	8	2		
350						93	27	11			
400						123	35	14	3		
500						191	54	21	5	2	
600							78	31	7		
750							122	48	11	3	
1,000								86	19	6	2
1,500								194	43	15	5
2,000									76	22	10

*For Rough Pipe, add 50%

TABLE 5-13. APPROXIMATE HEAD LOSS EQUIVALENTS FOR PIPE FITTINGS
(Johnson Division, UOP Inc, 1975) (expressed in fit of straight pipe)

Type of Fitting	Nominal Pipe Size, in inches											
	1.25	1.5	2	2.5	3	3.5	4	5	6	8	10	12
90° elbow	3.5	4.5	5.5	6.5	8	9.5	11	14	16	21	26	32
45° elbow	1.5	2	2.5	3	4	4.5	5	6	8	10	13	15
90° elbow (long radius)	2.5	3	4	4.5	5	6	7	9	11	14	17	20
Tees	See note below											
Reducer (1 step) large end	2	2.5	3	3.5	4	5	6	8	10	12	15	18
Gate valve (open)	1	1	1.5	1.5	2	2	2.5	3	4	5	6	7
Gate valve (1/2 open)	20	25	35	42	50	58	65	80	100	130	170	200
Globe valve (open)	35	45	55	67	80	100	115	140	160	215	285	340
Angle valve (open)	18	24	30	35	40	50	55	70	80	110	140	175
Swing check	9	11	13	16	20	23	25	32	45	52	65	80

Tees vary with the direction of flow through them. For straight-through flow, the head loss is about equal to that of a long radius 90° elbow; when flow is through the branch, the head loss is approximately 3 times as great.

FIGURE 5-20.
PERFORMANCE CURVES
(FLINT & WALLING INC.)

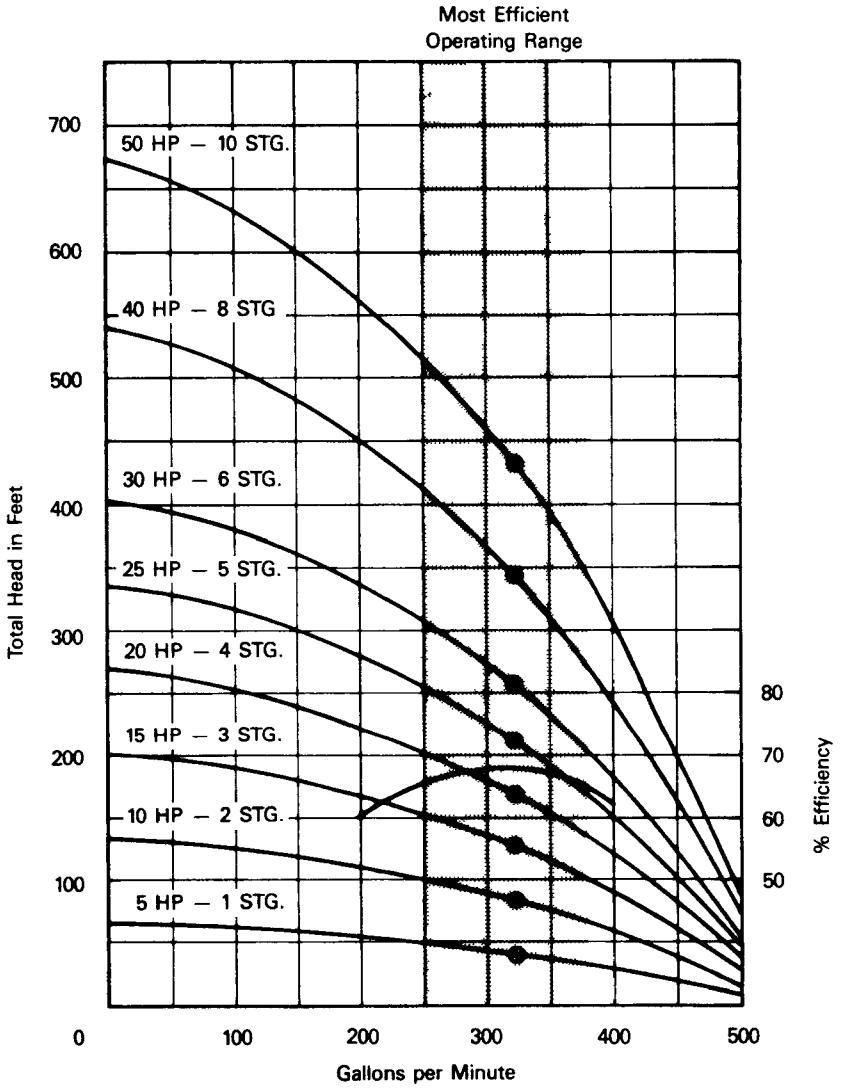


TABLE 5-14. RECOMMENDED WELL CASING DIAMETERS (JOHNSON DIVISION, UOP INC., 1975)

Well Yield (gpm)	Nominal Size of Pump Bowl (in)	Optimum Size of Well Casing (in)	Minimum Size of Well Casing (in)
<100	4	6 ID	5 ID
75-175	5	8 ID	6 ID
150-400	6	10 ID	8 ID
350-650	8	12 ID	10 ID
600-900	10	14 OD	12 ID
850-1,300	12	16 OD	14 OD
1200-1,800	14	20 OD	16 OD
1600-3,000	16	24 OD	20 OD

ID = inside diameter

OD = outside diameter

with a 6 inch diameter, an optimum casing size would be 10 inches (inside diameter).

Well depth is usually determined from the geologic data (e.g., aquifer thickness) available for the site and the plume's characteristics. In most cases wells will be completed to the bottom of the aquifer. This is typically the case for plumes that contain contaminants that are miscible with groundwater or are denser than groundwater or both. The exception to full penetration of the aquifer would be where contaminants are less dense than groundwater such that they float at the top of the aquifer. In this case partially penetrating wells may be desirable.

5.4.1.1.3 Well Screens

The purpose of a well screen is to (Department of the Interior, 1977):

- Stabilize the sides of the hole
- Keep soil particles out of the well
- Facilitate flow into and within the well.

Screens can range from blank pipe that is perforated inplace to manufactured, continuous-slot well screens with accurately sized openings. The types of screens commercially available and commonly used for dewatering are listed and described in Table 5-15. Most are made in lengths ranging from 5 to 20 feet so that any length of screen can be created by joining individual sections. The options in selecting well screens are the type, length, diameter, and opening size. The following sections provide information for making these choices.

The type of well screen selected is generally based on the availability of screens having the proper diameter and opening size that are made of a suitable material. The selection of screen diameter and opening size is controlled by several factors, which are discussed later. Material selection is governed by the corrosive and incrusting properties of the groundwater and contaminant plume, and the strength requirements imposed by column load and collapse pressure. Corrosion of the screen causes enlargement of openings, which could permit excessive sand entrance and premature pump wear. If corrosion is severe, collapse could occur. Incrusting is caused by mineral deposits on the screen surface which tend to plug the openings of the screen and formation. Removal by acids is often used to remedy incrustation problems. Bacteria growth can also cause well incrustation in the same manner as mineral deposits. Removal of deposits is typically done using strong chlorine solutions. These problems necessitate the use of screens (and casing) made of corrosion resistant materials. Table 5-16 lists some common materials and their applications.

TABLE 5-15. WELL SCREEN TYPES (After Powers, 1981)

Type	Diameters (in)	Openings (in)	Advantages	Disadvantages
Slotted	4 - 12	0.010 - 0.100	• Reasonable cost • Easy installation • Resistant to corrosion	• Type I PVC is brittle • PVC susceptible to degradation by chlorinated hydrocarbons • Can have loading probelms with thinner walled pipe • Slots can become partly clogged by sand • Generally minimal opening area available
Continuous Slot	4 - 36	0.003 - 0.250 (vertical rods not accounted for in area calculation)	• Screen has high open area • Precise slot dimensions • More effective development • Can be reused	• Must be welded to casing • Corrosion problem can occur depending upon material used
Louvered	6 - 48	0.032 - 0.250	• Reasonable cost • Reusable • Effective in gravel	• Susceptible to corrosion • Limited open area
Wire Mesh	4 - 24	>0.020	• Effective in jetted wells • Effective in fine soils • Precise slot dimensions	• Not suitable for drilled wells • Limited open area because of holes behind screen

TABLE 5-16. WELL SCREEN MATERIALS AND APPLICATIONS (Johnson Division, UOP Inc., 1975)

Material	Nominal Composition	Cost Factor	Suggested Applications
Monel	70% Nickel 30% Copper	1.5	Waters with high sodium chloride and dissolved oxygen levels such as sea water
Stainless Steel	74% Steel 18% Chromium 8% Nickel	1.0	Waters containing hydrogen sulfide, dissolved oxygen, carbon dioxide, or iron bacteria; excellent strength
Everdur	96% Copper 3% Silicon 1% Manganese	1.0	Waters having a high total hardness, high sodium chloride content where dissolved oxygen is absent, or a high iron content; extremely resistant to acid treatment
Silicon Red Brass	82% Copper 16% Zinc 1% Silicon	0.9	Same as Everdur but nor quite as good nor as strong; used in relatively inactive waters
Armco Iron	99.84% Pure Iron (double galvanized)	0.6	Not corrosion resistant; used in neutral water
Steel	99.35/99.72 Iron 0.09/0.15 Carbon 0.20/0.50 Manganese (double galvanized)	0.5	Not corrosion resistant; used for temporary wells
PVC	Polyvinylchloride	0.25	Highly corrosion resistant; Type I PVC is brittle and will crack when strained; not useful with chlorinated organics and some organic solvents

The well screen choice is also determined partly on strength requirements. Two loads are imposed on the screen which must be considered in screen selection; column load and collapse pressure (Johnson Division, UOP Inc., 1975). Column load results from the screen supporting the weight of the casing. These forces can be considerable in deep wells. Collapse pressure results from earth pressure and caving materials squeezing the screen. Resistance to these loads is directly proportional to the modules of elasticity of the screen material. For example, the strength of a stainless steel screen is twice that of a copper alloy screen of the same size (steel modules = 30 million psi, copper alloy modules = 15 million psi). Screens should be chosen to have:

- Strength to resist load and collapse
- Maximum open area consistent with the strength requirements.

The optimum length of a screen is based on the type and thickness of the aquifer, the available drawdown, and the stratification of the aquifer. Table 5-17 gives some general rules that should be applied when determining screen length. The criteria given in Table 5-17 is for contaminants that completely mix in the aquifer or are denser than water. For contaminants that are less dense than water (i.e., floating contaminants), screen length and placement should be determined on the basis of plume boundaries and the desired drawdown.

The opening size of the well screen requires that the entrance velocity of the groundwater be less than a critical value. Some references suggest using a critical value equal to or less than 0.1 feet per minute (Johnson Division, UOP Inc., 1975), while others suggest choosing the critical value of entrance velocity based on the hydraulic conductivity of the filter material in contact with the screen (Powers, 1981). Table 5-18 gives recommended

TABLE 5-17. CRITERIA FOR WELL SCREEN LENGTH SELECTION
(After Johnson Division, UOP Inc., 1975)

Aquifer Type	Homogeneity	Criteria
Water Table (unconfined)	Homogeneous	• Screen bottom one third of aquifer • Drawdown should not exceed the top of the screen or a point slightly above
	Heterogeneous	• Screen the most permeable sections, especially in lower part of the aquifer • Screen sections that contain contaminants regardless of permeability • Drawdown should not exceed the top of the screen
Artesian (confined)	Homogeneous	• 70% to 80% of the aquifer's total thickness <25 ft - 70% 25-50 ft - 75% >50 ft - 80% • Maximum allowable drawdown should be the top of the aquifer
	Heterogeneous	• Screen as in a homogeneous aquifer to extract contaminants • Drawdown should not be lower than the top of the screen

TABLE 5-18. RECOMMENDED ENTRANCE VELOCITIES FOR VARIOUS FILTERS (Powers, 1981)

Hydraulic Conductivity (gpd/ft^2)	Entrance Velocity (ft/min)
>6,000	12
6,000	11
5,000	10
4,000	9
3,000	8
2,500	7
2,000	6
1,500	5
1,000	4
500	3
<500	2

entrance velocities for various filters. Regardless of the method chosen to obtain critical entrance velocities, the open area can be calculated from:

$$A_o = 19.2\ Q/V_s$$

where:

A_o = open area of well screen (in/ft)

Q = expected or desired yield of well per unit length of screen (g/min-ft)

V_s = entrance velocity (ft/min)

Once the open area has been determined, the only parameter needed to correctly select a well screen is the slot size. Typical open areas of some commercially available screens are given in Table 5-19.

TABLE 5-19. OPEN AREAS OF COMMERCIALLY AVAILABLE WELLSCREENS (Powers, 1981)

Nominal Diameter (in.)	Slot Size (in.)	Approximate Open Area (in^2/ft)*			
		Continuous Wire	Double Louvre	Slotted PVC	Wire Mesh
4	0.015	27.6	--	8.9	76.3
	0.030	46.6	5.9	14.4	--
	0.060	71.2	--	27.1	--
	0.090	86.4	--	43.4	--
	0.120	96.7	--	57.6	--
6	0.015	41.2	--	10.1	112.4
	0.030	60.0	6.9	20.2	--
	0.060	84.8	13.9	40.3	--
	0.090	106.9	20.7	60.5	--
	0.120	123.2	28.7	80.6	--
8	0.015	39.3	--	13.0	146.3
	0.030	69.3	9.1	25.9	--
	0.060	113.0	18.6	51.8	--
	0.090	142.6	27.6	77.8	--
	0.120	164.3	38.3	103.7	--
12	0.015	59.2	--	20.2	216.3
	0.030	81.5	13.7	40.3	--
	0.060	117.4	27.8	80.5	--
	0.090	155.9	41.3	121.0	--
	0.120	186.5	57.4	161.3	--
18	0.015	53.5	--	28.8	305.4
	0.030	99.2	19.8	57.6	--
	0.060	169.6	40.2	115.9	--
	0.090	228.1	59.7	172.8	--
	0.120	271.3	82.9	217.5	--
24	0.015	83.2	--	--	407.2
	0.030	117.5	25.9	--	--
	0.060	172.3	52.6	--	--
	0.090	235.1	78.1	--	--
	0.120	289.4	108.4	--	--

*Open area may be less than manufacturer's specifications because of design; refer to Table 5-14.

Determining the required slot size for the screen depends on the well development method to be used and the filter pack. A mechanical analysis (i.e., sieve analysis) must be performed on a sample representing the water-bearing formation to select the proper slot size. This approach differs from that used for monitoring wells, i.e., the use of a 10-slot screen (Sisk, 1981), because wells used in remedial actions are typically pumped extensively. Extensive pumping with improper screens and development can result in low yields, premature pump wear, and possible well failure.

Most wells used for remedial actions will probably be drilled, which normally requires the area around the well screen to be filled with a filter or gravel. This filter performs several important functions (Powers, 1981):

- Fills the annular space preventing the uncontrolled collapse of the formation against the screen
- Retains a sufficient percentage of fines thus preventing them from being pumped continuously
- Passes some amount of fines and mud cake that have built-up on the sides of the hole
- Transmits water freely from the aquifer to the screen during pumping.

Procedures for selecting filters and screens have been developed by numerous groups, however, a method developed by Powers (1981) is described below. The first step in the procedures is to obtain a sample at the aquifer and conduct a mechanical analysis of it. Results of the analysis are plotted as shown in Figure 5-21a. A filter range is thus established such that:

- The filter is of uniform material, preferably with a uniformity coefficient (C_u) of $C_u<3.0$
- The filter C_u is not greater than the aquifer C_u unless it is a graded filter
- The filter D_{50} is four to eight times greater than the aquifer D_{50}.

FIGURE 5-21. RANGE OF FILTER SELECTIONS (POWERS, 1981)

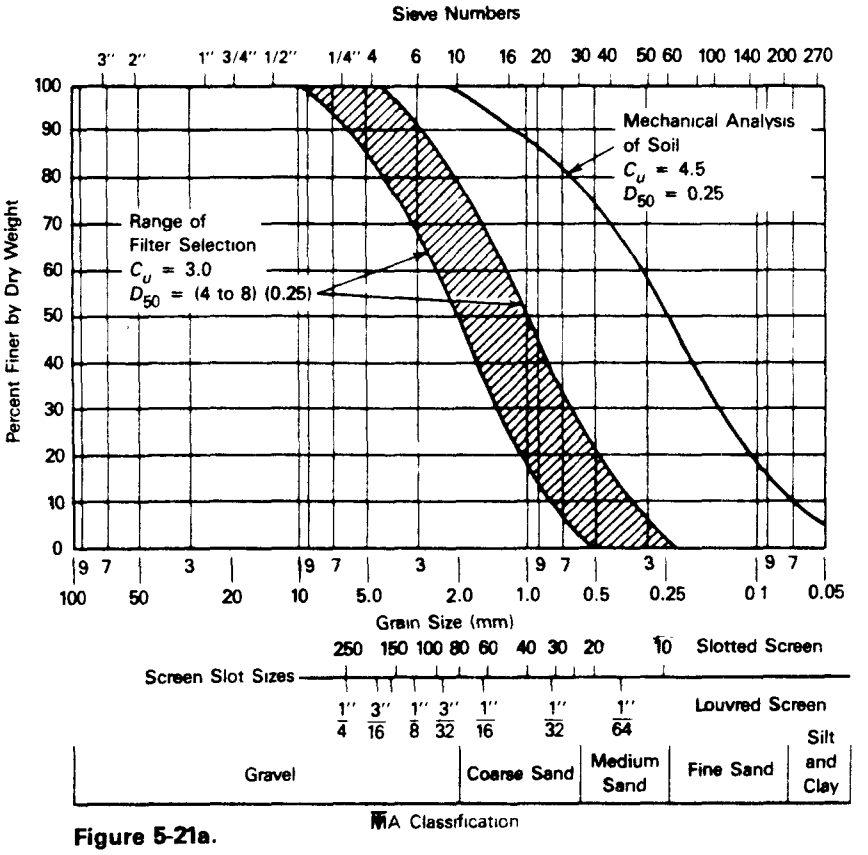


Figure 5-21a.

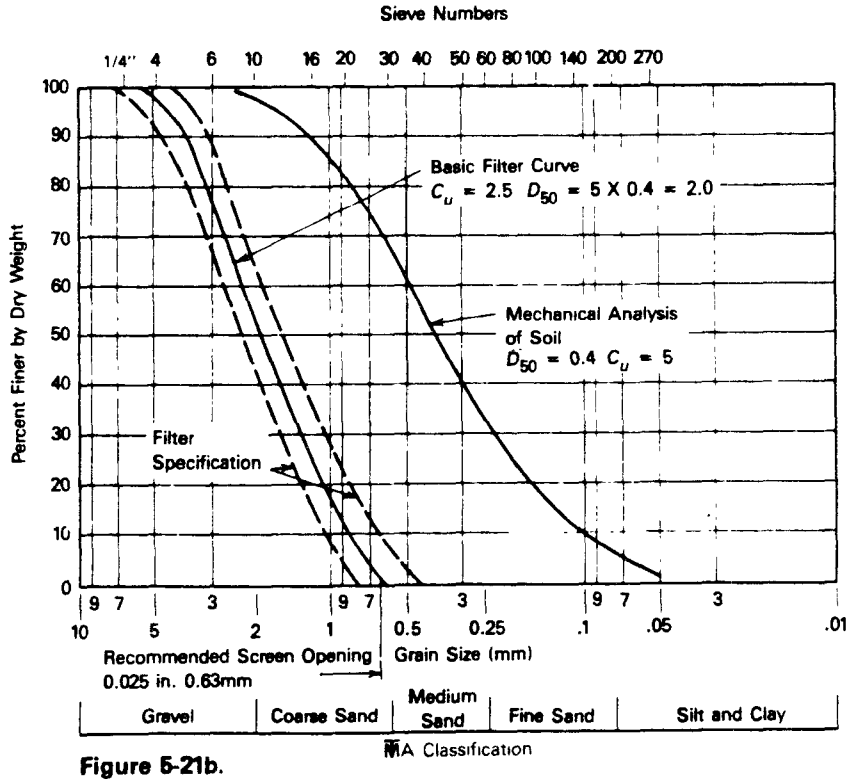


Figure 5-21b.

The range of suitable filters is then plotted as shown in Figure 5-21a. A rather broad range of filters is established for a soil D_{50} and selection of the proper filter within this range is based on the gradation of the soil and the desired yield of the well. Table 5-20 gives criteria for determining filter requirements more precisely.

Once the range of acceptable filters has been narrowed to fit the specific application, the results of the mechanical analysis is replotted as shown in Figure 5-21b. Typically, a set of limits within which the filter material must fall is furnished to a supplier. The mechanical analysis of the supplied filter material is then plotted. Well screen slot size is finally selected to pass about 10 percent at the fine limit and 0 percent of the coarse limit (Figure 5-21b). While this procedure appears to be fairly simple, a significant amount of judgement is required in applying it to specific site conditions.

For naturally developed wells (i.e., wells without a gravel pack), well screen opening size is selected from a mechanical analysis of the water-bearing formation (Figure 5-22). The correct slot opening size is chosen so that the screen will retain from 40 percent to 50 percent of the particles (Johnson Division, UOP Inc., 1975). The 40 percent size is chosen for wells in non-corrosive water and where sample analysis is considered reliable. The 50 percent size is chosen for wells in corrosive waters (i.e., to allow for some enlarging of screen openings) and where sample analysis reliability is in doubt. For the grain size analysis shown in Figure 5-22, the screen opening selected would be between 0.040 inch and 0.050 inch (50 percent and 40 percent retention, respectively).

Where stratified soils are encountered, the screen slot opening is chosen according to the gradation of materials of the different strata. Table 5-21 gives some criteria for selecting and placing screens in stratified soils. The examples given in the table are generalizations and should be viewed as such. For example, cutting off silty zones with blank casing may be preferable. The use of variable or graded filters may also be necessary to achieve good results. However, a rule of thumb used by drillers is to allow

TABLE 5-20. FILTER SELECTION CRITERIA (AFTER POWERS, 1981)

Characteristic	Criteria
Gradation of Soil	For uniform soils ($C_u \leq 3$): the D_{50} of the filter should be in the low range, about 4 to 5 times the D_{50} of the soil. For graded soils ($4 \leq C_u \leq 6$): the D_{50} of the filter can be 5 to 6 times the D_{50} of the soil. For very well graded soils ($C_u \geq 7$) where it may be desirable to develop some fines from the soil: the D_{50} of the filter can be as great as 8 times the D_{50} of the soil.
Yield of Well	For low well yields per linear foot of screen in relation to hydraulic conductivity: the multiples of the D_{50} of the filter can be increased by a factor of 1 or perhaps 2 because pore velocities are low enough to preclude the movement of fines. For high well yields per linear foot of screen in relation to hydraulic conductivity: the multiplier of the D_{50} filter should not be increased.

FIGURE 5-22.
GRAIN-SIZE ANALYSIS CURVE
(JOHNSON DIVISION, UOP INC, 1975)

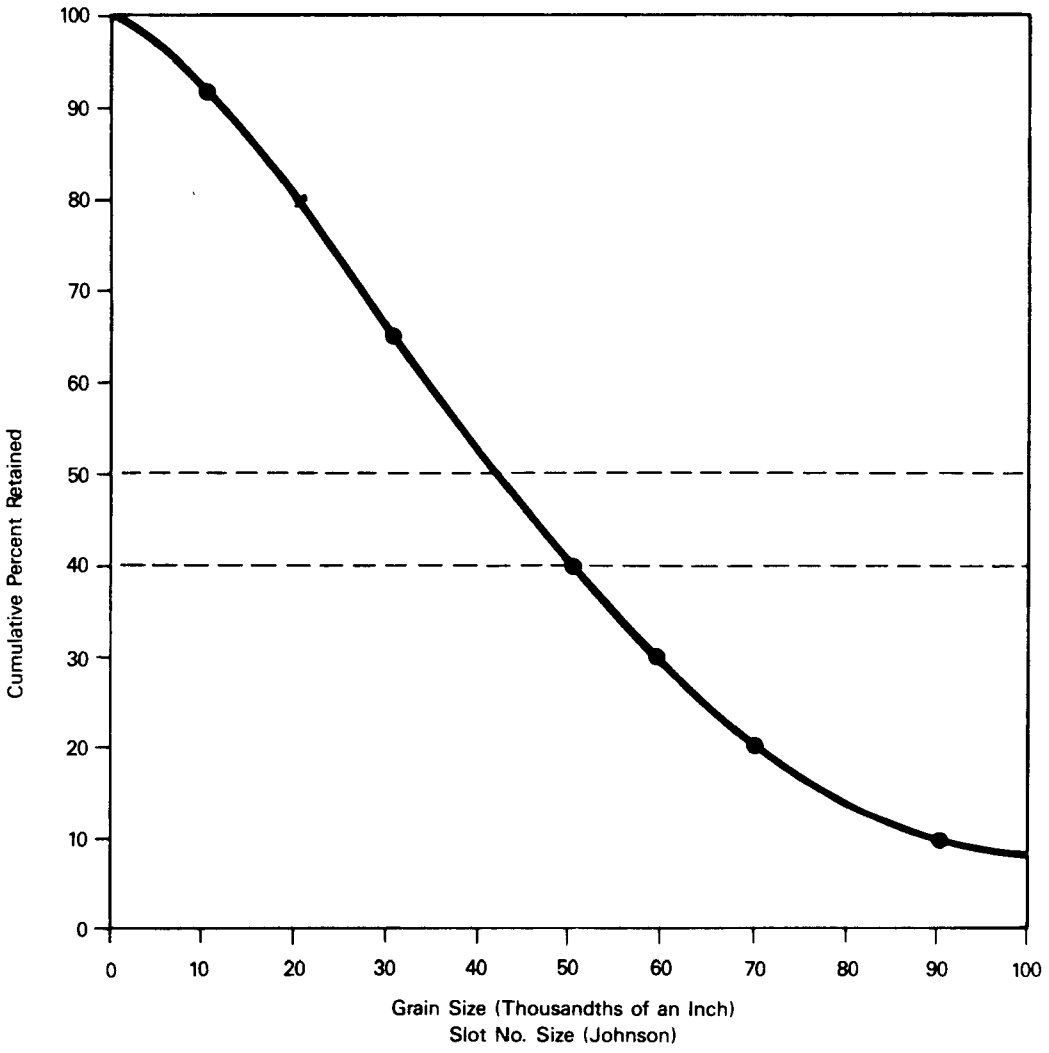


TABLE 5-21. WELL SCREEN SELECTION FOR STRATIFIED SOILS
(Johnson Division, UOP Inc, 1975)

Development Type	Stratification Scheme	Criteria
Filter Pack	Coarse material over fine	Filter and screen selected should be suitable for the finer material
	Fine material over coarse	Filter and screen selected should be suitable for the coarse material because the coarse material will probably draw the fine material
Naturally	Coarse material over fine	Screen should be selected according to each soil material
	Fine material over coarse	Screen should be selected according to each soil material; screen in the fine material should extend at least 2 feet into the coarse material and the slot size in the coarse material should not be twice that of the screen in the fine material

the finer material to dictate screen size over the various strata because mixing packs in the field is difficult and graded packs can differentiate in place.

5.4.1.2 Ejector Wells

Ejector wells or systems have certain advantages over wellpoints and deep wells because they are not typically depth limited as wellpoints are and they are less expensive than deep wells when close spacing is required. The biggest drawback to the use of ejector wells is that they are very inefficient (typically less than 15 percent efficiency). Ejector wells can be used

independently of each other or arranged so that they utilize a common pumping system.

The typical components of a two-pipe and a single-pipe ejector well system are illustrated in Figure 5-23. The lift principle for the two-pipe model is (Powers, 1981):

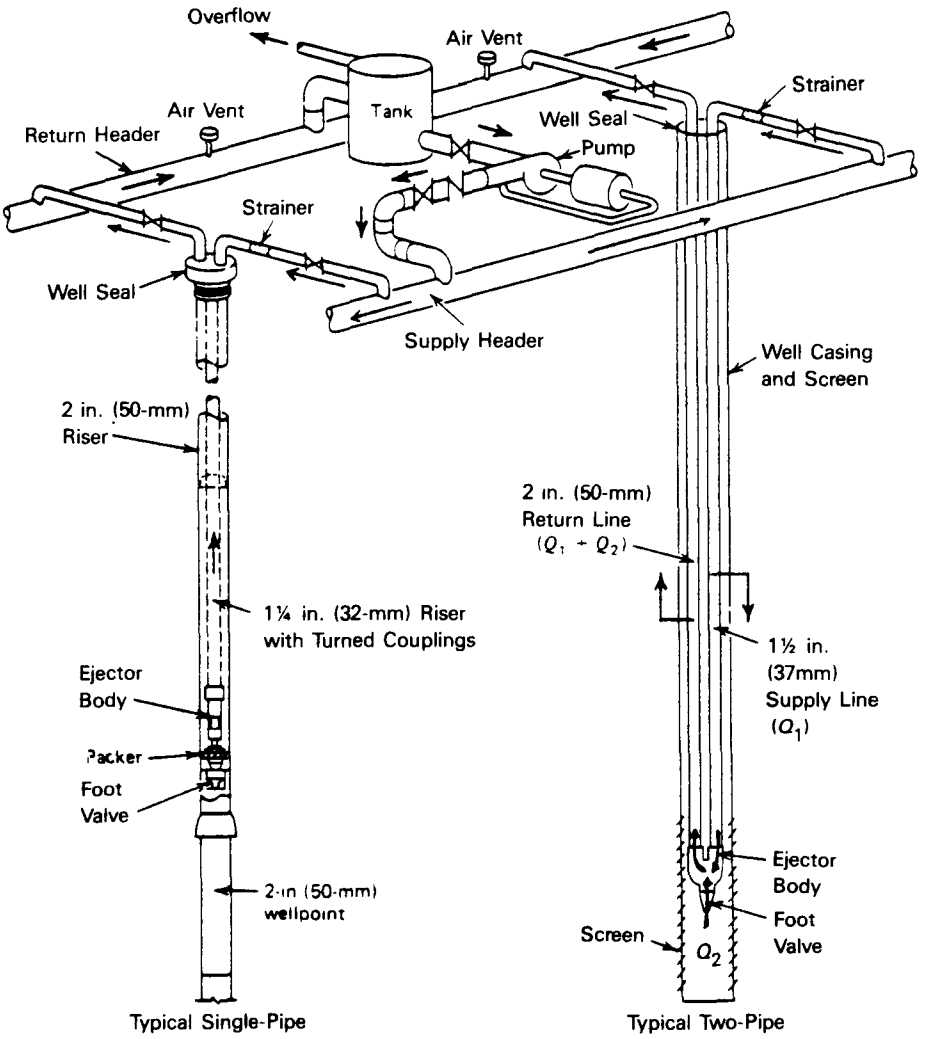
- High pressure supply water (Q_1) moves down the supply pipe through ports in the ejector body to the tapered nozzle where the pressure head is converted to water velocity
- Supply water exits the nozzle at less than atmospheric pressure creating a vacuum in the suction chamber
- Groundwater (Q_2) is drawn into the chamber through the foot valve because of the pressure differential
- Supply water and groundwater ($Q_1 + Q_2$) are mixed in the suction chamber
- The mixed water enters the venturi where the velocity decreases because of divergence resulting in increased pressure
- The increase in pressure develops sufficient head to return the combined flow to the surface.

The lift principle for the single-pipe model, illustrated in Figure 5-23, is (Powers, 1981):

- Supply water under pressure (Q_1) flows downward between the well casing and the inner ejector return pipe
- The nozzle, suction chamber, and venturi perform as described for the two-pipe system
- A packer assembly separates the supply water from the groundwater so that different pressures are developed
- Foot valves located below the packer are used as in the two-pipe ejector system.

Casing and screens used by ejector systems are the same as those utilized by deep wells or wellpoints. Design recommendations for casing and riser sizes are given in Table 5-22. Single-pipe ejector wells are most commonly used because they require less piping and yields are greater for smaller diameter casings.

FIGURE 5-23.
COMPONENTS OF ONE-PIPE AND TWO-PIPE EJECTOR WELLS
(POWERS, 1981)



Typical Single-Pipe Ejector Installed in a 2-in. (50-mm) Wellpoint

Typical Two-Pipe Ejector Installed in a 6-in. (150-mm) Well

TABLE 5-22. RECOMMENDED CASING AND RISER SIZES FOR EJECTOR SYSTEMS (Powers, 1981)*

Groundwater Flow (Q_2) (gpm)	Single Pipe Ejector		Two Pipe Ejector		
	Well Casing (in)	Return Pipe (in)	Well Casing (in)	Supply Pipe (in)	Return Pipe (in)
12	2.0	1.25	4	1.00	1.25
20	2.5	1.50	5	1.25	1.50
40	4.0	2.00	5	1.50	2.00
70	5.0	2.50	6	2.00	2.50

*Recommended pipe sizes are for a setting of 40 feet with a supply pressure of 120 psi; deeper settings or lower pressures require larger pipe sizes.

Ejector pumps consist of a water tank and a pump with the required valves and piping. Each well can have its own pump or one pump can be used by a number of wells. Water from the storage tank is pumped under pressure through a header pipe that supplies the ejector wells. Return flow, a mixture of supply water and groundwater, recharges the tank through the header system. Excess water in the tank is discharged to a treatment system.

Selecting the proper ejector pump and educers (i.e., nozzle and venturi) can be made using manufacturer's data or through calculations. An example of manufacturer's data is given in Table 5-23. The procedures for calculating the size of an ejector are to calculate head ratios, estimate the capacity ratios, and calculate the diameters of the nozzle and the venturi. The procedure is fairly complex and is described in detail by Powers (1981).

Materials for risers, swings, headers, tanks, screens, and pumps should be selected to minimize corrosion and encrustation. Selection criteria are the same as for deep well systems, however, ejectors must also be protected to maintain well performance. Ejectors used in corrosive environments can be made of plastic to eliminate the problem. Of greater importance is the ejectors' sensitivity to clogging especially if clogging results from encrustation, caused by the pressure reduction. Reduction in pressures at the throat of the venturi can cause accelerated rates of chemical precipitation. A continuous water supply to eliminate recirculation and filters may be necessary to prevent encrustation and clogging. A water analysis prior to designing the system is advisable to test for problems that may limit ejector well use.

5.4.1.3 Wellpoints

Wellpoint systems consist of a group of closely spaced wells connected to a header pipe and pumped by a suction pump. Wellpoints are best suited for groundwater extraction in stratified soils where total lift or drawdown will not exceed approximately 22 feet. The advantages to using wellpoints are that the system design is flexible and the wellpoints are relatively inexpensive even when closely spaced.

TABLE 5-23. EJECTOR SYSTEM PERFORMANCE SPECIFICATIONS (Burks HNAZ Series Pumps)

One-Pipe Educer

Pump HNAZ Series	Well Dia.	Educer No.	Pressure Switch Setting	Control Valve Setting	Capacity in Gallons Per Hour At Vertical Depth To Pumping Level														Max. Shutoff Pressure At Deepest Set'g
					30'	40'	50'	60'	70'	80'	90'	100'	110'	120'	130'	140'	150'	160'	
5 HNAZ 1/2 hp	2"	22183	20-40	38	675	660	545	425	330	190									55
		22184		35	515	480	450	400	350	290	250	200	150						62
7 HNAZ 3/4 hp	2"	22185	30-50	32	840	800	680	560	425	310									55
		22171		35	495	480	465	450	420	390	330	270	210	150					62
10 HNAZ 1 np	2"	22185	30-50	35	965	920	790	660	530	400	285	140							55
		22186		42	550	520	505	485	450	410	360	320	285	155					66
		22176		50			350	350	330	320	295	260	230	200	175	150	125	100	72

Two-Pipe Educer

Pump	Min. Well Dia.	Educer No.	Pressure Switch Setting	Control Valve Setting	Capacity in Gallons Per Hour At Vertical Depth To Pumping Level														Max. Shutoff Pressure At Deepest Set'g
					30'	40'	50'	60'	70'	80'	90'	100'	110'	120'	130'	140'	150'	160'	
5 HNAZ 1/2 hp	4"	22199	20-40	32	585	570	475	380	305	225									58
		22200		32	405	400	395	390	365	320	275	230	190	150					61
7 HNAZ 3/4 hp	4"	22196	30-50	28	860	850	730	610	505	385									58
		22197		30			480	480	455	395	330	270	220	180	140				63
10 HNAZ 1 hp	4"	22201	30-50	33	990	970	820	665	525	375									53
		22194		38	575	570	565	555	525	465	405	335	275	210	155				59
		22202		38	385	380	375	370	365	360	355	345	315	260	210	175	140	105	62

A suction (vacuum) pump is typically used in wellpoint systems to lift water. Suction pumps accomplish lift by developing a negative pressure head at the pump intake rather than by applying force to the water source as in ejector pump systems. Four factors limit the maximum lift attainable by suction: atmospheric pressure, vapor pressure, head losses caused by friction, and the required inlet head of the pump. Because of these factors, the maximum lift theoretically attainable is about 25 feet. In practice, however, lifts between 15 and 22 feet are more common when centrifugal pumps are used.

Two types of suction pumps are commonly used, the oil sealed and the water sealed vacuum pump. Pumps are usually rated by the volume of air handled at various vacuums under standard conditions of temperature and pressure. Oil sealed pumps are capable of producing vacuums of 30 to 100 cubic feet per minute at 25 inches of mercury; water sealed pumps produce vacuums of 50 to 500 cubic feet per minute at 25 inches of mercury.

Wellpoints are specially made well-screens that are typically 1.5 to 3.5 inches in diameter and are capable of yields up to 35 gallons per minute. Large wellpoints are available, up to 8 inches in diameter, which have capacities greater than 35 gallons per minute. These large wells are generally called suction wells rather than wellpoints. Wellpoints can be installed using a variety of methods including jetting and driving. Figure 5-24 shows the basic types of wellpoints. Wellpoint screens can be made of heavy wire mesh, continuous wire, slotted plastic, or perforated plates. The materials selected should minimize the potential for corrosion and encrustation. The guidelines for wellpoint materials are the same as for deep wells.

The depth at which wellpoints are set is dependent upon the hydrogeology of the site. Drawdown is limited to approximately 22 feet below land surface, but the wellpoint can be set at almost any depth depending on the situation. Where contaminants float at the top of the aquifer, wellpoints can be set at shallow depths. Where contaminants sink or mix with the groundwater, the wellpoints can be set deeper. The only criterion for wellpoint depth is to

FIGURE 5-24.
DRIVEN WELLPOINT (a), JETTED WELLPOINT (b),
AND DRILLED WELLPOINT (c)
(JOHNSON DIVISION, UOP INC., 1975 a & b; POWERS, 1981c)

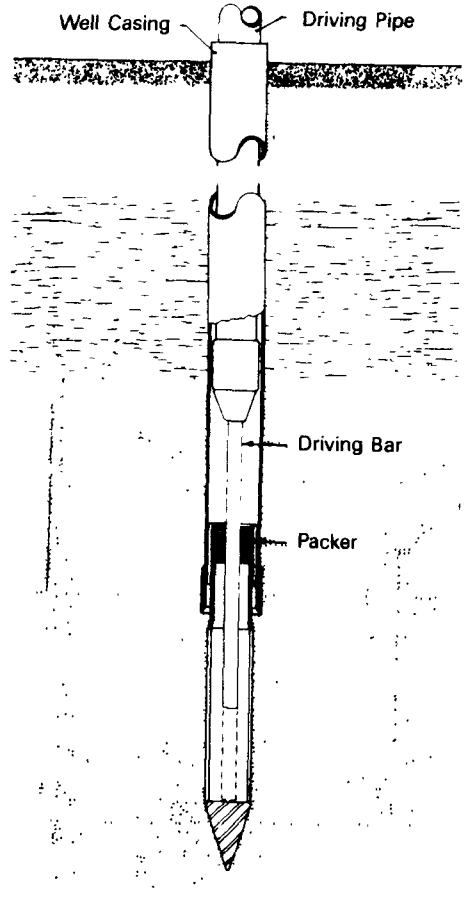


Figure 5-24a. Wellpoint Driven

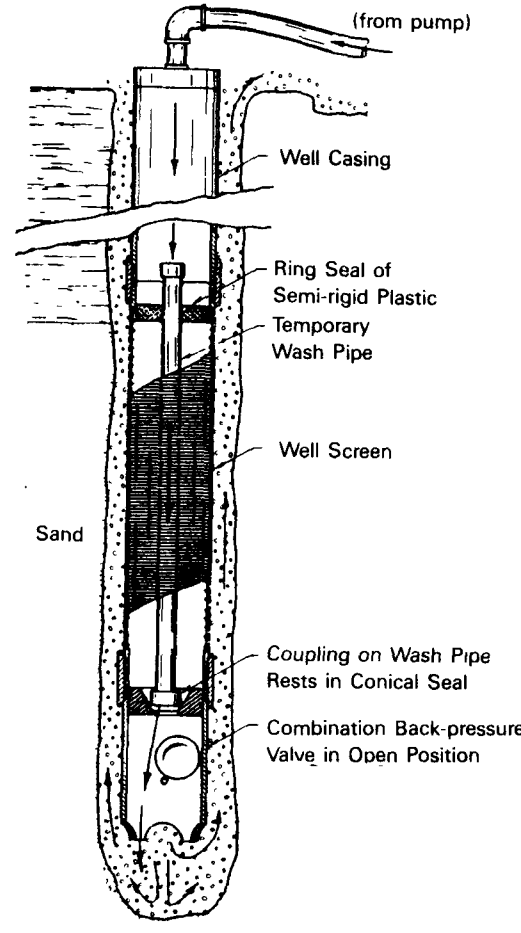


Figure 4-24b. Jetted Wellpoint

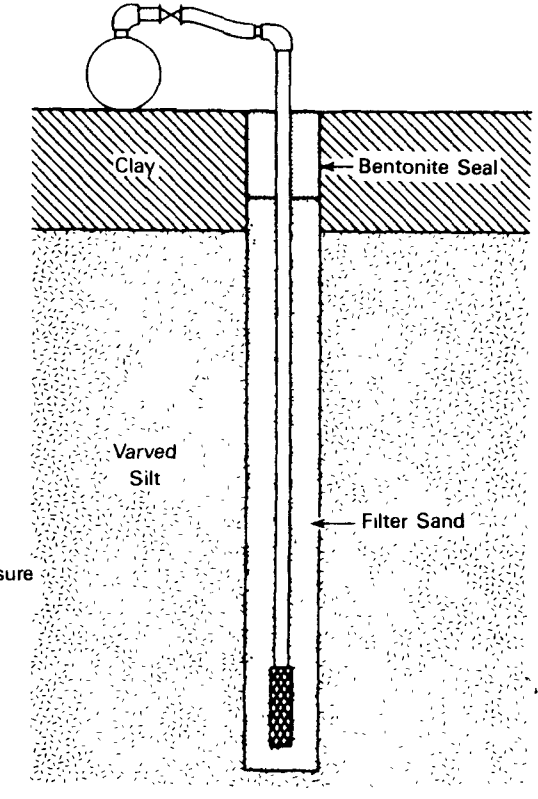


Figure 5-24c. Drilled Wellpoint

avoid dewatering below the screen. If this occurs, air enters the system and reduces the vacuum and therefore the drawdown. During normal operation of the system, dewatering below the top of a wellscreen can sometimes occur. The problem can be minimized in the field by adjusting the values that control individual wells.

Because wellpoints are typically installed in oversized boreholes, filter sands are placed around the wellpoint to fill the annular space. Besides filling the annular space, the sands perform other functions, including (Powers, 1981):

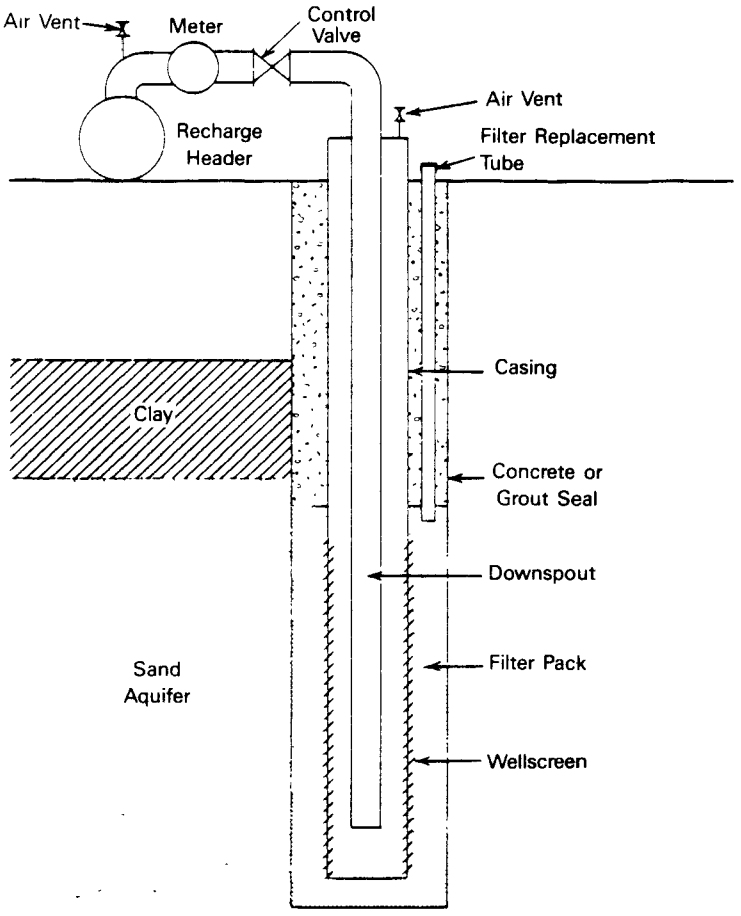
- Increasing the effective diameter of the wellpoint
- Decreasing the entrance velocities of water
- Preventing clogging of the screen with fines
- Providing vertical drainage from overlying layers.

Commercially available wellpoints are typically designed with openings suitable for use with washed concrete sand filters. This type of filter performs well when the soil penetrated is finer than the concrete sand. If soils are very fine and have little cohesion, however, they may migrate towards the wellpoint when concrete sand filters are used. In this instance, mortar sand filters may improve well yields and prevent clogging. For some applications, selecting the filter material and the screen opening specifically for each wellpoint's application may be necessary as described in the deep well section.

5.4.1.4 Recharge Wells

Recharge wells are sometimes used in plume management to create groundwater mounds or to act as depositors for treated water. The design of a recharge well is similar to an extraction well (Figure 5-25) with a few variations. Pumps, casing, filter packs, and screens should be designed in a

FIGURE 5-25.
BASIC INJECTION WELL (POWERS, 1981)



similar manner as outlined in the previous sections. The additional features that are normally needed are (Powers, 1981):

- A downspout to prevent air entrapment from cascading water when the well is operated at a low level
- An air vent to release trapped air at start-ups
- A concrete or grout seal to prevent water from flowing along the casing to the surface when the well is pressurized.

Development and periodic redevelopment of an injection well is necessary to maintain efficiencies because they do not continue to develop themselves with use as extraction wells do.

5.4.2 System Design

System design includes a determination of the number of wells needed, the patterns and spacing of the wells, the design of the individual wells, the pumping cycles and rates needed, and the method of handling discharges. The following sections provide some general guidelines for well system design.

5.4.2.1 Preliminary Requirements

Prior to designing the well system, a complete hydrogeologic understanding of the site must be established. A potentiometric surface map (i.e., a map depicting contours of equal head) and a geologic cross-section of the site should be developed. Development of these two tools is very important to well system design. Figure 5-26 gives an example of these for a gasoline pipeline leak. Other parameters that are required for system design are the coefficients of transmissivity (T) and storage (S) and the discharge (Q) and drawdowns ($H-h_w$) from the pump tests. Once data on these parameters are established, the design process can proceed.

FIGURE 5-26.
POTENTIOMETRIC SURFACE MAP AND GEOLOGIC CROSS-SECTION OF GASOLINE POLLUTION SITE (WILLIAMS AND WILDER, 1971)

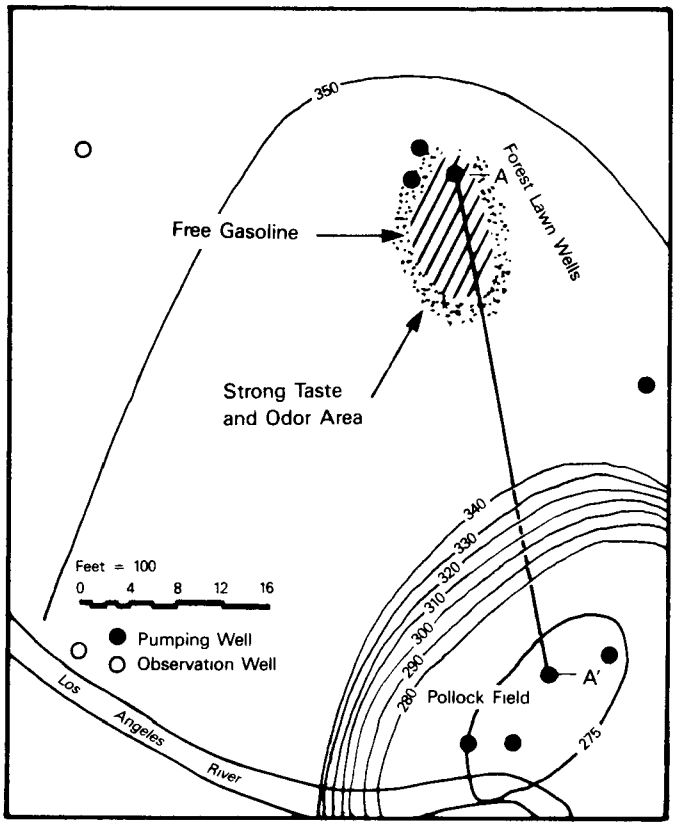


Figure 5-26a. Plane View

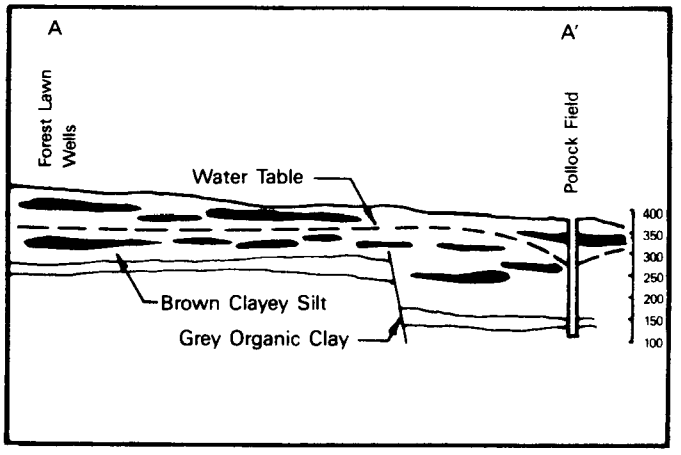


Figure 5-26b. Cross-sectional View

5.4.2.2 Equilibrium vs. Non-Equilibrium Pumping

A choice has to be made whether equilibrium or non-equilibrium pumping is to be utilized at the site because this affects the extent of the radii of influence. As pumping time decreases the radii of influence also decrease (refer to Figure 5-15) and this will result in a larger number of wells needed for a given area. Using the example given in Figure 5-15, increasing the pumping time from 300 minutes to 1,000 minutes causes the radius of influence to increase from 350 feet to 650 feet.

Equilibrium pumping will probably be utilized in most plume management sytems, but non-equilibrium pumping does have advantages in some cases. Table 5-24 lists the advantages and disadvantages of both approaches. Typically, cyclic pumping systems cost more because of the need for a larger number of wells with greater capacities and the greater operation and maintenance requirements.

5.4.2.3 Well Spacing

Determination of the proper spacing of wells to completely capture a groundwater plume is probably the most important item in system design. Field practitioners have long had a standing "rule of thumb" for estimating well spacing; adjacent cones of depression should overlap (i.e., radius of influence should overlap). This method is valid for aquifers that have low natural flow velocities but will not be valid for aquifers with large natural flow velocities. For these latter cases (and probably all cases) velocity distribution plots should be developed to determine well spacing and ensure capture of the plume (Keely and Tsang, 1983). Both of these methods are discussed in the following sections.

5.4.2.3.1 Radius of Influence

The most accurate method for estimating the radius of influence is by pumping test analysis. Pumping tests can identify recharge boundaries, barrier boundaries, and slow-storage release conditions. The pumping test

TABLE 5-24. ADVANTAGES AND DISADVANTAGES OF PUMPING METHODS

Method	Advantage	Disadvantage
Equilibrium	• Greater well spacing is feasible, thus reducing the number of wells • Constant discharges to treatment • Reduced pumping rates may be possible • Better suited to aquifers with high hydraulic conductivities • Operation and maintenance cost may be lower because cycling is hard on system components • Easier to design and operate effectively	• May require long pumping times before cones intersect, which may be a problem with fast moving contaminants • Not as workable in aquifers with low hydraulic conductivities • May not work well where the flow of contaminants is low, where the plume floats, or sinks, or where recharge is scant
Non-equilibrium	• Better suited to aquifers with low hydraulic conductivities • May be more applicable to floating and sinking plumes • May be more suitable for sites that have groundwater barriers and scant recharge	• Closer spacing of wells is required resulting in a greater number of wells • Design is fairly complicated • May have higher operation and maintenance costs because of cycling • Greater capacity pumps are usually required

should be performed until equilibrium conditions are reached. Typical test durations for a confined aquifer are about 24 hours, while for an unconfined aquifer they may be several days. Once equilibrium conditions have been reached, the radius of influence for equilibrium or non-equilibrium conditions can be estimated using the methods described previously and the equations in Table 5-25.

When pumping test data are lacking or incomplete, rough estimates of the radius of influence can be obtained from the values of transmissivity or hydraulic conductivity, pumping times, and coefficient of storage and the equations presented in Table 5-25. Coefficients of storage values typically range from 0.01 to 0.35 for water-table aquifers and from 0.00001 to 0.001 for confined aquifers. Typical values are 0.2 for water-table aquifers and 0.001 for confined aquifers.

When using equilibrium formulas, the radius of influence appears as a logarithmic function so that percision in estimating is not necessary. However, values of R_0 measured in the field can vary from 100 to 100,000 feet and greater orders of magnitudes are possible. Therefore, gross errors in actual R_0 values are possible.

When estimates are made without pumping data, the values obtained are rough approximations that do not take recharge into account. These estimates should be adjusted on the basis of judgments made on the locations of possible recharge and discharge barriers. Adjusting values of R_0 downward is advisable in most cases because the greater overlap of the cones of depression will result in a lower probability of containment escape between wells. The tradeoff, however, is a greater number of wells and higher costs.

A comparison of the results obtained for R_0 when pumping test data are used and when estimates are made is given below. A well is an unconfined aquifer pumped under equilibrium conditions provided the following information:

- $Q = 500$ gpd
- $K = 500$ gpd/ft^2

TABLE 5-25. RADIUS OF INFLUENCE EQUATIONS

Pumping Method	Unconfined Aquifer	Confined Aquifer
Equilibrium	$\ln R_o = (K(H^2-h_w^2)/458Q) + \ln r_w$	$\ln R_o = (T(H-h_w)/229Q) + \ln r_w$
Non-equilibrium	distance - drawdown plots or Theis Method	
Estimates	$R_o = r_w + (Tt/4790S)^{0.5*}$	
	$R_o = 3(H-h_w)(0.47K)^{0.5}$	

R_o = radius of influence (ft)
K = hydraulic conductivity (gpd/ft^2)
H = total head (ft)
h_w = head in well (ft)
Q = pumping rate (gpd)
r_w = well radius (effective radius) (ft)
T = transmissivity (gpd/ft)
t = time (min)
S = storage coefficient

*For unconfined aquifers H-h cannot be a large percentate of H

- $r_w = 0.5$ ft
- $H = 100$ ft
- $h_w = 80$ ft.

Using the exact equation for R_o given in Table 5-25:

$$\ln R_o = [K(H^2 - h_w^2)/458Q] + \ln r_w$$
$$= [500(100^2 - 80^2)/458(500)] + \ln 0.5$$
$$R_o = 1296 \text{ ft.}$$

Using the first estimation equation for R_o given in Table 5-25:

$$R_o = r_w + (Tt/4790S)^{0.5}$$

$t = 7$ day or 10080 min (assumed)

$C_s = 0.2$ (assumed)

$$= 0.5 + [50000(10080)/4790(0.2)]^{0.5}$$
$$R_o = 725 \text{ ft.}$$

Using the second estimation equation for R_o given in Table 5-25:

$$R_o = 3(H - h_w)(0.47K)^{0.5}$$
$$= 3(100-80)(0.47(500))^{0.5}$$
$$R_o = 920 \text{ ft.}$$

This comparison illustrates the importance of exercising good judgment in selecting well spacing.

5.4.2.3.2 Velocity Plots

Well spacing for complete plume removal can more accurately be determined by developing velocity plots for the pumping well (Keely and Tsang, 1983).

Velocity of flow through an aquifer can be expressed as (i.e., from Darcy's Law: Section 5.2.1):

$$V = Q/An = KI/n$$

where:

V = pore velocity (ft/day)
Q = flow rate (gpd)
A = area normal to flow direction (ft^2)
n = effective porosity (dimensionless)
K = hydraulic conductivity (gpd/ft^2)
I = hydraulic gradient (dimensionless)

Using Darcy's Law for calculating pore velocity, two important derivations can be made: water velocity toward a pumping well and natural flow velocity in the aquifer. Velocity toward a well can be represented by:

$$V_{pumping} = Q/An = Q/2\pi rhn$$

where the new terms are:

r = distance from well center to point where drawdown is measured (ft)
h = drawdown (ft)

This expression is typically used for pumping wells because Q is usually known for the well and A can be easily estimated. The area, A, in the above equation represents the area of the curved face of an imaginery cylinder (refer to Figure 5-2) for some distance away from the well. This area can be calculated by $A = 2\pi rh$ and substituted into the original Darcy equation.

For calculating natural flow velocities the following equation is typically used:

$$V_{natural} = KI/n$$

This equation is chosen because, for the aquifer under investigation, the average hydraulic conductivity (K) and hydraulic gradient (I) are usually known or can be estimated with a relative degree of accuracy.

Utilizing these two equations the downgradient stagnation point, i.e., the distance downgradient the pull of waters back toward the well by pumping is exactly countered by the natural flow velocity of water away from the well (Keely and Tsang, 1983), can be calculated by setting the expression for $V_{pumping}$ equal to the value of $V_{natural}$. This equality is represented by (Keely and Tsang, 1983):

$$V_{pumping} = V_{natural}$$

$$Q/2\pi rhn = V_{natural}$$

and then solving for r to provide the stagnation point:

$$r = (Q/2\pi hn)\ V_{natural}$$

where h is represented by saturated thickness of the aquifer. This relationship shows that the stagnation point is directly related to the pumping rate of the well (i.e., the higher the pumping rate the further downgradient the stagnation point) and inversely related to the natural flow velocity (i.e., the greater the natural flow velocity the closer the stagnation point is to the well).

The maximum distance to either side of the well perpendicular to groundwater flow, and therefore the well spacing, that contaminants are not drawn completely into the well has been defined by Todd (1980) as π times the downgradient stagnation point (πr). Hence, any contaminants that lie outside

the πr distance will not be captured by the well. These boundaries, πr downgradient and πr perpendicular to flow, define the areal limits of a pumping well's capture zone. Keely and Tsang (1983) have noted that for only the extremely rare case of zero natural flow are the areal boundaries of the capture zone identical to the calculated cone of depression. This means that even though the cones of depression of two pumping wells intersect they may not be completely capturing the plume unless their capture zones intersect. The ramifications of the this finding on well spacing for plume removal is paramount.

The use of the preceding equation can be used to calculate the capture zone of a pumping well and well spacing (πr) using a simple hand-held calculation. However, numerous computer models are available that can perform these calculations and provide velocity plots. Two such models are the radial flow time series model and the RESSQ model. Readers are referred to Keely and Tsang (1983) for a detailed explanation of these models.

An example of the use of the equality equation is provided below. Assume that the following data are available for the aquifer under investigation:

$Q = 100{,}000\ ft^3/day$
$K = 600\ ft/day$
$h = 100\ ft$
$I = 0.0015\ (15\ ft/10{,}0000\ ft)$
$n = 0.30$

Using these data in the equality equation for the downgradient stagnation point provides the following:

$$r = (Q/2\pi hn)\ V_{natural} = Q/2\pi hn(KI/n)$$
$$= (100{,}000/2\pi\ (100)(0.30))[(600)(0.0015)/0.30)]$$
$$= (100{,}000/2\pi\ (100)(0.30))(2.6)$$
$$= 204\ ft$$

The well spacing can therefore be calculated (as previously defined) by πr = 604 ft. This means that a line of extraction wells placed perpendicular to groundwater flow would have to be spaced at distances of 1208 ft (i.e., 2 πr) to capture the plume.

5.4.2.4 Pumping Rates

Pumping rates and drawdowns are related by:

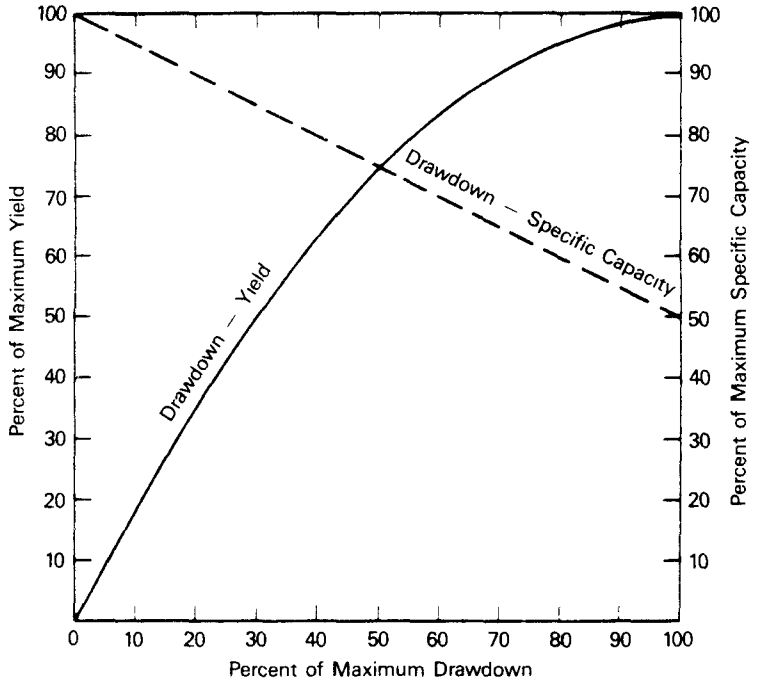
- Q/H^2-h^2 = constant for unconfined aquifers
- $Q/H-h$ = constant for confined aquifers.

This means that increasing the pumping rates will not affect the radius of influence, but will affect the time of pumping (refer to Figure 5-15). However, in reality, this may or may not be true for a field situation.

For unconfined aquifers, the optimum operating characteristics have been shown to occur at about 67 percent of the maximum drawdown. This can be demonstrated mathematically from plots of specific yield versus maximum drawdown, and specific capacity versus maximum drawdown (Figure 5-27). When the product of yield and capacity is a maximum, optimal operating conditions are obtained. Drawdowns greater than 70 percent of the maximum are uneconomical. This means that for an unconfined aquifer with a saturated thickness of 100 feet, the drawdown should be approximately 67 feet for optimum operating conditions to be obtained. Drawdowns less than 67 percent of the maximum may be desirable in cases when steady state, long term pumping is anticipated because the cones of depression will be the same.

For confined aquifers, the drawdown is directly proportional to the discharge as long as the drawdown does not exceed the top of the aquifer. Therefore, optimum operating conditions vary. Because pumping rates are directly related to drawdowns and will not affect the cone of depression, pumping rates can be obtained to suit the situation. In situations where the contaminate plume floats, drawdowns and pumping rates will probably be small. Large drawdowns and high pumping rates are desirable where contaminants are

FIGURE 5-27.
DRAWDOWN VERSUS YIELD AND SPECIFIC CAPACITY FOR A WATER TABLE AQUIFER (JOHNSON DIVISION, UOP INC., 1975)



dispersed throughout the aquifer, where quick removal is desired and natural groundwater flow rates are large. Close spacing of wells with large drawdowns and high pumping rates can very quickly dewater an aquifer.

5.4.2.5 System Integration

Once the well spacing, pumping rate, and drawdown have been determined, the system can be designed as a unit. At this point, a decision must be made on the pattern and type (i.e., injection or extraction) of wells to be installed. Numerous patterns of extraction wells or injection wells or both are available. The choice is typically based on whether the design is for containment or removal, the time available for recovery, and the amount of dewatering that is allowable. Straight line extraction or injection patterns would probably be used in containment, while the more complex patterns of extraction and injection wells would be used for removal. Patterns that combine extraction and injection wells allow for more rapid contaminant removal without greatly affecting groundwater levels. These patterns are also advantageous because the treated water extracted can be reinjected.

After a well pattern is chosen, the number of wells needed to control the plume must be determined. This is based on the estimated well spacing and the drawdown required. In spacing wells for plume control, it is necessary to have the well's capture zones intersect each other so that contaminants will not flow between wells and escape. Where low natural flow velocities exist, cone overlap may be small; where large natural flow velocities exist, cone overlap should be greater to ensure capturing the plume.

The number of wells needed can be determined by plotting the chosen map pattern of wells with their required spacing on the potentiometric surface map of the site. After this is done, the drawdowns within the radii of influence should be plotted and cumulative drawdowns determined. This will result in a new potentiometric surface map of the site that can be used to identify dead-spots or where contaminants can escape. Monitoring the system's effectiveness in contaminant removal should also be performed utilizing wells. The location of these wells can be determined from the revised potentiometric surface map by identifying deadspots or areas where cones of depression overlap. An example of a system design for an ideal, low natural flow velocity, water-table aquifer (i.e., homogeneous, anisotropic, unlimited extent) is given below and shown in Figure 5-28.

A contaminant plume originating from a landfill has been discovered and delineated as shown in Figure 5-28. Removal of the existing plume and containment of any new contaminants is desired. A pumping test conducted at the site determined the hydrogeologic parameters to be:

- $K = 25\ gpd/ft^2$
- $Q = 100\ gpm$
- $H = 100\ ft$
- $h_w = 46\ ft$
- $r_w = 0.5\ ft.$

FIGURE 5-28.
WELL PLACEMENT AND DRAWDOWN SUPERIMPOSED
ONTO GROUNDWATER MAPS

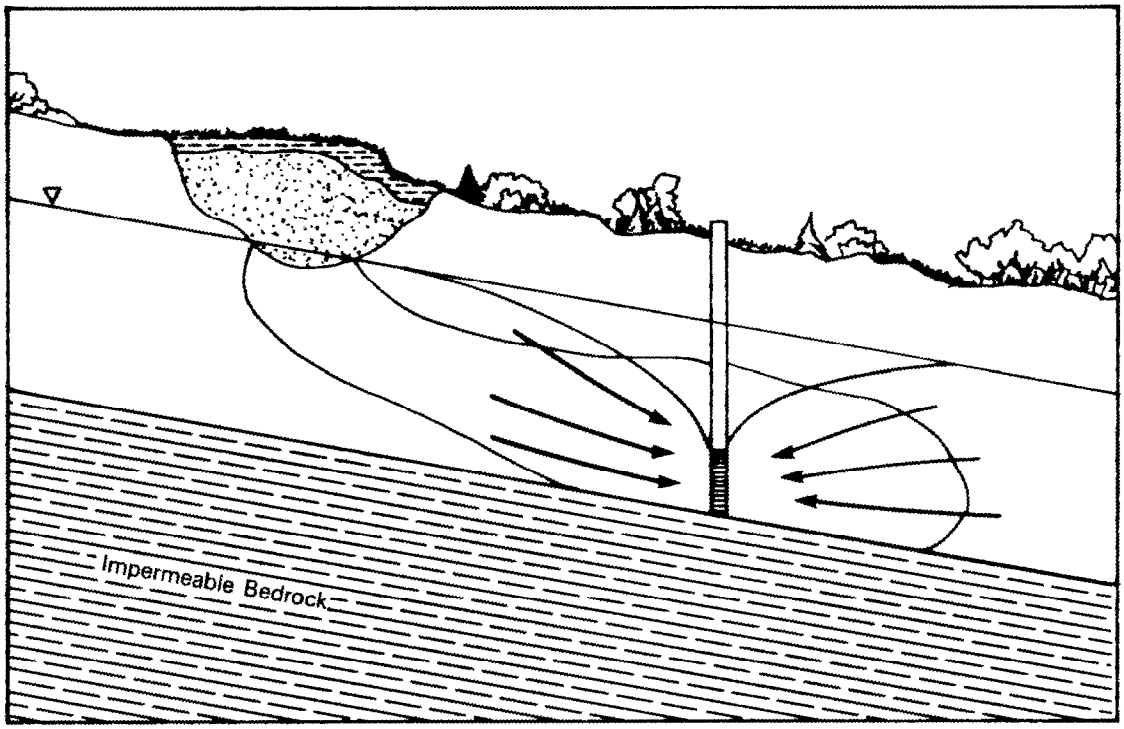


Figure 5-28a. Cross-sectional View

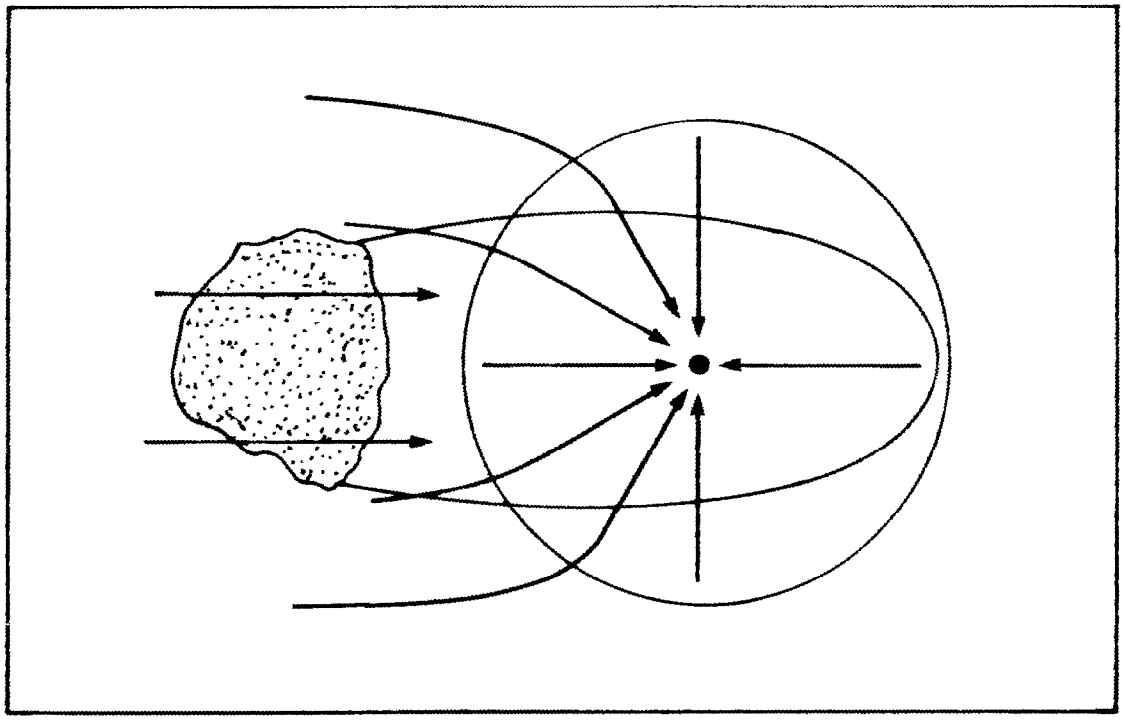


Figure 5-28b. Plane View

Continuous pumping is to be used at the site because the source of contaminants cannot be eliminated effectively. Based on this, the radius of influence for a pumping well is determined by:

$$R_o = e^x$$

$$\text{where } x = [K(H^2-h_w{}^2)/458(Q)] + \ln r_w$$

$$= [25(100^2-46^2)/458(100)] + \ln 0.5$$

$$= 4.996$$

$$R_o = e^{4.996}$$

$$= 150 \text{ feet (approximately).}$$

Because pumping is to be continuous, the optimum operating conditions for the well should be approximately 67 percent of the total drawdown. Using this drawdown ($H-h_{wm}$), pump rating (Q_m) and screen length can be estimated by:

$$(H-h_{wm}) = 0.67(H)$$

$$= 0.67(100)$$

$$= 67 \text{ feet}$$

$$h_{wm} = 33 \text{ feet}$$

$$Q_m = (Q/(H^2-h_w{}^2)(H^2-h_{wm}{}^2)$$

$$= (100/(100^2-46^2)(100^2-33^2)$$

$$= 113 \text{ gpm}$$

Using the calculated information, a single 6-inch well with a radius of influence (R_o) equal to 150 feet would provide plume removal and containment at the site (Figure 5-28). Optimum pumping conditions can be achieved with a continuously operated pump rated for approximated 113 gallons per minute. The length of screen installed should be less than 33 feet because drawdown will be approximately 67 feet. The material used for the well would be determined by the type of contaminants present.

5.4.3 Installation and Maintenance of Wells

Well installation typically consists of four steps:

- Opening the borehole
- Installing the casing and screen (if used)
- Completing the well
- Developing the well.

The first of these steps consists of opening a borehole for the well by dislodging and removing earth materials. Variations in the method used for dislodging and removing the earth materials are the main differences in well construction techniques. Screens and casings may be installed simultaneously with the opening of the borehole or after the borehole has been completed. The sequence of events depends on the technique used to open the hole and the geologic characteristics of the materials in the immediate vicinity of the hole. Installation of screens, filters, pumps, and grout completes the construction of the well. Well development is the last step and consists of dislodging and removing fines that have built up on the well screen, the filter, and walls of the aquifer during the previous steps. Removal of these particles maximizes well yield and prevents damage to the pump.

Well maintenance generally is performed periodically after installation of the well to maintain yields. Maintenance programs consist of recording performance, evaluating the need for maintenance, and treating the well to improve yields. The three basic causes of reduced yield that require well maintenance are pump failure, corrosion, and incrustation.

5.4.3.1 Opening Boreholes and Installing Casing

There are numerous methods for opening holes and installing casing which can be classified according to the well depth:

- Shallow wells (<50 feet)--dug, bored, driven, and jetted wells

- Deep wells (>50 feet)--drilled (e.g., hollow-core, hydraulic-rotary, air-rotary), cable tooled, and jetted wells.

The method chosen depends on the geologic characteristics of the soil and rock, the well diameter, and the hydrologic characteristics of the aquifer. Wells can also be classified by the type of geologic materials they penetrate (i.e., consolidated and unconsolidated). Regardless of the method chosen for well installation, exploration techniques should be employed prior to and during the opening of the borehole. While installation is occurring, continuous testing to monitor changes in the soil and water being removed should be performed so that installation techniques can be modified as necessary. The various means of opening a borehole and installing casing and screen are described in Table 5-26.

5.4.3.1.1 Hand Methods

Shallow wells up to six inches in diameter can be pushed into soft soils by hand methods. Depths of these wells is generally limited to between 20 and 30 feet, but 90 foot wells have been installed in this manner. Wells installed by hand methods are usually wellpoints attached to a riser pipe. The maximum depth to groundwater in these wells is generally less than 20 feet because suction pumps are utilized. Manual methods for opening a hole are typically driving and augering. The use of augers is limited because they are ineffective in water saturated sands and coarse silts. A combination of the two methods is often used successfully to reach greater depths. Deeper and larger diameter wells can be installed if heavier driving block assemblies are utilized. These blocks can weigh 75 pounds or more and can be operated by a block and tackle or with a drill rig.

5.4.3.1.2 Boring

Wells can either be bored with large diameter auger buckets or with continuous flight, spiral augers. Rotary bucket augers are capable of opening holes up to 48 inches in diameter and 90 feet deep in very soft soils. Diameters are typically limited to about 24 inches in most materials.

TABLE 5-26. METHODS OF WELL-INSTALLATION

Method		Applications			Casing/Screen Treatment	Comments
Basic	Variations	Geologic Material	Max. Well Dia. (in)	Max. Well Depth (ft)		
Hand	Augers	Soft, without excess sand and water, no boulders	6	20	Driven after opened	
	Driving Points	Soft soil free of boulders	3	30	Driven as hole proceeds	Wider and deeper holes can be machine driven
Boring	Rotary Auger Bucket	Soft soils without excess boulders	48	90	Driven after hole opened	Typically limited to 24-in diameter wells or less
	Spiral Auger	Soft soils without excess sand and water, no boulders	6	90	Driven after hole opened	Can not be used below water table, must be used in combination with other techniques
Jetting	Self jetting	Soft soils free of boulders	8	50	Drives as hole opened	Jetted wells require less development
	Wellpoint/Riser Unit	Soft soils free of boulders	8	50	Driven as hole opened	Method uses large quantities of water
	Separate Temporary Jetting Pipe	Soft soils free of boulders	8	100	Placed after hole opened	
	Separate Permanent Jetting Pipe	Soft soils free of boulders	8	100	Driven as hole opened	

(Continued)

TABLE 5-26. (Continued)

Method		Applications			Casing/Screen Treatment	Comments
Basic	Variations	Geologic Material	Max. Well Dia. (in)	Max. Well Depth (ft)		
Jetting (cont.)	Rotary Hole-puncher	Soft soil, sandstone, schist	24	200	Driven as hole opened	
Rotary Drilling	Conventional Hydraulic	Any type	--	--	Placed after hole opened	Typically used for deep wells, quickest drilling method
	Reverse Hydraulic	Any type, boulders may be a problem	60	--	Placed after hole opened	Requires large quantities of water
	Air Rotary	Any type	12	--	Placed after hole opened	Very fast method
	Air Rotary with Pneumatic Hammer	Any type	8	--	Placed after hole opened	
Cable Tool	Conventional	Any type	--	--	Driven during or after hole opened	Typically used for deep wells, provide ability to closely monitor and log well
	California Stove Pipe	Any type	--	--	Pushed with jacks as hole opened	Quicker and less expensive than conventional cable tool

However, this type of auger can be used in soils with minor caving problems if the boreholes are filled with light drilling mud. The mud not only prevents caving but helps keep porous materials in the bucket. The use of drilling muds is not recommended for use in contaminant monitoring wells because of possible analytical water quality testing interferences.

Spiral screw augers are typically limited to diameters between 4 and 6 inches and depths less than 90 feet. They can be ineffective when used below the water table or in sandy soils because the screw will not lift the soil. Spiral augers are used most commonly in conjunction with other methods.

Tools used for rotary bucket augers include the bucket, drilling pipe or rod, a kelley bar, mechanical pull downs, and boulder handling equipment. The cylindrical bucket has cutting blades on the bottom and is attached to the kelly bar by rods. Drill rods supply the length necessary to reach into deep holes and can be eliminated in shallow holes. The kelly bar transfers the circular motion from the rotary table and the downward force of the mechanical pull-downs to the rods and the bucket. Any large boulders in the path of the borehole must be removed using orange peel buckets, stone tongs, or ram horn tools.

Spiral augers are less complicated and easier to use than bucket augers. The bucket, rods, and kelly bar are replaced by a screw auger which does the cutting. Cuttings are pushed to the surface by spiral action. Spiral augers are attached directly to the rotary table through the drill rods.

Casing installation for augered wells can normally be done after the hole has been opened to its entire depth. One exception is when bucket augers and drilling mud are used in sandy soils. In this case, casing should be driven closely behind the progress of the bucket. For applications where the borehole is not in danger of caving, the casing can be installed after the hole is completed. Casing is normally installed by pushing it with a drive block similar to the ones used for driving wells. The top of the casing is protected with a drive head and the bottom with a drive shoe.

5.4.3.1.3 Jetting

Jetting involves opening the borehole using the force of a water jet. Water jets typically have flow rates of 100 to 250 gallons per minute and develop pressures of 50 to 300 pounds per square inch. The force of the water jet not only opens the hole but also conveys the waste materials back to the surface. Jetted wells can range in diameter from a few inches to about 24 inches and can reach a depth of up to 200 feet. Jetting can be done in geological materials as hard as sandstone and schist, but most often is used in soil (unconsolidated) materials. Since the equipment required for jetting is fairly sophisticated and equipment set-up times are long, jetting is most commonly used for installing many closely spaced wells (e.g., wellpoint systems).

There are six basic jetting methods. Four of the jetting methods are used for installing small diameter, shallow wells such as wellpoints. These four methods are:

- Self jetting, permanant drop tube--The entire wellpoint is jetted down and remains in place. The permanant drop tube eliminates the need for temporary jetting pipes. Riser tubes are used to extend the length of the well as jetting proceeds and to act as casings.

- Self jetting, separate drop tube--Same as the self jetting, permanant drop tube method except that the drop tube is removed after jetting is completed.

- Separate temporary jetting pipe--A capped pipe with cutting teeth at the bottom is jetted down by water forced through openings in the cap. If soil becomes difficult to remove, the pipe is rotated back and forth to aid the jetting process. Once the well is pushed to the final depth, the wellpoint and riser are placed in the jetting pipe and the jetting pipe is removed.

- Separate permanent jetting pipes--Similar to the separate temporary method except that the wellpoint with screen is packed into the jetting pipe, the pipe is lifted to expose the screen, and the jetting pipe then becomes the riser casing.

Deeper wells can be installed using two jetting methods; jet-percussion and rotary holepunchers. In the jet-percussion method, a chisel bit is used

that has jetting nozzles attached. The nozzles direct water onto the bit for cleaning and at the bottom of the hole to loosen material. The chisel bit is lifted and dropped with a block and tackle or motorized assembly. The combined action of the bit and the water jet open the wellhole. Drilling water is circulated in a closed loop to minimize water consumption. Once the water emerges from the bit, it rises along the outside of the drilling pipe carrying cuttings with it. Cuttings are settled out in a pit and the water is recycled. Casing is driven as the hole progresses, and well screens are installed through the casing. The rotary holepuncher method is similar to the jet-percussion method except that the bit is rotated by rotary drilling methods instead of being lifted and dropped. Rotary drilling methods are described in the next section.

5.4.3.1.4 Rotary Drilling

There are two basic components of rotary drilling--the drill bit and stem, and the drilling fluid. The rotary action of the drill bit and stem being forced down by mechanical or gravitational means opens the borehole. A continuous flow of drilling fluid prevents the hole from collapsing and removes the cuttings. Casings and screens are typically installed after the borehole has been completed. Conventional rotary drilling and its variations can be used to install wells of any diameter and depth in any geologic material. While this is one of the fastest drilling methods, equipment set-up time and labor costs can prohibit its use for shallow wells.

In conventional hydraulic rotary drilling, the flow of the drilling fluid is similar to that for jet-percussion where fluid flows down the drilling rod, out the drill bit, and up the sides of the hole. Besides removing cuttings and keeping the hole from collapsing, the drilling fluid:

- Prevents the bit from sticking when operations are interrupted
- Seals the well hole to prevent fluid loss
- Cools and cleans the drill bit
- Lubricates bits, pumps, and pipes.

The properties of the drilling fluid that can be controlled to ensure satisfactory performance are density, viscosity, sand content, gel strength, and filtration properties. The disadvantages of using a drilling fluid is greater drill rig decontamination time and well development expense.

Reverse circulation, rotary drilling is a variation of the conventional method that is appropriate for soft soils where water tables are greater than 10 feet below the surface. Reverse rotary is the least expensive method for wells ranging from 18 to 60 inches in diameter. The drilling fluid is more of a water than a mud in this method and fluid losses can be as great as 500 gallons per minute. Because of this, an ample water supply must be available and a settling pit of three times the final borehole volume should be designed. The major difficulties with the method are high fluid loss and caving when drilling in loose, permeable soils.

Another variation of conventional rotary methods is the air rotary method. The drilling fluid in this case is high pressure air rather than a water-based fluid. Air rotary methods are good for consolidated rock drilling and will allow faster penetrations and longer bit life as long as water infiltration into the hole is small. Typical hole diameters are 12 inches or less but larger bits are available. This method is frequently used in combination with conventional hydraulic rotary methods to complete a single hole.

A variation of the air rotary method is air rotary with pneumatic hammers. This method may be the quickest when drilling through very hard geologic formations. Well diameters with this method are typically 8 inches or less. The pneumatic hammer requires air pressures greater than 300 pounds per square inch when drilling larger holes. Both air rotary methods have the advantage of being able to observe water inflows into the well as drilling progresses.

5.4.3.1.5 Cable-Tool

The cable-tool method (also known as the "percussion," "solid-tool," "standard," and "churn" method) opens a hole by lifting and dropping a heavy string of tools which loosen and crush the geologic material. The resulting particles or cuttings are mixed with water from the well or water pumped into the well and the resulting slurry is removed with a bailer. When drilling in soft soils, the casing closely follows the bit to prevent caving. In hard rock formations the casing can be installed after the hole has been completed. The cable tool drilling method is typically used for deep well installation, is relatively slow (5 to 10 ft/day in solid rock, 50 to 100 ft/day in soft soil), and is high in labor costs. Conventional cable-tool methods and a single variation are discussed below.

Conventional cable-tool equipment consists of a bit, a stem for length and weight, jars to loosen stuck stem and bits, and a rope socket which connects the entire string to the drill cable. The up and down motion of the tools results from the motorized lever called the walking beam or the spudder (i.e., winch). The bailer is on a separate cable called the sandline. The bailer is comprised of a pipe with a check valve that opens as the pipe enters the hole and closes as it is lifted, thereby removing cuttings.

One variation on the standard method is the California Stovepipe. The major differences are:

- A mud scow acts as both bit and bailer
- A thin pipe within a pipe ("stovepipe") is used as a casing as opposed to standard line pipe
- The casing is pushed with jacks rather than hammered with tools.

A unique feature of the casing is that it is too weak to pull back out of the hole to expose the well screen. Therefore, perforations are blasted into the bottom of the casing to allow water flow and act as a screen.

5.4.3.2 Well Completion

Once the borehole has been opened installing a screen, filter pack, and grout is necessary to complete the well prior to well development. The order that these installations occur is dependent on whether the borehole is oversized and requires a filter pack or not. The majority of the wells that are drilled for contaminant removal will require filter packs.

5.4.3.2.1 Filter Pack Wells

The method typically utilized for completing a well with an artificial filter is the double-casing method. In this method, a string of outside casing, corresponding to the size of the outside diameter of the filter pack (i.e., the borehole), is installed as the hole is drilled or after it is completely opened. A second string of casing containing the well screen is then centered within the outer casing. The selected filter material is then placed between the inner and outer casings. After placement of a few feet of filter material, the outer casing is pulled back an equal amount and the procedures is repeated until the filter material is brought to the desired level above the screen.

The outer casing may be removed completely or left in place above the level of the screen. In either case, the top of the annular space above the filter is sealed with grout (e.g., cement, clay). If the outer casing is to be permanent, the inner casing above the screen may be removed as long as the two overlap a few feet. The top of the inner casing should be sealed using a lead slip packer. The annular space left between the outer casing and the aquifer should also be sealed with grout. Pumps are then installed into the inner casing and the well is developed.

Jetted wellpoints are completed in a similar manner as a drilled well. In this case, filter sands are packed around the wellpoint and grout is installed from the top of the filter to the surface. The grout prevents surface water infiltration into the well and minimizes the chances of air entering the wellpoint.

5.4.3.2.2 Naturally Developed Wells

The method used for completing a naturally developed well is the opposite of the method for filter pack wells. Grouting and sealing, if necessary, occur first followed by installation of the well screen. The two basic approaches to grouting are:

- Slurry placement method in which grout is pumped or gravity fed into the annular space
- Casing method in which well casing is used to keep the borehole open while installing grout.

Regardless of the method used, the grout must always be placed from the bottom of the well up. If a temporary outside casing has been installed during drilling, the casing must be removed while the grout is still fluid. This allows for a good seal between the borehole walls and the grout. Once the grout has set, the plug can be drilled out and the well screen installed.

Numerous methods are available for installing well screens depending on the type of well screen used, the drilling equipment, the geologic material, and the presence of grout. The types of well screen installation methods are:

- Pull back method--Casing is sunk to the full depth of the borehole, the well screen is lowered inside the casing and the casing is pulled back to expose the screen. The casing can not be grouted.
- Setting in open hole--The casing is grouted in place, a hole is drilled through the grout to below the level of the casing and the well screen is installed inside the casing in the drilled hole.
- Bail-down procedure--The well screen is fitted with a bail down shoe attached to a bailing pipe, the assembly is sunk into the formation below the well casing by operating the bailer and drilling tools through the screen, the screen moves into the hole formed by displacing the soil, and a packer at the top of the screen is expanded after the pipe is removed. This method is good only in soft, sandy soils.
- Wash-down method--The well screen is installed using the same method as for jetting techniques and a packer is used to seal the screen and the casing.

Once the well screen is installed the pump can be placed within the casing. Well development can then take place to ensure adequate yield.

5.4.3.3 Well Development

Well development is the process where fine soil materials are removed from in and around the screen allowing water to flow freely. This process is accomplished by surging water or air through the well screen and into or out of the surrounding material. The well development process:

- Removes materials that have built-up in the openings of the screen during the well drilling and installation processes
- Removes fines from the sides of the borehole that resulted from the drilling procedure, e.g., drilling mud
- Increases the hydraulic conductivity of adjacent geologic materials and the filter pack by removing fine materials
- Stabilizes the fine materials that remain in the vicinity of the well and retards their movement into the well.

The benefits of well development are increased yields, reduced pumping of fines which can damage pumps, and decreased corrosion and encrustation. The results of the development process is a layer of coarse particles adjacent to the screen. The percentage of finer particles increases with distance away from the well. Well development is necessary in any well because clogging can occur regardless of the drilling method used or the formation being penetrated.

Wells can be developed by natural groundwater flow or by artificial means. Prior to developing a well, the well should be bailed to remove sand that has accumulated during construction. Bailing also ensures that water will flow into the well. Development procedures can last from about two hours to two days depending on well depth and formation properties.

Natural development of wells is accomplished by alternately pumping and allowing the well to recover. This method does not cause much of a surging

action and is relatively ineffective in alleviating particle bridging and in removing fines. There are seven additional methods that can be used in well development. They are:

- Overpumping--This method develops the aquifers by pumping at high rates and assumes that the system will be stable at normal pumping rates. When the pumps are stopped, backwash helps overcome bridging. While this method is the simplest and quickest, it is also the least effective.

- Rawhiding--This method involves pumping intermittently to lift water out of the well and then adding water back into the well causing a surging action. Rawhiding is limited to wells without check valves that prevent backwashing. The method is not normally recommended as a primary development method but is used commonly as a finishing method.

- Recirculation--In this method, water is pumped from a high point in the well and discharged at a low point in the well screen. This results in a mild turbulence at the screen which frees fines. It is not generally used as a sole well development method.

- Surge blocking--In this method, a surge block or surge plunger is pushed in and pulled out of the well in a plunger-like fashion. The plunger can be solid or valved. Valved plungers allow action on the downstroke and strong action on the upstroke, but care must be taken on the upstroke because screens can be collapsed. This method is the most common and highly effective method of development.

- Hydraulic jetting--In hydraulic jetting, a device with two or more nozzles is pushed to the wellscreen and water is jetted through the screen openings. The water reverses direction returning through the screen and carries fines into the well for removal. Pumping the well while jetting helps flow reversal and fine removal, and provides water for jetting.

- Compressed air surging--This method is a combination of air jetting and pumping with water. Air is gently pumped into the well and water is pumped out of the well to start the circulation. The airline is then closed off and pushed down into the screen area while air pressures build to 100 to 150 pounds per square inch. An air blast is then released surging through the system. This cycle is repeated several times.

- Sonic development--In this method, high frequency sound waves are sent through the well screen and the surrounding formation where they vibrate and loosen particles. Because the waves effectively penetrate beyond the well, this method is very attractive for filter packed wells.

The methods previously discussed can be aided with the use of dispersing agents and acids. Dispersing agents (such as polyphosphates) act as deflocculants and disperse clay. Mild acids can be used to dissolve limestone and open crevices. In rock formations, blasting has been used to enhance well yields. However, blasting is not a recommended procedure for most hazardous waste release sites.

5.4.3.4 Well Maintenance

Pumps, casing, and screens must be maintained to ensure a constant reliable flow of water from the well. Proper well maintenance is especially important in plume management because the loss of a well could result in contaminant escape. The causes of well yield loss and failure are:

- Encrustation--The build-up of scale or other chemical or biological material
- Corrosion--A breakdown of well hardware by turbulent flow, chemical reactions, galvanic reactions, or fatigue stress
- Pump failure--Typically caused by sand intrusion, wear, or mechanical or electrical failure.

Prior to beginning any well maintenance, a preliminary evaluation must be conducted to identify whether or not the problem can be corrected. Operational records will determine the normal operating conditions of the well and aid in evaluating the problem. Removing the pumps and checking the casing and screens may be necessary. This can also be accomplished by downhole video equipment.

General maintenance procedures that can be used for encrustation, corrosion, and related pump problems include:

- Chemical treatment of casing and screens
- Redevelopment of the well
- Chemical treatment of pumps, and mechanical and electrical maintenance.

Chemical treatments are designed to dissolve encrustation and corrosion that has formed or have been deposited on the well components. Three classes of chemicals are generally used; acids, biocides, and phosphates. Acids are used to dissolve inorganic substances that have formed, such as calcium, magnesium, and iron. Biocides are used to remove organic materials that may have developed such as iron bacteria. The effectiveness of both of these treatments is directly related to retention times on the affected components. Additives are available to aid in the treatment including:

- Inhibitors--Allow for the selective removal of some substances
- Chelates--Keep metals in solution to prevent redeposition
- Wetting agents--Reduce surface tension allowing chemicals to penetrate smaller openings.

Phosphates or surfactants act as dispersing agents which help break-up clays, colloids, and some metals. The effectiveness of surfactants depends only on initial contact; prolonged contact does not increase their effectiveness. Whenever surfactants are used (especially phosphates), chlorine should be used to prevent bacteria and algae growths.

The best and most common application method used in chemical treatment is the double surge block. This method allows the application of concentrated chemicals directly to the required areas. If chemicals are necessary to clean the surrounding aquifer they can be injected through the screen using compressed air or surge blocking techniques.

Mechanical redevelopment of the well can also be used to increase yields and remove encrustation. Redevelopment should always follow chemical treatments. The seven development methods described previously can be used.

5.5 Costs of Well Systems

Costs of well systems for plume management can vary greatly from site to site. Some of the factors that detemine these costs are the geology, the

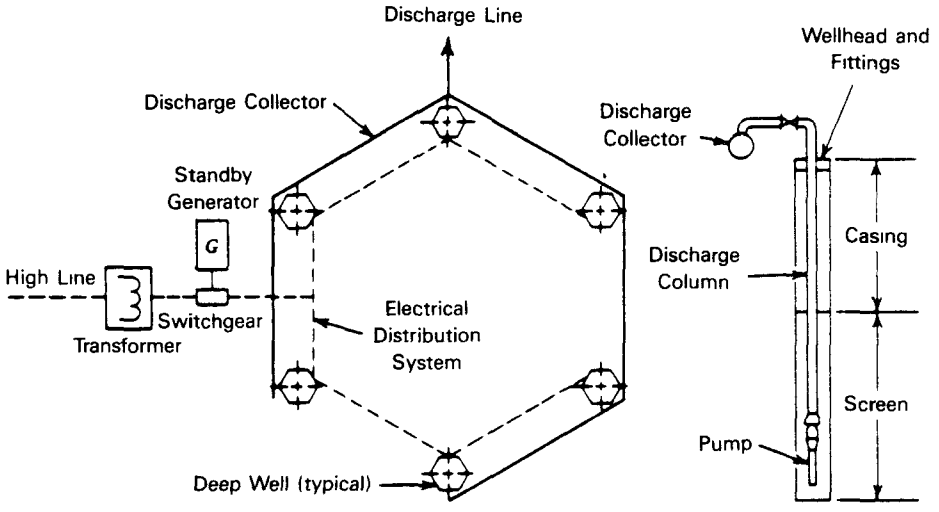
characteristics of the contaminant and naturally occurring groundwater, the extent of contamination, the periods and durations of pumping, local wage rates, the availability of supplies and equipment, and the electrical power required. Costs associated with a well system can be grouped into the following categories (Powers, 1981):

- Mobilization costs
- Installation and removal costs
- Operation and maintenance costs.

Mobilization costs include all costs incurred in obtaining equipment and having it available at the site. Some of the items included in mobilization costs are (Powers, 1981):

- Well components--including well screens, casing, well pumps, motors, controls, discharge columns, well heads, fittings, collector pipes, and power lines (Figure 5-29)
- Installation equipment--including drilling rigs, jetting equipment, and well development equipment
- Pumping equipment--including pumps, contacts, hoses, and cables
- Standby equipment--including generators, switches, pipes, and cables
- Equipment rental and repair
- Delivery and handling charges for equipment
- Utility installation
- Enclosures for storing equipment
- Engineering and geotechnical services--including the design of the system, submittal preparation, field testing, and on-site supervision during installation
- Waste, water, and soil treatment--including transport, treatment, and disposal
- Decontamination of drill rigs and tools
- Health and safety precautions.

FIGURE 5-29.
TYPICAL WELL SYSTEM COMPONENTS
(POWERS, 1981)



Installation and removal costs include the costs for crews and equipment necessary to install the well system at the site. These costs should also include allowances for set-up, clean-up, weather, and other miscellaneous delays that typically occur. Removal costs will probably be incurred at all sites at some point when pumping is no longer required. Removal costs can be off-set somewhat by salvage costs of the removed equipment. However, decontamination of the equipment may be more costly than salvage value.

Operation and maintenance costs are typically high for pumping systems. In some cases, these costs can be greater than the initial installation and mobilization costs. The following items should be considered in preparing cost estimates (Powers, 1981):

- Operating labor--continuous or intermittent manning of equipment

- Maintenance labor--includes servicing for pumps, well cleaning, and maintaining engines and electric equipment
- Supervision
- Energy--includes fuel and electricity
- Maintenance materials--includes materials for surface components and for well rehabilitation
- Repair and overhaul of equipment.

The above list of cost items is not all inclusive; some sites will require additional items. If long term operations are expected, operation and maintenance, and removal costs should include escalating factors. Tables 5-27 through 5-29 give some typical costs that may be incurred for the items mentioned above. However, because the costs of many of these items will vary with site location, specific estimates from local contractors and suppliers should be obtained.

TABLE 5-27. RANGE OF COSTS FOR SELECTED PUMPS AND ACCESSORIES
([1/] STA-RITE INDUSTRIES, INC.)

Pump/Accessory	Description	Cost Range (1981 $)
Jet Pump[1/]		
- shallow well	pumping depths 25 ft horsepowers 1/3 to 1-1/2 HP capacities 60 to 27000 gph	$220 to $430
- deep well	pumping to depths 320 ft horsepowers 1/3 to 2 HP capacities 60 to 1000 gph	$220 to $570
- jets and valves	single pipe jets double pipe jets	$40 to $80 $30 to $50
- seals	single or double pipe	$15 to $25
- foot valves		$10 to $40
- air volume controls		$10.00
Submersible Pumps[1/]		
- 4 in. pump	pumping depths 900 ft horsepowers 1/3 to 3 HP capacities 50 to 2000 gph	$350 to $1400
- control boxes		$60 to $120
- magnetic starters		$160 to $250
- check valves		$14 to $330
- well seals		$15 to $100
Vacuum Pumps		
- diesel motors	800 - 7000 gpm (48,000 -- 420,000 gph)	$13,000 to $40,000
- electric motor	800 - 7000 gpm (48,000 -- 420,000 gph)	$9,000 to $31,000

TABLE 5-28. TYPICAL RANGE OF COSTS FOR WELLSCREENS AND WELLPOINTS (Johnson Division, UOP Inc., and Gator Plastics Inc.)

Type	Description	Costs (1981 $)
Drive Wellpoints	stainless steel 1-1/4 to 2-in ID	$28.00 to $40.00/ft
	low carbon steel 1-1/4 to 2-in ID	$15.00 to $35.00/ft
	PVC plastic 1-1/4 to 2-in ID	$5.00 to $6.00/ft
Wellscreens	stainless steel 1-1/4 to 36-in ID	$28.00 to $610.00
	low carbon steel 1-1/4 to 36-in ID	$15.00 to $170.00/ft
	PVC plastic 1-1/4 to 12-in ID	$10.00 to $60.00/ft
Jetting Screens (fittings)	cast iron or mild steel 2 to 12-in ID	$30.00 to $270.00
Baildown Shoe (fittings)	mild steel 4 to 12-in ID	$180.00 to $800.00

TABLE 5-29. AVERAGE DRILLING COSTS (1981) FOR UNCONSOLIDATED MATERIALS (Stang Drilling and Exploration)

Drilling Technique	Drilling Costs[3/] ($/hr)	Average Production Rates (ft/hr)	Drilling Cost ($/ft)
Conventional Hydraulic Rotary	$100/hr	40 ft/hr	$2.50/ft
Reverse Hydraulic Rotary	$220/hr	40 ft/hr	$5.50/ft
Air Rotray[1/]	$175/hr	50-60 ft/hr	$3.50 - $3.00/ft
Air with Pneumatic Hammer	$180/hr	40-50 ft/hr	$4.50 - $3.60/ft
Auger	$90/hr	20-40 ft/hr	$4.50 - $2.50/ft
Bucket Auger	$100 to $140/hr	50 ft/hr	$2.00 - $2.80/ft
Cable-Tool	$60-$70/hr	4 ft/hr	$15.00 - $17.50/ft
Hole Puncher (jetting)[2/]			$35/ft
Selfjetting[2/]			$18/ft

[1/] consolidated material

[2/] includes rental of all necessary equipment; e.g., wellpoints, pumps and headers

[3/] drilling costs approximately equal mobilization costs

CHAPTER 6

SUBSURFACE DRAINS

6.1 Introduction

Subsurface drains include any type of buried conduit used to collect liquid discharges (e.g., contaminated groundwater) by gravity flow. The key components of a typical subsurface drainage system are:

- Drain pipe or gravel bed--for conveying flow to a storage tank or wet well
- Envelope--for conveying flow from the aquifer to the drain pipe or bed
- Filter--for preventing fine particles from clogging the system
- Backfill--to bring the drain to grade and prevent ponding
- Manholes or wet wells--to collect flow and pump the discharge to a treatment plant.

The theory of groundwater flow and the effects of various aquifer properties on flow are similar for drains and wells. Well theory is discussed in Chapter 5. Specific differences and similarities between flow toward drains and wells are described in Section 6.2.

Section 6.3 discusses site specific variables which determine the applicability and performance of subsurface drains. Soil properties, leachate characteristics, size and flow rate of the plume, aquifer properties, and site geology are considered relative to the design and performance of subsurface drains. The design considerations discussed in this section include drain depth and spacing, hydraulic design of pipes (pipe diameter and gradient, and flow velocity), and envelope and filter materials. This section also includes

a discussion of design criteria for a sump and pumping station for collection and transfer of leachate to a treatment system.

Equipment, methods, and unit costs for the construction, installation and maintenance of subsurface drainage systems are described in Section 6.4. Construction and installation can be divided into two major activities: (1) excavation of the trench to the required depth and gradient, and (2) installation of drain materials. The discussion of trench excavation in this section includes methods and equipment for trench excavation, wall stabilization, dewatering, and grade control. Drain installation includes materials and procedures for installing bedding, drain pipes, envelopes, and filter material. This subsection also includes procedures for backfilling the trench. Unit costs are provided throughout this section for equipment, methods, and procedures. The section also provides guidance on the selection of equipment and methods based on depth requirements, subsurface geology, and soils.

Section 6.4.3 addresses procedures and methods for maintenance and inspection of subsurface drains. This section includes a discussion of problems which might arise during construction and operation of the system. Inspection, preventative maintenance, and corrective action procedures to ensure against system failure are also discussed. Potential problems which can impact the performance of drains include clogging of envelopes and filters by siltation; chemical build-up or incompatibility; clogging as a result of sediment build-up in manholes; root clogging of tile drains; and development of sinkholes.

Subsurface drainage can be used to control leachate plume migration for a wide range of site conditions. They essentially function like an infinite line of extraction wells. That is, they create a continuous zone of depression in which groundwater within this zone of influence flows toward the drain. Accordingly, subsurface drains can perform many of the same functions as pumping technologies. The decision to use one or the other should be based upon a cost-effective analysis. However, drains may be more cost-effective than pumping at sites with substrata of low or variable hydraulic conductivity

and where pumping systems cannot provide a continuous hydrologic barrier. They are limited to relatively shallow applications.

Subsurface drains can be used to contain or remove a leachate plume or to lower the groundwater table near a disposal site in order to prevent contact of water with waste materials. Containing or removing a plume located hydraulically downgradient of a site typically requires less extensive drainage than lowering the groundwater table. However, containment or removal generally requires deeper drains than systems used to lower the groundwater table in order to ensure that the entire plume is intercepted. This can be seen by comparing Figures 6-1 and 6-2.

Subsurface drains can be applied with liners or other barrier materials to restrict inflow to one side of the drain. A typical situation where this technique would be particularly applicable is when a surface water body located near a subsurface drain was contributing a substantial flow of clean water into the drainage system. The use of conventional subsurface drainage would result in large flow capacity requirements for the drain and high costs for treating the additional water. One sided drainage could be used primarily to collect the contaminated portion of groundwater and cutoff recharge flow from the surface water body. The use of one sided subsurface drainage requires that the system be keyed into a low permeability formation so that groundwater does not travel under the barrier material (Figure 6-3). One sided subsurface drainage may be applicable to many other conditions where the main objective is to restrict the flow of clean water, and thus, minimize water treatment requirements.

Subsurface drains are also used in conjunction with groundwater cutoff barriers to prevent the buildup of groundwater upgradient of the barrier. In this application, they are not the main control technique, but rather, serve an ancillary function. When used with barriers, subsurface drains may be designed to handle the reduced flow conditions of a totally encapsulated site (Figure 6-4) or to limit the up-gradient head increase preventing overflow and minimizing differential hydraulic pressures on the barrier (Figure 6-5).

FIGURE 6-1.
THE USE OF SUBSURFACE DRAINAGE TO CONTAIN A LEACHATE PLUME

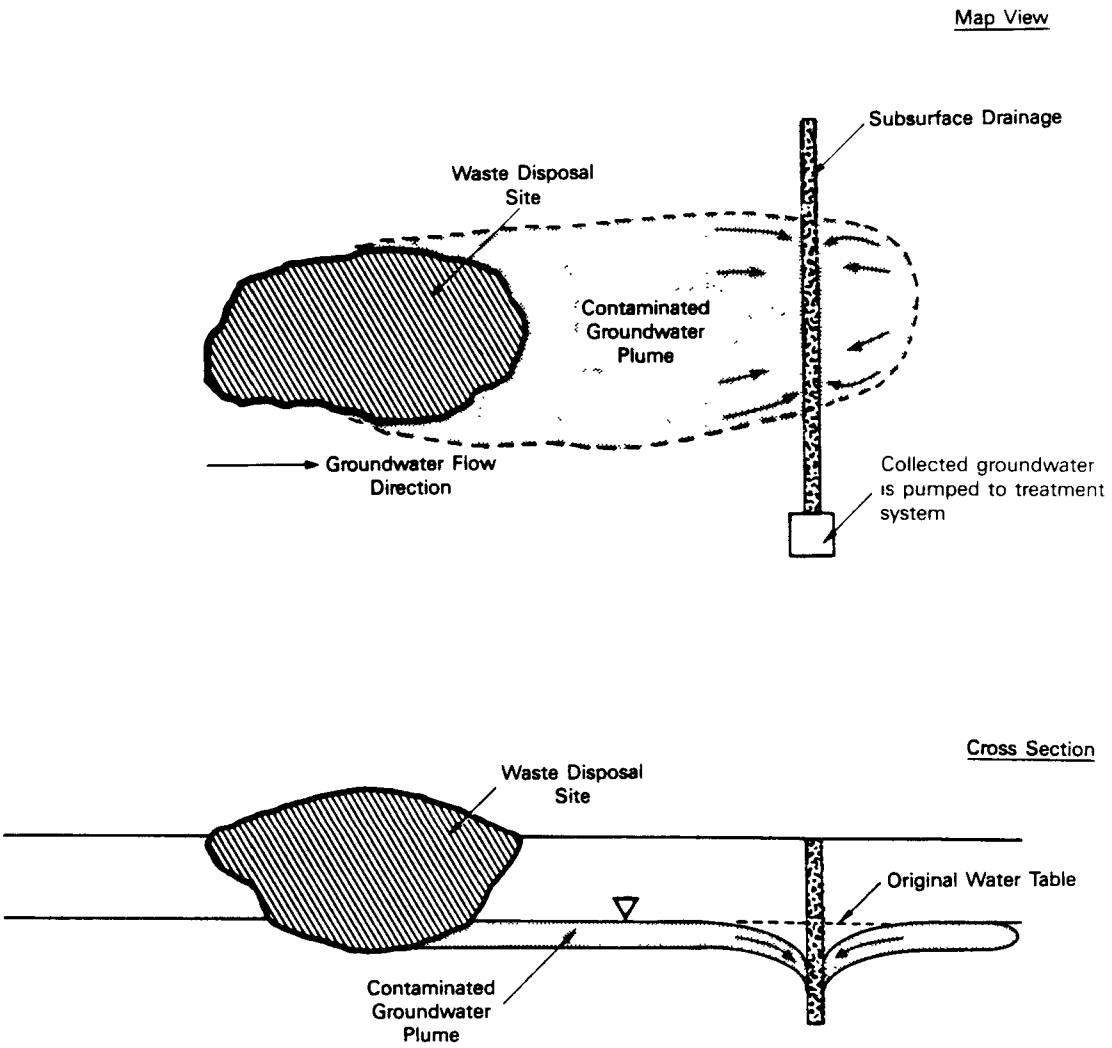


FIGURE 6-2.
THE USE OF SUBSURFACE DRAINAGE TO LOWER GROUNDWATER LEVELS

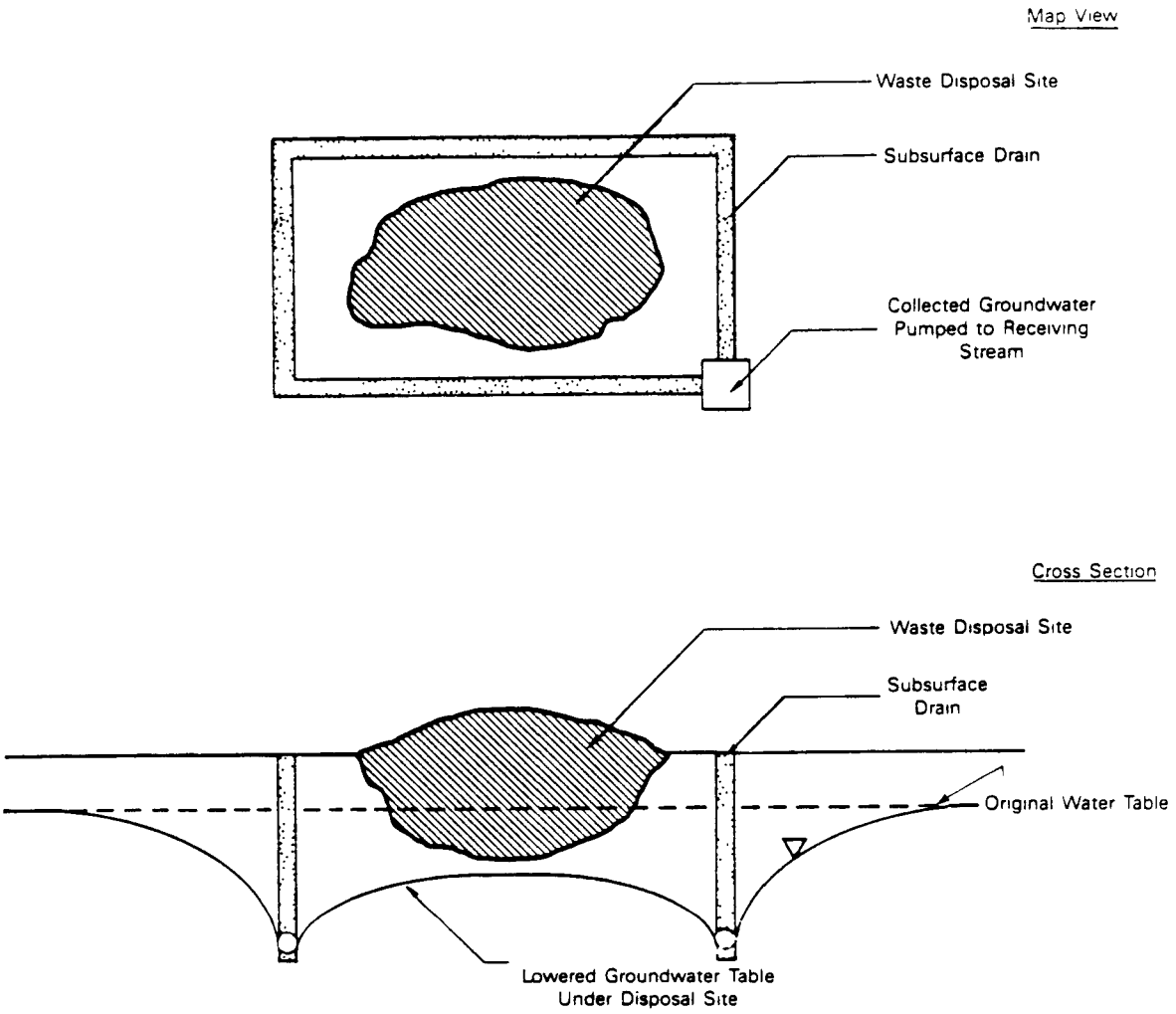
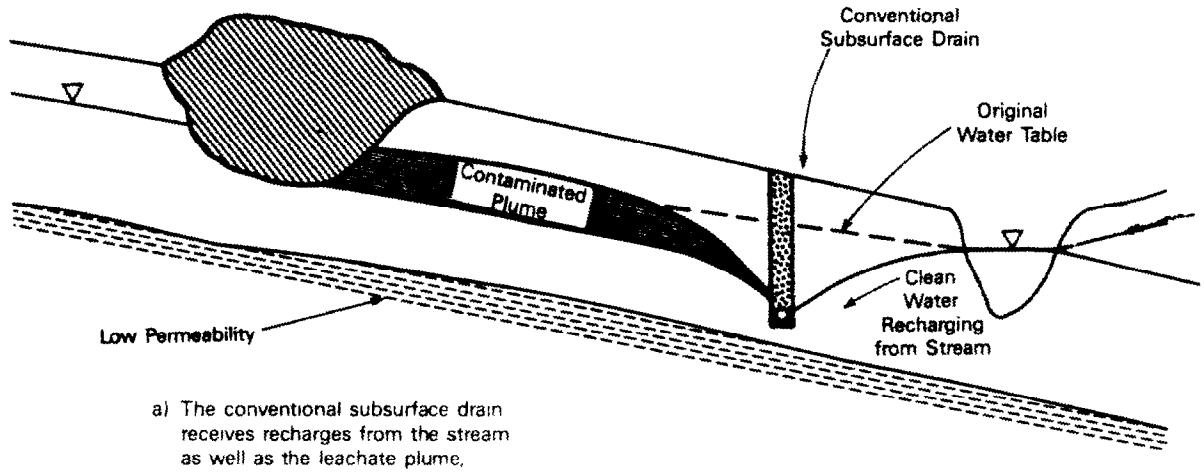
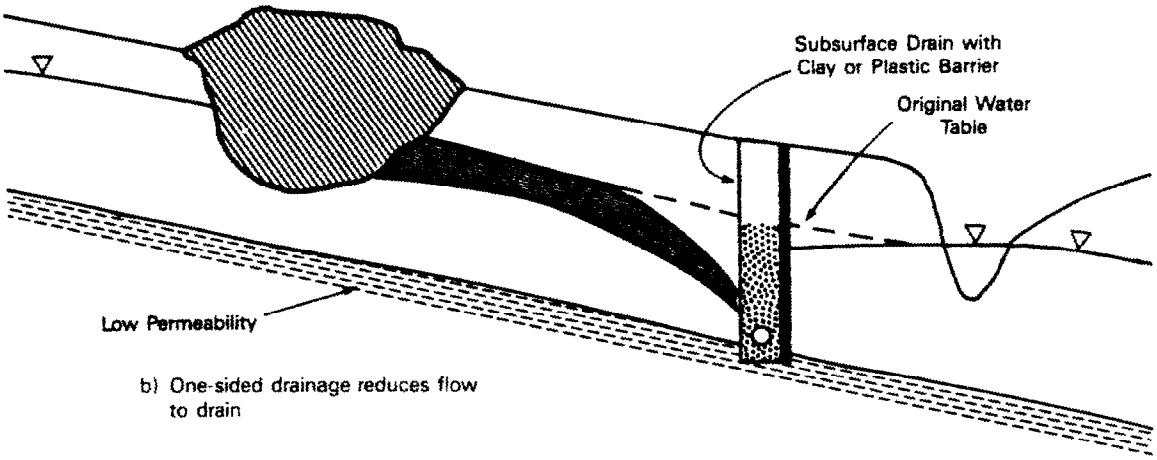


FIGURE 6-3.
THE USE OF A ONE SIDED SUBSURFACE DRAIN FOR REDUCING FLOW FROM UNCONTAMINATED SOURCES



a) The conventional subsurface drain receives recharges from the stream as well as the leachate plume, resulting in larger collection and treatment requirements



b) One-sided drainage reduces flow to drain

FIGURE 6-4.
THE USE OF SUBSURFACE DRAINAGE IN A COMPLETELY ENCAPSULATED SITE

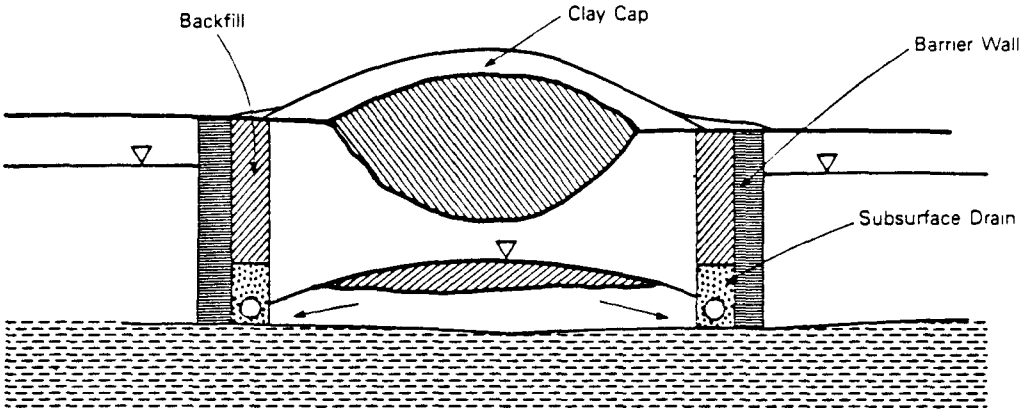
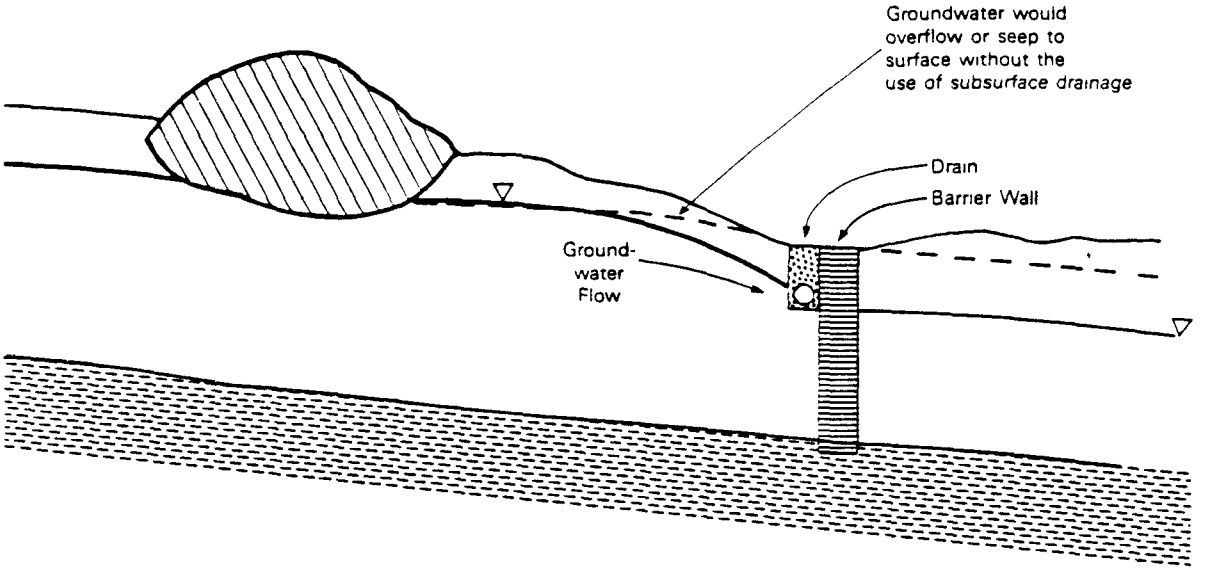


FIGURE 6-5.
THE USE OF SUBSURFACE DRAINAGE TO PREVENT
OVERFLOW AND PONDING



There are a number of limitations to the use of subsurface drains as a remedial technique. For example, an aquifer with a high hydraulic conductivity and high flow rate may preclude the use of a subsurface drain because of the requirement for a large system capacity. However, use of a barrier containment system together with subsurface drains, to reduce flow, may be effective under conditions of high flow. Also, contamination at great depths may cause construction costs to be prohibitive, particularly if a substantial amount of hard rock would have to be excavated to install the drain.

Other limitations to the use of drains may include the presence of viscous or reactive chemicals in the contaminant plume which could clog the drain. Also, high concentrations of iron and manganese in the plume or in the groundwater could cause the eventual clogging of the drain system because of the buildup of insoluble compounds. Drains are fairly difficult and costly to rehabilitate.

6.2 Theory

The principle of flow toward a subsurface drainage trench or pipe is similar to that discussed in Chapter 5 for flow toward a well. However, instead of the cone of depression which is observed around a pumping well, a water table trough develops which runs the length of the drainage trench. This difference is illustrated in Figure 6-6.

The similarities and differences in flow between a well and a drain can be better appreciated from the theoretical expressions for flow. Flow in an unconfined aquifer, for example, can be most simply described by the following formulas:

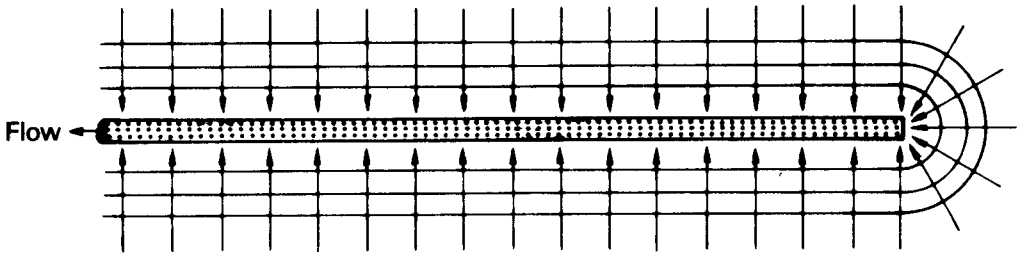
(1) Flow toward a drain (two sided flow)--

$$Q = xK\ (H^2 - h_d^{\ 2})/L$$

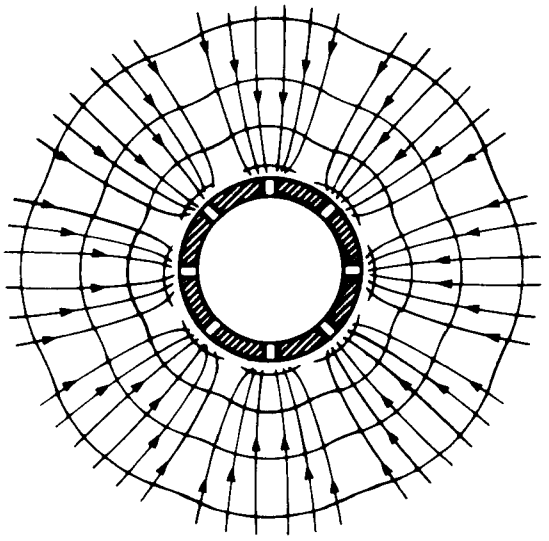
(2) Flow toward a well--

$$Q = \pi\ K(H^2 - h_w^{\ 2})/(\ln R_o/r_w)$$

FIGURE 6-6.
DIFFERENCES BETWEEN GROUNDWATER FLOW TOWARD DRAINS VERSUS WELLS



Flow to a Perforated Drain Pipe



Flow to a Slotted Well Screen

where:

Q = discharge rate (ft^3/sec)
K = hydraulic conductivity (ft/sec)
H = total head at aquifer prior to pumping (ft)
h_d = desired drawdown (ft)
h_w = height of water in well after pumping (ft)
R_o = radius of influence at well (ft)
r_w = radius at well, including gravel pack (ft)
L = influence of drain (ft)
x = unit length of pipe (ft)

Figure 6-7 illustrates how h_d in the equation $Q = xK(H^2-h_d^2)/L$ and h_w in the equation $Q = \pi K(H^2-h_w^2)/(\ln R_o/r_w)$ are similar and how in both cases $H-h_d$ or $H-h_w$ is equal to the drawdown.

Whereas flow from a given point to a well is inversely proportional to the radius of influence divided by well radius (R_o/r_w), flow to a drain is inversely proportional to the influence of the drain (L) only. That is, the diameter of the drain does not significantly affect flow as does the radius of the well. This is because the diameter of the drain is only a matter of inches whereas the length of the flowlines are many feet. Therefore total frictional resistance to groundwater flow through the entire system caused by the pipe is usually ignored.

Flow to drains in a confined aquifer is also similar to flow to wells, and can be described by the following formulas:

(1) Flow toward a drain--

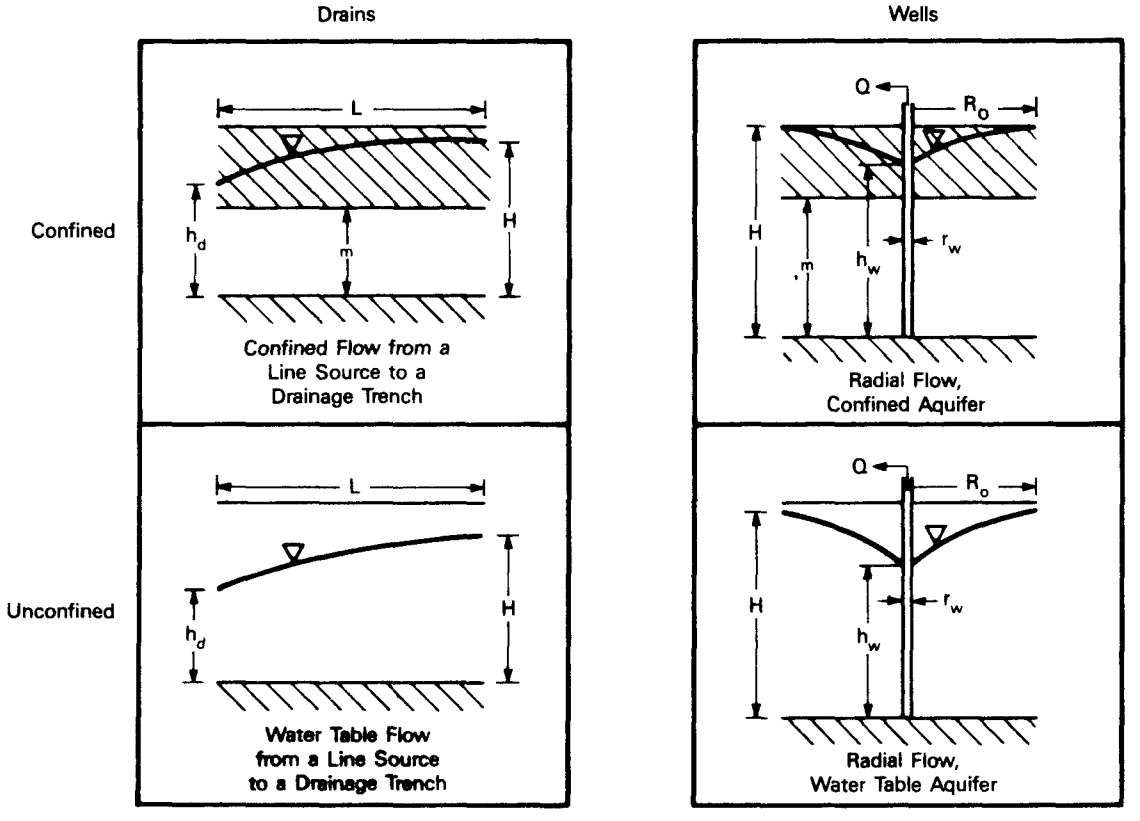
$$Q = 2Km\ (H-h_d)/L$$

(2) Flow toward a well--

$$Q = 2\pi Km(H-h_w)/(\ln R_o/r_w)$$

where m is the thickness of the confined aquifer. Figure 6-7 illustrates the similarities between flow to drains and wells under confined and unconfined conditions.

FIGURE 6-7.
RELATIONSHIP BETWEEN DRAWDOWN (H-h) IN A DRAIN AND A WELL
(AFTER POWERS, 1981)



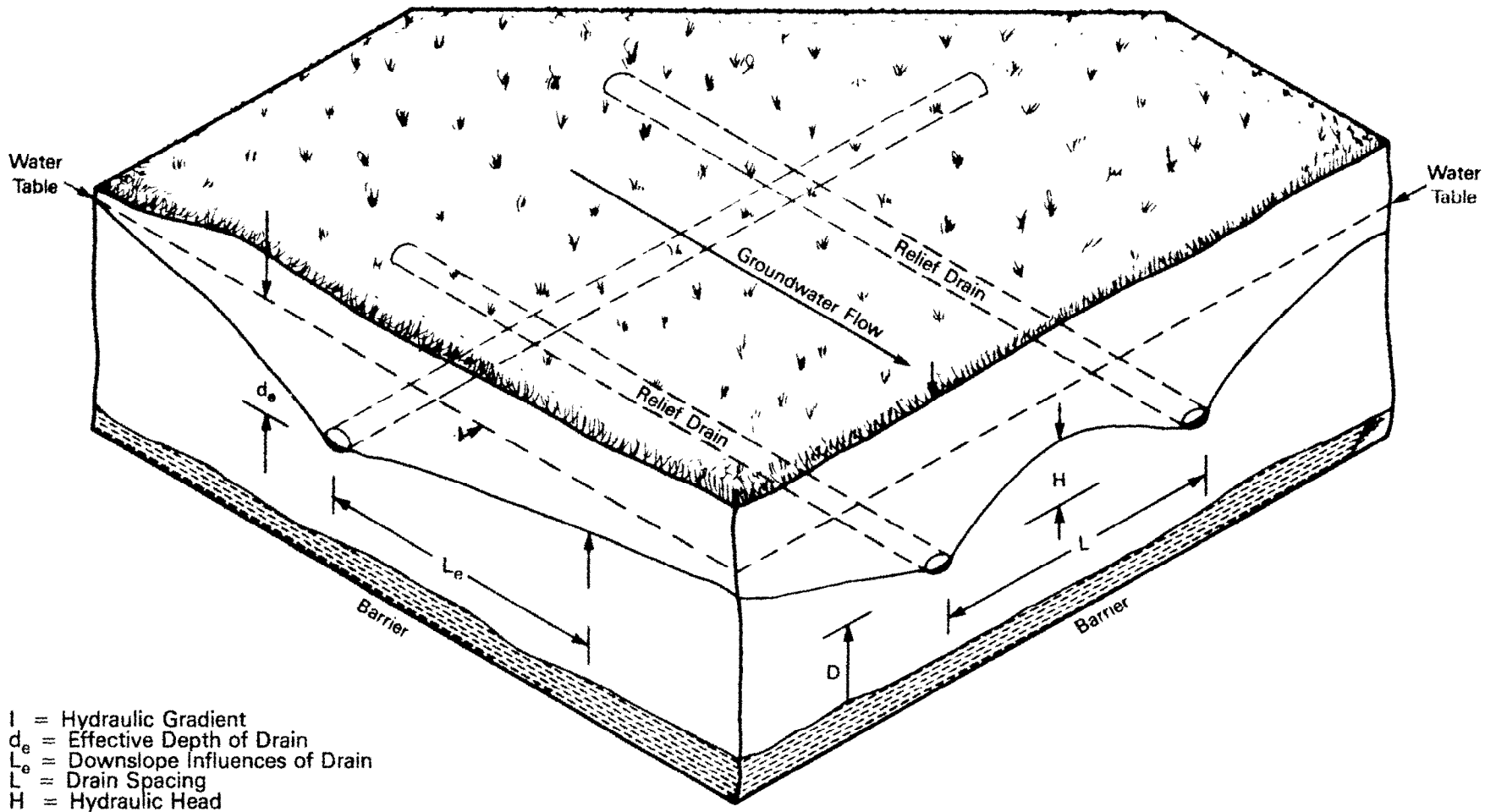
6.2.1 Drainage System Terminology

Functionally, there are two types of subsurface drains which can be used for collecting leachate--interceptor drains and relief drains. Interceptor drains are installed perpendicular to groundwater flow and are used to collect groundwater from an up-gradient source. Interceptor drains can be used to prevent contaminated leachate or groundwater from reaching wells or surface waters hydraulically downgradient from the site. Relief drains are installed in areas where the water table is relatively flat. Relief drains can be used to lower the water table beneath a waste site and are usually installed around the perimeter of the site or parallel on either side of the site such that their areas of influence overlap. Figure 6-8 illustrates the function of relief drains and interceptor drains in altering the configuration of the groundwater table. Interceptor drains can also be used where infiltration from a landfill or a surface impoundment results in a groundwater mound which induces lateral flow at the edges of the mound. Figure 6-9 illustrates this situation. In general, interceptor drains are more widely used than relief drains.

The major difference in the way these two types of drains function is that the drawdown created by interceptor drains is proportional to the hydraulic gradient, whereas the drawdown created by relief drains is a function of the hydraulic conductivity and depth to the impermeable barrier below the drain. Hydraulic gradient is the key consideration in choosing which type of drain system to install. Relief drains are used if the hydraulic gradient is slight (about three percent or less), whereas interceptor drains are used if the gradient is steep. The rationale for the way these drains function is discussed in Section 6.2.2.

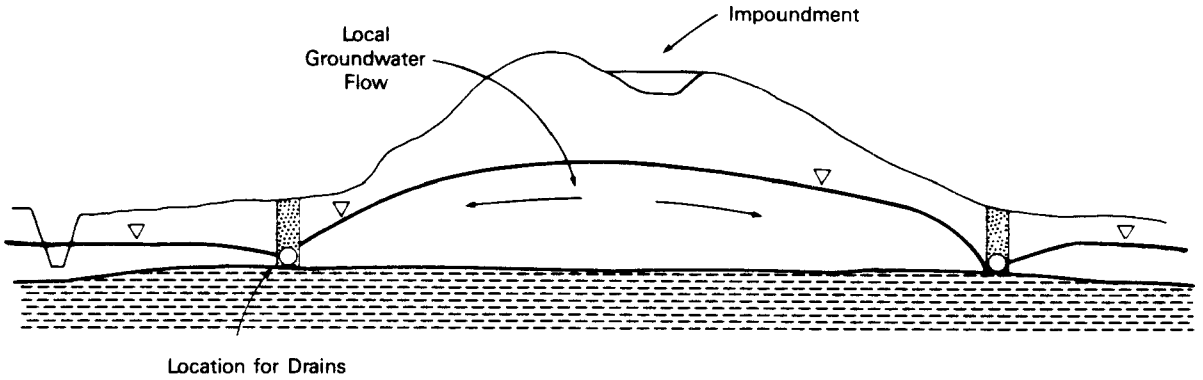
The components of a relief or interceptor drainage system are classified by their size and function as laterals, collectors, or mains. Laterals are the smallest diameter component of drainage systems and collect groundwater directly. Collectors are larger than laterals and receive water from one or more lateral drains. Mains are the largest of the three components and

FIGURE 6-8.
THE EFFECT OF RELIEF AND INTERCEPTOR DRAINS IN ALTERING THE CONFIGURATION OF THE WATER TABLE (SCS, 1973)



I = Hydraulic Gradient
d_e = Effective Depth of Drain
L_e = Downslope Influences of Drain
L = Drain Spacing
H = Hydraulic Head
D = Depth Below Drains to the Impermeable Barrier

FIGURE 6-9.
THE USE OF INTERCEPTOR DRAINS TO COLLECT FLOW INDUCED BY GROUNDWATER MOUNDING

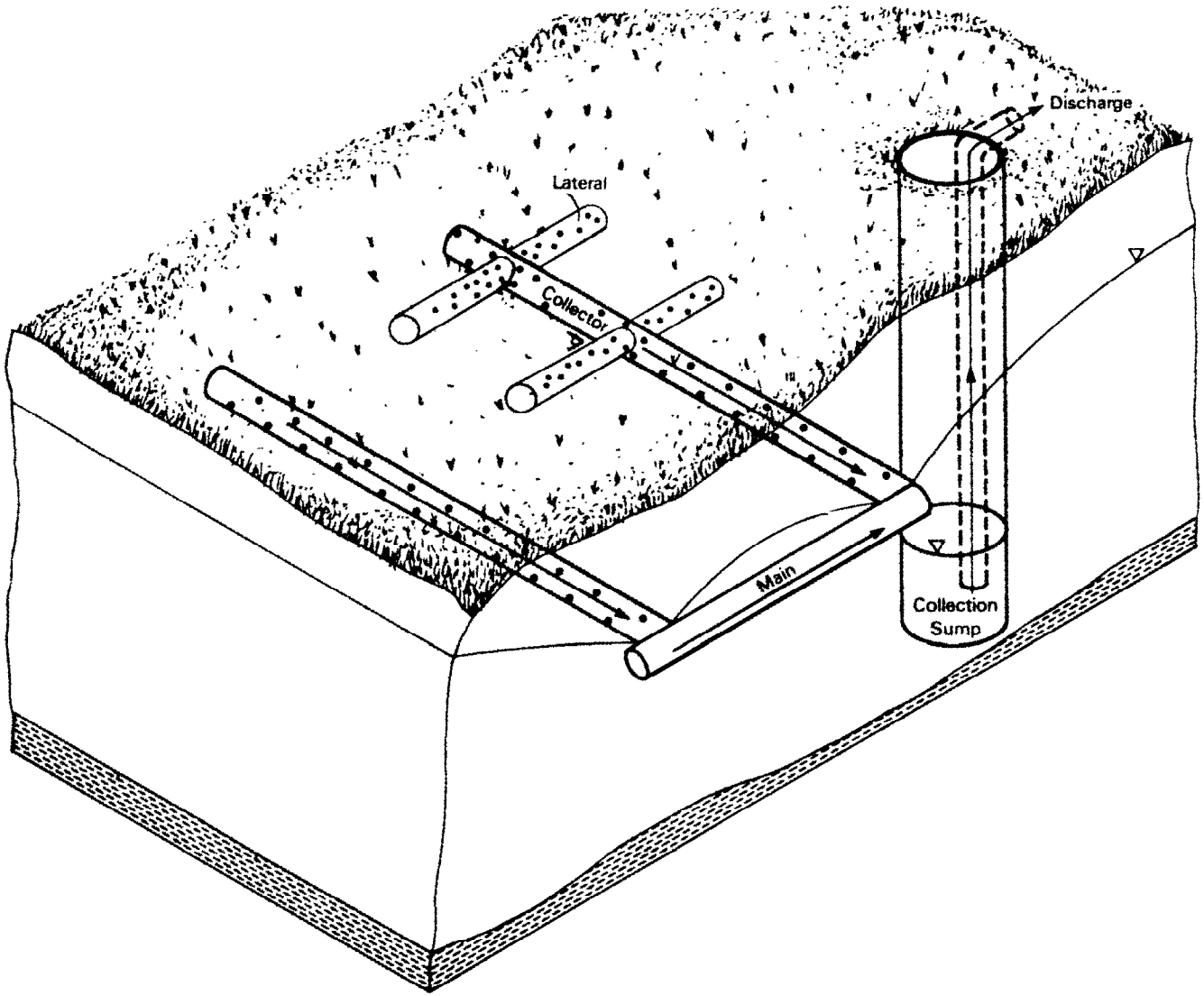


receive water from one or more collector drains. These components are illustrated in Figure 6-10.

Subsurface drains can be designed to use either pipe or a gravel layer to transmit flow. Drains that use clay or cement tiles or plastic pipes are called tile or pipe drains. Drains that use a high permeable bed only are called french or gravel drains. Pipe drains are somewhat more difficult and expensive to install than french drains but are less susceptible to clogging in most instances. While both pipe and french drains have been used for waste site remediation, pipe drains are much more common.

In summary, drainage systems are classified by function (i.e., as relief or interceptor systems), by their components (i.e., as pipe or gravel drainage systems), and by their configuration (i.e., as a single component or singular system, or a multi-component or composite systems). In a singular system, each lateral drains into its own sump or collection tank. In a composite

FIGURE 6-10.
COMPONENTS OF A DRAINAGE SYSTEM



system, each lateral drains into a collector, which may in turn drain into either a main or a collection tank. Selection of a singular or a composite system design depends upon the size of the leachate plume, the hydraulic conductivity of the subsurface strata, the hydraulic gradient, and other factors.

When a drainage system is being designed, the following four elements must be determined:

- Drain depth (i.e., depth of water table lowering)
- Parallel drain spacing
- Drain pipe diameter and gradient
- Envelope and filler materials and design.

Drainage system design generally requires using a combination of theoretical design considerations, field test results, and practical experience.

Theoretical equations not involving the use of computer programs or electric analogues are based on various idealized conditions. For example, some equations are based on the assumption of steady state conditions (i.e., where the hydraulic head does not vary with time). Others assume unsteady state conditions (i.e., a fluctuating water table). Steady state conditions require that recharge rates equal the drain discharge rates so that the water table remains relatively constant. Non-steady state conditions take into account some fluctuation of the water table with time under the influence of non-steady recharge. Since solutions to drainage problems using the steady state assumption are simpler to derive, most systems are designed based on these conditions. Steady state assumptions are generally valid when considering average discharges over extended periods of time. However, in areas subject to periodic, high-intensity rainfalls followed by long dry periods, the use of steady state equations can result in over design (i.e., larger diameter pipes or narrower drain spacing than is actually necessary). Another idealized condition often assumed in drainage formulas is that the earth materials are homogeneous and isotropic. Such conditions rarely exist, but the use of average hydraulic conductivities generally results in

reasonable estimates for drain design. Experience has shown however that the efficiency of subsurface drains is generally less than predicted by the theoretical equations. As a result, systems are often overdesigned on purpose by placing drains closer together and by using larger drain pipe than estimated from the equations.

6.2.2 Depth and Spacing

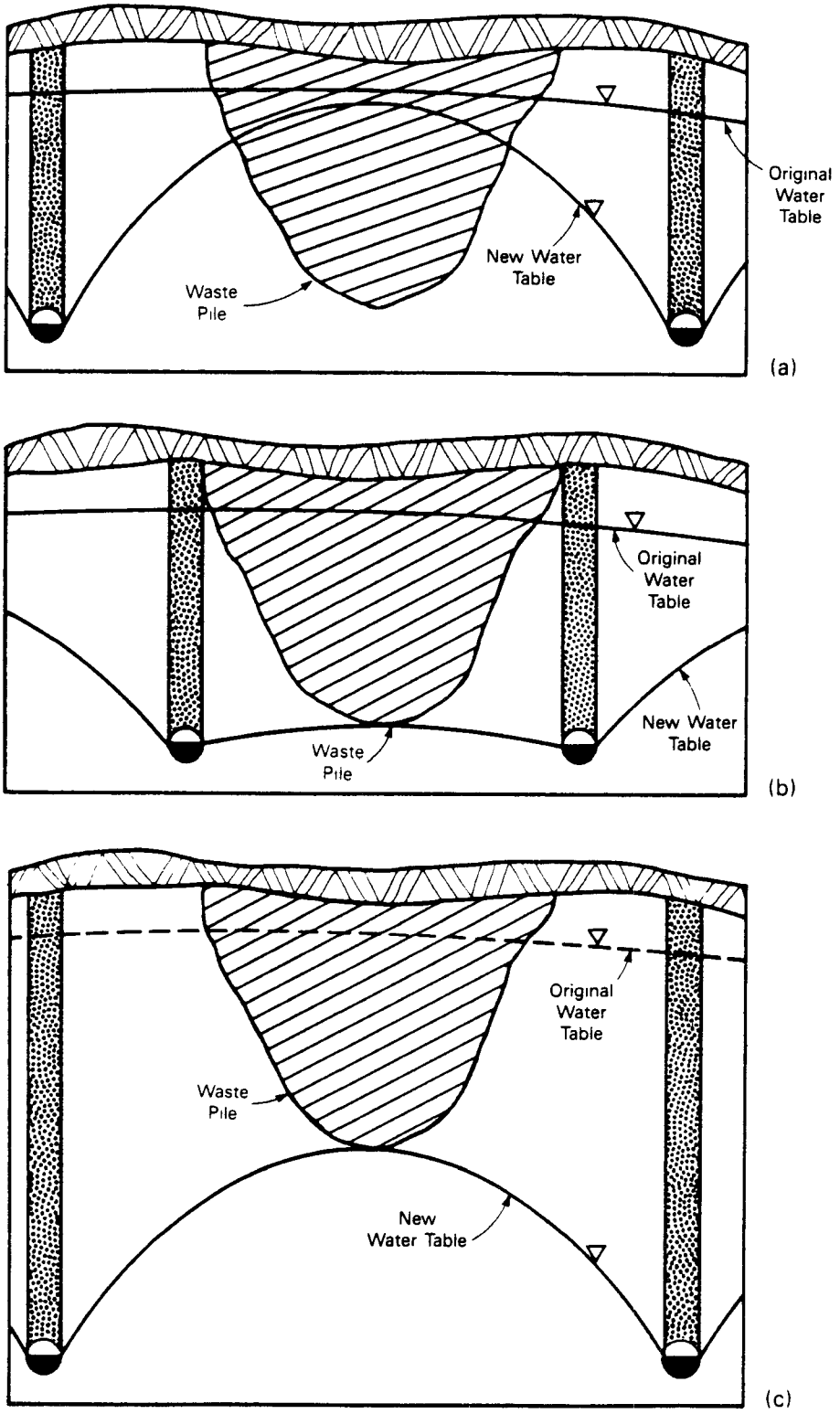
This section discusses both theoretical approaches and field methods for determining the depth and spacing of relief and interceptor drains. In many instances where interceptor drains are used, a single drain is adequate and spacing is not a consideration.

6.2.2.1 Relief Drains

When two parallel drain lines are installed, each one exerts an influence (L) on the water table (see Figure 6-7). If the drains are designed properly such that their influences intersect each other, the drawdown each exerts is additive, resulting in an increased total drawdown of the water table. This effect is greatest midway between the two drains. For a site where the drain will not rest on an impervious clay layer, the zone of influence and the depth are interdependent design variables, which depend upon hydraulic conductivity, depth to an impervious layer, and the discharge rate. If the drains could be placed deeper, the effective drawdown would increase, assuming homogeneous earth materials, or the drains could be placed further apart while maintaining a constant effective drawdown. If the drains can not be placed deeper, for example because of shallow bedrock, the effective drawdown could be increased by spacing the drains closer together. The relationship between depth and spacing is critical to the design of effective parallel drainage systems.

In designing a subsurface drain system for a hazardous waste site, it must be determined whether two parallel lateral drains on either side of the site will be able to effectively lower the groundwater beneath a site. The importance of this is illustrated in Figures 6-11a through 6-11c. If the drains were designed with the depth and spacing shown in Figure 6-11a, the

FIGURE 6-11.
THE RELATIONSHIP OF DRAIN DEPTH AND SPACING TO WATER TABLE DRAWDOWN



zone of influence of the drains would not be adequate to dewater the waste pile. By placing the drains closer together as illustrated in Figure 6-11b, their combined drawdown would be below the level of the waste pile so that it would effectively dewater the pile. However, a minimum spacing is often imposed by the boundaries of the waste because excavation through the waste material can be extremely hazardous.

If a drainage system has been designed so that the drains are as closely spaced as possible and the plume is not totally intercepted, there are two possible alternatives to improve the design. First, the drains may be placed deeper (Figure 6-11c), provided the depth to bedrock and hydraulic conductivity of soil layers is adequate. Alternatively, the system can be redesigned as a composite drainage system. The choice between these two alternatives depends on the depth to an impermeable barrier, the boundaries of the plume and waste site, and the cost.

As mentioned previously, drain spacing is also influenced by the hydraulic conductivity of the earth material in which it is installed and the depth to the impervious barrier. In theory, the deeper the impervious barrier below the drain the greater the thickness of the water transmitting layer, and consequently, the wider the drain spacing can be to achieve the same drawdown. This phenomenon can be explained by the fact that there is radial flow (i.e., flow from all sides) to the drain from underlying soils, and the extent of this flow is dependent upon the distance beneath the drain to the impervious layer. The effect of the depth to the impermeable barrier on drain spacing is illustrated in Figure 6-12. Determining this depth as part of field testing is essential.

The effect of hydraulic conductivity on drain spacing and the importance of placing drains in a permeable layer can be appreciated by the example shown in Figure 6-13. If drains are laid in the layer of lower hydraulic conductivity (K_1 = 1.64 feet/day) a drain spacing of only 591 feet could be used. If, on the other hand, the drains are placed in the uppermost part of a sandy aquifer having a hydraulic conductivity of 32.8 feet/day, the drains may be

FIGURE 6-12.
THE EFFECT OF DEPTH TO A LOW PERMEABILITY BARRIER ON DRAIN SPACING (DE RIDDER AND VAN AART, 1974)

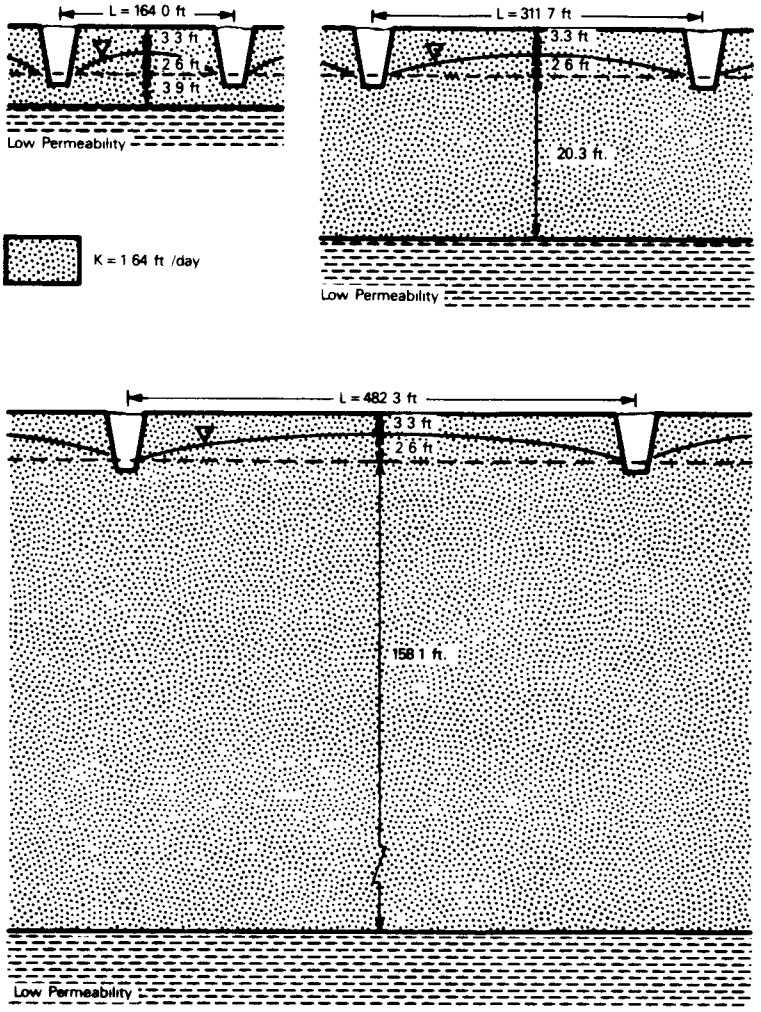
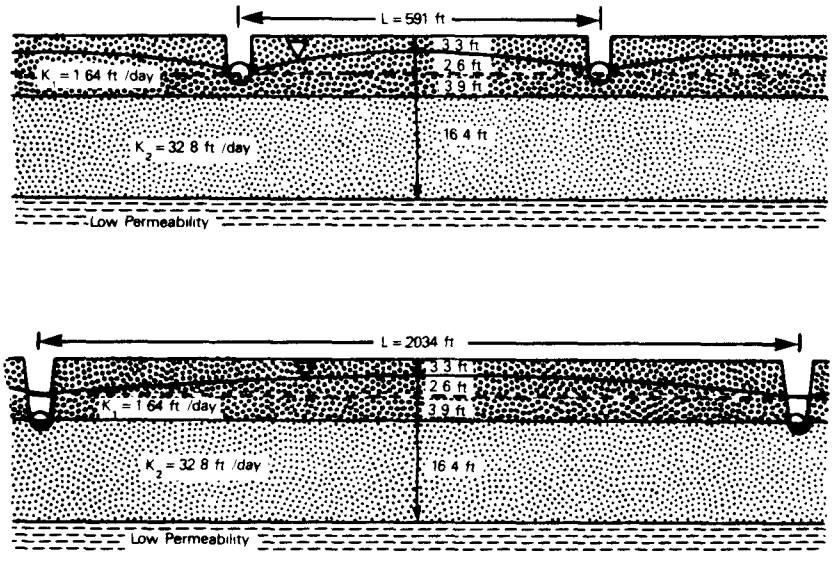


FIGURE 6-13.
THE EFFECT OF HYDRAULIC CONDUCTIVITY DRAIN SPACING
(DE RIDDER AND VAN AART, 1974)



spaced 2034 feet apart (DeRidder and Van Aart, 1974). This example illustrates the importance of investigating both the layer at and below the envisioned drain depth.

Drain spacing can be determined based either on field test data using experimental plots or on theoretical design formulas. Numerous equations and models have been developed for determining drain spacing for various idealized conditions. Drain spacing equations for relief drains generally relate the following factors:

- Precipitation and other sources of recharge
- Evaporation
- Hydraulic conductivity
- Depth to the impermeable barrier
- Cross-sectional area of the drain
- Water level in the drain.

In areas with a large natural flow velocity, spacing the drains so that their zones of influence just overlap may not be sufficient. High flow velocity is not accounted for by these equations and it may be necessary to place the drains closer together than predicted by theoretical equations to intercept the entire plume.

6.2.2.1.1 Flow to Drains Reaching an "Impermeable" Barrier

Drains are often used where the depth to a low permeability barrier is relatively shallow and the drains can be laid just above the barrier. In developing and using drain spacing formulas for this case, an underlying soil layer is considered to be "impermeable" if the hydraulic conductivity is less than one tenth that of the above soil layer (Wesseling, 1973).

An equation for flow to parallel ditches resting on an impermeable barrier was developed by Donnan (1946) and described by Wesseling (1973). The flow conditions illustrated in Figure 6-14 can be described by the equation:

$$q = (8KDH + 4KH^2)/L^2$$

where:
- q = drain discharge rate per unit surface area (ft/sec)
- K = hydraulic conductivity (ft/sec)
- H = height of the water table above the water level in the drain midway between two drains (ft)
- D = distance between the water level in the drain and the "impermeable" layer (ft)
- L = drain spacing (ft).

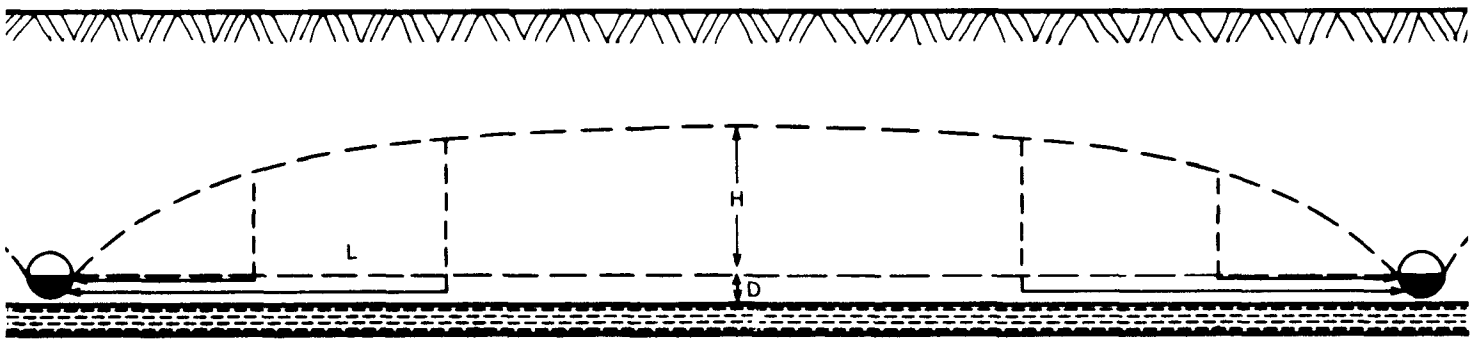
When D is very small compared to H, as it is in the case of pipe drains resting on an impermeable barrier, the formula can be simplifed to:

$$q = 4KH^2/L^2$$

6.2.2.1.2 Flow to Drains Not Reaching an Impermeable Barrier

In many instances, relief drains will not be installed at the top of the impermeable barrier. This may be because the plume can be completely intercepted by shallow drains or because of the high cost of installing drains to the full depth.

FIGURE 6-14.
FLOW TO A DRAIN RESTING ON A LOW PERMEABILITY BARRIER

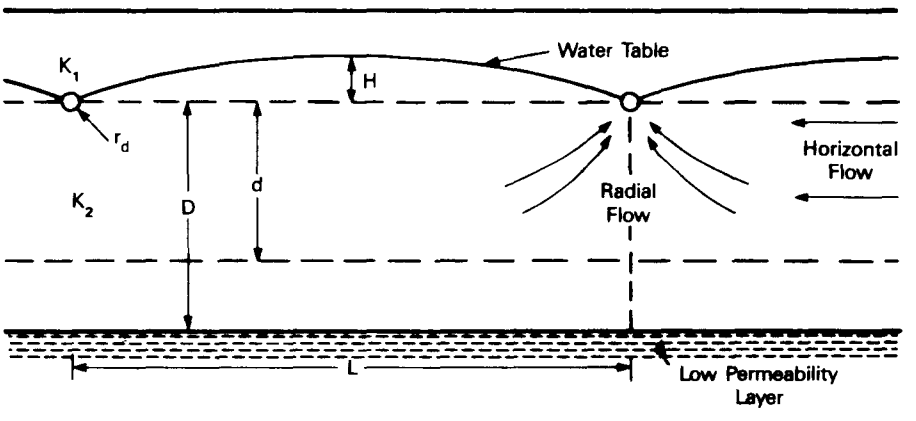


The Donnan (1946) equation described in the previous section does not adequately describe the flow to a drain which does not reach an impermeable barrier. If the drain does not reach an impermeable barrier (Figure 6-15), the flowlines will not be parallel and horizontal (as shown in Figure 6-14), but will converge towards the drain (i.e., radial flow). This convergence causes a more than proportional head loss in the groundwater system which must be accounted for in the drain spacing formula. Hooghoudt (1940), as described by Wesseling (1973), developed a modified drain spacing formula for a two layered soil given by:

$$q = (8K_2dH + 4K_1H^2)/L^2$$

where: $K_1;K_2$ = hydraulic conductivity above (K_1) and below (K_2) the drain (ft/day)
d = equivalent depth of the aquifer below the drain (ft).

FIGURE 6-15.
FLOW TO DRAIN NOT RESTING ON A LOW PERMEABILITY BARRIER (VAN SCHILFGAARD, 1974)



In the Hooghoudt (1940) equation both drain spacing, L, and equivalent depth, d, are unknowns. The value of d is typically calculated from a specified value for L, so that the Hooghoudt (1940) equation cannot 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 of the equation are equal or the use of nomographs which have been developed specifically for equivalent depth and drain spacing. Table 6-1 gives values of the equivalent depth (d) as a function of drain spacing (L) and saturated thickness below the drains (D). This table shows values of d for a drain pipe with a radius (r_d) of four inches. Similar tables have been prepared for other values of r_d (Wesseling, 1973). For saturated thicknesses (D) greater than 32.8 feet, the equivalent depth can be calculated from drain spacing using the following equation (Repa et al., 1983):

$$d = 0.057\ (L) + 0.845$$

This equation was developed by linear regression from the values given in Table 6-1.

6.2.2.1.3 Flow to Drains in a Two-layered Soil

Although Hooghoudt's equation is widely used to approximate drain spacing, it is only accurate in two-layered soils when the level of the drain corresponds with the interface between the two soil layers. Ernst (1962), as described by Wesseling (1973), developed an equation which offers a considerable improvement over the Hooghoudt equation for two layered soils insofar as the interface between the two soil layers can be above or below the drains. This formula is especially useful when the upper layer has a considerably lower hydraulic conductivity than the lower layer. The flow to drains as described by Ernst is illustrated in Figure 6-16 and written as follows:

$$H = (qD_v/K_v) + (qL^2/8\Sigma(K_i m_i)) + (qL/\pi K_r)\ \ln\ (\alpha D_r/u)$$

where:

H	= total hydraulic head or water table height above drain level midway between drains (ft)
q	= drain discharge rate per unit surface area (ft/day)
L	= drain spacing (ft)
K_r	= hydraulic conductivity of the layer with radial flow (ft/day)

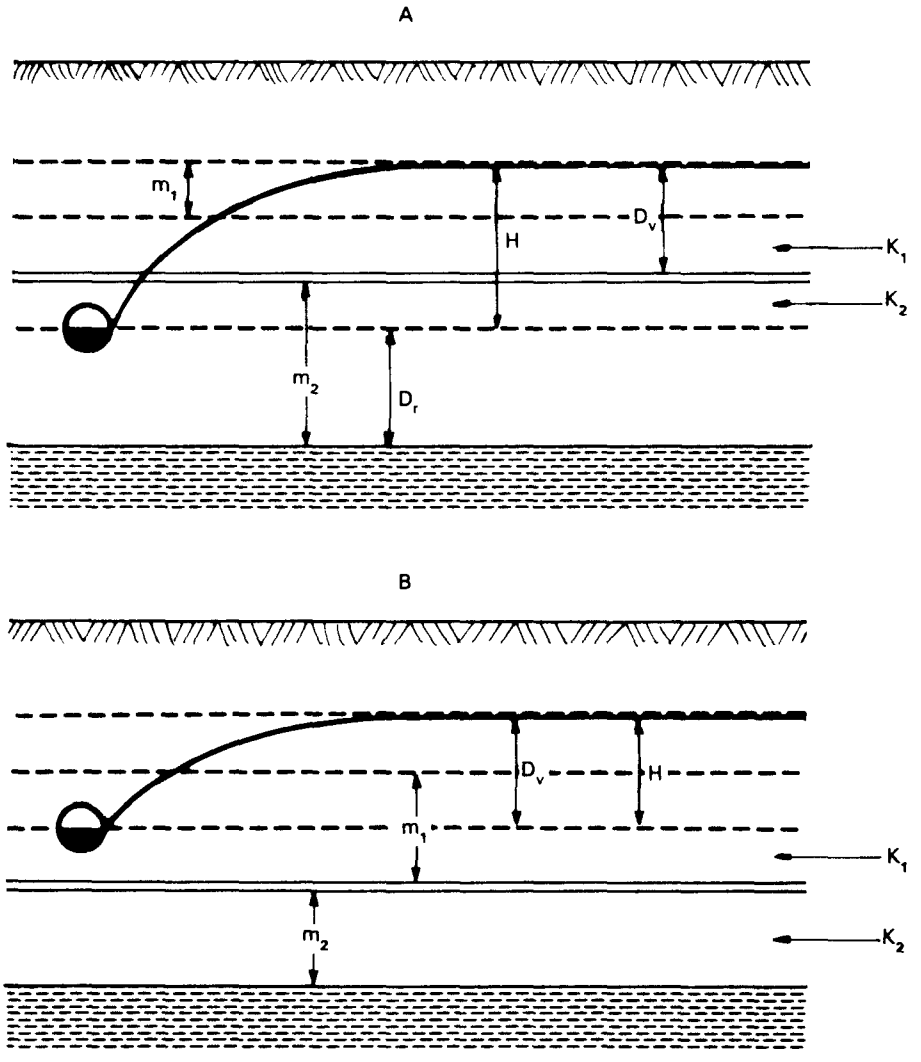
TABLE 6-1

VALUES FOR EQUIVALENT DEPTH d(m) FOR r_d = 4 inches
CALCULATED FOR DIFFERENT VALUES OF DRAIN SPACING (L)
AND SATURATED THICKNESS BELOW DRAINS (D)
(Wesseling, 1973)

L(m)	5	7.5	10	15	20	25	30	35	40	45	50
D(m)											
0.5	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

L(m)	50	75	80	85	90	100	150	200	250
D(m)									
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

FIGURE 6-16.
SYMBOLS FOR THE ERNST EQUATION FOR FLOW IN A TWO-LAYERED SOIL WITH (A) THE DRAIN IN THE LOWER LAYER AND (B) THE DRAIN IN THE UPPER LAYER



K_v = hydraulic conductivity of the layer with vertical flow (ft/day)

D_v = thickness of the layer over which vertical flow is considered (ft)

D_r = thickness of the layer in which radial flow occurs or thickness below the drain level of the layer in which the drain is located (ft)

$\Sigma(K_i m_i)$ = Transmissivity of the soil layers through which horizontal flow is considered (ft^2/day)

u = wet perimeter of the drain (ft)

α = geometry factor for radial flow which depends upon flow conditions (dimensionless).

Values of u, $\Sigma(K_i m_i)$, and α require further discussion. For pipe drains in an area of high hydraulic conductivity, u is determined from:

$$u = (b + 2)(2r_d)$$

where the new terms are:

b = width of the trench (ft)
r_d = radius of the drain (ft)

If envelope material is used, it is advisable to replace $2r_d$ by the height of the envelope. $\Sigma(K_i m_i)$ is equal to the transmissivity of the entire aquifer for a two-layered soil or $\Sigma(K_i m_i) = K_1 m_1 + K_2 m_2$. Values for the geometry factor, α, vary depending upon the relative hydraulic conductivity of the two soil layers and whether the drains are installed in the upper or lower layers. The value, α, for various conditions is discussed further below.

The Ernst equation can be used to determine drains spacing in homogeneous or two-layered soils. However, under some conditions, drain spacing estimates using the Ernst equation will not necessarily be any more valid then those obtained using Hooghoudt's formula. These situations include:

- Where all soil layers are homogeneous.
- Where drains are installed in the upper layer of a two-layer aquifer such that 0.1 K_1 is greater than K_2. In this case the lower layer is considered to be an impervious barrier and the soil is considered homogeneous.

- Where the depth from the drain bottom to the impermeable barrier is larger or suspected to be larger than 0.25L. When this occurs, the hydraulic conductivity and thickness of the second layer does not have an effect on flow and the two layers may be considered homogeneous.

The Hooghoudt equation should be used under these circumstances to simplify the calculations.

Conditions where the Ernst equation is most useful include:

- Where the drains are to be situated in the lower layer of a two-layer soil and $K_1 < K_2$ (the vertical resistance in the second layer can be neglected against that in the first), so that (1) The geometry factor, α, can be neglected, (2) $\Sigma(K_i m_i)$ is approximately equal to $K_2 m_2$, (3) $K_r = K_2$ and $K_v = K_1$ (K_r = hydraulic conductivity of radial flow and K_r = hydraulic conductivity of vertical flow), (4) $D_v = 2m_1$ (average thickness below the water table of layer with permeability K_1) and (5) $D_r = D_o$. In this case, the formula can be expressed as follows:

  $$H = (q2m_i/K_i) + (qL^2/8K_2m_2) + (qL/\pi K_2) \ln (D_o/u)$$

  where D_o is the distance from the water level in the drain to the next soil layer below having different hydraulic properties.

- Where the drain is entirely in the upper layer of a two-layer soil and $K_2 > 20\ K_1$, such that $\alpha = 4$, K_r and $K_v = K_1$, and $D_v = H$. In this case, the formula becomes:

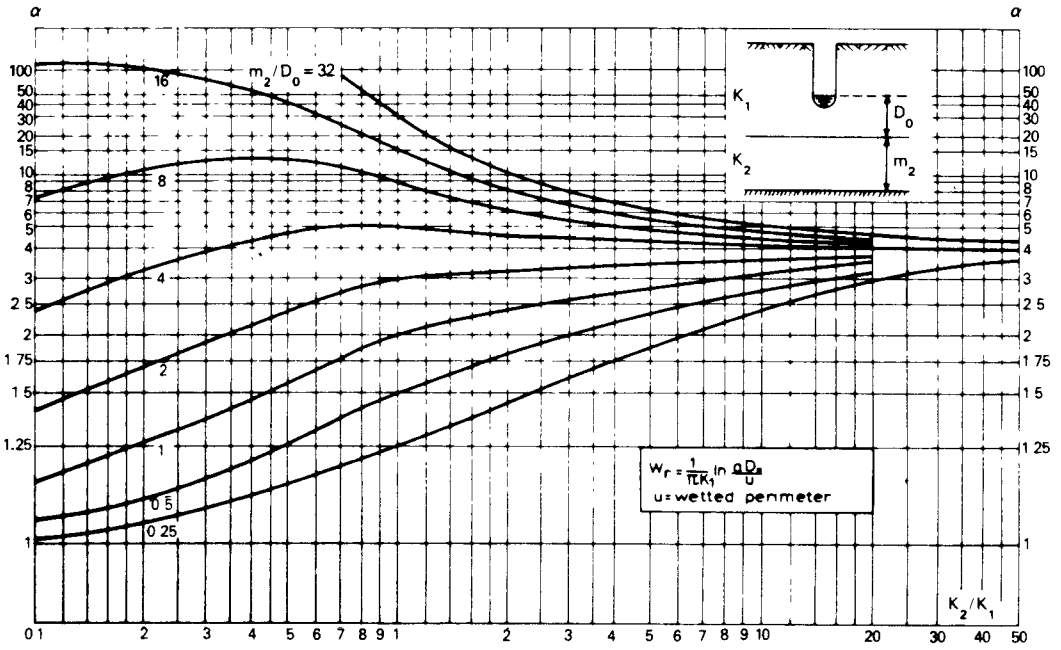
  $$H = (qH/\pi K_i) + (qL/8\ \Sigma(K_i m_i)) + (qL/\pi\ K_1) \ln (4D_o/u)$$

- Where the drain is entirely in the upper layer of a two-layer soil and $0.1\ K_1 < K_2 < 20\ K_1$. In this case, the geometry factor, α, has to be determined from the nomograph in Figure 6-17 and introduced into the Ernst equation (Wesseling, 1973).

6.2.2.1.4 Drain Spacing Estimates Based on Field Experiments

Experimental or trial drains can be used to determine or verify drain spacing calculations. They are also used at times to determine drain spacing where theoretical estimates are difficult to obtain because of variable soil conditions. For example, in areas where soils are heterogeneous, reliable data on hydraulic conductivity may be difficult to obtain from conventional sampling methods. In such cases, determination or verification of these

FIGURE 6-17.
NOMOGRAPH FOR DETERMINING THE GEOMETRY FACTOR α, IN THE ERNST EQUATION (WESSELING, 1973)



factors may be more economical or efficient using experimental drain fields. A necessary prerequisite to using experimental drain fields is that a site can be located which has similar hydrologic, pedologic, geologic, and topographic conditions to the waste site.

The dimensions of the test site are governed by:

- The drain spacing to be tested--Spacings which are narrower and wider than those calculated or estimated should be included and the intervals should be chosen in distinct steps. Thus, if the spacing is estimated by calculations to be 164 feet, spacings of 82 feet and 328 feet should be included as well.

- A length to width ratio of at least 5 and preferably 7 to 10--Thus, when the test spacing is 164 feet, the test length should be at least 820 feet. If the plots are too wide in relationship to the length, then boundary effects (i.e., inflow from an undrained part of the area) may be significant.

The depth to which experimental drains are installed can be approximated from theoretical estimates and information on the depth of the plume and depth to permeable and impermeable layers.

Water observation wells should be installed at various points in the experimental field to aid in the evaluation as follows (Dieleman, 1974):

- Midway between drains to measure the hydraulic head or the water table height above the drain
- Near one or more of the drains in each field plot to measure the shape of the water table. Shape should be measured at varying points (e.g., 10, 20, and 50 feet from the drain)
- At the upper and lower ends of some plots to observe boundary effects
- On top of drain tubes to determine drain function.

The adequacy of the drain spacing can be determined from the measured hydraulic head and shape of the water table. These data are also used to determine hydraulic conductivity using the Hooghoudt or other appropriate equation. In using the Hooghoudt equation:

$$q = (8K_2dH + 4K_1H^2)/L^2$$

the discharge rate per unit area drained ($q=Q/A$), is measured directly in the field using a discharge recorder attached to the drain outlet.

6.2.2.2 Interceptor Drains

Interceptor drains are designed to cut off the flow of groundwater originating from an up-gradient source and are generally more applicable to plume management than relief drains.

There are basically two major applications for interceptor drains in plume management. In the first situation (Figure 6-18a), the natural gradient of the water table causes significant contaminant movement away from the site. Here, interceptor drains could be installed downgradient of the contaminant source to cut off the contaminants flow to streams or wells.

In the second situation (Figure 6-18b), contaminants flow outward in several directions from a site because of a groundwater mound which has built up beneath the site. This situation would require installation of parallel interceptor drains located on either side of the site or a drainage system which completely encircles the site.

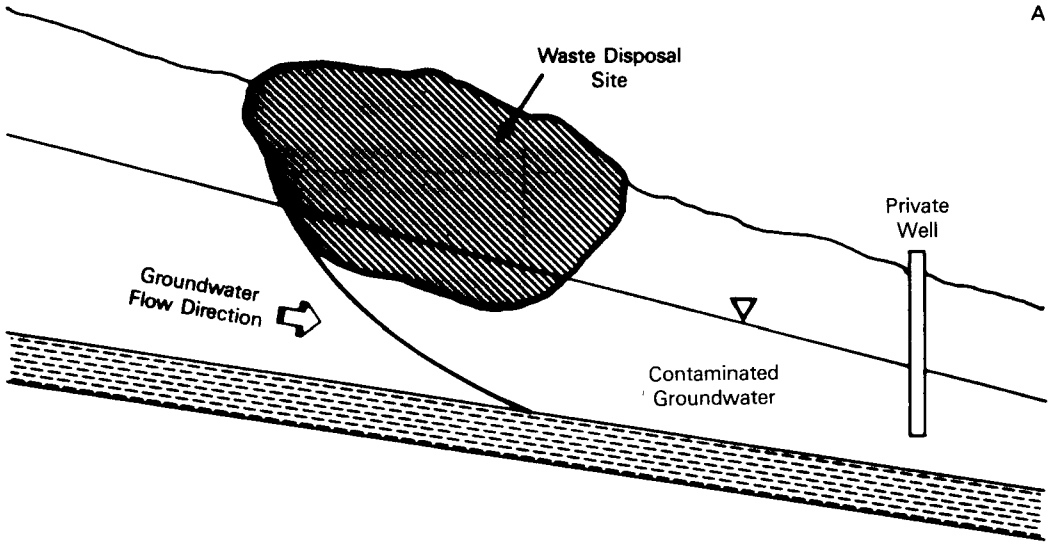
6.2.2.2.1 Interceptor Drains to Cut Off Natural Groundwater Flow (Figure 6-18a)

The design of interceptor drains is more often based on practical experience than on theoretical design. Preliminary field investigations are undertaken to determine the direction of flow, the hydraulic gradient, the hydraulic conductivity, location of subsurface strata, and the boundaries of the plume. These data are typically obtained through an on site boring program. With the results of the field survey and a knowledge of how interceptors capture flow, the depth and position of the drains are estimated. After the line is staked on the site, additional borings should be taken along and across the staked line, and the alignment shifted, if needed, to obtain proper interception.

Interceptor drains are generally installed perpendicular to groundwater flow and parallel to the contours of the land. Figure 6-19 illustrates the positions of an interceptor drain relative to groundwater flow and topography.

If soil borings indicate that stratified soils having greatly different hydraulic conductivities exist, the drain should be installed with a sand and gravel envelope resting on the layer of lower hydraulic conductivity, if possible. If the trench line is cut through an impervious stratum, there is danger that a significant percentage of the water moving laterally will bridge

FIGURE 6-18.
SITE CONDITIONS REQUIRING INTERCEPTOR DRAINS FOR PLUME MANAGEMENT



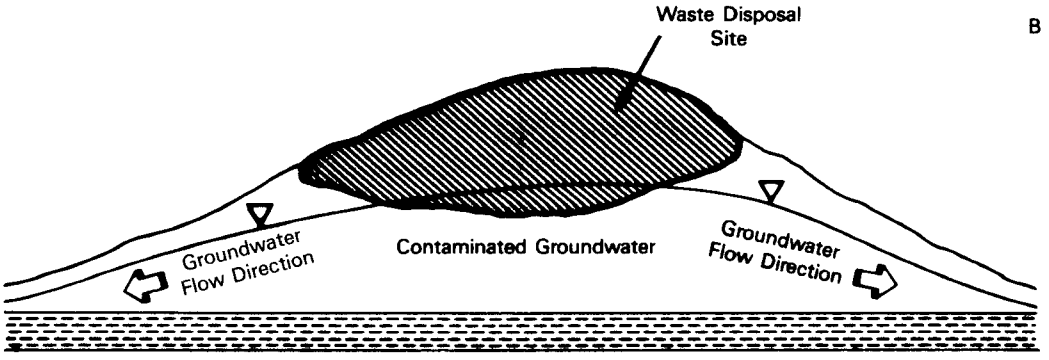
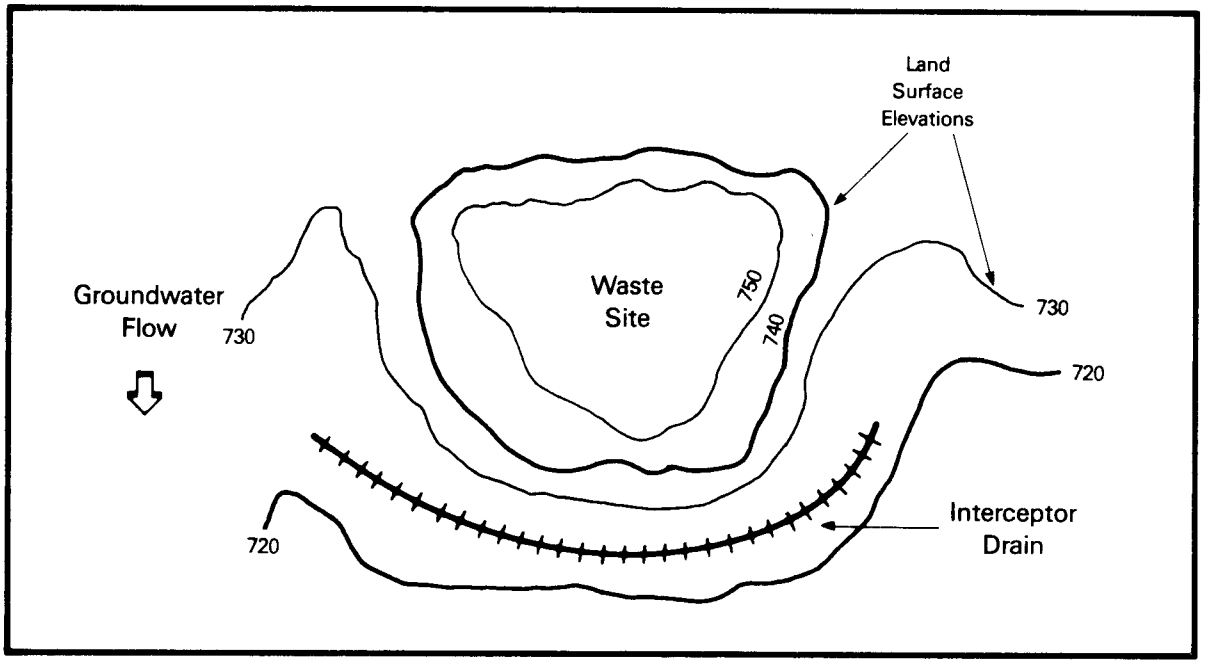


FIGURE 6-19.
LOCATION OF A SUBSURFACE DRAIN WITH RESPECT TO TOPOGRAPHY AND THE DIRECTION OF GROUNDWATER FLOW



over the drain and continue downgradient. This is especially important where a permeable layer sandwiched between two relatively impermeable layers outcrops along the side of the slope such that surface seepage is a problem. When this situation occurs, the flow can be intercepted by a drain located just upgradient of the seep.

For the purpose of installing an interceptor drain, a layer can be considered "impermeable" if its hydraulic conductivity is less than one tenth that of an adjacent, more permeable layer.

In order to decide where to position a drain (or drains) to intercept the entire leachate plume, the relationship between depth and flow, and the up-gradient and down-gradient influence of the interceptor drain must be known.

If other boundary conditions are fixed, the shape of the drawdown curve upgradient of the site is independent of hydraulic conductivity, but is a function of head. The upgradient influence on drawdown extends for a distance which is greater the more gradual the water table gradient. In general, the upgradient influence is small compared to the downgradient influence.

The depth to which the water table is lowered downgradient of the interceptor is proportional to the depth of the drain. Theoretically, a true interceptor drain lowers the water table downgradient to a depth equal to the depth of the drain. The distance downgradient to which it is effective in lowering the water table is infinite provided recharge is not occurring. This however is never the case since infiltration from precipitation always recharges the groundwater.

If a leachate plume extends from the water table to the depth of the impermeable layer, and an interceptor drain is placed at the midpoint between the water table and the impervious layer, a little less than 50 percent of the flow will be intercepted. If the drain is placed at the top of the impermeable layer nearly all of the flow will be intercepted (Van Schilfgaarde, 1974; SCS, 1973).

In summary, the major considerations in the design of an interceptor drain are:

- To place the drains deep enough to intercept the entire plume, preferably to the top of an impermeable barrier
- To ensure that the upgradient influence of the drain is adequate to intercept the plume
- To ensure that the downgradient influence is adequate to intercept the plume, if necessary. In many cases it will be adequate to cut off the upgradient source of the plume and allow any contaminated water already downgradient to continue on its course. This will depend upon the size of the plume and the use of the groundwater downgradient.

Interceptor drains must be properly designed so that their upgradient and downgradient influences completely capture and remove the contaminant plume. If after installation, a drain is not operating properly, effective plume management can still be achieved by:

- Installing a cut-off wall downgradient of the drain to limit plume migration
- Installing a second drain up- or downgradient of the first to cause further reductions in head levels
- Installing a second drain above or below the first to intercept leachate which bypasses the original drain.

The upgradient and downgradient influence of an interceptor drain can be determined theoretically or in the field. Evaluating the upgradient and downgradient influence in the field requires that the first interceptor drain be installed at a position and depth determined from soil borings and groundwater monitoring. A period of time should be allowed for the system to come to equilibrium before piezometers are monitored to determine the drawdown at various points upgradient and downgradient of the interceptor.

The theoretical determination of the upgradient influence of an interceptor drain involves the use of an equation developed by Glover and Donnan (1959), as described by Van Hoorn and Vandermolen (1974):

$$D_u = 1.33\, m_s I$$

where: D_u = Effective distance of drawdown upgradient (ft)

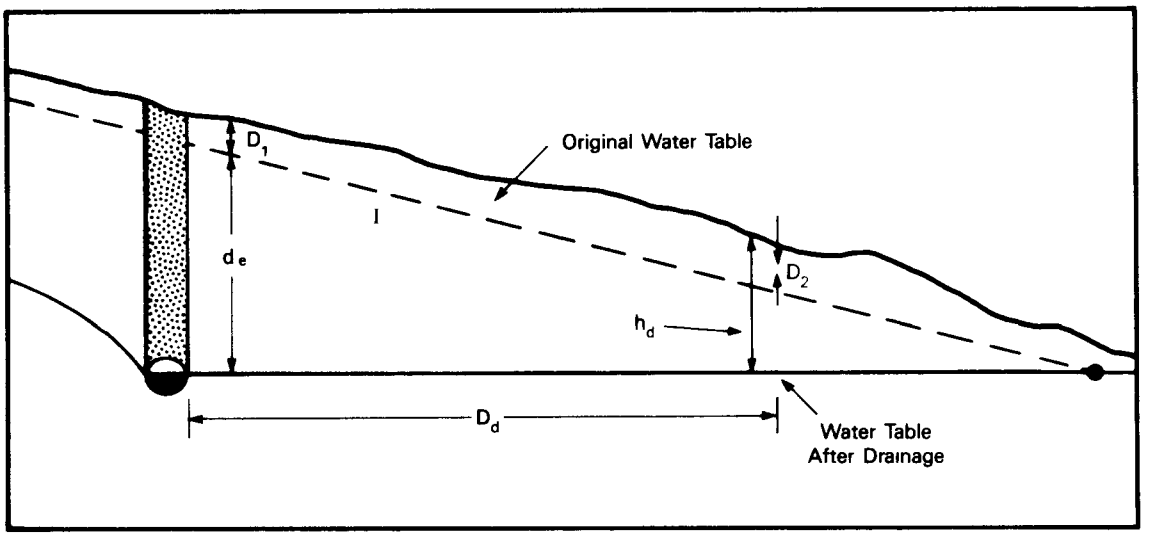
m_s = Saturated thickness of the water bearing strata not affected by drainage (ft)

I = hydraulic gradient (dimensionless).

The theoretical determination of the downgradient influence can be obtained from the following equation (Figure 6-20):

$$D_d = (KI/q)\,(d_e - h_d - D_2)$$

FIGURE 6-20.
SYMBOLS FOR THE GLOVER AND DONNAN EQUATION FOR CALCULATING THE DOWNGRADIENT INFLUENCE OF AN INTERCEPTOR DRAIN



where:
- K = hydraulic conductivity (ft/day)
- I = hydraulic gradient (dimensionless)
- q = drainage coefficient (ft/day)
- d_e = depth of drain (ft)
- h_d = desired depth of drawdown (ft)
- D_2 = distance from ground surface to water table prior to drainage at the distance D_d downgradient from the drain (ft)
- D_d = downgradient influence (ft).

In the equation given above, D_d and D_2 are interdependent variables. In obtaining the solution to the equation, estimating the value of D_2 is necessary and then trial computations are made. If the actual value of D_2 at distance D_d is appreciably different, a second calculation is necessary. Where I is uniform throughout the area, D_2 can be considered equal to D_1 (i.e., the distance from the ground surface to the water table measured at the drain). If a second interceptor is needed to lower the water table to the desired depth, it would be located D_d feet downgradient from the first.

6.2.2.2.2 Interceptor Drains for Controlling a Groundwater Mound (Figure 6-18b)

The design of subsurface drains for managing a plume generated from a site containing a groundwater mound depends on the direction(s) of flow from the site, the direction(s) of natural groundwater flow, the hydraulic gradient(s) and the depth to a low permeability barrier. For this situation, drains can be placed completely encircling the mound, or in parallel on either side of a mound.

If the drains are installed so that they rest on top of a low permeability barrier and the water table is drawn down to the level of the drain piping, drain spacing is not important because any part of the plume that lies outside the influence of the drains will nevertheless be contained beneath the site boundaries. If the site overlies fractured bedrock or if the cost of construction precludes installation of the drains to the top of the low permeability layer, then spacing becomes an important design criteria. This is because the drawdowns of the drains may not intersect, and leachate could flow beneath the drains or through fractured bedrock. If the hydraulic gradient is not too steep, the drain spacing formulas presented for relief drains can be used to approximate spacing. These formulas (Hooghoudt and Ernst equations) can provide a reasonable approximation of spacing where the hydraulic gradient is less than three percent. Gradient becomes a much more important factor in determining flow and drain spacing when gradients are greater than three percent. Under these circumstances, the formulas for determining the upgradient and downgradient influence of the interceptor drains provide a more accurate approach for drain spacing. Computer models are generally required for these situations to ensure proper design.

6.3 Design

In designing a closed pipe drain, the pipe is assumed to be able to accept the drainage water when it arrives at the drainline and that the pipe will carry away the water without a build up of pressure. To meet the first criterion, the relationship between the hydraulic conductivity of the gravel

envelope, the perforations in the drain pipe, and the base material must be assessed as described in the following subsection on filters and envelopes. To meet the the second criterion, the pipe size and drain slope must be adequate to carry away the water after it enters the pipe.

6.3.1 Flow Capacity

6.3.1.1 Total Drainage Discharge

In order to estimate drain diameters (i.e., the hydraulic design) and volume of storage required by the sump, the total discharge (Q) from the laterals, collectors, and mains must be determined. Estimates of total discharge can be obtained using the water balance method. This method provides an estimate of the amount of percolation that will recharge the water table between the lines of the drain. Once the percolation rate has been calculated, discharge can be obtained by multiplying the rate times the drainage area: Q (ft^3/day) = q (ft/day) x area (ft^2). Where water balance data are not available, the discharge from parallel relief drains can be approximated from the following formulas (Bureau of Reclamation, 1978):

$$Q = 2\pi KHdx/L \quad \text{(for drains above a barrier)}$$

$$Q = 4KH^2x/L \quad \text{(for drains on a barrier)}$$

where:
- Q = total discharge from two sides (ft^3/day)
- H = maximum height of the water table above the drain, midway between the drains (ft)
- K = weighted average hydraulic conductivity (ft/day)
- d = equivalent depth (ft)
- L = drain spacing (ft)
- x = length of pipe (ft).

Guidance for calculating H, d, and L is given in section 6.2.2.1.

The rate of flow (Q) from an interceptor drain can be estimated quantitatively using the equation:

$$Q = KIA$$

where:

Q = unit flow (ft^3/second)
K = hydraulic conductivity (ft/second)
I = slope (dimensionless)
A = area through which flow occurs (ft^2).

The cross sectional area intercepted by an interceptor drain is equal to the effective depth of the drain (i.e., the vertical distance from the bottom of the drain to the water table) times the length of the drain:

$$Q = KI\ d_e x$$

where:

d_e = average effective depth of the drain (ft)
x = length of the drain (ft).

The use of the above equation should be reserved for situations in which the hydraulic conductivity, soil profile, and cross sectional area are uniform and accurately known. In some instances constructing a pilot drainage system may be more desirable to more accurately determine discharge and drain size (SCS, 1973).

6.3.1.2 Gradient and Velocity

The proper installation and function of pipe drains requires rigid control of grade and alignment in order to prevent siltation. The minimum grades for a closed pipe drain where siltation is not likely to be a hazard are given in Table 6-2. Steeper grades are generally more desirable. With steeper grades, the control required during construction is less exacting and there is also less chance of the drains clogging. The selected grade should be great enough to result in a flow velocity that prevents siltation yet will not cause turbulence. Where the velocity is less than 1.4 feet/second and siltation is a hazard, preventative measures including filters and silt traps should be considered. The velocity (i.e., critical velocity) at which turbulence results varies with soil type. Critical velocities of various soil types are shown in Table 6-3. Table 6-4 gives the grades for various drain sizes which result in the critical velocity discussed above. (SCS, 1973; Bureau of Reclamation, 1978).

TABLE 6-2

MINIMUM GRADES FOR VARIOUS PIPE SIZES (SCS, 1973)

Pipe diameter (in)	Grade (%)
4	0.10
5	0.07
6	0.05

TABLE 6-3

CRITICAL VELOCITY OF VARIOUS SOIL TYPES (SCS, 1973)

Soil Types	Velocity (ft/sec)
Sand and Sandy Loam	3.5
Silt and Silt Loam	5.0
Silty Clay Loam	6.0
Clay and Clay Loam	7.0
Course Sand and Gravel	9.0

TABLE 6-4

DRAIN GRADES FOR SELECTED CRITICAL VELOCITIES

Drain Size Inches	VELOCITY (ft/sec) 1.4	3.5	5.0	6.0	7.0	9.0
	Grade--feet per 100 feet					
	For drains with "N" = 0.011[a]					
	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

(a)--"N" is the roughness coefficient.

6.3.1.3 Drain Diameter

The size of the drain for a given capacity is dependent on the flow, hydraulic gradient, and the roughness coefficient, N, which is a function of the hydraulic resistance of the drain material. The formula for hydraulic design is based on the Manning formula for pipes, which is written as follows:

$$Q = AV = k_m A R^{0.67} I^{0.5}$$

in which: Q = discharge (ft^3/sec)
R = hydraulic radius (feet) equal to the wetted cross-sectional area, A_w, divided by the wetted perimeter,
(1/4 the diameter for full flowing pipes)
I = hydraulic gradient
k_m = roughness factor (1/N)
V = velocity (ft/sec)
A = area (ft^2).

To apply the Manning formula in the hydraulic design of drain pipes, use is made of the equation:

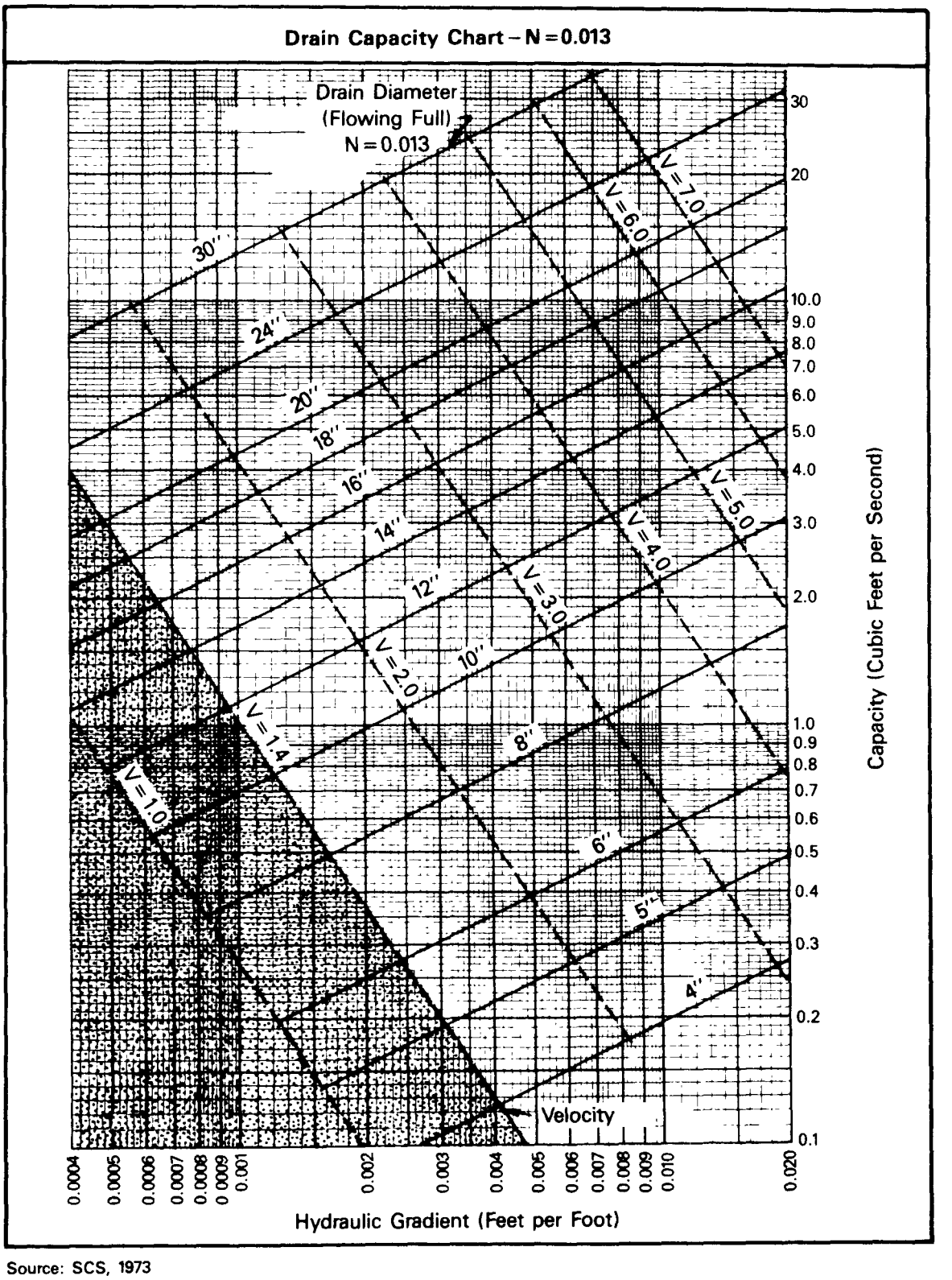
$$Q = AV = qA_d$$

in which q is the specific discharge of the drain (feet/sec) and A_d is the area affected by the drain (ft^2).

Using the calculated discharge and knowing the gradient of the drain line and the roughness coefficient, the pipe size can be determined from nomographs based on the Manning Formula. Figures 6-21 and 6-22 are nomographs for estimating drain size for values of N = 0.013 and N = 0.015 respectively.

These nomographs provide a means of selecting proper drain size at the starting point in the drainage system. However, in drainage systems with long laterals or mains, common practice is to change to a larger pipe size after some distance to allow for the increased quantity of water to be carried. The following example illustrates a method for determining at what distance a change to the next larger pipe diameter should be made.

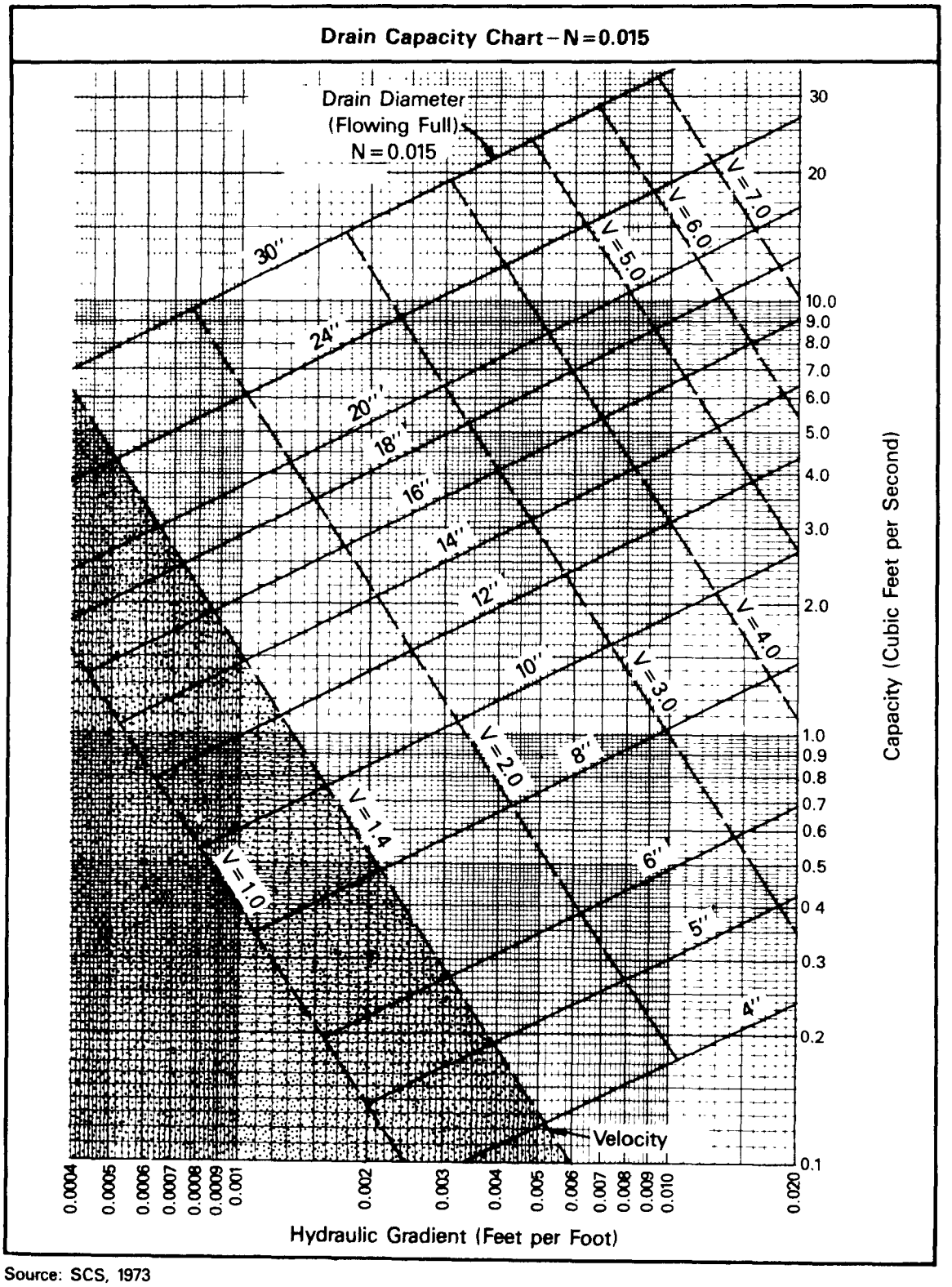
FIGURE 6-21.
CAPACITY CHART FOR N = 0.013



Source: SCS, 1973

Note: The shaded area indicates where the velocity of flow is less than 1.4 feet per second to indicate where drain filters may be required.

FIGURE 6-22.
CAPACITY CHART FOR N = 0.015



Source: SCS, 1973

NOTE: The shaded area indicates where the velocity of flow is less than 1.4 feet per second to indicate where drain filters may be required

Assume that the length of a drain is 6000 feet and that the total discharge at a constant grade of 0.2 percent is 2.40 feet^3/second. The increase in flow or accretion would be given by:

$$2.40\ (ft^3/sec)/6000\ (ft) = 0.0004\ (ft^2/sec)$$

Assuming that a 6 inch drain pipe is used at the upper end of the system, one can determine to what length down drain the 6 inch pipe would be adequate by referring to Figure 6-22 (N=0.015). From Figure 6-22, a 6 inch diameter pipe on a grade of 0.20 percent has a maximum capacity of 0.22 ft^3/sec. Letting X equal the distance downdrain that the 6 inch pipe would be adequate, then:

$$X = 0.22\ (ft^3/sec)/0.0004\ (ft^2/sec) = 550\ ft$$

The 6 inch drain is therefore adequate for 550 feet of line. At this point, the switch to an 8 inch pipe is made. These computations should be continued progressively for the total 6000 feet of drain pipe. If laterals enter the drain, the estimated yield of these should be added at the proper section (SCS, 1973).

6.3.2 Filters and Envelopes

Performance of a drainage system is based on the assumption that the drainage system will accept drainage water when it arrives at the drainline. Filters and envelopes are used to ensure that this requirement is met.

6.3.2.1 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 is more permeable 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 material such as gravel around the pipe, the number of pore connections at the boundary between the soil and the envelope will increase, thereby decreasing the flow velocity (Wesseling, 1973).

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 entrance resistance, which can cause the water level to rise above the drain (Wesseling, 1973).

Although filters and envelopes have distinctly different functions, well graded sands and gravels can be used to meet the requirements of both a filter and an envelope. Depending on the design manual that is consulted, there may or may not be a distinction made between envelopes and filters.

6.3.2.2 Types of Filters and Envelopes

Well graded sands and gravels can function both as an envelope and a filter. The specifications for granular filters, however, are more rigid than those for envelopes. Usually filter materials must 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, 1973).

A variety of soil stabilizers have been developed as a substitute for sand and gravel envelopes. They are added to local soils in low concentrations to make stable aggregates which function as envelopes when backfilled around the trench. Artificial aggregates have been produced using portland cement, asphalt emulsions, and various polymer solutions and emulsions (Wesseling, 1973). With the exception of portland cement aggregates, which have been used successfully at some sites, soil stabilizers are generally not an economical substitute for sand and gravel. Also in many instances, these aggregates can react unfavorably with some components of the leachate thus altering the hydraulic conductivity of the envelope materials and increasing the hydraulic resistance.

Synthetic materials are used mainly for filtration. The most widely used materials for synthetic filters are nylon, polypropylene, polyvinyl chloride, polyethylene, and polyester fibers. The most common manufacturing processes for synthetic filters include weaving, which produces a window screen type of material and bonding in which the density and open spaces are controlled by fiber diameter and the number of fiber layers. Spun bonded filters have a large number of openings with a range of opening sizes throughout, whereas woven filters have openings of a fixed size. Spun bonded fabrics are most widely used in drainage systems and are recommended for gravel and sand soils but not for silts and clays.

6.3.2.3 Design

6.3.2.3.1 Sand and Gravel Filters and Envelopes

Detailed design procedures are available for both gravel and sand envelopes. SCS (1973) has distinct design criteria for filters and envelopes, whereas the Bureau of Reclamation (1978) has developed one set of standards for a well graded envelope which meets the requirements of both a filter and an envelope. The separate SCS (1973) design criteria will be considered below for the following reasons:

- Site specific conditions may warrant the use of only a filter or an envelope, but not both

- Where both a filter and an envelope are needed, the SCS design criteria for a filter can generally be used
- Use of fabric filters may be desirable with a gravel envelope.

To provide general guidelines for the use of gravel filters, SCS has divided soils into three groups depending on their need for a filter (Table 6-5). Soils with a high percentage of fines (the first grouping in Table 6-5) always require a filter. The second group may or may not need a filter and the last group seldom needs a filter.

The general procedure for designing a gravel filter is to: (1) make a mechanical analysis of both the soil and the proposed envelope 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 have adopted similar criteria which set size limits for a filter material based on the size of the base material. These limits are as follows:

D_{50} filter/D_{50}base = 12 to 58%

D_{15} filter/D_{15} base = 12 to 40%

Multiplying the 50 percent grain size (D_{50}) of the base material by 12 and 58 percent gives the limits the 50 percent grain size of the filter should fall within. Multiplying the 15 percent grain size (D_{15}) of the base material by 12 and 40 percent gives the limits the 15 percent grain size of the filter should fall within. Figure 6-23 shows the upper and lower limit curves of drain envelope suitability for a specific soil. In this example, only filter "No. 3" falls within the 15 percent and 50 percent limits. When filter and base materials are more or less uniformly graded, a generally safe filter stability ratio is given by:

$$D_{15} \text{ filter}/D_{85} \text{ filter} < 5$$

TABLE 6-5

A CLASSIFICATION TO DETERMINE THE NEED FOR DRAIN FILTERS OR ENVELOPES (SCS, 1973)

Unified Soil Classification	Soil Description	Filter Recommendation	Envelope Recommendation
SP (fine)	Poorly graded sands, gravelly sands.		
SM (fine)	Silty sands, poorly graded sand-silt mixture.		Not needed where a sand and gravel filter is used but may be needed with flexible drain tubing and other type filters.
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.	Filter needed	
MH (diatomaceous fine)	Inorganic silts, micaceous sandy or silty soils, elastic silts.		
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.		Not needed where a sand and gravel filter is used but may be needed with flexible drain tubing and other type filters.
SC	Clayey sands, poorly graded sand-clay mixtures.	Subject to local on-site determination.	
GM	Silty gravels, poorly graded gravel-sand-silt mixtures.		
SM (coarse)	Silty sands, poorly graded sand-silt mixtures.		
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.		
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.		
SP,GP (coarse)	Same as SP & GP above.		Optional.
GW	Well graded gravels, gravel-sand mixtures, little or no fines.	None	May be needed with flexible drain tubing.
SW	Well graded sands, gravelly sands, little or no fines.		
CH	Inorganic, fat clays.		
OL	Organic silts and organic silt-clays of low plasticity.		
OH	Organic clays of medium to high plasticity.		
Pt	Peat.		

FIGURE 6-23.
MECHANICAL ANALYSIS OF A GRAVEL FILTER MATERIAL
(SCS, 1973)

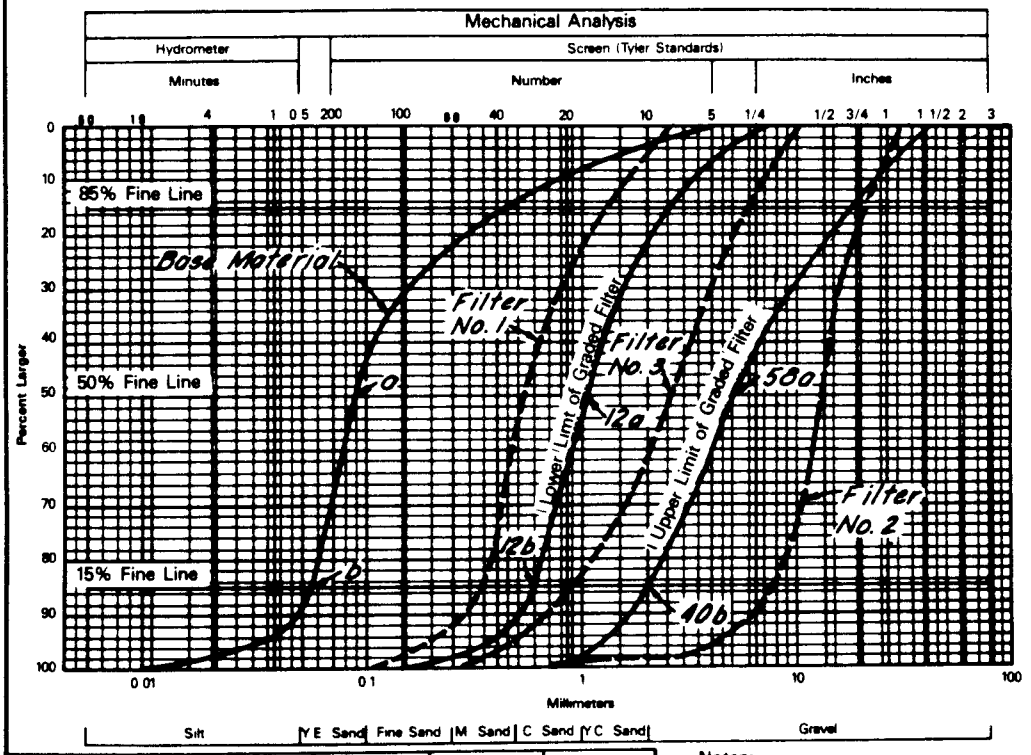
State _____ Area _____ District _____

Property *John Jones*

Location *T.13S., R.10E., SW¼ Sec.8* Date *4-9-70*

Base Soil Sample: No. 1 *4 to 6 Feet* No. 2 _____

Proposed Filters
- No. 1 *McGregor Pit*
- No. 2 *Wilson Pit*
- No. 3 *Jacob Pit*



Mech Analysis		Base No 1 Ret	Base No 1 Acc	Base No 2 Ret	Base No 2 Acc	Filter No 1 Ret	Filter No 1 Acc	Filter No 2 Ret	Filter No 2 Acc	Filter No 3 Ret	Filter No 3 Acc
Sieve	1 1/2							0	0		
	1							2	2		
	1/2							44	46	0	0
	1/4	0	0					44	90	10	10
	5	1	1			0	0	9	99	18	28
	10	2	3			3	3	0	99	36	64
	20	7	10			23	26	0	99	23	87
	40	6	16			44	70	1	100	10	97
	60	6	22			24	94			2	99
	100	8	30			4	98			1	100
	200	34	64			2	100				
Hydrometer	0 5	23	87								
	1	5	92								
	4	4	96								
	19	4	100								
	60										

Notes:
Filter No. 3 fits the base material curve better than 1 or 2 and should be specified as 1 and 2 do not fall within the 15 and 50 % bracket

Recommended Filter *No. 3*

For perforated pipe drains, the requirement for the minimum size of the envelope material is affected by the size of the perforations. SCS recommends that the 85 percent grain size (D_{85}) of the envelope material should not be smaller than half the diameter of the perforations, and not more than 10 percent of the filter material should pass the No. 60 sieve. SCS (1973) also recommends a minimum filter thickness of three inches or more.

6.3.2.3.2 Sand and Gravel Envelopes

Envelopes are generally recommended in order to provide a permeable path for water to move into the pipe openings. They are also recommended to improve the stability of certain soils during pipe installation and to provide stability and support for corrugated flexible pipe. The design approach recommended by SCS (1973) is to first determine whether the drainage requires a filter and then to determine the need for an envelope. This is because a well graded filter can generally also serve the function of an envelope.

The first requirement of sand and gravel envelopes is that the envelope have a hydraulic conductivity 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 (0.01-inch). 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 greater the inflow. 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 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 recommends a minimum thickness of four inches around the pipe, while SCS (1973) recommends a three-inch minimum thickness.

6.3.2.3.2 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 (Dupont, 1981):

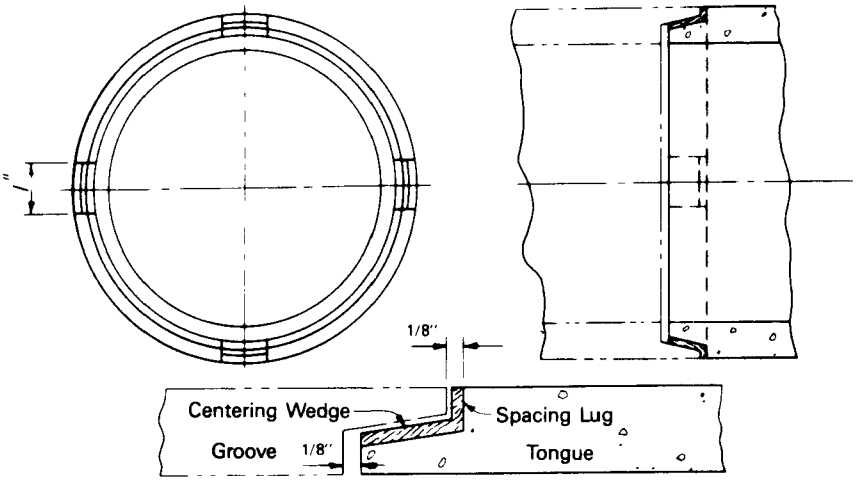
$$\frac{P_{85}\ (85\%\ \text{pore size of the filter fabric})}{D_{85}\ (85\%\ \text{grain size of the subgrade material})} \leqslant 1$$

$$\text{or } P_{85} \leqslant D_{85}$$

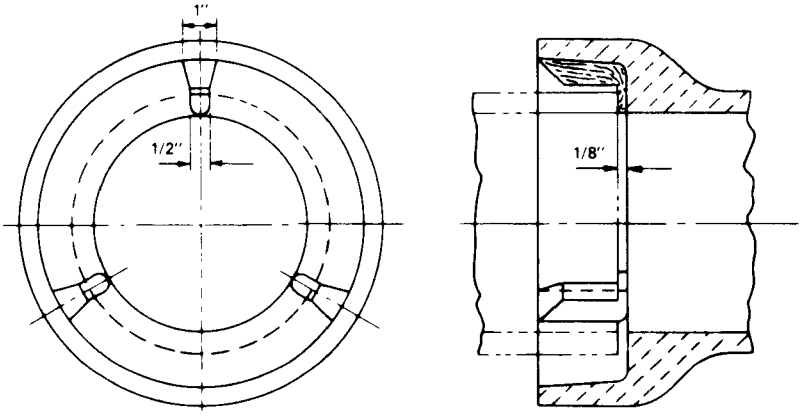
6.3.3 Design and Selection of Pipes

Materials commonly used for subsurface drains include ceramic and concrete tile and plastic pipe. Clay and concrete tiles may be perforated, or they may have openings or joints between the segments through which water may enter. Jointed concrete tiles may be manufactured with plain, tongue-and-groove, or bell-and-spigot ends as shown in Figure 6-24. Tongue-and-groove and bell-and-spigot joints interlock, making them easier to place and hold alignment. Perforated pipe is easier to install than jointed pipe and is more widely used for subsurface drainage at waste disposal sites. Plastic pipe is available as perforated and flexible corrugated pipe. Subsurface drains may be installed at considerable depth and the ability of the pipe to carry the load of the backfill will be an important consideration in some cases. The American Society for Testing and Materials (ASTM) standards lists allowable crushing strengths of rigid pipe drains. These standards are listed below under the discussion of specific pipe types. For corrugated plastic tubing,

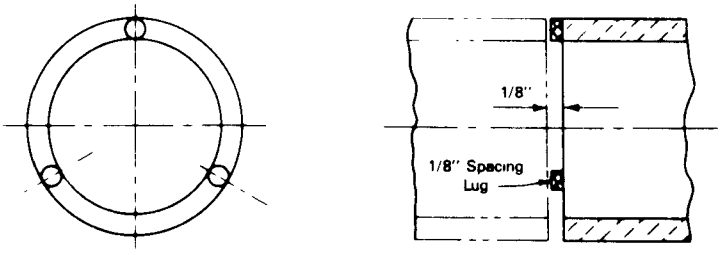
FIGURE 6-24.
JOINT DESIGN FOR RIGID DRAIN TILES
(BUREAU OF RECLAMATION, 1978)



Tongue and Groove Type for Concrete Pipe



Bell and Spigot Type for Clay or Concrete Pipe
Use 4 Wedges and Lugs for Concrete Pipe (Clay Pipe Shown)



Plain End Type for Clay or Concrete Pipe

the strength depends upon the bedding material. All plastic drains should be installed in at least a four-inch gravel envelope. Flexible pipe deflects when loaded, which results in a transfer of the load to the bedding material. Safe loads are those that will cause 10 percent or less deflection. Equations for determining deflection are presented in the Bureau of Reclamation's Drainage Manual (1978).

6.3.3.1 Clay Pipes

There are basically two types of clay drains manufactured for subsurface drainage--clay drain tile, which is no longer widely used, and perforated, vitrified clay pipe. Standard specifications can be found in ASTM C4 and C700 (ASTM, Part 16, 1982), respectively. Clay drain tile is considerably less expensive than vitrified clay pipe, but vitrified clay is stronger, more durable, and more chemically resistant than clay tile. Vitrified clay piping tested over a six month exposure period was found to be resistant to acids, bases, chlorinated and aromatic solvents, chromic acid, copper sulfate, and numerous other chemicals. However, vitrified clay piping was found not to be resistant to hydrofluoric acid (Logan Clay, 1983). Vitrified clay pipe is available in standard strength and extra strength, ranging in diameter from 3 to 42 inches. Standards require that perforations be 0.25 inch in diameter.

6.3.3.2 Concrete Pipes

Unreinforced concrete drain pipes suitable for drainage of industrial wastes must comply with ASTM C14, C412, or C444 (ASTM, Part 16, 1982). Standard C412 covers drain tile and C14 and C444 deal with perforated concrete pipe. Where concrete pipe will be exposed to a pH of less than six or to sulfate concentrations in excess of 400 parts per million or both, acid and sulfate resistant cement should be used since regular types of cement are subject to disintegration by these materials. Perforated asbestos cement pipe (C-508-81) is also resistant to acids and may be used for drainage.

6.3.3.3 Plastic Pipes

The most commonly used materials in the manufacture of plastic pipes are polyvinyl chloride (PVC) and high density polyethylene (HDPE). At an equivalent weight and size, PVC pipes have a somewhat higher resistance to outside pressure than HDPE pipes. PVC is generally recommended for large pipes in excess of six inches because of its greater strength. However, HDPE is less impact resistant especially under low temperatures (Cavelaars, 1974). PVC and HDPE have good resistance to dilute acids and bases. PVC is not recommended for chlorinated and aromatic solvents.

Plastic pipes come in smooth and corrugated varieties and there are advantages and disadvantages to each. Corrugated wall plastic tubes are stronger, lighter weight, less expensive, and easier to handle because of their flexibility than are smooth-walled plastic pipes. Smooth pipes, on the other hand, have a considerably lower hydraulic resistance than do corrugated pipes. The outside diameter of smooth pipes can be about 25 percent less than that of corrugated pipes (Cavelaars, 1974).

Flexible corrugated tubes have gained considerable popularity in agricultural drainage in recent years. A flexible drain tube gains part of its vertical soil load carrying capacity by lateral support from the soil at the sides of the conduit. Thus, the stiffness of the conduit wall as well as the rigidity of the soil surrounding the tube are both structural parameters (Van Schilfgaarde, 1974). This implies that the stability of the surrounding soil is an important factor in the performance of flexible corrugated tubing. For conventional ceramic or concrete tile, conduit wall rigidly is the principal parameter.

6.3.3.4 Manholes

Manholes are located in pipe drains to serve as junction boxes, silt and sand traps, observation wells, discharge measurement points, access sites to the drain for maintenance, and for the easy location of drains. There are not

set criteria for spacing manholes. In general, they should be used at junction points on a drain and at major grade changes. Sometimes a manhole is not required at a junction and a simple "Y" or "T" section may be adequate.

6.3.4 Drainage Sump and Pumping Plant

The main steps in the design of a drainage sump and pumping plant (Figure 6-25) are to (Bureau of Reclamation, 1978):

- Determine the maximum inflow to the sump
- Determine the amount of storage required
- Calculate the pumping rate
- Determine the start, stop, and discharge levels
- Determine the type and size of the storage tank
- Select the pump and the motor.

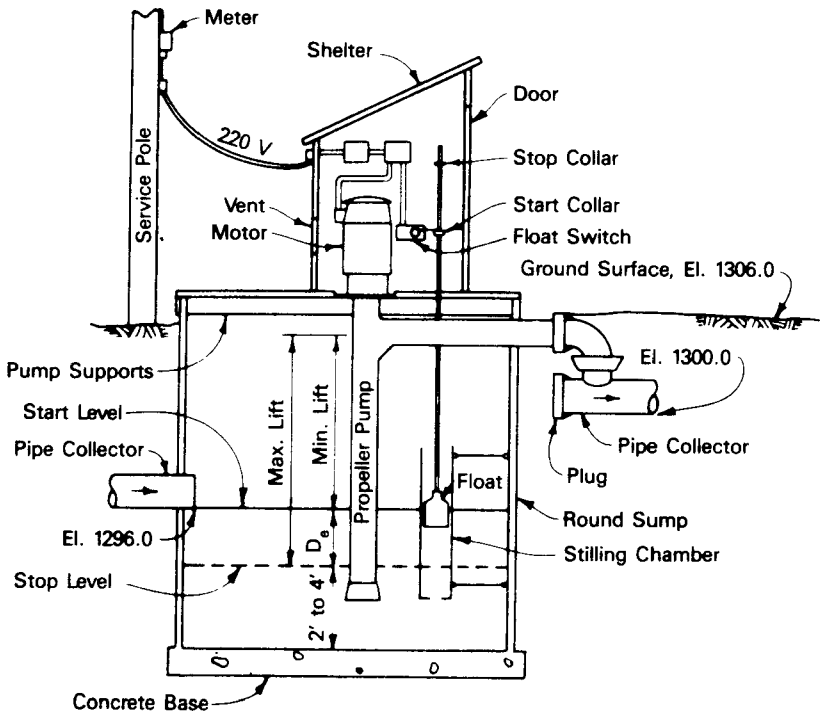
The maximum inflow to the sump is determined from the drainage coefficient and the area served by a drain or drains discharging to the sump. As a precaution, a 20 percent allowance should be made for flow that may occur in excess of the design rate. Therefore:

$$Q_p = 1.2\ Q_g$$

where: Q_p = pumped discharge capacity (ft^3/day)
Q_g = gravity discharge capacity (ft^3/day)

The amount of storage required depends on the inflow and the cycling operation of the pump. The pump and motor are most efficient when run continuously, although an 8 to 12 minute cycle is almost as efficient. Assuming a 12 minute cycle, for example, during maximum inflow there will be 5 cycles per hour, or 5 starts per hour, with equal on and off times of 6 minutes each. During low flow periods, the off time will be much longer than the running time. For a motor to have equal on and off times, the storage capacity must be equal to the amount of water that would run into the

FIGURE 6-25.
TYPICAL DESIGN OF AN AUTOMATIC DRAINAGE PUMPING PLANT
(BUREAU OF RECLAMATION, 1978)



sump in one-half the cycling time (t_c), or 6 minutes for the example used above. Therefore:

$$S_v = t_c \, Q_p$$

where: S_v is the volume of storage required (ft^3).

The pumping rate, Q_m (ft^3/min), can then be determined using the following expression:

$$Q_m = (S_v + Q_p t_p)/t_p$$

where: t_p is the running time of the pump for maximum inflow (min).

In order to estimate the size of the storage sump required, the inflow and the minimum and maximum water levels in the sump must be estimated. In general, the maximum water level for starting the pump should be at the top of the pipe drain discharging into the sump. Maximum water level should never exceed one-half of the pipe diameter over the top of the drain. The mimimum elevation should be from 2 to 4 feet above the base of the sump. The difference in elevation between the water level in the sump and the discharge elevation is called the pump lift (Figure 6-25).

The volume required for storage plus the requirement for a minimum water level above the bottom of the sump determines the size of the sump. The distance between the pump cut-off and cut-on elevation (D_e) should be small to keep the depth of the sump reasonable. Assuming a D_e of 2 feet and an inflow volume (S_v) of 150 feet^3, the required diameter (D_x) for a vertical cylindrical sump can be estimated as follows:

$$D_x = (S_v/0.7854D_e)^{0.5}$$

$$D_x = (150\ ft^3/(0.7854(2))^{0.5}$$

$$D_x = 10\ ft$$

The next step in the design is to determine pump type, number, and size. The type and size of the pump and motor can be selected from reliable pump and motor manufacturers. Suitable pumps must be able to handle relatively large flows and substantial amounts of sediment. The performance of the pump varies with head (static lift plus frictional losses in the pump), speed, discharge, and horsepower. Axial flow, mixed flow, or radial flow centrifugal pumps are generally recommended.

The size and number of pumps are determined from the system's required capacity. If the discharge volume is large, having two pumps may be advantageous to provide more efficient pumping over a wider range of pumping rates. Also, if a breakdown of one pump occurs the system operation will not be upset. In a plant with two pumps, one pump operation is generally recommended to have about half the capacity of the other. When the pumps are not of equal

capacity, storage should be allowed for the capacity of the largest pump (Bureau of Reclamation, 1978; SCS, 1973).

6.4 Installation and Maintenance

The following section describes the equipment and procedures used in the construction and maintenance of subsurface drain systems. A large portion of this section is devoted to trench excavation methods and equipment because this portion of the drain installation process is often the most complex and costly. Included are sections on excavation equipment, wall stabilization, dewatering, and grade control. Drain installation is the next major section, which includes the material and procedures most commonly used for placing bedding, drain pipe, the gravel envelope, geotextile filter fabric, and backfill material.

6.4.1 Trench Excavation

Trench excavation is the most significant step in the construction of a subsurface drain. The ease or difficulty of excavation can have a dramatic effect on the cost of the total installation. A difficult trench excavation could even result in the exclusion of subsurface drainage as a viable technique because of prohibitive costs.

A number of different work elements may be involved in excavating a trench for a subsurface drain. Preliminary exploration may be carried out by direct excavation, drilling, or seismic testing. The presence of rock may require that the rock be mechanically ripped, blasted, or otherwise fragmented. Actual excavation of earth or rock may be carried out by a variety of excavation equipment, the optimum being determined by the depth, width, and length of the trench and the material being removed. Also, a number of alternatives are available to prevent wall failure. These alternatives are related to the size of the trench and the stability of the surrounding soil. Provisions for dewatering and proper grading are also important aspects of trench excavation.

6.4.1.1 Subsurface Exploration

The selection of an excavation technique is intimately related to the characteristics of the unconsolidated material or rock being removed. Some form of subsurface exploration should be conducted to define the various earth layers so that proper excavation techniques can be selected, accurate cost estimates can be developed, and proper planning can occur.

Generally, three methods exist for subsurface exploration--direct excavation, drilling, and seismic testing. Direct excavation is carried out with a backhoe or ripper. Drilling will most probably be occurring at a hazardous waste site for the purposes of determining the local geology and the extent of groundwater and soil contamination. For subsurface exploration purposes, only drilling techniques in which a drilling log can be prepared are appropriate. Techniques for drilling are discussed in Chapter 5.

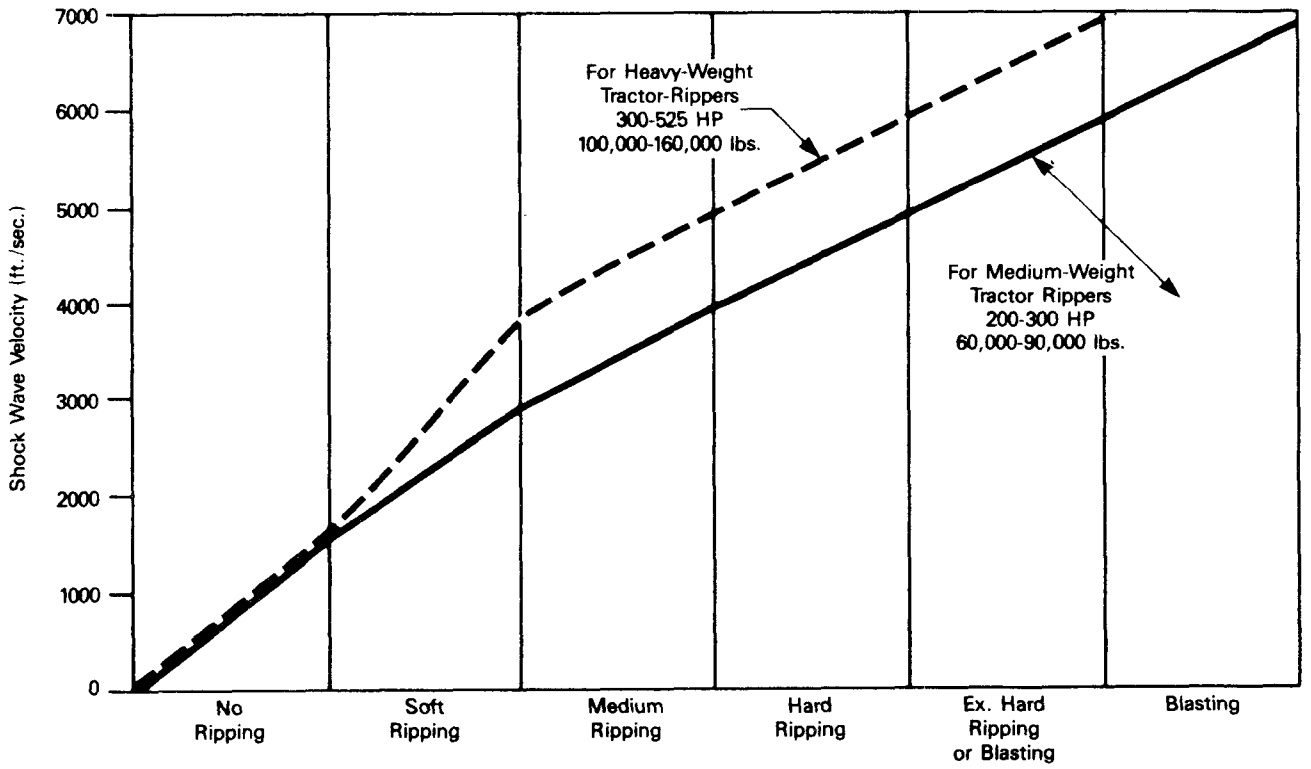
Seismic techniques have been used for about 75 years as a standard method for oil exploration and for construction planning. Seismic testing is based on measuring the velocity of a shock wave through earth formations at various depths. The shock wave velocity varies considerably with respect to the condition of the earth materials through which the wave passes. In seismic testing, the shock wave can be generated by a sledge hammer and striking disk (for limited explorations) or by blasting for larger explorations. The sledge hammer-disk method can be used for distances up to 300 feet and at working depths of about 100 feet. Blasting, although the preferred techniques for large scale seismic studies, should only be used with extreme caution near hazardous waste sites, if at all. The shock wave generated from either technique is measured by a sensing device called a geophone, which is located a designated distance from the generation point. The time interval between shock wave generation and measurement and the distance between the two points determines velocity.

Seismic profiles are used in the construction industry to classify the type of excavation required at various depths. Generally, excavation can be classified into three categories: common (non-ripping), mechanical ripping,

and blasting. Ripping is further divided into soft, medium, hard, and extremely hard ripping, which helps to describe the types of equipment required, production rates, and unit costs. General guidelines for classifying the type of excavation and specifying the equipment required with respect to seismic velocities have been developed over the years by many correlative studies. The guidelines are summarized in Figure 6-26.

As shown in Figure 6-26, if seismic velocities are over 7,000 feet per second, rock can not be mechanically ripped and must be fragmented in another manner. Blasting and other techniques greatly increase the cost of trench excavation. Thus, the distance to the "drill and shoot" or blasting zone and the amount of rock requiring blasting are major considerations with respect to the feasibility of installing a subsurface drain.

FIGURE 6-26.
CORRELATION OF REQUIRED EXCAVATION METHODS WITH SEISMIC TESTING DATA (DERIVED FROM CHURCH, 1981)



The operations of mechanical ripping and blasting are obviously quite different, and excavation by either means mandates a different set of considerations. The equipment, methodology, and limitations with respect to each of the techniques is briefly discussed in later sections.

6.4.1.2 Rock Fragmentation

6.4.1.2.1 Ripping

Mechanical ripping can now fragment rock of considerable consolidation where previously only blasting was feasible. The ripper is basically composed of a shank with a replaceable alloy steel tip attached to a tool bar. The tool bar can be raised, lowered, or inclined by a series of arms and hydraulic cylinders known as the power assembly. The ripper is attached to and pulled by a tractor, which usually doubles as a push dozer. Rock ripping usually takes place by lowering and angling the tip into the rock while the tractor moves slowly along at a range of 0.75 to 1.50 miles per hour. Ripping occurs in igneous and metamorphic rocks by a crushing and breaking action and in sedimentary rocks by splitting and cleaving.

Rippers for trench excavation are composed of a single shank and tip assembly, while rippers for other types of applications may have two or three assembles mounted on the tool bar.

The maximum vertical reach of the largest trench ripper available is a little over six feet (Alban Tractor Co., 1982). Therefore, the use of ripping is limited to a trench depth of six feet. However, if the ripper can enter the trench to rip the lower lifts, a greater depth can be reached. At some trench depths, ripping may become relatively uneconomical to continue to use because the trench width needed for clearance increases the volume of material

to be excavated. Analysis of alternate construction techniques as a function of depth is essential to minimize project costs.

6.4.1.2.2 Impact Methods

Impact methods can be used on a smaller scale in trench work to break rock. The hourly outputs of impact methods are only about one percent of the output of a ripper. However, they may be preferred in some cases even though they are a more expensive option. Three types of equipment are available for impact or percussive rock fragmentation--the hand-held jackhammer, the backhoe mounted Hobgoblin, and the Darda rock and concrete splitter.

The hand held jackhammer is a small percussion drill operating on about 90 pounds per square inch of air pressure and imparting about 2,000 blows per minute (Church, 1981). A jack hammer's use is limited to a depth of 16 inches. The average output of a jackhammer in limestone is about six cubic yards per hour (Richardson, 1980). The Hobgoblin is a pneumatically driven impact tool that is mounted on a backhoe arm and acts similar to a jackhammer. A Hobgoblin also has a relatively low production rate (i.e., six cubic yards per hour). The Darda rock and concrete splitter uses water as the percussive force in splitting rock. The splitter exhibits deeper penetration and higher output than the jackhammer or Hobgoblin, however, the addition of water could exacerbate pollution from a disposal site. The Darda splitter has a penetration depth of 26 inches and an average production rate of 13 cubic yards per hour (Richardson, 1980).

6.4.1.2.3 Blasting

Blasting is not recommended for rock excavation purposes around hazardous waste sites for a number of reasons. Blasting, even if well controlled, could result in unwanted fracturing below the subsurface drain such that a bypass for the contaminant plume is created. Ground vibration from blasting may also cause unwanted fracturing of impermeable layers such as a previously installed clay cap. The compression wave set off by blasting could cause secondary detonation in a hazardous waste site that contains explosive materials.

Blasting also requires extensive safety provisions, causes general citizen concern (because of noise and flyrock and air pollution) and may cause damage or failure of trench side slopes and shoring.

Blasting to excavate rock for a trench would be conducted using the blasthole technique. In this technique, holes are drilled in rock in a specified arrangement and explosives are loaded into the holes. The percussive force of the detonation fractures the rock between the holes. Controlled blasting is a technique to reduce unwanted fracturing of nearby rock. This method involves drilling a line or lines of open holes that are not loaded with explosives between the blastholes so to create a plane of weakness for rock fragmentation. This action helps to contain fracturing. However, even controlled blasting does not completely eliminate unwanted fracturing. Blasting ranges from $38 to $44 per cubic yard of trench (Godfrey, 1981).

6.4.1.2.4 Non-explosive Demolition

Non-explosive demolition is a viable alternative to blasting as a means of fragmenting rock. In non-explosive demolition, holes are drilled into the rock in a preselected pattern. The dry demolition agent is mixed with the appropriate amount of water and is poured into the holes. As the material dries it expands and exerts an expansive stress on the rock of more than 3,000 metric tons per square meter (A.H. Harris and Sons, Inc., 1982). The rock then fractures because of stress induced by the material's expansion and the pattern of the drill holes. The fracture mechanism is composed of three phases--crack initiation, crack propagation, and the increase of crack width. The direction of cracking can be planned by appropriately arranging the hole spacing, depth, and inclination. Maximum hole depth is approximately 33 feet.

Non-explosive demolition is a process free from flyrock, noise, ground vibration, gas, dust, and the other environmental and safety problems associated with blasting. However, the method is expensive compared to other fragmentation methods because of the amount and cost of the materials needed. Costs range from $300 to $470 per cubic yard of fragmented material. The time

needed for crack initiation and propagation also limits the alternative's feasibility.

6.4.1.3 Excavation Equipment

The following sections describe equipment that can be used to remove earth or fragmented rock for the purposes of excavating a trench. Equipment includes trenches, backhoes, clamshells, cranes, and bulldozers. In each of the following sections, the specific applications, limitations, and production costs are discussed.

6.4.1.3.1 Trenchers

Trenchers or ditchers are designed to provide continuous excavation in soil and well fragmented or weathered rock. They consist of a series of buckets mounted on a wheel (bucket-wheel type) or a chain sprocket and ladder (bucket-ladder type). In continuous trenching, the wheel or ladder is lowered as the revolving buckets excavate the trench to the appropriate depth. The trench assembly may be mounted on wheels or on semi-crawler or full-crawler frames. The trencher moves forward simultaneously as the trench is excavated resulting in a trench of neat lines and grades. The bucket wheel types are generally used to dig shallow trenches for agricultural drainage. The maximum depth for a large wheel trencher is about 8.5 feet (Church, 1981). Bucket-ladder type trenchers can excavate trenches up to 27 feet deep and about 6 feet wide, although 4 feet is the maximum economical width (Church, 1981).

Generally, continuous trenching in suitable materials is much faster than trenching via backhoe. Hourly production rates for wheel and ladder trenchers operating at 100 percent efficiency in various materials is given in Table 6-6. Actual efficiencies for the entire spectrum of job conditions may range from 20 to 90 percent (Church, 1981). Trenchers are much more efficient in rural settings where fewer obstructions are present.

TABLE 6-6

APPROXIMATE HOURLY PRODUCTION IN CUBIC YARDS FOR LADDER AND WHEEL TRENCHERS OPERATING AT 100 PERCENT EFFICIENCY (CHURCH, 1981)

Rock-earth formation	Engine horsepower of trencher			
	50	100	150	200
Alluvium, sand-gravels, lightly cemented	210	420	630	840
Weathered rock-earth:	180	360	540	720
Maximum weathering	120	240	360	480
Minimum weathering	60	120	180	240

Trenchers can be equipped with back-end modifications to provide shoring, install a geotextile envelope, lay either tile or flexible piping, blind the piping, and backfill with gravel or with excavated soil. Thus, they can be designed to perform practically all drain installation functions. Costs for trenching will vary according to the specific site conditions. Costs for deep-trenching using a ladder trencher at 31 percent efficiency (urban setting) were estimated at $1.25 per cubic yard in 1978, including the cost of the trencher, operation and maintenance costs, and manpower (Church, 1981).

6.4.1.3.2 Backhoes

The backhoe is an inverted shovel operated mechanically, hydraulically, or by cable and used for trenching and other subsurface work. A backhoe is used at job sites where the excavator must be kept at ground level and where subsurface structures are numerous. Backhoes can excavate earth and fragmented rock up to one-half the bucket diameter. Large modified backhoes can have a digging depth range of over 70 feet; however, their large size may exclude their use in urban areas.

Production rates for various sizes of backhoe dipper buckets are given in Table 6-7. These rates are based on 100 percent working efficiency. Actual working efficiencies range from 33 to 83 percent depending on the particular site conditions (Church, 1981).

Unit costs for excavating trenches at various depths using various sizes of backhoes are given in Table 6-8.

6.4.1.3.3 Clamshells and Draglines

Clamshells and draglines are crane mounted buckets which are sometimes used in trench excavation operations. Clamshells are used when the trench depth is below the limit of other equipment (about 70 feet) or when access is a problem that prevents the use of other equipment. The use of clamshells is limited to loose rock or earth materials. Production rates for clamshells are low compared to other methods. Typical production rates are from 20 to 35 cubic yards per hour. Costs for clamshell excavation have been estimated to be $2.93 to $4.34 per cubic yard. If excavation is taking place along sheeting or a cofferdam, costs could rise to above $14 per cubic yard excavated.

Draglines are sometimes used to remove loose rock and earth in trenchwork. The dragline bucket is cast outward by the crane and pulled back as it scrapes the rock or earth into the bucket. Draglines are generally used when a large reach is required for loading or casting. Hourly production rates at 100 percent efficiency are on the same order as a backhoe, as shown in Table 6-9. Costs for excavating by dragline for 0.75 and 1.5 cubic yard buckets have been estimated to be $2.47 and $2.76 per cubic yard respectively (Godfrey, 1981).

6.4.1.4 Wall Stabilization Methods

Deep trenching requires that some form of wall stabilization technique be used to prevent cave-in during installation of drain pipe. The most common technique used to reinforce trench openings is to use shoring, which is a network of wood or steel braces or both. Freezing the ground making up the

TABLE 6-7

THEORETICAL HOURLY PRODUCTION OF A HYDRAULIC BACKHOE (EPA, 1982)

Soil Material	Bucket Size (cubic yards)						
	1	1.5	2	2.5	3	3.5	4
Moist loam, sandy clay	85	125	175	220	275	330	380
Sand and gravel	80	120	160	205	260	310	365
Common earth	70	105	150	190	240	280	330
Clay, hard dense	65	100	130	170	210	252	300

TABLE 6-8

PRODUCTION COSTS FOR TRENCHING USING BACKHOES (GODFREY, 1982)

Trench Depth (feet)	Hoe Bucket Size (cubic yards)	$/cu.yd.
8	0.5	2.92
10	0.75	2.34
12	1	1.95
14	1.25	1.88
16	2	1.57
18	2.5	1.53
20	3.5	1.27

TABLE 6-9

THEORETICAL HOURLY PRODUCTION RATE
OF A DRAGLINE EXCAVATOR (EPA, 1976)

Type of soil	Bucket size (cubic yards)						
	1	1.5	2	2.5	3	3.5	4
Moist loam, sandy clay	130 (6.6)	180 (7.4)	220 (8.0)	250 (8.5)	290 (9.0)	325 (9.5)	385 (10.0)
Sand and gravel	130 (6.6)	175 (7.4)	210 (8.0)	245 (8.5)	280 (9.0)	315 (9.5)	375 (10.0)
Common earth	110 (8.0)	160 (9.0)	190 (9.9)	220 (10.5)	250 (11.0)	280 (11.5)	375 (10.0)
Clay, hard dense	90 (9.3)	130 (10.7)	160 (11.8)	190 (12.3)	225 (12.8)	250 (13.3)	280 (12.0)

Note: Numbers in parentheses represent the maximum digging depth.

trench wall is another common technique usually applied to larger underground construction jobs. In many cases the trench is cut with walls sloped to the angle of soil stability.

6.4.1.4.1 Shoring

Shoring involves supporting the trench walls with wood or steel structures. A number of different types of shoring exist. Many construction firms use slip shields, which are also known as sliding caissons. Slipshields are constructed on-site by welding I-beams between two parallel pieces of sheet steel.

The slipshield is constructed for a specific trench width. When a section of trench is excavated, the slipshield is lowered into the trench using a crane. Pipe installation then begins. After piping is installed and

the trench is filled with gravel, the slipshield is eased out of the trench with the crane and moved to the next excavated section. Use of the slipshield is limited to the load capacity of the crane.

Adjustable aluminum bracing can also be used for trench wall stabilization. Bracing is available for trenches up to 3 feet wide at depths from 5 to 10 feet and spaced on 4 foot centers. Costs for rented aluminum bracing has been estimated to be $1.95 to $2.75 per linear foot of trench (Richardson, 1980). Aluminum bracing is much more economical than using wooden braces, which for the same type excavation would cost about $2.50 to $3.25 per linear foot of trench.

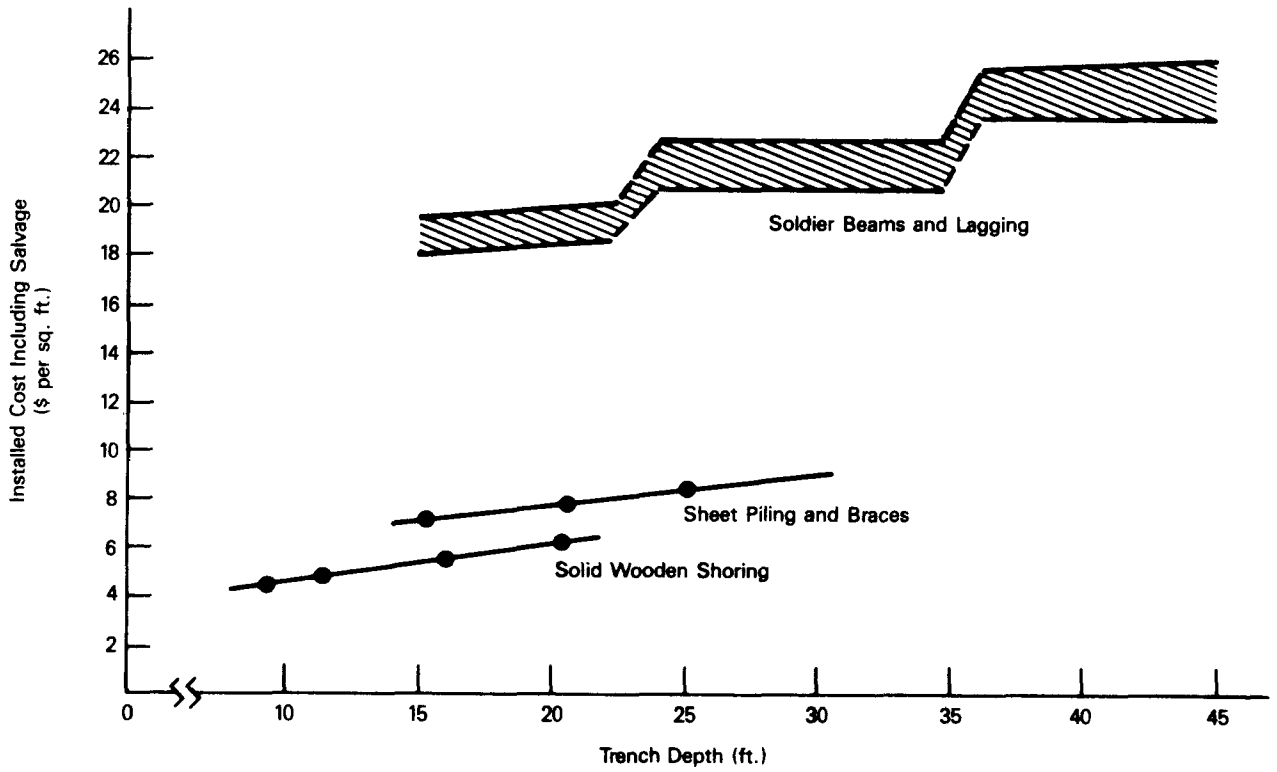
For deeper trenching, other methods of shoring are available. Steel sheet piling can be driven and braced to support the trench walls. Also, steel H piles can be driven with horizontal wooden beams inserted between them.

Solid wooden shoring is also used for deeper trenches. Wooden shoring consists of vertical wooden planks supported by cross pieces called wales, which are braced across the trench opening. Costs for the above methods are summarized in Figure 6-27.

6.4.1.4.2 Ground Freezing

Ground freezing is a method of wall stabilization and groundwater cutoff using the technique of refrigeration. The method has been practiced for a number of years by some specialty firms, but has not been applied widely to earth or rock excavation (Church, 1981). Groundwater freezing involves installing a series of vertical refrigeration pipes into the ground next to the area of trenching. Heat energy is removed from the soil via recirculating brine in the refrigeration pipes until the ground freezes. Granular soils achieve excellent strength only a few degrees below the freezing point. Clay soils may require substantially lower temperatures (down to -20°F) to attain the required strength. Although very expensive, ground freezing can also serve as a groundwater cut-off wall. The combination of both these functions

FIGURE 6-27.
COSTS OF SHORING FOR DEEP TRENCHES
(GODFREY, 1981)



may make it a suitable technique for trenchwork depths of 25 feet and greater (Freezwall, Inc., 1980).

6.4.1.4.3 Open Cuts

In shallow trenches, trenches may possibly be dug with sloped walls so that a stable angle is attained and shoring is not required. This is called an open cut. The application is of course limited to shallow trenches in most areas, since the width of the trench at the surface will usually be limited by other construction. Railroad and highway design standards specify a 1.5 (horizontal) to 1 (vertical) slope for most conditions and a 2 to 1 slope for very soft soils. Open cuts require that more earth is excavated, so costs for

excavation and backfilling will be higher than for a vertical wall trench. However, there may be an overall savings because of the lack of shoring, which is generally a more expensive endeavor.

6.4.1.5 Dewatering

Construction of a subsurface drain to intercept or divert groundwater requires excavation to a depth below that of the water table. Maintaining proper grade, placement, and alignment of drainage pipe are operations in subsurface drain installation which are best carried out in a "dry" environment. Three basic options are available for dewatering the subsurface:

- Open pumping
- Predrainage using wellpoints or well systems
- Groundwater cutoff.

These techniques may be used separately or in combination to accomplish dewatering. Open pumping and predrainage require the pumping of water from the trench excavation. In the case of installing a subsurface drain downgradient of a leachate plume, there is a strong likelihood that these dewatering operations will be handling contaminated groundwater. Treatment of contaminated groundwater is required for those dewatering technologies requiring pumping. This may include pumping or trucking to an existing wastewater treatment plant, using on-site treatment equipment, reinjecting the contaminated groundwater into the waste site, or installing and operating the permanent on-site treatment plant which will treat the total flow when the drainage system is operational. Selection of the proper treatment scheme depends on many factors, including the size of the excavation, the volume of contaminated pumpage, the duration of excavation, the concentrations and kinds of contaminants, and the costs for the various alternatives.

6.4.1.5.1 Open Pumping

Open pumping is the direct removal of water that has seeped into the excavation. This method is the least expensive of the dewatering technologies. Open pumping requires the construction of a sump hole or pit at the lowest point of the excavation, so that water can flow towards and collect in the pit. A sump pump, which may be one of several types, pumps the accumulated water from the sump out of the excavation.

Open pumping is applicable to shallow trench excavations with stable soils of low hydraulic conductivity where groundwater seepage into the excavation is minimal (Powers, 1981). There is a possibility of side wall slumping in open dewatering, therefore, the method is most applicable in excavations near open, undeveloped areas. Open dewatering is used frequently with groundwater cutoff techniques. Open dewatering should not be used in areas where soils are unstable, such as uniform granular soils without plastic fines or soft granular silts and clays because of the danger of slope instability. The flow of water in an excavation can also cause erosion, which may limit the use of open dewatering. Conditions of high flow caused by materials with moderate to high hydraulic conductivities or a large head also limit the use of open dewatering as a viable method (Powers, 1981).

As mentioned above, there are two basic components in an open pumping system--the sump hole and the pump. The sump consists of a pit dug several feet below subgrade that is large enough to provide ample settling time to remove sediments. A submersible pump or inlet line from a surface pump is placed in the pit about a foot above the pit bottom. The pit is surrounded by a large diameter corrugated steel pipe which has been perforated. The sump area around the corrugated steel piping is packed with gravel or crushed rock to provide protection against erosion and to trap sediments that can damage the pump. Cleaning and maintenance of sumps is an ongoing chore during excavation. In large trench excavations, there may be multiple sumps, each handling a portion of the total flow.

Three types of pumps are available for removing water from a sump: centrifugal suction, centrifugal submersible, and diaphragm pumps. Centrifugal suction pumps are used most commonly for removing large volumes of water, such as during springtime, after flooding, and after shutdown operations. Specifications for various sizes of centrifugal suction pumps are given in Table 6-10.

Centrifugal submersible pumps operate underwater and are used commonly in sump holes. Specifications for various size submersible pumps are shown in Table 6-11. Diaphragm pumps have the ability to handle many materials ranging from clean water to water containing muds, sands, small rocks, and miscellaneous trash. They are used commonly in the most difficult dewatering operations because of their ability to handle a wide range of materials. Specifications for diaphragm pumps are given in Table 6-12. Costs for sump pit construction and pumping are given in Table 6-13.

6.4.1.5.2 Predrainage Using Well Points or Wells

Well points and deep wells can be used to lower the water table near a trench excavation. Well points are one of the most widely used and most versatile dewatering technologies. The use of well points and deep wells and associated costs are discussed in Chapter 5.

6.4.1.5.3 Groundwater Cutoff

Groundwater cut-off barriers are used to reduce groundwater flow into an excavation. Open pumping is generally used in conjunction with cutoff barriers to handle the low rate of seepage through the barrier. Cutoff barriers may consist of steel sheet piling, concrete, or a bentonite slurry. Frequently these techniques are used as leachate control technologies in themselves as discussed in Chapter 7. Ground freezing is also considered a groundwater cutoff technique, and although expensive, may be a viable technique considering it serves the dual purpose of shoring and acting as a groundwater cutoff.

TABLE 6-10

TYPICAL CHARACTERISTICS OF SELF-PRIMING CENTRIFUGAL PUMPS
(DELIVERY IN GALLONS PER MINUTE ACCORDING
TO TOTAL HEAD; CHURCH, 1981)

Total Head Including Friction	Height of Pump Above water	
	10 ft	20 ft
Model 4-M, 1 1/2 in		
15 ft	67	
20 ft	66	
25 ft	65	47
30 ft	63	47
40 ft	54	44
50 ft	37	34
55 ft	25	25
Model 10M, 2 in		
25 ft	166	
30 ft	164	115
40 ft	157	113
50 ft	145	107
60 ft	122	97
70 ft	85	75
Model 15-M, 3 in		
30 ft	250	170
40 ft	230	165
50 ft	200	155
60 ft	160	138
70 ft	110	100
Model 20-M, 3 in		
30 ft	333	235
40 ft	310	230
50 ft	275	220
60 ft	220	195
70 ft	160	155
80 ft	90	90
Model 30-M, 4 in light		
30 ft	500	350
40 ft	495	345
50 ft	475	340
60 ft	450	325
70 ft	415	300
80 ft	355	270
90 ft	250	215
100 ft	100	100

Total Head Including Friction	Height of Pump Above Water	
	10ft	20 ft
Model 40-M, 4 in heavy		
25 ft	665	
30 ft	660	475
40 ft	645	465
50 ft	620	455
60 ft	585	435
70 ft	535	410
80 ft	465	365
90 ft	375	300
100 ft	250	195
110 ft	65	50
Model 90-M, 6 in		
25 ft	1500	
30 ft	1480	1050
40 ft	1430	1020
50 ft	1350	970
60 ft	1225	900
70 ft	1050	775
80 ft	800	600
90 ft	450	265
100 ft	100	100
Model 125-M, 8 in		
25 ft	2100	1570
30 ft	2060	1560
40 ft	1960	1520
50 ft	1800	1450
60 ft	1650	1360
70 ft	1460	1250
80 ft	1250	1110
90 ft	1020	940
100 ft	800	710
110 ft	570	500

NOTE: Shaft or brake horsepower must be taken from manufacturer's performance curves or calculated by the equation:

$$hp = \frac{gal/min \times 8.34 \times head}{efficiency \times 33{,}000}$$

TABLE 6-11

REPRESENTATIVE SPECIFICATIONS AND PERFORMANCES OF CENTRIFUGAL SUBMERSIBLE PUMPS (CHURCH, 1981)

Specification	Nominal Discharge Diameter (inches)			
	2	4	6	8
Pump:				
Discharge, diameter, in	2	4	6	8
Impeller, diameter, in	11	23	8	28
Speed, r/min	3450	1750	1750	1750
Motor, electric, AC, 60 cycle:				
Horsepower	2	25	60	95
Speed, r/min	3450	1750	1750	1750
Phase	Single	Three	Three	Three
Voltage	115-230	230-460	230-460	230-460
Complete machine:				
Cable length, ft	50	50	50	50
Strainer:				
Open area, in^2	38	48	48	48
Openings size, in	1/4	3/8	1	1
Dimensions, in:				
Height	23	46	57	57
Diameter	16	32	39	39
Weight, approximate, lb	50	150	450	800
Performance in gal/min for total head of:				
20 ft	130	930	2160	2800
40 ft	100	870	1950	2720
60 ft	70	780	1720	2570
80 ft	...	680	1480	2370
100 ft	...	550	1100	2120
120 ft	...	360	400	1800

TABLE 6-12

REPRESENTATIVE SPECIFICATIONS AND PERFORMANCES OF DIAPHRAGM PUMPS (CHURCH, 1981)

Specification	Nominal Discharge Diameter (inches)		
	2	3	4
Pump, single-acting:			
Strokes per minute	60	60	52
Length of stroke, in	2 1/2	2 13/16	3 3/4
Usable diameter of diaphragm, in	7	11 1/2	12 3/4
Displacement, in^3	71	221	381
Prime mover:			
Diesel or gasoline engine, hp	3	6	10
Electric motor, as 60-cycle:			
Horsepower	1/2	1 1/2	3
Speed, r/min	1750	1750	1750
Phase and voltage	115-230	115-230	115-230
Complete machine:			
Weight, wheels-tires-mount, lb	230	350	500
Diesel engine	150	200	400
Gasoline engine	150	200	350
Performances, gal/min versus total head:			
5-ft suction head with discharge head of:			
5 ft	26	78	125
15 ft	22	66	113
25 ft	18	64	104
15-ft suction head with discharge head of:			
5 ft	21	68	97
15 ft	19	64	74
25 ft	17	60	70
25-ft suction head with discharge head of:			
5 ft	18	56	82
15 ft	16	60	66
25 ft	15	56	52

TABLE 6-13

COST FOR OPEN PUMPING (GODFREY, 1981)

Item	Unit Cost
Sump hole construction, includes excavation, and gravel	$0.80/cu ft
Corrugated pipe collar (installed)	
12" pipe	$18/ft
15"	$22/ft
18"	$25/ft
24"	$35/ft
Pumping 8 hr., attended 2 hrs. per day, including 20 feet of suction, hose, and 100 feet discharge hose (rental)	
2" diaphragm	$77/day
4" diaphragm	$85/day
Pumping 8 hr., attended 8 hrs. per day (rental)	
2" diaphragm	$310/day
4" diaphragm	$340/day
3" centrifugal	$315/day
6" centrifugal	$380/day
Automatic Submersible sump pumps (centrifugal)	
Bronze	
1-1/4 inch, 26 GPM at 8 foot head	$180/ea
1-1/4 inch, 45 GPM at 8 foot head	$190/ea
1-1/4 inch, 63 GPM at 8 foot head	$280/ea
1-1/4 inch, 87 GPM at 8 foot head	$375/ea
Cast Iron	
1-1/4 inch, 26 GPM at 8 foot head	$115/ea
1-1/4 inch, 45 GPM at 8 foot head	$250/ea
1-1/4 inch, 63 GPM at 8 foot head	$280/ea
Cast iron diaphragm pumps (starter & level control included)	
2" discharge	
10 GPM at 20 foot head	$370/ea
60 GPM at 20 foot head	$460/ea
120 GPM at 20 foot head	$850/ea
160 GPM at 20 foot head	$1450/ea
3" discharge	
220 GPM at 20 foot head	$1950/ea

6.4.1.6 Grade Control

Proper grade in a subsurface drain ensures against ponding of water and provides for a non-silting velocity in the drainage pipe. Specifications for grading were discussed previously. The control of grade during trench excavation is important so that the grade of the as-built drainage system is as close as possible to the design grade. Poor grade control can result in siltation or the need for extensive backfilling and leveling, which drives up costs.

There are two categories of grade control that are presently used with mechanical trench excavators--visual and automatic laser control. The visual method is known as target grade control. In target control, grade stakes (of equal length) are driven to the design subgrade along every 100 feet of the trench line. A line drawn through the tops of the grade stakes would be parallel to the design grade of the trench. Targets are driven next to the grade stakes and are adjusted to a fixed distance above the elevation of the grade stakes. The selection of this distance depends on the depth of the trench and the line of sight between the machine operator and a reference sighting rod on the machine. When the trenching machine is cutting on grade, the targets will align with the reference sighting rod.

Accuracies of plus or minus 0.1 foot are easily obtainable with the target method. If the depth of the trenches is also checked with respect to the design grade at each target point, the target grade control method can attain an accuracy of within plus or minus 0.02 foot of the design grade (Taylor and Willardson, 1971).

Target grade control enables the early detection of errors in setting grade stakes or targets because any misalignment will be apparent. Changes in grading can be accomplished quickly by resetting targets. Target grade control always proceeds trench excavation and even with modifications, does not hinder the progress of conventional trench excavation machines.

Target grading has a few disadvantages. The precision and accuracy of grade control depends on the machine operators skills and alertness in checking grades constantly to ensure proper control. Fog and other visual obstructions may require that targets be placed closer together. Large numbers of targets are difficult to handle and they will tend to have very limited lifespans because of breakage and accidental burial.

An alternative to the target control system is the use of automatic grade control with laser beams. Many conventional trenchers (including some backhoes) are equipped with this control system. The basic system consists of a portable, tripod-mounted, low-power laser beam projector and a machine-mounted electronic tracker-receiver (Van Schilfgaarde, 1974). The laser beam projector emits a beam set to an elevation or grading datum, or both. The receiver-tracker adjusts the trenching unit hydraulically to automatically control depth and trench grade. A number of commercial systems are available, some eliminating the need to ever reset the laser projector, thus enabling change of grade at any point along the trench excavation.

Costs for automatic grade control vary widely. A laser beam sending unit with tripod mounting costs approximately $8,500 (Lazer Plane Corp., 1982). A machine-mounted receiver for a backhoe, which simply indicates depth, is estimated to cost approximately $3,500 (Lazer Plane Corp., 1982). The most sophisticated receiver-tracker for a continuous trencher, enabling grade change at any point during excavation, without resetting the sender would cost from $7,500 to $11,500 (Lazer Plane Corp., 1982).

6.4.2 Drain Installation

Once trench excavation is completed, the components of the subsurface drain can be installed. The process consists of the installation of drain pipe bedding, the drainage pipe or bed, the gravel or soil envelope, the filter fabric, the backfill, and any auxiliary components. Modified continuous trenching machines exist which accomplish all excavation and drain installation operations simultaneously. However, simultaneous excavation/installation machinery is limited to small diameter drains in which the pipe

can be inserted into an automatic pipe feeder and can be carried along on rolls or wagons. Larger diameter pipe drains require separate operations. Installation of the various components of a subsurface drain are discussed below.

6.4.2.1 Drain Pipe Bedding

The bedding for a subsurface drain depends somewhat on the type of drainage pipe to be installed and the expected loading on the pipe from the backfill. The simplest bedding material used is gravel or crushed rock. Laying a gravel bed simply consists of bringing the trench to design grade by excavation and backfilling and then spreading an even layer of gravel over the trench floor. The pipe is set by placing additional gravel around the pipe. In the case of partial dewatering of an unstable trench floor, the trench is often over excavated and backfilled with a larger sized stabilizing gravel, as specified in Table 6-14. This enables proper drainage to the sump without soil loss and instability. The bedding material can then be laid on top of the stabilized floor.

TABLE 6-14

SPECIFICATIONS FOR STABILIZING GRAVEL

Gradation of stabilizing material	Percent
Retained on 5-inch screen	0
Retained on 4-inch screen	0 to 20
Retained on 3-inch screen	0 to 30
Retained on 2-inch screen	20 to 50
Retained on 3/4-inch screen	20 to 50
Passing No. 4 sieve	Less than 8

Gravel beddings are used with larger diameter drainage pipes of perforated concrete, vitrified clay, and rigid plastic construction. For alignment purposes, trench floors are often cut in a semicircle so that the gravel fill forms a cradle for the pipe. Costs for gravel are approximately $8.60 to $10.10 per cubic yard (Godfrey, 1981).

In tile drainage systems where proper alignment is crucial, the trench floor is brought up to grade with soil compacted to its original density. A small V-notch is then excavated along the trench bottom to serve as an alignment guide for laying the tile. Soil must be removed around bell-and-spigot joints for proper alignment. Bedding for corrugated flexible pipe consists of digging a semi-circular groove instead of a V-notch to prevent pipe collapse during backfilling. A groove may not be required if backfilling is very well controlled, such as on continuous excavation and installation machines. As mentioned earlier, bedding may be part of a continuous excavate, install drain and backfill procedure or it may be conducted separately from the other operations.

When unstable soils are encountered (oftentimes because of dewatering problems), timber cradles may be used to ensure proper grade and alignment. Timber cradles consist of wooden supports on which the pipe is laid. These cradles are supported by piles or long stakes driven into a solid base. This is an expensive operation since each pile or stake must be checked to be sure it is aligned, driven into a solid base, and cutoff at the proper grade. Timber cradles are used mostly for concrete and clay pipe. Concrete pipe cradles are also used for these purposes.

6.4.2.2 Installation of Drain Pipe

Installation procedures for drain pipe vary widely according to the type, diameter, and length of pipe being used. Pipe installation is usually initiated at the lowest trench elevation and proceeds upgrade. Flexible plastic pipe can be installed from rolls mounted on a trenching machine and continuously fed into the trench as the excavation proceeds. Flexible pipe

can also be fed by hand from rolls mounted on wagons. Lengths of pipe can be joined at the surface between rolls.

The installation of flexible plastic pipe has some inherent disadvantages. Stretching the pipe will cause a loss of structural strength and possible collapse. Stretching should be guarded against, especially in periods of hot weather. Some drain contractors use sun shields over their pipe rolls in hot weather work. Brittleness and difficulty in unbending are problems that are encountered in cold weather. Also, flexible plastic pipe tends to float and partial dewatering may present problems.

The installation of rigid pipe can not be done automatically. Laying tile pipe can be a semi-automatic operation in which a worker feeds tile into a tile chute which properly aligns, spaces, and holds each tile until the backfill is added. Where bell-and-spigot type joints are used, the bell end should always be installed upgrade. When tiling manually, care must be taken to ensure proper spacing between tiles. A tile should always be held by hand or other means until the next tile is ready to be placed. Cracked or broken tile should be discarded (Van Schifgaarde, 1974). If a tile is warped, the tightest fit should be made on the top. Tile should be cut to maintain even spacing when working around curves.

Long lengths of rigid pipe are either hand carried or lowered by crane into the trench for installation depending on their size. Perforated pipes of PVC, polyethlene, concrete, bituminous fiber, and vitrified clay commonly come in long lengths.

When extending drainage systems under roadways, structures, root zones, or areas not requiring drainage, unperforated pipe (or in the case of tile, cemented joints) should be specified.

Installed costs for various types and sizes of pipe, excluding excavation and backfilling, are given in Table 6-15.

TABLE 6-15

INSTALLATION COSTS FOR DRAINAGE PIPE
(GODFREY, 1981)

Item	$ Per Linear Foot
Asbestos Cement, class 4000 underdrain, perforated	
4" diameter	3.30
6" diameter	4.33
8" diameter	6.40
10" diameter	7.25
12" diameter	9.60
Unperforated, 4" diameter	2.57
Bituminous fiber, perforated underdrain	
3" diameter	2.10
4" diameter	2.32
5" diameter	3.37
6" diameter	3.94
Corrugated steel or aluminum perforated, asphalt coated	
6" diameter, 18 ga.	4.16
8" diameter, 16 ga.	5.45
10" diameter, 16 ga.	6.90
12" diameter, 16 ga.	8.60
18" diameter, 16 ga.	12.15
Plain steel or aluminum, corrugated	
6" diameter, 18 ga.	3.78
8" diameter, 16 ga.	5.10
10" diameter, 16 ga.	6.50
12" diameter, 16 ga.	8.15
18" diameter, 16 ga.	11.35
Porous wall concrete underdrain, std. strength	
4" diameter	3.17
6" diameter	3.35
8" diameter	4.80
12" diameter	7.70
15" diameter	9.15
18" diameter	12.30

(Continued)

TABLE 6-15 (continued)

Item	$ Per Linear Foot
Porous wall concrete underdrain, std. strength (continued)	
Extra strength	
6" diameter	3.57
8" diameter	4.96
10" diameter	7.90
12" diameter	9.10
15" diameter	10.35
18" diameter	14.00
Vitrified clay, perforated, 2' lengths (C-211)	
4" diameter	3.09
6" diameter	4.45
8" diameter	7.10
12" diameter	10.10
Vitrified clay sewer pipe, premium joint (C-200)	
4' and 5' lengths, 4" diameter	4.10
4' and 5' lengths, 6" diameter	5.60
8" diameter	7.20
10" diameter	9.45
12" diameter	12.25
15" diameter	19.75
18" diameter	26
24" diameter	55
30" diameter	73
36" diameter	110
3' lengths, add 30% to above	
2' lengths, add 60% to above	
PVC, perforated underdrain	
3"	$0.64[a]
4"	$0.82[a]
6"	$1.85[a]
Flexible PVC, perforated, 4"	$0.29[b]

[a] Material cost only; from Plastic Piping Systems, 1982.
[b] Material cost only; from L.C. Smith, Inc., 1982.

6.4.2.3 Gravel or Soil Envelopes

Gravel envelopes are installed around the pipe drain to increase flow into the drain and reduce the buildup of sediments in the drain line. Gravel envelopes may be placed by hand, backhoe, or by a hopper cart or truck. In continuous trencher-drain installation machines, gravel filling may be ongoing along with other operations.

When placing the gravel envelope, the gravel must be in complete contact with the pipe. Specifications for gravel envelopes around clay pipe call for a minimum of 12 inches of gravel envelope above the pipe for structural support. The horizontal thickness of envelopes varies according to design considerations, as discussed previously.

When fine grained soils are encountered, a dual layered aggregate gravel envelope can be installed. To install the dual layered envelope, fine aggregate is placed to form the bottom of the outer envelope. A narrow strip of coarse aggregate is then placed to form the bottom of the inner envelope. After the drainage pipe is installed, the sides and tops of the two layer system are then completed.

Costs for placing gravel fill are $8.60 and $10.10 per cubic ~~foot~~ yard, for bank run and screened gravel, respectively (Godfrey, 1981).

If the existing soil hydraulic conductivity is sufficient, the existing soil may be backfilled as the envelope material. This practice is commonly used in agricultural drainage, however, a greater degree of control is desired for drainage at waste disposal sites. Therefore, for leachate plume control, envelope materials should consist of a hydraulic conductivity significantly higher than the surrounding strata.

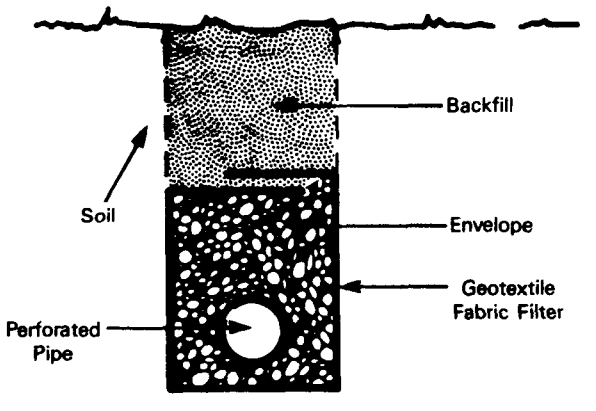
6.4.2.4 Filter Fabrics

Filter fabrics are sometimes installed around the gravel envelope to prevent fines from clogging the envelope and drain pipe. Fabrics function by

creating a graded soil filter against the fabric. Non-woven filter fabrics can be used successfully in sand and gravel base materials containing little or no silt or clay, i.e., up to 1 percent (Wolbert-Master, Inc., 1982). When soils contain substantial percentages of silt or clay, the non-woven fabric eventually forms a filter cake and prevents the groundwater from entering the envelope. Woven fabrics can be used in strata with significant concentrations of silt, i.e., 5 to 10 percent (Wolbert-Master, 1982).

When constructing a drain using a fabric filter wrapping, the fabric is installed first, followed by the bedding, the pipe, and the envelope in that order. The fabric filter is then wrapped around the top of the envelope prior to backfilling with soil. A schematic of an installed pipe drain with filter fabric is shown in Figure 6-28. Fabric filters can be installed manually or by machine.

FIGURE 6-28.
PIPE DRAIN WITH FILTER FABRIC



The costs for installing geotextile fabrics vary somewhat depending on the difficulty of installation. Material costs are typically $0.75 to $0.85 per square yard of fabric for non-woven, spun bound synthetics, and about

$1.60 to $2.70 for the heavier, traditional woven-type fabric (Wolbert-Master, Inc., 1982). Costs for simple installations are typically $0.10 to $0.15 per square yard of material. In deep trenches, where installation is more difficult, rolls of material may have to be sewn together and installation costs have been known to range from $2.00 to $5.00 per square yard (Wolbert-Master, Inc., 1982).

6.4.2.5 Backfilling

After the gravel envelope has been installed, the trench must be backfilled to the original grade. Almost any type of excavation equipment can be used to backfill trenches. In addition to the previously mentioned excavation equipment, bulldozers, bucket loaders, scrapers, or front end loaders can be used. Many times a backhoe combined with a front end loader will be used on the job site, with excavation being done by the hoe end (i.e., the back end) and loading or backfilling being done by the front end. Backfilling is done simultaneously in some continuous trenchers. The use of a geotextile fabric on the top of the envelope may be advisable to prevent siltation of the envelope from the backfill material. In order to prevent settling of the backfill after construction, periodic compaction of soil lifts is also required. This may be accomplished using air tamping, or a vibrating or sheepsfoot compactor. An additional compaction technique developed for agricultural drainage is called puddling. Puddling involves inundating a soil lift with water to cause compaction. Dams are built across the trench every 200 feet to provide suitable conditions for puddling (Van Schilfgaarde, 1979). Puddling may not be advisable because the water applied may contact contaminated water or soil near the trench and thus require treatment.

The top two feet of trench should be backfilled with topsoil (if originally present) to reestablish the ambient vegetation. Estimated costs and daily outputs for backfilling by dozer with and without compaction is given in Table 6-16.

TABLE 6-16

ESTIMATED COSTS AND OUTPUTS FOR BACKFILLING BY DOZER
(Godfrey, 1981)

Item	Daily Output (yd^3)	Cost/yd^3
Dozer backfilling in trench, up to 300 foot haul, no compaction	825	$1.09
Air tamped	235	$4.92
Compacted Backfill, 6" to 12" lifts, vibrating roller	460	$2.11
Sheepsfoot roller	410	$2.39

6.4.2.6 Manholes and Wet Wells

Manholes are used in subsurface drainage systems to serve as junction boxes between drains, silt and sand traps, observation wells, and access points for pipe location, inspection, and maintenance. Manholes should be located at junction points, changes in alignment or grade, and other designated points. There is not a set criteria for manhole spacing.

A manhole should extend from 12 to 24 inches above the ground surface for ease of location. The base of the manhole should be a minimum of 18 inches below the lowest pipe to provide a trap for sediments. Manholes are typically designed to have a drop in elevation between the inlet and outlet pipes, to compensate for head losses in the manhole (Bureau of Reclamation, 1978). A typical manhole design is shown in Figure 6-29.

A manhole is not required for the upper end of a drainage line, however, the end should be plugged to prevent debris from entering. Riser pipes may be put at the upper end of a drainage pipe for cleanout purposes. These should be installed at an angle enabling access for pipe cleaning equipment. Costs for the installation of concrete manholes is given in Table 6-17.

FIGURE 6-29.
TYPICAL MANHOLE DESIGN FOR A CLOSED DRAIN
(BUREAU OF RECLAMATION, 1978)

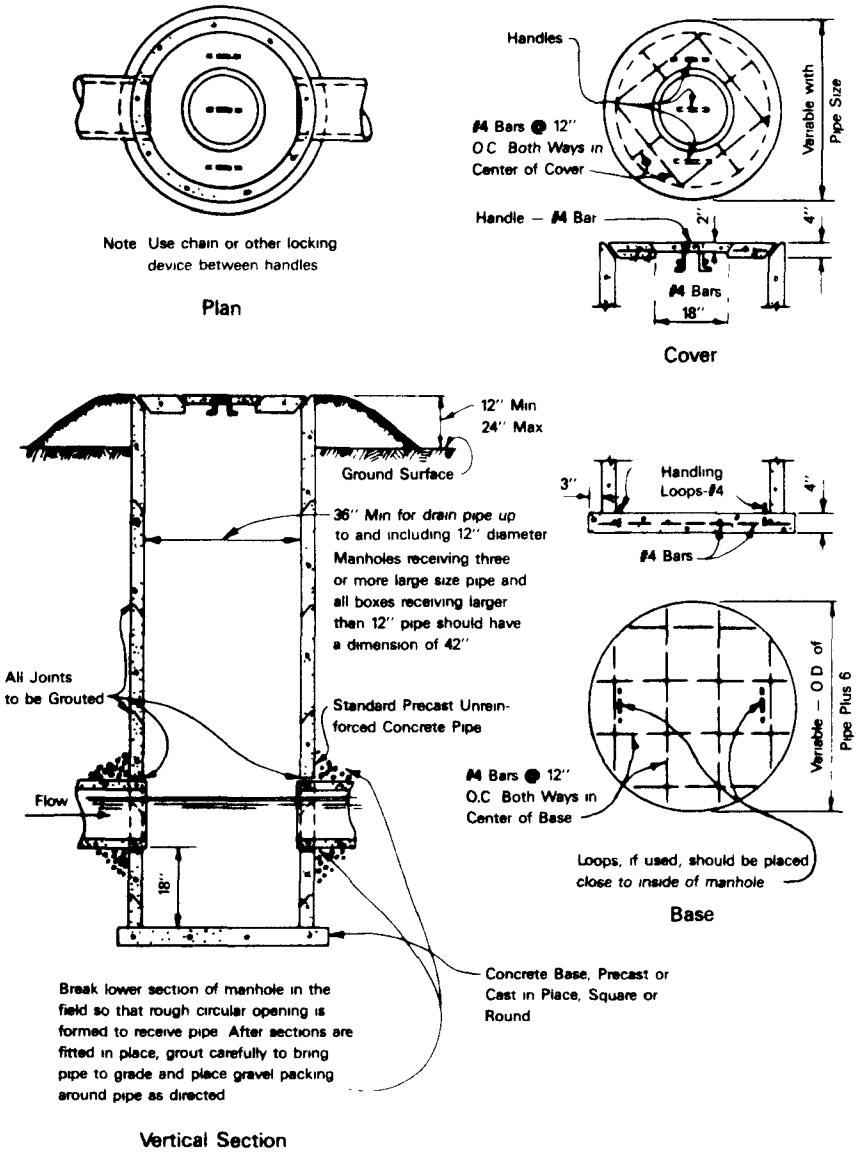


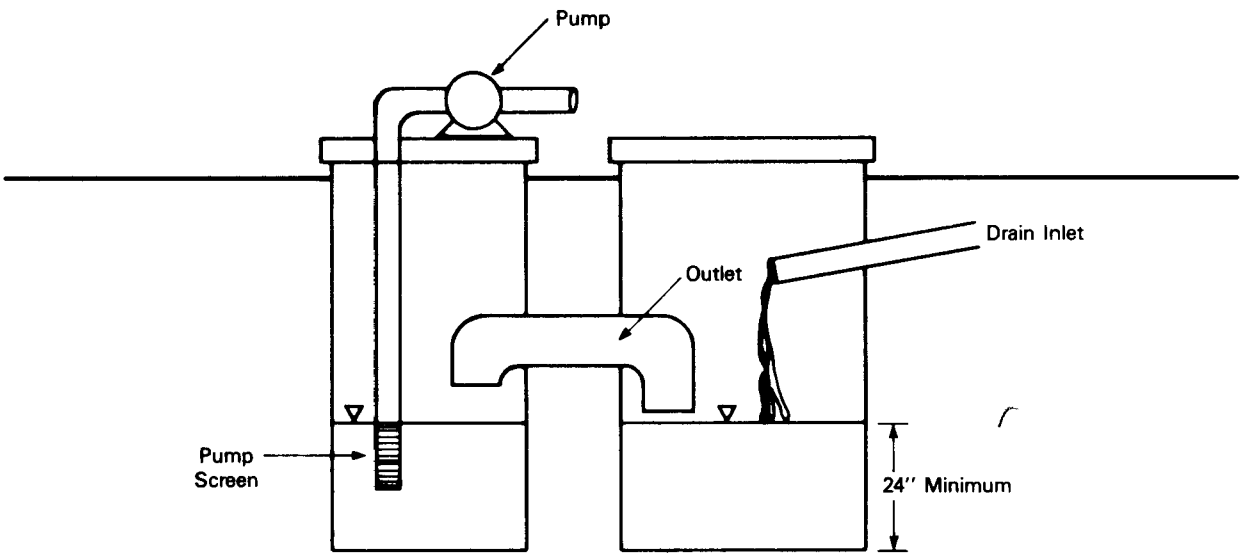
TABLE 6-17

INSTALLED COSTS FOR MANHOLES
(Godfrey, 1981)

Item	Cost ($)
Concrete slab, cast in place, 8" thick,	
6' deep	630
8' deep	865
12' deep	1,345
16' deep	1,825
20' deep	2,305
Precast concrete riser pipe,	
4' inside diameter,	
6' deep	510
8' deep	690
12' deep	781
16' deep	872
20' deep	963
6' inside diameter,	
6' deep	1,075
8' deep	1,475
12' deep	1,665
16' deep	1,855
20' deep	2,045
Slab tops, precast, 8" thick,	
4' diameter	122
5' diameter	136
6' diameter	170
Frames and covers,	
watertight,	
24" diameter	310
32" diameter	405
light traffic,	
24" diameter	185
36" diameter	355

Manholes are also used as wetwells and as silt traps in conjunction with a wetwell, so that pumps can be protected from abrasive sediments. A typical design of a combined sediment trap and wet well for pumping collected groundwater to the surface is shown in Figure 6-30.

FIGURE 6-30.
TYPICAL DESIGN OF SEDIMENT TRAP AND WETWELL



6.4.3 Inspection and Maintenance

Proper and frequent inspection of the subsurface drain during construction will often uncover potential problems that might otherwise remain undetected for years. The following items should be checked during construction:

- o Quality of tile, tubing, pipe and other materials
- o Alignment, depth, and grade of drain and bedding, (although minor variations are acceptable, there should never be a reverse grade)

- Trench width at the top of the drain
- Spacing of tile joints and integrity of other connections
- Bedding, filter, and envelope materials and installation
- Backfilling and compaction
- Auxiliary structures including manholes, cleanout riser pipes, sediment traps and wet wells, and pumps and piping.

After all components of the drain are in place the drain should be tested for obstructions. This can be done visually by shining a high powered flashlight through a drain from one manhole and observing the beam in another. If an unobstructed view cannot be obtained by this method because of an actual obstruction, slight misalignment, or great distance, an air filled rubber ball or plug about one inch less than the drain diameter should be flushed or pulled through the drain (Bureau of Reclamation, 1978). Only a moderate amount of water pressure or pulling force should be used so as not to deform the ball. Any obstruction which causes the floating ball to stop or which significantly increases the pulling force (greater than 25 lbs; Van Schilfgaarde, 1979) will justify digging up and replacing the pipe, removing the obstruction, or taking some other corrective action. In large diameter drains, TV-camera inspection can be used after construction and periodically during operation.

Manholes and silt traps should be checked frequently for the first year or two of operation for sediment buildup. Less frequent inspection is required as the system ages.

Piezometers may be installed in the various parts of the drainage system to identify operational problems with the filter, envelope, pipe, or other components of the system. Piezometers can measure the loss of head through a medium, and thus, can identify obstructions to flow, such as a clogged envelope or filter.

There are a number of problems that can develop in drainage systems that will require maintenance or corrective action, including:

- Clogging by sediment build-up in manholes or drains
- Root clogging of tile drains
- Clogging by build-up of chemical compounds
- Sinkholes developing in the soil above the drain.

Problems caused by the above conditions are usually apparent at the surface above the drain. Inspection of the area will reveal soft or ponded surface conditions, areas of subsidence, and areas of accelerated vegetative growth.

Sediments that enter the system will usually be scoured through the drain to the manhole where they collect. If manholes are not cleaned out periodically, sediments can build-up to the point of clogging the drains. Drains can also clog because of breaking or partial collapse of pipe, a partial obstruction, or an adverse change in alignment caused by settling in a poorly compacted soil or inadequately installed bedding.

Subsurface drainage pipes can also be clogged by plant roots, particularly trees and shrubs. The roots of a single plant may extend over a distance of several meters in a drainage pipe, given the right conditions. Cottonwoods and willows are particularly troublesome.

The build-up of chemical compounds may also occur in a drain pipe. Iron and manganese can form insoluble precipitates with sulfur via microbial oxidation. If drains are located in areas with high deposits of reduced iron and manganese and the flow to the drains is sufficiently constant for a long period of time, these compounds can build-up to the point of clogging the drain.

Sinkholes, or blowouts, may also develop over the drain. These may be caused by improper spacing of tile drains, pipe breakage, insufficient cover material, or high pressures within the drain.

Clogged drainage pipe can be corrected by either hydraulic jetting or mechanically scraping the drain. Flushing methods use high pressure water jetting equipment that can travel inside the drain pipe. The water jets can remove sediment and chemical compounds from the walls as well as from perforation in the drainage pipe. Specially designed water jets can cut through dead roots up to one-half inch in diameter (Winger, 1979). Some water jetting apparatus also contain special, closed-cage, rotating root cutters for cutting larger roots. Cleaning speed by water jetting is about 1,000 feet per hour, with few roots, to 800 feet per hour, with a moderate number of roots, (Winger, 1979).

Mechanical scraping removes deposits from the pipe wall using such tools as augers, pull or push blades, brushes, and hollow pipe bailers. Costs for mechanical and hydraulic pipe cleaning were about $2.00 per foot in 1980 (USEPA, 1982).

Chemical means are often used for removing iron and manganese deposits where hydraulic or mechanical methods are ineffective. Usually, sulfur dioxide gas or a solution composed mainly of sulfuric acid is used to soften the deposit to the point where hydraulic jetting can be used. Non-corrosive sulfuric acid pellets have recently been developed which prevent the initial formation of iron and manganese compounds when added to the gravel envelope.

In cases where there is a structural problem, such as drain breakage or improper drain spacing causing a sinkhole, the drain must be dug up and the condition corrected. Malfunctioning perforated pipe drains located near root systems should be dug up and replaced with non-perforated pipe (or in the case of tiles, sections of cemented joints).

CHAPTER 7

LOW PERMEABILITY BARRIERS

Low permeability barriers can be used to divert groundwater flow away from a waste disposal site or to contain groundwater contaminated by a waste site. There are two major types of low permeability barriers which find application in leachate plume control--slurry walls and grout curtains. A slurry wall is a subsurface barrier that is formed through the excavation of a trench using a high density slurry, typically bentonite and water, to support the sides. The trench is then backfilled with materials having lower permeability than the surrounding soil. This backfilled trench or slurry wall reduces or redirects the flow of groundwater. A grout curtain is a subsurface barrier formed through the pressure injection of one of a variety of special fluids known as grouts into a rock or soil body to seal and strengthen it. Once in place, these fluids set or gel in the rock or soil voids to greatly reduce the permeability of and impart increased mechanical strength to the grouted mass. Because a grout curtain can be three times as costly as a slurry wall and is incapable of achieving uniformly low permeability, it is rarely used when groundwater has to be controlled in loose overburden. Grouts are primarily used to seal voids in porous or fractured rock when other methods of controlling groundwater flow are impractical.

Impeded groundwater flow may cause an increase in the upgradient hydraulic head, which may in turn affect the rate of vertical movement of the water. Therefore, the probable effects of a locally heightened water table should be carefully considered before deciding to apply a barrier wall. Also, unless measures are taken to reduce infiltration, a site surrounded by a barrier wall could easily fill up in the so-called "bathtub effect."

Several types and variations of slurry wall and grout implacement methods can be used to control or reverse leachate plume migration. The following sections describe the applications and limitations, the theory, the design and construction, and the cost of slurry walls and grouts.

7.1 Slurry Walls

Slurry walls are formed through a slurry trench construction process. Slurry trenching is a means of placing a low permeability, subsurface cut-off or wall near a polluting waste source in order to capture or contain leachate plumes. A trench is excavated through or under a slurry of bentonite clay and water. The trench is then backfilled with an engineered mixture of earth materials having the desired permeability properties. The width of slurry trenches vary, but are typically from two to five feet (D'Appolonia, 1979).

Slurry trenches are usually excavated down to and often into a natural low permeability layer (called an aquiclude) in order to shut off groundwater flow. However, when only lowering the water table is required, the trench may not have to be keyed into an impervious layer.

7.1.1 Applications and Limitations

Slurry walls are classified by the materials of which they are composed and by the position in which they are placed with respect to the pollution source. Slurry wall material and configuration determine the applications and limitations of different slurry wall alternatives for leachate plume management.

There are three major types of slurry walls that are categorized according to the material used to backfill the trench. These three types are:

- Soil-bentonite
- Cement-bentonite
- Diaphragms.

Soil-bentonite walls are composed of soil materials (often the trench spoils) mixed with small amounts of the bentonite slurry. Cement-bentonite cut-off walls are excavated using a slurry of Portland cement and bentonite which is left to set or harden to form the final wall. Diaphragm walls are composed of pre-cast or cast-in-place reinforced concrete panels (diaphragms) installed using slurry trenching techniques. Each of these, as well as hybrids of the three, has different characteristics and applications.

In general, soil-bentonite walls can be expected to have the lowest permeability, the widest range of waste compatibilities, and the lowest installation cost. They also offer the least structural strength (highest compressibility), usually require the largest work area, and are restricted to sites that can be graded nearly level.

Cement-bentonite walls can be installed at sites where there is insufficient work area to mix and place soil-bentonite backfill. Also, by allowing wall sections to harden and then continuing the wall at a higher or lower elevation, they are more adaptable to an extreme topography. Although cement-bentonite walls are stronger than soil-bentonite walls, they are at least an order of magnitude more permeable, resistant to fewer chemicals, and more costly.

Diaphragm walls are the strongest of the three types as well as the most costly. Provided the joints between panels are installed correctly, diaphragm walls have lower permeability and about the same chemical compatibilities as cement-bentonite walls. Because of the higher expense of diaphragm walls and the rarity of a need for a high strength barrier, they are seldom used for pollution control, thus they will not be treated in detail here.

Combinations of these three major backfill types may be included within the same wall. For example, a soil-bentonite backfill may be used for the majority of a wall with cement-bentonite being used for a portion, such as a road or rail crossing that requires greater strength. Being able to combine

the various types of walls makes this technique adaptable to a wider range of site characteristics.

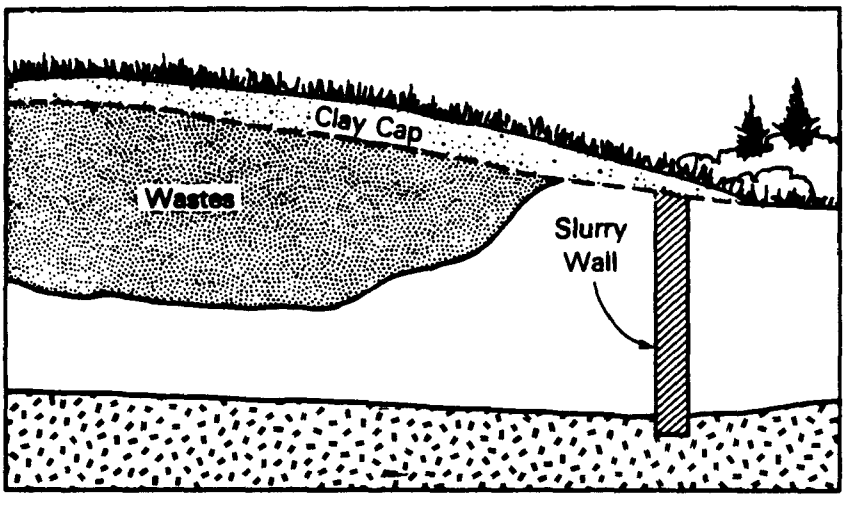
The vertical and horizontal positioning of a slurry wall with respect to the location of the pollution source and groundwater flow characteristics is known as configuration. There are two types of configurations--vertical and horizontal. Wall configuration, combined with associated remedial measures, determine in theory how effective a slurry wall will be in controlling leachate migration. Although configuration depends greatly on geologic and topographic setting, waste characteristics, and the nature of the environmental problems caused by the site, generalizations on the applications of different slurry walls can be useful in understanding and evaluating a slurry wall as a part of a remedial action.

7.1.1.1 Vertical Configuration

Vertical configuration refers to the depth of the slurry wall with respect to both geologic formations and the water table. Based on vertical positioning, walls are either "keyed" into a low permeability formation below an aquifer or placed to intercept only the upper portion of the aquifer. This latter type is commonly referred to as a "hanging" slurry wall.

Keyed-in slurry walls are excavated to a confining layer below, to contain contaminants that mix with groundwater or sink to the bottom of the aquifer. This layer may be a low permeability formation such as a clay or silty clay or may be the underlying bedrock (Figure 7-1). In either case, the connection between the wall and the low permeability zone is critical to the overall effectiveness of the wall. If the low permeability zone is some easily excavated material, such as a clay layer or weathered rock, basic construction quality control should be sufficient to ensure a good key-in. In cases were the low permeability zone is hard bedrock, however, the excavation process may be much more complicated and costly.

FIGURE 7-1.
HANGING SLURRY WALL (SPOONER et al., 1984)

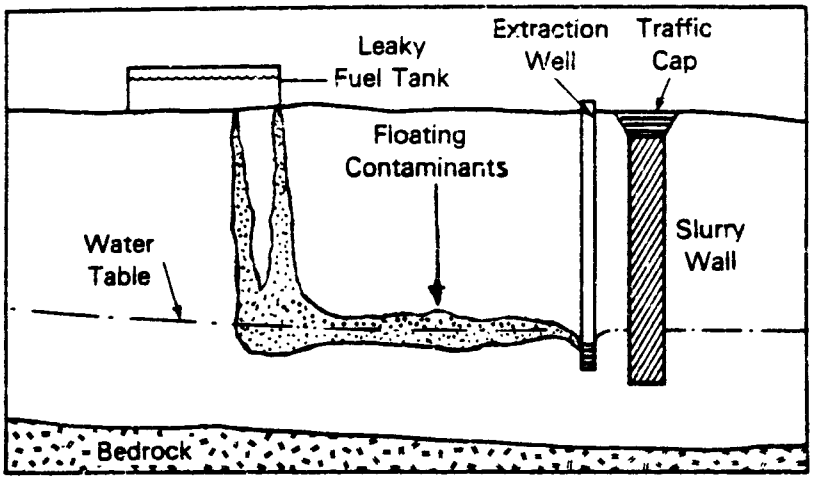


Hanging slurry walls are not keyed into a low permeability layer. This configuration is used to control contaminants, such as petroleum products, which do not mix with the groundwater but float on top of it. In such cases, the slurry wall need only extend into the water table to intercept the contaminants (Figure 7-2). The exact depth of the wall will depend on the thickness of the floating contaminant layer and the lowest water table elevation. Other considerations include the extent to which the weight of the contaminant might have depressed the water table and the effect removal of the contaminants would have on the water table.

7.1.1.2 Horizontal Configuration

Horizontal configuration refers to the positioning of the wall relative to the location of the pollution to be controlled and the direction of groundwater flow. Depending on desired horizontal configuration, slurry walls may

FIGURE 7-2.
HANGING SLURRY WALL (SPOONER et al., 1984)



completely surround the polluted area or be placed upgradient or downgradient from it.

Circumferential placement refers to placing a slurry wall completely around a contaminated area (Figure 7-3). Although this requires a greater wall length than either upgradient or downgradient placement alone, it does offer many advantages and is a common practice. A circumferential slurry wall, when used with a surface infiltration barrier (cap), can greatly reduce both leachate movement and generation. If a leachate collection system is used, the surrounded area can be virtually dewatered. This offers the advantages of vastly reduced leachate amounts and can help increase the longevity of the wall by reducing the amount of leachate and wall contact. As shown in Figure 7-4, groundwater levels can be adjusted to maintain the direction of groundwater flow inward, and thus, prevent the escape of contaminants.

FIGURE 7-3.
PLAN OF CIRCUMFERENTIAL WALL PLACEMENT
(SPOONER et al., 1984)

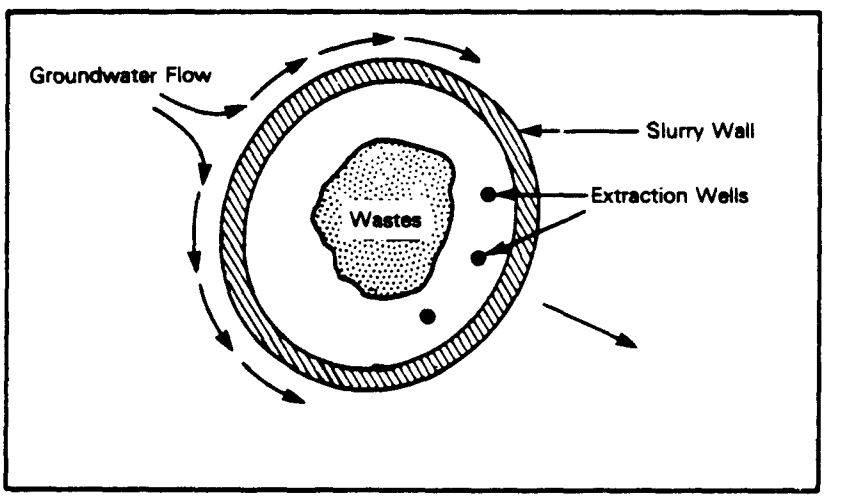
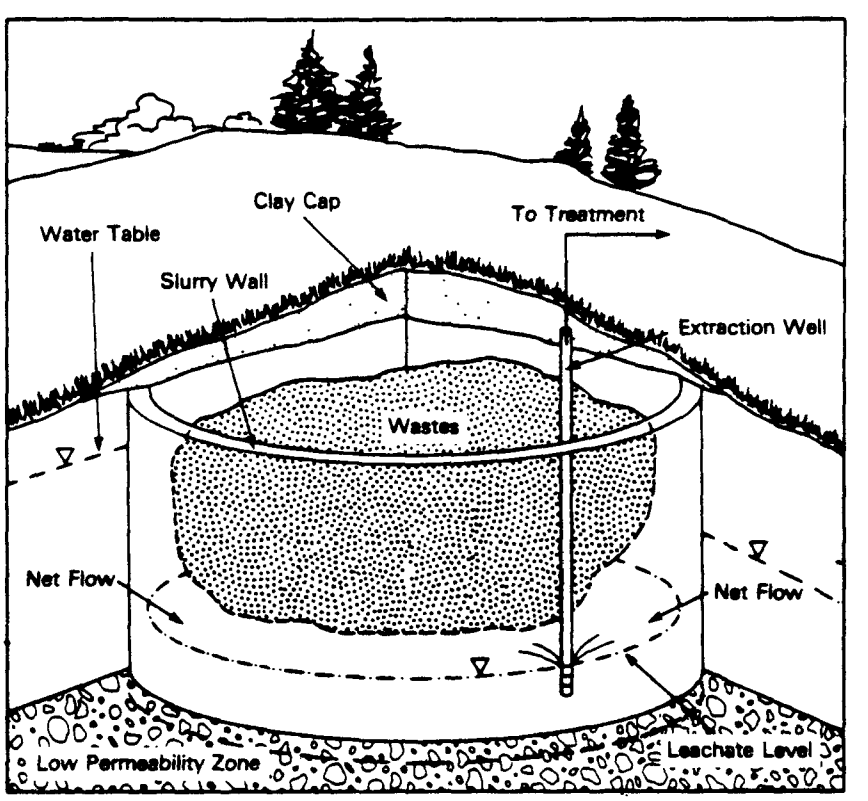


FIGURE 7-4.
CUT-AWAY CROSS-SECTION OF CIRCUMFERENTIAL
WALL PLACEMENT (SPOONER et al., 1984)



Upgradient placement refers to the positioning of a wall on the groundwater source side of a leachate source. This type of placement can be used where there is a relatively steep gradient across the site such that groundwater flow can be diverted around the wastes. In such cases, leachate generation is reduced. As can be seen by Figures 7-5 and 7-6, a high water table gradient is required for upgradient placement to be effective. Depending on the site setting and the contaminants involved, an upgradient wall may be keyed-in or left hanging. In either case, drainage and diversion structures are likely to be needed to successfully alter the flow of clean groundwater. The use of upgradient barrier walls is not common.

Placement of a slurry wall at a site on the side opposite the groundwater source is referred to as downgradient placement. This placement configuration does nothing to limit the amount of groundwater entering the site and is practical only in situations where there is a limited amount of groundwater flow, such as near drainage divides (Figures 7-7 and 7-8). This positioning does not reduce the amount of leachate being generated, but acts as a barrier to contain the leachate so it can be recovered. Although this placement may be used as a keyed-in wall for miscible or sinking contaminants, it is most often used to contain and recover floating contaminants. In either case, the compatibility between the leachate and the wall backfill is important because of contact between the two. In addition, care must be taken in designing a downgradient wall installation to ensure that contaminated groundwater does not overtop the wall.

As outlined above, slurry walls can be applied to a plume migration problem in several ways. The possible combinations of configurations and their typical uses are summarized in Table 7-1.

7.1.2 Theory

During construction of slurry walls, a slurry containing bentonite is placed in the open trench to support the trench walls. After excavation is completed, a mixture of bentonite slurry and soil, or a mixture of cement, bentonite, and water is placed in the trench to form the completed wall. This

FIGURE 7-5.
PLAN OF UPGRADIENT PLACEMENT WITH DRAIN
(SPOONER et al., 1984)

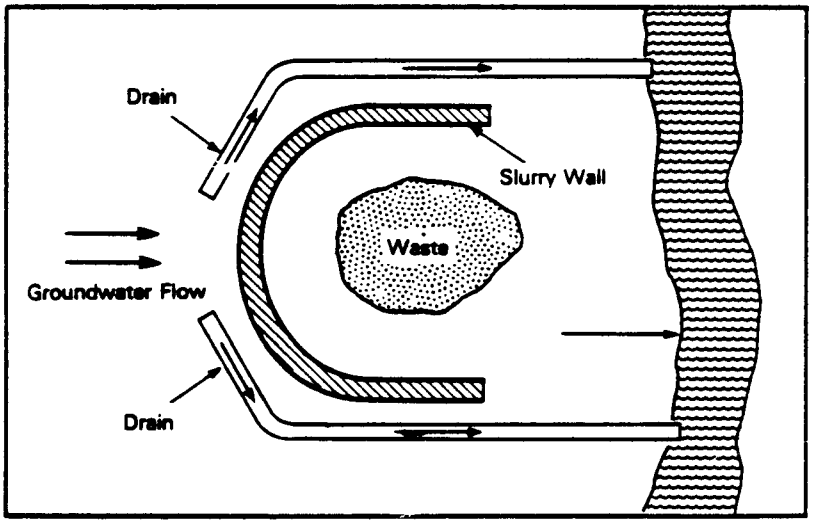


FIGURE 7-6.
CUT-AWAY CROSS-SECTION OF UPGRADIENT
PLACEMENT WITH DRAIN (SPOONER et al., 1984)

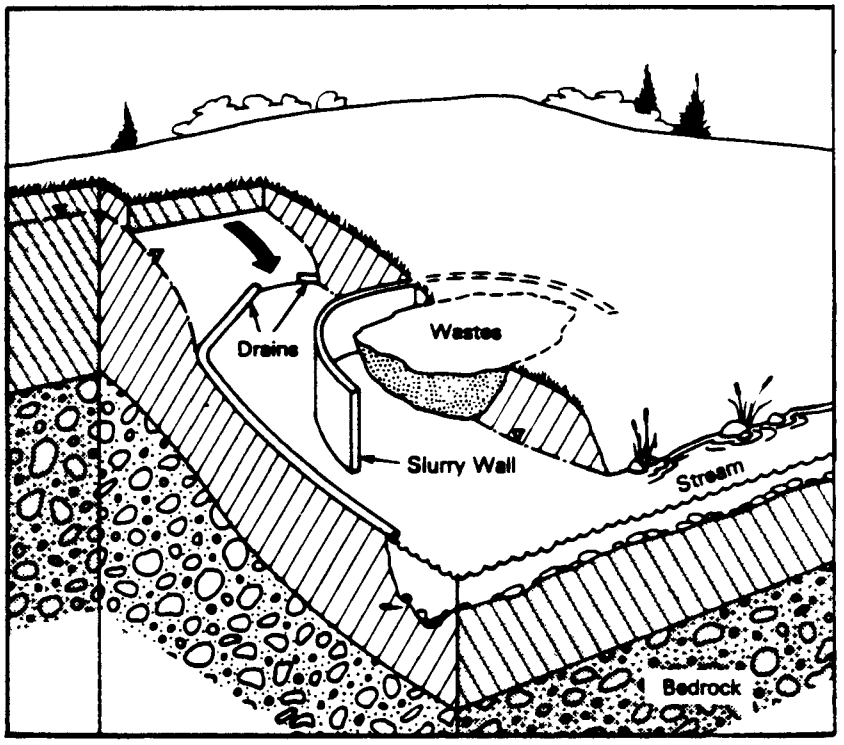


FIGURE 7-7.
PLAN OF DOWNGRADIENT PLACEMENT
(SPOONER et al., 1984)

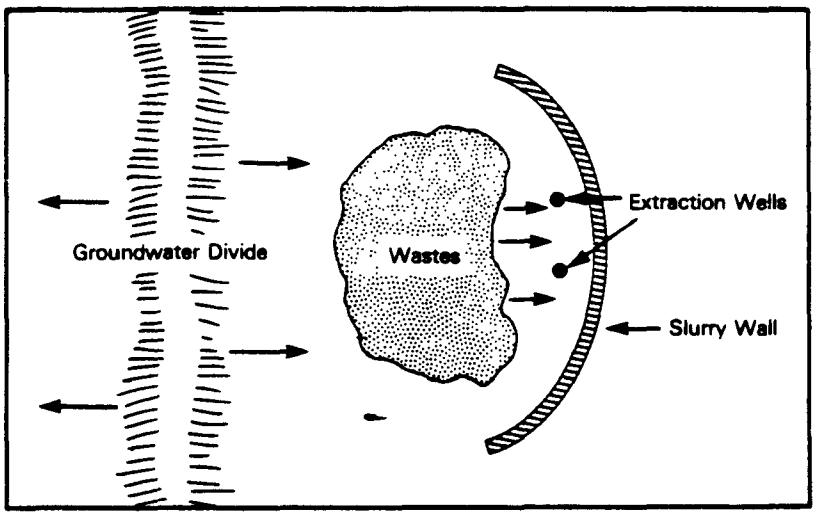


FIGURE 7-8.
CUT-AWAY CROSS-SECTION OF DOWNGRADIENT
PLACEMENT (SPOONER et al., 1984)

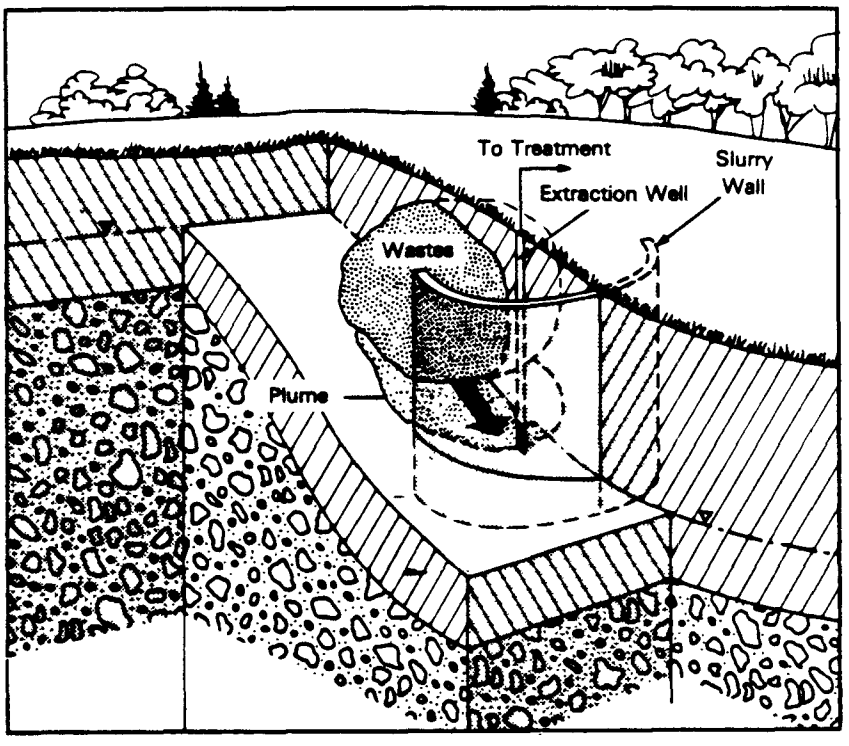


TABLE 7-1. SUMMARY OF SLURRY WALL CONFIGURATIONS (Spooner et al., 1984)

Vertical Configuration	Horizontal Configuration		
	Circumferential	Upgradient	Downgradient
Keyed-in	Most common and expensive use	Not common	Used to capture miscible or sinking contaminants for treatment
	Most complete containment	Used to divert groundwater around site in high gradient situations	Inflow not restricted, may raise water table
	Vastly reduced leachate generation	Can reduce leachate generation but not movement	Compatibility very important
		Compatibility not critical	
Hanging	Used for floating contaminants moving in more than one direction (such as on a groundwater divide)	Not common	Used to capture floating contaminants for treatment
		May temporarily lower water table behind it	Inflow not restricted, may raise water table
		Can stagnate leachate but not halt flow	Compatibility very important

section presents the current-held theories on the mechanisms by which the slurry and the backfill perform this function.

7.1.2.1 Slurry Materials and Functions

To properly construct a barrier wall for pollution migration control, a high quality slurry is necessary which:

- Has a high viscosity when allowed to stand and a low viscosity when agitated (termed thixotrophy)
- Forms a low permeability layer called a filter cake on the trench sides
- Has a density that is lower than that of the backfill.

Slurry specifications, along with ranges normally encountered during construction, are presented in Table 7-2.

7.1.2.1.1 Clay

Both soil-bentonite and cement-bentonite slurries rely on bentonite clay to maintain the required slurry properties. Bentonite is a soft, soapy-feeling deposit composed primarily of the clay mineral montmorillonite and about 10 percent impurities, such as iron oxides and native sediments (Boyes, 1975). Montmorillonite's structure and chemical composition give bentonite its unique properties.

Crystals of montmorillonite are composed of three distinct layers. The outer layers consist of a tetrahedral arrangement of silicon and oxygen molecules. Some of the silicon atoms in these layers have been replaced by aluminum. Sandwiched between the silica layers is a layer of aluminum atoms surrounded by six hydroxyls or oxygen atoms in an octahedral configuration. Some of the aluminum atoms in this layer have been replaced by magnesium. Because of the substitutions in the three layers, unsatisfied bonds exist within the crystal, resulting in a net negative charge. To satisfy this charge, cations and water molecules are adsorbed onto the internal and external surfaces of the clay crystals. These surfaces comprise the exchange

TABLE 7-2. SPECIFIED PROPERTIES OF BENTONITE AND CEMENT BENTONITE SLURRIES (Spooner et al., 1984)

Parameter	Bentonite Slurry		Cement-Bentonite Slurry	
	Fresh-Hydrated	During Excavation	Fresh-Hydrated	During Excavation
Density (g/cm^3)	1.01-1.04	1.10-1.24	1.03 to 1.4	
(p.s.f.)	65	69-84		
Viscosity, apparent				
(Seconds Marsh)	38-45	38-68	40-45	38-80
(centipoise)	15		15	>130
Viscosity, plastic	<20*		9	30-50
Filtrate Loss, ml	<30 range 15-30	range 15-70 apparent average 40-60	100-300	
pH	7.5 to 12	10.5-12	12-13	
Fluid Content, % of Total Wt.	93-97	78-82	76	55-70
Bentonite Content, %	4-7	6	3-6	3-6
Other Ingredients, % by weight	sand <1* solids 2	sand <5* solids 3-16	cement 18 solids 15-30	--- 30-45
Gel Strengths				
10 seconds, Pascals			15	10
10 minutes, Pascals	7-30	20-40	18	22
10 minutes, $lb/100\ ft^2$ ($24-72\ dynes/cm^2$)	5-15	---	---	---

*Specification for construction of tremied concrete diaphragm walls.

Adapted from Case, 1982; Xanthakos, 1979; Millet and Perez, 1981; U.S. Army Corps of Engineers, 1976; Guertin and McTigue, 1982b; Boyes, 1975; Jefferis, 1981a; Ryan, 1976

complex of the clay. The types of cations adsorbed on the exchange complex have a great influence on the properties of the clay (Brady, 1974).

There are two major types of montmorillonites; sodium-saturated and calcium-saturated. Sodium montmorillonites are preferable to calcium montmorillonites because sodium montmorillonites swell more and flocculate less than calcium montmorillonites when exposed to water. Not all of the bentonites used for slurry trench construction, however, consist chiefly of sodium montmorillonites because there are limited quantities of natural sodium bentonites. In some cases, calcium bentonites are used after being exposed to sodium-containing materials such as sodium hydroxide to force some of the calcium ions off of the exchange complex of the montmorillonite and then replace them with sodium ions (Grim, 1968).

Where high concentrations of calcium salts occur in the soil or groundwater and where cement-bentonite is used, calcium ions can easily displace sodium ions on the clay. The ease of replacement of sodium by calcium increases as the concentration of calcium in the solution and on the clay exchange site increases. Once about 30 percent of the exchange sites become occupied by calcium, the clay acts like calcium montmorillonite rather than the sodium variety (Grim, 1968).

Clays other than montmorillonite have occasionally been used in slurries to maintain trench stability, particularly in areas where bentonite is difficult to obtain. Table 7-3 lists several properties of montmorillonite as compared to other common clay minerals. The primary drawback to the use of other clays in the slurry during trench construction is the inability of these clays to form a filter cake under relatively low hydraulic heads and short formation times.

An exception to this general rule concerns situations where a high concentration of salts in the slurry trench construction area is anticipated. When sodium bentonite is exposed to salt water, undesirable property changes occur; the slurry begins to flocculate, viscosity decreases, fluid loss

TABLE 7-3. COMPARISON OF SELECTED PROPERTIES OF CLAYS (Spooner et al., 1984)

Parameter	Montmorillonite	Kaolinite	Illite	Other Clays or Sheet Silicates
Volume change caused by hydration	2-11 cm^3/g			vermiculite >montmorillonite >beidellite >kaolinite >halloysite
Hydration rate	Water sorption continues for about 1 week	--	--	water sorption for most colloidal clays is complete in 1 to 3 days
Particle shape	thin, flat, irregular plates	irregular, flat hexagonals	--	attapulgite-fibrous, sepiolite-fibrous, most others irregular and flat
Theoretical specific surface area, m^2/g	700-800	5-20	100-200	vermiculite 300-500
Cation Exchange Capacity, meg/100g	60-150	3-15	10-40	vermiculite 100-150, attapulgite/sepiolite 3-15
Liquid limit	150-700	29-75	59-90	--
Plastic limit	65-97	26-35	34-43	--

Continued

Adapted from Baver, Gardner and Gardner, 1972; Grim, 1968; Grim and Guven, 1978; Xanthakos, 1979; Spangler and Handy, 1973

TABLE 7-3 (continued)

Parameter	Montmorillonite	Kaolinite	Illite	Other Clays or Sheet Silicates
Percentage of clay by weight in water to produce a 15 centipoise colloidal suspension	5.5-12	--	--	25-36 for typical native clays attapulgite-same as montimorillonite
Approximate negative charge per formula unit	0.5-1.3	0	1.2-2.0	vermiculite 1.2-1.8, muscovite 2.0
Density of charge $meq/m^2 \times 10^{-3}$	1.1-1.9	6-7.5	1.0-2.0	vermiculite 3.0-3.3
Layer thickness in angstroms	expansive >10 air dry 15	7.15	10	vermiculite 14, muscovite 10, biotite 10, halloysite 10,
Particle density, g/cm^2	2.5 Wyoming bentonite 2.2 Japanese bentonite	--	--	mica 2.8-3.2

Adapted from Baver, Gardner and Gardner, 1972; Grim, 1968; Grim and Guven, 1978; Xanthakos, 1979; Spangler and Handy, 1973

increases, and the slurry may gel completely and lose its thixotropic properties (Xanthakos, 1979). In these situations, a type of clay called attapulgite has been used.

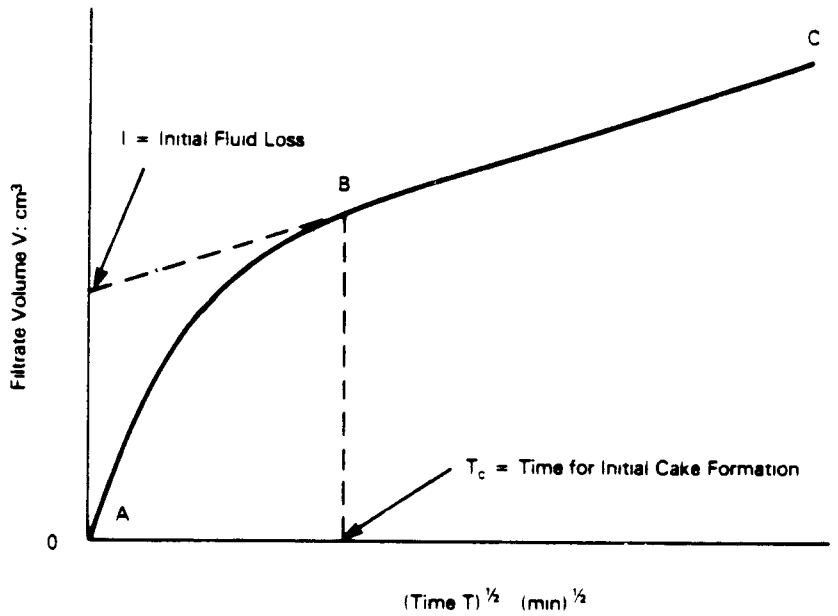
Attapulgite is not composed of plates as is montmorillonite. Instead, the attapulgite crystals are composed of linked silica chains that form a fibrous structure; chains of water molecules fill voids between the tetrahedral and octahedral silica layers (Grim, 1968). Attapulgite is apparently not negatively affected by the electrolyte concentration in the water to which it is added and does not require any initial hydration time as does montmorillonite.

Wyoming bentonites are the most commonly used bentonites for slurry trench construction. They contain about 60 percent sodium and 40 percent calcium and magnesium cations on the exchange complex of the clay (Connybear, 1982). These bentonites are mixed at a rate of 4 to 7 percent bentonite in 93 to 96 percent water (Boyes, 1975).

7.1.2.1.2 Filter Cake

When trench excavation is initiated, the slurry is pumped into the trench to maintain a slurry level at or near the initial ground level (Millet and Perez, 1981). As the slurry is introduced into the trench, it flows into pores in the strata through which the trench is cut. Leakage of slurry into these voids continues until the flat clay particles in the slurry begin to accumulate in layers, which grow large enough to bridge the gaps between the soil particles or until gelation of the slurry within the pores occurs. Figure 7-9 illustrates the relatively rapid initial slurry loss, followed by a reduction in the rate of loss. This reduction is caused by the formation of a layer of clay particles "plastered" on the trench sides, which reduces lateral liquid flow out of the trench and into the adjacent soil. This layer is called a filter cake.

FIGURE 7-9.
FLUID LOSS DURING FILTER CAKE FORMATION
(HUTCHINSON et al., 1983 AS CITED BY SPOONER et al., 1984)



The filter cake is a thin membrane composed of plate-like bentonite particles (Case, 1982). The solids content of the newly formed filter cake ranges from 10 to 50 percent, with higher solids contents found in filter cakes from calcium than from sodium bentonites (Grim and Guven, 1978). The filter cake from a slurry containing 5 percent bentonite typically contains 15 percent bentonite (Hutchinson et al., 1975). During active excavation, the filter cake is usually less than three millimeters thick. This thick layer of clay is, however, an effective barrier to water movement as the permeability of the filter cake can be as low as 10^{-6} ft/day (Xanthakos, 1979).

Formation of the filter cake is of critical importance in slurry trench construction. This membrane performs numerous functions including:

- Minimization of slurry loss into surrounding soils
- Stabilization of the soil that is in contact with the slurry by gelling in the soil pores and by plastering the particles against the trench walls

- Providing a plane on each trench wall against which the hydraulic pressure and dead weight of the slurry can act to stabilize the excavation.

Desirable characteristics of filter cakes include rapid formation and reformation when necessary, resistance to shearing, and low permeability. Experience has shown that a thin filter cake is an indication of a tough, impermeable membrane, but a thick "flabby" filter cake is likely to allow high fluid losses (Boyes, 1975).

Filter cakes must be formed rapidly when the initial soil contact occurs in order to avoid excessive slurry losses. These membranes must resist mechanical disruption by the backhoe bucket or clamshell during trench excavation. If inadvertently scraped off the trench wall during excavation, the slurry must be of such composition as to allow rapid formation of a new filter cake to avoid possible collapse of the trench.

A high gel strength is desirable in the filter cake because the gel structure contributes to shear strength, and the filter cake must resist shearing forces from both the excavation equipment agitated slurry and the soil comprising the trench wall. In addition, high gel strength indicates rapid formation of a gel structure. When the slurry penetrates soil pores, rapid gelation assists in restricting further slurry flow and thus minimizes slurry losses (Xanthakos, 1979).

A final desirable filter cake characteristic is low permeability. The movement of water through the filter cake should be minimized to:

- Avoid wetting and thus softening and lubricating unstable layers that may be present in the soil surrounding the trench (Boyes, 1975)
- Avoid increasing pore water pressure, because this increases the total stress on the system and reduces the angle of friction in the soil surrounding the trench (Hutchinson et al., 1975)
- Maintain the slurry level in the trench well above the groundwater level. This sustains the thrust of the slurry on the trench side walls by restricting the pressure losses caused by filter cake leakage (Xanthakos, 1979).

7.1.2.1.3 Cement-Bentonite

Cement-bentonite (CB) slurries normally contain less than 6 percent by weight bentonite, 18 percent ordinary Portland cement (o.p.c.), and 76 percent water (Jefferis, 1981b). Typical ranges of CB slurry contents are presented in Table 7-4.

TABLE 7-4. TYPICAL COMPOSITIONS OF CEMENT-BENTONITE SLURRIES (Jefferis, 1981b as cited by Spooner et al., 1984)

Constituent	Amount of Slurry (Percent by weight)
Bentonite	4-7
Water	68-88
Cement	
without replacements	8-25
when blast furnace slag added, minimums	1-3
when fly ash added, minimums	2-7
Blast furnace slag, maximums (not commonly available in U.S.)	7-22
Fly ash, maximums, if used	6-18

When bentonite slurries are compared to CB slurries, the differences become evident. Most of these differences are caused in part by the effects of the calcium from the cement on the sodium montmorillonite in the bentonite. The three most important differences between the properties of CB slurries and bentonite slurries are:

- Physical properties, including viscosity and filter cake formation
- Setting times
- Filter cake permeability.

Because of the calcium in the cement, the properties of the sodium bentonite in CB slurries are permanently altered. For example, the initial viscosity is somewhat higher because of the flocculation of the slurry and the higher solids concentration; 15 to 30 percent in CB slurry, opposed to about 6 percent in bentonite slurries (Millet and Perez, 1981; Jefferis, 1981b). The results of the filtrate loss test are also higher. This indicates that the time required for filter cake formation is longer and the permeability of the cake formed is higher (Hutchinson et al., 1975; Millet and Perez, 1981).

The primary differences betwen CB and soil-bentonite (SB) slurries that is of practical importance in slurry trench construction is the fact that CB slurries begin to harden within two to three hours after mixing, while bentonite slurries do not set (Case, 1982). This often necessitates the use of construction techniques different from those used during construction of SB walls, as described below.

CB walls may be constructed in a series of panels but are more commonly installed as a continuous trench. When CB panels are constructed, alternate panels are excavated under a CB slurry, then allowed to partially set. When they have obtained a sufficient shear strength, the intervening panels are excavated also under a slurry. A portion of the initial panel ends are removed during this second stage to ensure continuity between the initial panels and the intervening ones. This technique is used only for very deep installations.

7.1.2.2 Backfill Materials and Functions

Two major types of backfill have been used in barrier walls: soil-bentonite and cement-bentonite. Soil-bentonite (SB) backfills are composed of bentonite slurry and soil materials, often the spoils excavated from the trench. Cement-bentonite (CB) backfills are the result of the in-place hardening of cement-bentonite slurries. The design factors affecting the permeability of completed cement-bentonite and soil-bentonite walls are backfill composition and slurry properties.

7.1.2.2.1 Soil-Bentonite Walls

Soil-bentonite walls are excavated under a bentonite slurry in a continuous trench. The trench is then backfilled with a soil-bentonite mixture having the desired properties.

Two primary components make up the backfill: the bentonite slurry and a suitable soil mixture. The soil mixture used in backfilling is often soil material from the trench excavation, augmented with selected soil from a suitable borrow source. Regardless of the source, the soil material must meet three requirements:

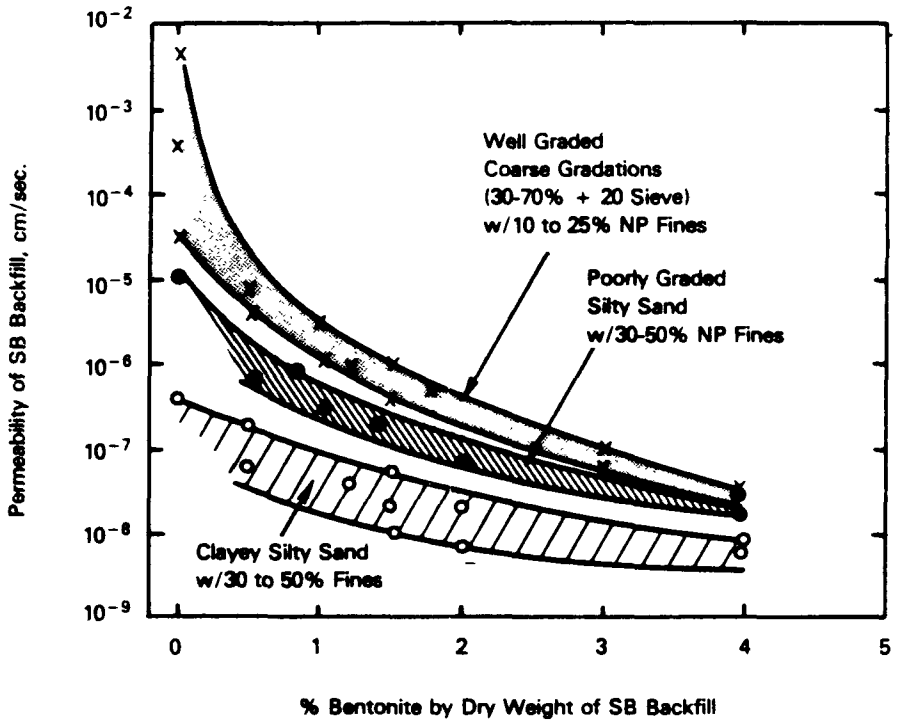
- Must be free of deleterious materials
- Must have a suitable particle size distribution
- Must mix with the slurry to form a more-or-less homogeneous paste.

The backfill should not contain materials that would react with the pollutants being contained. For example, calcium-containing materials (gypsum, chalk, or caliche) should not be used in walls intended to contain highly acidic groundwater.

Another requirement for backfill material is that it contain a suitable particle size distribution. To ensure a low permeability, the backfill must have from 20 to 40 percent fine particles, preferably plastic fines. As shown in Figure 7-10, backfill permeability will be lower when the backfill material contains a higher proportion of fines (D'Appolonia, 1980). Fine particles, particularly clays, contribute to low permeability by assisting in bridging the pores between larger particles.

A final requirement of the soil material used in the backfill is that it be mixed with the slurry to form a more-or-less homogeneous paste. This is largely a function of water content and blending technique. According to D'Appolonia (1980), this requirement is relatively easy to meet as a wide variety of materials have been successfully mixed at slurry trench construction sites. In general, clayey soils will require a greater blending effort and higher water content than more coarse materials.

FIGURE 7-10.
RELATIONSHIP BETWEEN PERMEABILITY AND QUANTITY OF BENTONITE ADDED TO SB BACKFILL (D'APPOLONIA, 1980 AS CITED BY SPOONER et al., 1984)



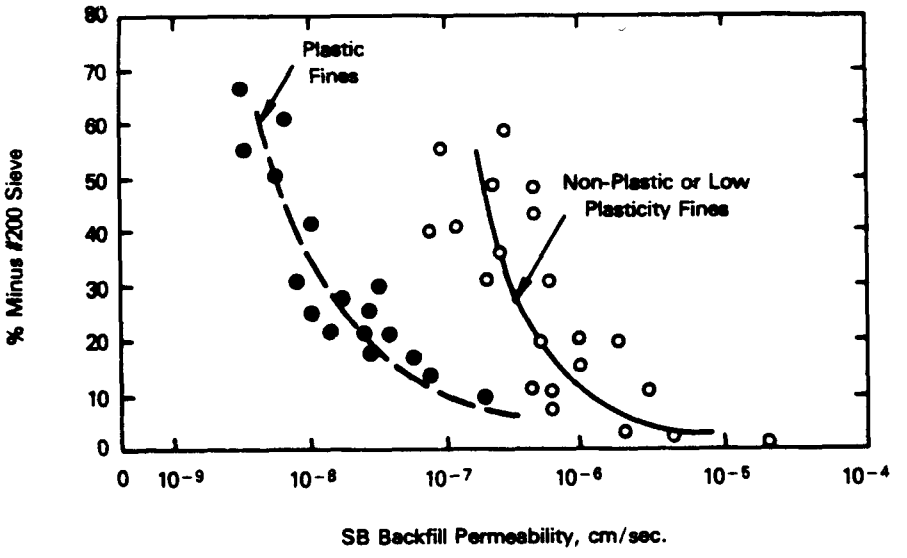
Once the backfill material has been selected, it is mixed with the slurry until the slurry attains the necessary slump. The slump of the backfill is the vertical distance a cone-shaped mass of concrete or other plastic material will settle and is a measure of the backfill's shear strength. The slump should range from two to seven inches on the ASTM C143-74 "Slump of Portland Concrete" Test (D'Appolonia, 1980). The shear strength of the backfill mixture, as indicated by the slump cone test, should be high enough that the backfill easily displaces the slurry, but not so high that it folds over itself rather than flowing into the trench. The density of the backfill should be at least 15 pounds per cubic foot (240 kilograms per cubic meter) greater than that of the slurry in the trench (D'Appolonia, 1980). Preferably, the shear strength should be lower than that of the filter cake, yet high enough to allow it to stand on a 5:1 to 10:1 slope (Millet and Perez, 1981).

The primary requirement of a slurry wall for controlling leachate plumes is low permeability. Permeabilities of completed soil-bentonite walls can be as low as 1.5×10^{-5} ft/day, although higher permeabilities are more common (Xanthakos, 1979). Several factors affect the permeability of the completed wall including:

- Quality of the filter cake
- Fines content of the backfill
- Bentonite concentration of the backfill
- Plasticity of the fines in the backfill
- Voids ratio of the backfill.

D'Appolonia (1980) found that plastic fines inhibit water movement more effectively than non-plastic fines. This is logical since the plastic fines are composed mostly of clays, which are less pervious than the non-plastic fines, which are primarily silts. Since bentonite has a very small particle size, it also contributes to reduced permeability. The effect of plastic and non-plastic fines on backfill permeability is shown in Figure 7-11.

FIGURE 7-11.
EFFECT OF PLASTIC AND NON-PLASTIC FINES CONTENT ON SOIL-BENTONITE BACKFILL PERMEABILITY (D'APPOLONIA, 1980 AS CITED BY SPOONER et al., 1984)



The void ratio of a soil material is dependent on pore size and particle size. Small particle sizes and a low void ratio (high packing density) will yield low permeability.

7.1.2.2.2 Cement-Bentonite Walls

In contrast to soil-bentonite walls, cement-bentonite walls are normally used where strength, rather than low permeability is the primary consideration (Guertin and McTigue, 1982). The requirements for cement-bentonite wall performance are the same as for soil-bentonite walls.

Cement-bentonite wall strength is designed to be slightly greater than that of the surrounding ground and is typically comparable in strength to stiff clay (Jefferis, 1981a; Millet and Perez, 1981). Although strengths of cement-bentonite walls can range from 10 to 100 pounds per square inch, ultimate strengths are generally about 5 to 20 pounds per square inch and are achieved after 90 days (Xanthakos, 1979; Case, 1982).

Cement-bentonite walls can usually withstand compressive strains of about 2 percent without cracking. This is because they are not as brittle as typical concrete walls (Ryan, 1976). Moreover, they can withstand relatively high hydraulic gradients and are basically non-erodable. A cement-bentonite wall only 2 to 3 feet wide can satisfactorily withstand at least 100 feet of hydrostatic head because of its cohesive nature (Millet and Perez, 1981). Despite the fact that cement-bentonite walls are both non-brittle and cohesive, they are not indestructable and hydrofracturing has been reported (Millet and Perez, 1981).

The continuity of cement-bentonite walls is an especially important factor when these walls are constructed in panels rather than in a continuous trench. Thus, there is a possibility for gaps to remain between the panels which can allow for leachate escape. The time required for cement-bentonite walls to harden depends on the presence of set time retarders and cement replacements, among other factors. The speed of set time is of interest when panel construction techniques are employed.

The permeability of cement-bentonite walls is normally about 10^{-3} ft/day (Case, 1982). This can be decreased slightly by adding blast furnace slag or additional bentonite. Jefferis (1981a) reports that the permeability can also be decreased as much as an order of magnitude because of consolidation of the completed wall. This, however, can be deleterious as it is accompanied by cracking.

There are many factors which affect the performance of cement-bentonite walls including:

- Slurry contents
- Mixing methods
- Use of additives
- Cement content
- Bentonite content
- Use of cement replacements.

As the cement content in the backfill is increased, a stronger, more brittle wall is formed (Millet and Perez, 1981). Higher cement contents, however, may allow higher wall permeabilities because of consolidation cracking. The ultimate strength of cement-bentonite walls is low, on the order of 15 to 20 pounds per square inch, and the permeability is relatively high, usually around 10^{-3} ft/day (Case, 1982).

The ratio of the water to cement in the slurry also affects the characteristics of cement-bentonite walls. Generally, higher ratios produce weaker walls. Typical water-cement ratios range from 3:1 to 5:1. These are much higher than the ratios found in concrete mixes. The cement is kept in suspension while the slurry is liquid because of the presence of the bentonite in the mixture. The bentonite content in the cement-bentonite slurry walls contributes to their low permeability and resistance to chemical attack. Where very low permeability or resistance to aggressive chemicals are required, the bentonite content of the slurry can be increased (Jefferis, 1981a). However, as the bentonite content is increased, the viscosity goes up, and the slurry becomes unworkable.

Cement replacements, such as blast furnace slag or fly ash, can be used to replace up to one-third of the cement in the slurry (Jefferis, 1981a). Typical percentages of these constituents in cement-bentonite slurries are shown in Table 7-4. Cement replacements have several effects on the characteristics of the completed wall. The most significant effect is the reduction in the rate at which water separates from the cement mixture before and during hardening (bleeding rate).

When conventional cement-bentonite walls or walls with fly ash replacement are allowed to set for seven days, permeabilities of 2.8×10^{-3} to 1.4×10^{-2} ft/day (1 to 5×10^{-6} cm/sec) result. In contrast, cement-bentonite walls containing from 2.5 to 7.5 percent blast furnace slag exhibited permeabilities of about 2.8×10^{-4} ft/day (10^{-7} cm/sec) after seven days of hardening (Jefferis, 1981a). Fly ash replacements, on the other hand, can effectively increase a wall's chemical resistance while blast furnace slag replacements cannot (Jefferis, 1981a).

7.1.2.2.3 Diaphragm Walls

Precast concrete panels and cast-in-place concrete walls are known as diaphragm walls. Unlike soil-bentonite and cement-bentonite walls, these walls develop a great deal of strength over time and can be used as structural components.

Precast concrete panel walls are cast offsite in segments from 1 to 3 feet wide, from 10 to 20 feet long, and from 30 to 50 feet deep. The panels are lowered into a trench containing bentonite slurry and secured in place. Because of their dimensional limitations, concrete panel walls are usually only employed where depths of 50 feet or less are required (Guertin and McTigue, 1982). An exception to this general rule occurs where a cement-bentonite slurry that remains in place is allowed to set up around the panel. Using this technique, the trench can be extended lower than 50 feet and the panel suspended in the cement-bentonite slurry. The slurry then forms a cut-off both below the panel and on either side of it.

Cast-in-place concrete walls are constructed by excavating a short trench (or slot) under a bentonite slurry. The maximum allowable slurry density is 75 pounds per square inch and the maximum allowable sand content is 5 percent (Millet and Perez, 1981). Once the density and the sand content of the slurry in the trench are appropriate, the reinforcing bars are lowered into place. Concrete is then tremied into place using a funnel-like apparatus that directs the concrete to the trench bottom. The apparatus is raised as the concrete level rises. Slurry is pumped out as the concrete is tremied in and then filtered and used for the next panel. When set, the wall is composed of a concrete panel sandwiched between two bentonite filter cakes. One side of the wall is normally excavated during later construction thus destroying one of the filter cakes. The other side, however, retains its filter cake which serves as a low permeability seal (Guertin and McTigue, 1982).

Diaphragm walls are not normally used for pollution migration control because of their susceptibility to leakage through panel connections and their greater expense.

7.1.2.3 Failure Mechanisms

There are several mechanisms or processes that can affect the construction or functioning of slurry walls and cause failure. These failures may be the result of excavation and installation procedures or subsurface conditions.

The failure may occur during excavation and installation thus requiring re-excavation of the slurry trench. Once the wall is in place, a variety of factors can affect the integrity of the wall and its impermeability.

One failure mechanism is trench collapse caused by the loss of stability in the trench walls during excavation and before backfilling or cement-bentonite slurry hardening. Direct causes of trench collapse are (U.S. Corps of Engineers, 1978; Duguid et al., 1971; Nash and Jones, 1963).:

- Insufficient slurry head above groundwater
- Sudden or rapid loss of slurry caused by contact with gravel, large pores, or fissures

- Vibrations
- Surface runoff into open cracks
- Surcharges
- Insufficient agitation of the slurry.

A number of mechanisms can cause failure of the completed wall. Among these failure mechanisms are:

- Cracking
- Hydrofracturing
- Tunnelling
- Piping
- Chemical disruption.

Each of these mechanisms is discussed below.

Soils containing an appreciable concentration of clay can shrink when allowed to dry, forming cracks. Even cement bentonite walls can crack "drastically" when allowed to dry (Jefferis, 1981b). Some soils can, however, shrink and crack even when nearly fluid enough to flow. Nash (1976) and Tavenas et al. (1975) noted the formation of cracks in silty clay soil mixtures only slightly dry of their liquid limits.

These cracks can be caused by several processes, including:

- Consolidation
- Hydraulic fracturing
- Syneresis
- Tunnelling and piping
- Chemical disruption.

7.1.2.3.1 Consolidation

Consolidaton of soils occurs when water is squeezed from the soil pores (Hirschfeld, 1979). This process is accompanied by a decrease in the volume

of the soil mass caused by a decrease in the volume of voids in the soil (Baver, Baver and Gardner, 1972). The amount of consolidation is maximal in fine-textured soils and minimal in coarse-textured materials. The rate of consolidation depends on the soil permeability, the thickness of the layer being loaded and the magnitude of the volume decrease. In fine textured soils, consolidation occurs at a much slower rate than in coarse textured ones. Most consolidation occurs rather quickly, however, fine textured soils can continue slow minor consolidation for several months (Hirschfeld, 1979). The soil-bentonite backfill in slurry walls has been found to continue consolidating for about 6 months, with minimal decreases in volume thereafter (Xanthakos, 1979).

The amount of consolidation in slurry walls is limited by the backfill's arching action along the trench walls and by grain to grain contact in the backfill (Xanthakos, 1979; D'Appolonia, 1980). The amount of slurry wall consolidation depends on trench width as well as on the amount of fines in the backfill. As the fines content increases, consolidation increases because fine particles are more compressible than coarse ones (Mitchell, 1976). Wider trenches have been found to consolidate more than narrow ones. An eight-foot wide trench, for example, was reported by Xanthankos (1979) to have consolidated from one to six inches during the first few months after construction. In contrast to soil-bentonite walls, cement-bentonite walls consolidate very rapidly (Ryan, 1976).

The process of consolidation in soil-bentonite walls can produce many small cracks along shear zones in the soil mass (Mitchell, 1976). Horizontal cracks that extend through the backfill can also be produced by the arching action mentioned earlier (Nash, 1976). This cracking can occur even when the backfill is still quite fluid. Normally, the backfill contains from 25 to 30 percent water. This moisture content is slightly more than the liquid limit of the backfill (D'Appolonia, 1980a). When soil is at its liquid limit, it contains so much water that it will flow under the influence of an applied stress (Baver, Baver and Gardner, 1972). Even so, Nash (1976) observed the formation of horizontal cracks in a silty clay backfill that was only slightly below its liquid limit.

The arching that accompanies consolidation can also lead to the formation of another type of cracking called hydrofracturing. The relationship between hydrofracturing and consolidation is described below.

7.1.2.3.2 Hydrofracturing

When soils or rocks are subjected to excessive hydraulic pressures, cracks may form through which the excess water can flow. The pressure at which this cracking occurs can be less than the effective overburden pressure on the rock or soil in situ. This phenomenon has been used by petroleum geologists to fracture petroleum-containing strata and thereby increase the yield of oil wells (Bjerrum et al., 1972).

The amount of pressure required to induce fracturing depends on the depth of the point receiving the pressure and the ratio of vertical to horizontal pressures at that point. Other factors are also likely involved. Values used by petroleum geologists are 1 psi per foot of depth where the vertical pressures are less than the horizontal pressures, and 0.64 psi per foot of depth where the vertical pressures are greater than the horizontal pressures (Bjerrum et al., 1972).

The cracks caused by this pressure extend vertically where the vertical pressures are greatest and horizontally where the horizontal pressures are greatest (Tavenas et al., 1975). These cracks can continue to increase in length as long as the excess head is applied until they reach an area having greater permeability (Bjerrum et al., 1972). When the pressure is decreased, the cracks will partially close but will reopen when the pressure is again increased (Tavenas et al., 1975).

The effects of hydrofracturing on the functioning of a slurry wall can be severe. Where the vertical pressure exceeds the horizontal pressure (which is normally the case) the pressure exerted on the aquiclude can cause it to fracture. The vertical cracks will be immediately filled with slurry if this occurs during construction. Continued fracturing can, however, occur during backfilling. The backfill is less likely than the slurry to completely flow

in and fill the cracks in the aquiclude, and the aquiclude's permeability may thus be increased. The overall effect of this type of fracturing may be minimal except at sites where the aquiclude is thin or overlies a very permeable strata or both.

A more detrimental effect of hydrofracturing may occur where the horizontal pressures on the wall are greater than the vertical pressures. If more than 1 psi of pressure per foot of depth is applied to the wall, horizontal cracks may form. These could allow significant amounts of water or leachate to flow through the wall. Hydrofracturing has been reported in both soil-bentonite and cement-bentonite walls (Bjerrum et al., 1972; Millet and Perez, 1981).

The likelihood of slurry wall damage caused by hydraulic fracturing is highest where:

- Significant amounts of consolidation occur
- Piezometers are installed in the wall to monitor the wall's permeability using constant head tests
- Large vertical loads are applied to the soils on either side of the trench
- Large hydraulic gradients are allowed to develop across the wall.

When a great deal of consolidation and subsequent arching occur after backfilling, the vertical loadings on the wall can be reduced to levels less than the horizontal loadings. This allows the wall to become susceptible to horizontal cracking (Bjerrum et al., 1972). The amount of consolidation can be minimized by reducing the content of fine particles in the backfill (Mitchell, 1976). However, when additional coarse material is included in the backfill, the wall's permeability is likely to be increased (D'Appolonia, 1980b). The amount of permeability increase expected because of the inclusion of additional coarse material in the backfill should be weighed against the risk of hydrofracturing caused by the anticipated pressure differential across the trench.

Where piezometers are placed in the wall to test the wall's permeability, hydrofracturing may be induced when the excess head is applied. Hydrofracturing caused by use of piezometers occurred when permeability tests were being conducted on a series of dikes with soil-bentonite cores that were installed in Israel. The constant head permeability tests were designed so that the maximum head applied did not exceed the effective weight of the soil above the piezometer. However, even at very low hydraulic pressures, hydrofracturing occurred. This fracturing resulted in a thousand-fold increase in the measured permeability, i.e., from 10^{-4} ft/day initially to 10^{-1} ft/day after fracturing (Bjerrum et al., 1972).

Tests on an in situ, normally consolidated clay were conducted to see if hydrofracturing was caused by arching along the walls of the soil bentonite cores. The normally consolidated clay that was not influenced by arching along trench walls also fractured under the influence of an applied hydraulic pressure that was less than the effective overburden pressure (Bjerrum et al., 1972). This indicates that the occurrence of hydrofracturing is not dependent upon arching (and consolidation) along the walls of slurry trenches.

When large vertical loads are applied along the trench walls, the horizontal stresses on the slurry wall can be greatly increased. The vertical loading can occur because of placement of stockpiles or heavy equipment along the sides of the trench. Depending on the other stresses acting on the wall at the site, the horizontal stress may become greater than the vertical stress, thus making the wall susceptible to horizontal hydrofracturing.

A fourth potential cause of hydrofracturing is the presence of an excessive hydraulic gradient across the wall. If the pressure on the upgradient wall exceeds 1 psi per foot of depth and the horizontal pressures acting on the wall are greater than the vertical pressures, horizontal fracturing of the wall could occur. Excessively high hydraulic gradients could be induced by:

- Failing to provide subsurface drains or extraction wells upgradient of the wall
- Installing extraction or injection wells too close to the wall

- Dewatering a site without deflecting groundwater around the site and away from the wall (via drains, ditches, or extraction wells).

For this reason, the proper use and placement of auxiliary measures, such as wells, should receive careful attention during the wall's design stages.

7.1.2.3.3 Syneresis

Another process that can result in the formation of cracks in a slurry wall is syneresis. According to Mitchell (1976), syneresis is a "mutual attraction between clay particles" that causes the particles "to form closely knit aggregates with fissures between." Contractions that occur in the gel result in the extrusion of liquid (water). Syneresis is often observed in gelatin after aging (Mitchell, 1976). Syneresis may take place in slurry walls, however, the extent to which this phenomenon affects the performance of slurry walls is not known.

7.1.2.3.4 Tunnelling and Piping

Two processes that can result in extensive breaching of a slurry wall are tunnelling and piping. Both of these processes involve the formation of channels through the wall. However, the causes and solutions for the two problems are different.

Several dams have failed because of the formation of very large pores that extend completely through the dam from the upstream to the downstream face. These failures occurred where the earthen dams were constructed of low to medium plasticity native clays that contained appreciable amounts of sodium montmorillonite (Mitchell, 1976). The process by which the failures occurred is termed tunnelling and can be described as a series of interrelated steps, as are listed below (Mitchell, 1976):

1. Differential consolidation of the wet and dry portions of several earthen dams led to the formation of stress cracks below the water line.

2. Water that contained calcium ions flowed into the cracks.

3. Calcium ions from the water replaced sodium ions to the exchange complexes of the clay particles in the dam.

4. The calcium ions caused the clay particles to decrease in size (shrink) and to form packets or "flocs."

5. As the clay particles formed flocs, they became less dispersed and the space formerly occupied by the dispersed clay particles became filled with water.

6. As the sizes of water-filled spaces (pores) increased, the rate of water movement in the pores increased.

7. The increased rate of water movement allowed the water to carry more particles in suspension.

8. As the particle-carrying capacity of the water increased, the number of clay particles eroded by the flowing water increased.

9. As the number of clay particles eroded from the dams increased, the sizes of the pores in the dams increased.

10. As the pore space size increased, the speed of the tunnelling process increased until extensive tunnelling had occurred.

This tunnelling process has been found to take place in earthen dams and embankments that had initial permeabilities as low as 10^{-2} ft/day. One method that has been used to reduce the likelihood of tunnelling is to mix the soil with lime prior to dam construction. This causes the clay particles to shrink and become less easily dispersed before any cracking or particle erosion can occur (Mitchell, 1976).

The risk of tunnelling failures in slurry walls is greatest where groundwaters contain high concentrations of calcium. At these sites the calcium ions from the groundwater can disrupt the soil bentonite backfill and cause tunnelling failures that are similar to the ones experienced with the earthen dams described above. High calcium concentrations in groundwater are most commonly found in sedimentary aquifers, particularly limestone ones. Water from these aquifers can contain over 50 ppm dissolved calcium (Freeze and Cherry, 1979).

The presence of stress cracks, high hydraulic gradients, and permeable backfills are not prerequisites for the tunnelling process, but these factors are likely to speed failure rate considerably. The causes of stress cracking were described previously. The hydraulic gradients across slurry walls at hazardous waste sites are not normally as high as the gradients across dams, so the rate of tunnelling in slurry walls should be much slower than in earthen dams. Another factor that operates in favor of slurry walls is their low permeability, which is normally one to three orders of magnitude less than the minimum permeability of the materials through which tunnelling had been reported (Mitchell, 1976; D'Appolonia, 1980b). This low permeability indicates that the initial rate of water movement through a slurry wall will be much less than through the earthen dams, consequently the particle carrying capacity will also be severely restricted and the erosion rate will be minimal.

Despite the fact that tunnelling failures, if they occur in a soil-bentonite wall, are expected to require long time periods to develop, the potential for slurry wall disruption caused by the presence of calcium ions should be kept in mind when evaluating the feasibility and design criteria for a slurry wall installation at a particular site.

On the other hand, piping begins at the downstream face and proceeds toward the upstream face (Mitchell, 1976). Piping occurs because of the use of improper backfill materials or procedures. Variable wall thicknesses, poorly mixed backfill, or extensive quantities of coarse materials in the backfill can all contribute to piping failure.

Piping occurs where a high hydraulic gradient causes the rate of water movement through the wall to increase as the water nears the downgradient side of the wall. If the water movement is rapid enough, the water could conceivably force the downstream filter cake into the pores in the soil along the trench wall. As the rate of water movement out of the wall increases, the water can begin to erode the easily-dispersed backfill, creating even larger pores and allowing the water movement rate to increase further (Anderson and Brown, 1980). To avoid piping failures, the quality of the filter cake should

be maximized, the backfill materials should be properly selected and mixed, the backfill should be carefully placed to avoid fold-overs and permeable areas, and the hydraulic gradient across the wall should be monitored and kept within designed levels.

7.1.2.3.5 Chemical Disruption

Chemical substances in soil and groundwater can affect the durability of the slurry wall once it is in place. Chemical destruction can affect the cement in cement bentonite slurry walls as well as the bentonite in cement bentonite and soil bentonite walls. The effects of alkali salts on bentonite slurries were described previously.

The action of the chemicals on cement or bentonite are similar to that of the tunnelling process. The cement may become slurry solubilized and the bentonite may become entrained in the solution as the chemicals eventually create a solution channel through the wall into surrounding soil. Thus, chemical destruction processes may create, as well as accelerate, the tunnelling process. Chemicals may also prevent the slurry from forming an adequate filter cake along the sides of the slurry trench by interfering in the slurry gelation process.

7.1.3 Design and Construction

This section discusses design and construction considerations for pollution control slurry walls. The section also shows how the design must consider site-specific factors and discusses design components. Construction techniques for soil- and cement-bentonite and diaphragm walls are also covered.

7.1.3.1 Design Procedures and Considerations

The process of designing a slurry wall requires assessment of site-specific data and consideration of numerous design variables to determine the feasibility of a slurry wall and to select the most appropriate wall type for use at the site. This section describes these processes.

7.1.3.1.1 Feasibility Determination

Using the data from the site investigations, the designer must determine the feasibility and applicability of installing a slurry wall at the site. Factors to consider include:

- Potential waste incompatibility
- Anticipated hydraulic gradients and maximum allowable permeability in the completed wall
- Aquiclude characteristics, e.g., depth, permeability, continuity, and hardness
- Wall placement relative to wastes and leachates
- Cost and time considerations.

Each of these factors is discussed below.

Waste and leachate compatibility with proposed slurry wall backfill mixtures can be determined using laboratory tests. Where long-term permeability is crucial, clay mineralogy and geochemical testing is advisable. These tests provide an indication of which proposed backfill mixtures show the greatest resistance to long- and short-term permeation by the pollutants at the site.

Data on the anticipated hydraulic pressures on either side of the wall indicate the range of hydraulic gradients to which the wall may be exposed. When projected permeabilities and wall areas are known, the rate of subsurface movement through the wall can be determined using Darcy's Law. Darcy's Law states that

$$Q = KIA$$

where Q is the volume of water flowing through the wall, K is the hydraulic conductivity of the wall, I is the hydraulic gradient, and A is the cross-sectional wall area.

To illustrate how Darcy's Law can be used to estimate a slurry wall's effects on groundwater flow at a site, consider the following hypothetical situation: a proposed slurry wall is designed to be 164 feet (50 meters) long, 81 feet (25 meters) deep, and 3.2 feet (1 meter) thick. The hydraulic gradient at the site is estimated at 2, and the wall's hydraulic conductivity is designed to be less than 2.12×10^{-3} gpd/ft (1×10^{-7} cm/sec). According to Darcy's Law, the amount of water that will move through the wall is 57 gpd (0.216 m^3/day). Before the slurry wall was installed, the hydraulic conductivity of the same area was about 2.12 gpd/ft^2 (1×10^{-4} cm/sec), a low permeability for undisturbed soils. The amount of water flowing through the area each day prior to slurry wall installation was 57,000 gpd (216 m^3/day). Thus, this particular slurry wall would reduce the volume of water flow through this area by 99.9 percent. In areas having higher initial hydraulic conductivity the effect of the slurry wall would be even greater.

Another factor to consider when evaluating the feasibility of a slurry wall is the aquiclude at the site. Ideally, the aquiclude should be thick, impermeable, and unfractured, and should be soft enough for a backhoe or clamshell to excavate a 1- to 3-foot key-in to prevent seepage under the slurry wall. In areas where aquicludes are very hard, either slurry walls will be more expensive to install or the aquiclude wall union will be less certain. Where the aquiclude is thin, discontinuous, or fractured, slurry walls can be expected to be less efficient in pollution migration control because of seepage through the aquiclude. Consequently, other remedial measures may be called for.

An early assessment of possible wall locations and configurations can indicate the overall amount of wall exposure to wastes and leachates, the types and placements of auxiliary measures, and the actual length and depth of wall required. These factors, along with an estimate of the necessary wall durability, can assist designers in projecting the cost of the construction effort.

The need for rapid response at some sites necessitates an evaluation of the construction time required. According to Miller (1979), soil-bentonite

slurry walls are normally installed at a rate of 25 to 100 linear feet per day. Thus, the wall described previously could be installed in 2 to 7 days, assuming the work is accomplished in a nonhazardous environment. Hazardous conditions can more than double on-site work time.

7.1.3.1.2 Selection of Slurry Wall Type

If a slurry wall is determined feasible, the type of wall (soil- or cement-bentonite) that is required should be established. To decide whether a soil- or cement-bentonite cut-off wall should be installed at a particular site, several factors need to be considered. The following factors affect the suitability of soil- and bentonite-cement walls:

- Required permeability and hydraulic pressure
- Leachate characteristics
- Availability of backfill material
- Required wall strength
- Aquiclude depth
- Site terrain
- Cost.

These factors are discussed below.

Where low permeability is required, soil-bentonite walls are used and the wall width is determined by the hydraulic head across the trench. Case (1982) recommends that the trench should "have a width of 0.5 feet to 0.75 feet per 10 feet of hydrostatic head on the wall. Thus, for a 100-foot head loss, wall thickness should range from 5 to 7.5 feet." In comparison, a cement-bentonite wall only 2 to 3 feet thick will stand up to the same hydrostatic force (i.e., 100 feet). Deeper walls are generally wider then shallower ones because larger excavation equipment is used for deep walls, and this equipment generally digs a wider trench (Millet and Perez, 1981). Generally, cement-bentonite walls are designed to be narrower than soil-bentonite walls because of the greater shear strength and the higher cost of cement-bentonite walls (Millet and Perez, 1981; Ryan, 1976).

Soil-bentonite walls exhibit a lower permeability and a greater resistance to chemical attack, particularly to acids, than cement-bentonite walls. For this reason, soil-bentonite walls are favored for use as pollution migration cut-offs (Jefferis, 1982b; Xanthakos, 1979). Where floating rather than sinking contaminants are encountered, the slurry wall does not have to be extended down into the aquiclude. Instead, a "hanging" wall is installed. These walls are usually soil-bentonite types.

In some sites, the material excavated from the trench is contaminated because of contact with polluted groundwater. This contaminated soil may be unsuitable for use in the backfill. Samples of this material should be mixed with bentonite slurry and tested to determine the effects of the contaminants on the soil-bentonite wall permeability. In some cases, the contaminants will increase the permeability of the completed wall, but the increase may be less than the increase that could be caused by the sudden exposure of the wall to the polluted groundwater. At these sites, use of the contaminated material in the backfill may be advisable, providing the gradation is adequate (D'Appolonia, 1980b). If, however, the contaminated soil is discovered to be inappropriate for use as backfill material, a suitable borrow area should be found. Where such a borrow area is not available nearby, cement-bentonite walls may be more appropriate.

Generally, cement-bentonite walls are used where heavy vertical loadings are anticipated and large lateral earth movements are not expected. This is because cement-bentonite walls have a higher shear strength and lower compressibility than soil-bentonite walls. Cement-bentonite walls are, however, more likely to crack than relatively plastic soil-bentonite (Millet and Perez, 1981). If the wall must be extended beneath roads, rail tracks, or in close proximity to existing foundations, cement-bentonite walls can be used. In addition, cement-bentonite walls can be used in localized areas requiring strength and tied into soil-bentonite walls for the rest of the trench distance.

Cement-bentonite walls are more expensive than soil-bentonite walls because of the cost of the cement. For this reason, cement-bentonite walls

are not generally used where the aquiclude is deep or where very long cut-off walls are required (Ryan 1977).

At sites where slopes are steep, the areas for backfill mixing are limited or non-existent, and low permeability is not critical, cement-bentonite walls may be preferred. In general, soil-bentonite walls are limited to areas where the maximum slope along the trench line is on the order of 2 percent or less. At many sites, hills can be leveled and depressions backfilled with compacted soil prior to trench construction. The lack of sufficient backfill mixing areas can be overcome by hauling trench spoils to a central backfill mixing area, then hauling mixed backfill back to the trench. Pug mills can also be used for backfill mixing. These operations, site leveling use of pug mills and central backfill mixing, result in slower construction rates and higher costs.

In contrast to soil-bentonite walls, cement-bentonite walls can be constructed in areas of steeper terrain by utilizing the cement-bentonite panel construction technique described later in this section.

As mentioned earlier, a cement-bentonite wall is typically more expensive than a soil-bentonite wall of the same volume because of the cost of the cement. Where thick or deep walls are planned, cement-bentonite walls will, in most cases, be more expensive than soil-bentonite walls. Where wall thickness can be minimized and very low permeability is not essential, cement-bentonite walls can be considered.

7.1.3.2 Soil Bentonite Wall Construction

Following award of the construction contract, the selected firm will proceed with construction of the slurry wall. The major activities include:

- Preconstruction assessment and mobilization
- Site preparation
- Slurry preparation and control
- Slurry mixing and hydration

- Slurry placement
- Trench excavation
- Backfill preparation
- Backfill placement
- Site cleanup and demobilization.

Discussions of these activities follow.

7.1.3.2.1 Preconstruction Assessment and Mobilization

Three major activities occur during the mobilization phase of slurry trench construction. These are:

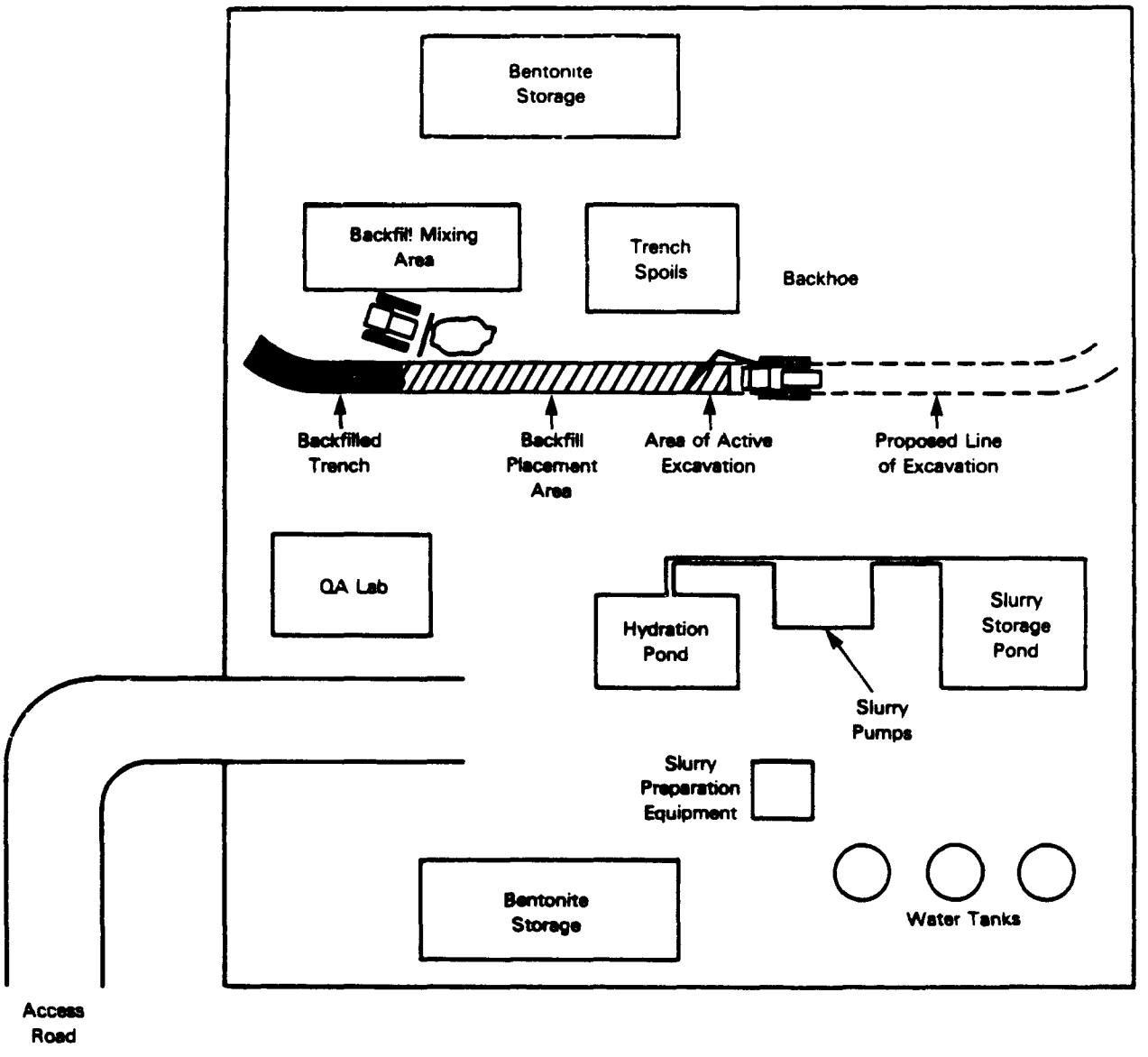
- Layout of the site plan
- Determination of the the equipment, type, amounts of materials, and facilities required.
- Determination of the number and source of personnel required.

These three activities are discussed below.

A preliminary layout is prepared based on drawings supplied by the engineer. Once the preliminary layout is developed, a close examination of the proposed construction site must be performed in order to ensure that all details of the plan are practical. After the on-site examination, a final layout of the worksite can be prepared. A diagram of a typical slurry wall construction site is shown in Figure 7-12.

The specifications and drawings, test boring records, subsurface exploration reports, and records of utility lines are the first sources of information for determining equipment, materials and facilities needs. In addition, an on-site inspection may be required to gain the detailed understanding needed for planning the construction activities.

FIGURE 7-12.
TYPICAL SLURRY WALL CONSTRUCTION SITE



The major work elements and equipment and facilities typically associated with each work element are:

- Excavation
  - hydraulic backhoe
  - mechanically operated clamshell
  - hydraulically operated kelly-mounted clamshell
- Slurry preparation and control
  - high speed colloidal batch mixer for small projects
  - flash mixer for large projects
  - pumps, valves, pipes, and tools
  - hydration ponds
  - desanders, hydrocylones, or screens
- Slurry placement
  - pumps
  - placement hoses and piping
- Backfill preparation
  - dozer or grader
- Backfill placement
  - dozer
  - mechanically operated clamshell
  - trucks and trough
- Supervision and quality assurance
  - shed or trailer
  - Marsh funnel
  - mud balance
  - standard sieves.

Numerous factors influence the types of equipment required as well as the final plan layout and the relative difficulty of construction activities. Some of these factors, along with their potential effects on slurry trench construction operations are listed in Table 7-5.

Determining the correct equipment applications for a particular project is based upon construction requirements and the constraints imposed by the job site. For example, choice of excavation equipment depends upon the depth of the slurry wall and the soil in which the wall is placed. The maximum excavation depth for a standard backhoe is about 50 feet, but larger, extended models are available to reach up to 80 feet in depth (D'Appolonia, 1980a). Both clamshell excavation systems will reach depths of more than 150 feet,

TABLE 7-5.
TYPES OF PHYSICAL CONSTRAINTS AND THEIR EFFECTS ON SLURRY WALL CONSTRUCTION
(Ryan, 1980a; U.S. Army Corps of Engineers, 1978; Xanthakos, 1979; Wetzel, 1982; Tamaro, 1980; Ryan, 1980b; Namy, 1980)

Physical Constraints		Possible Affected Areas	Approach Required
Topography:	Irregular contours Steeply sloping terrain	• Necessary equipment • Site access and work space • Type of wall selected	• Selection of equipment capable of operating in site specific terrain • Extensive site preparatory work - leveling of areas for site entry and work space • Use of cement-bentonite wall in panels or diaphragm wall
Site Access and Work Space:	Site congestion/ traffic Steep terrain Dense vegetation Lack of head room Insufficient space for mixing	• Extent of site preparation and pre-construction • Type of equipment selected • Type of wall selected • Wall construction process	• Special equipment needs; construction of access road; leveling of working area; clearing of dense vegetation • Amount of head room affects type of equipment selected or needed to relocate obstruction • Amount of work space affects wall type selected; soil-bentonite wall requires space for mixing; cement-bentonite wall requires less area for operations, but is more expensive; soil-bentonite can be mixed away from trench but this approach may mean cement-bentonite is cheaper for the site

(continued)

TABLE 7-5. (Continued)

Physical Constraints		Possible Affected Areas	Approach Required
Site Access and Work Space: (continued)			• Extra time needed for site preparation and construction • Appropriate easement clearances
Utilities:	Abandoned sewers Pipelines Leakage from water mains, sewers Power/telephone cables	• Equipment selection • Construction process (operations) • Problem control methods • Sudden slurry loss and possible trench collapse if unanticipated pervious zone, i.e., sewer piping is entountered and ruptured	• Special equipment necessary for excavation around piping and sewer lines, or need for manual excavation • Sequence of trench segment excavation may change if utility discovered; excavate other areas first • Watermain or sewer leakage may cause slurry contamination and loss of trench stability; a control plan necessary at outset of project • Sudden slurry loss requires immediate placement of solid materials (soil, debris) into trench
Cultural Features:	Old foundations Nearby structures Overhead structures	• Equipment selection • Construction process (operations) • Problem control methods	• Foundation penetration to isolate site • Excavation around foundations, or incorporate foundation into wall; if foundation support needed cement bentonite or diagraphm may be required

(continued)

TABLE 7-5. (Continued)

Physical Constraints		Possible Affected Areas	Approach Required
Cultural Features: (continued)		• Headroom	• Special equipment needed if breaking old foundations • Tall equipment may be restricted, e.g., cranes • More time may be necessary for operations • Experienced problem control personnel necessary
Other:	Availability of water Time of year; water table fluctuations, temperature Subsurface geology; large subsurface bounders Type of wall backfill	• Equipment selection • Slurry mixing • Time needed and available for project completion • Site preparation • Problem control methods	• Equipment selection for boulder destruction or excavation • Site may need de-watering system if water table is high or is expected to rise • Soil-bentonite backfill cannot be mixed in subfreezing temperatures • Cement-bentonite will not set in certain temperature ranges • Experienced problem control-personnel necessary • Transport of water to site if none available

with the hydraulic model sometimes preferred in more difficult digging conditions (Guertin and McTigue, 1982). Table 7-6 lists the typical equipment used for slurry trench construction.

Consideration must also be given to site access and obstructions. Access roads might limit the size equipment that can be brought to the site, while obstructions at the site might preclude the use of some types of equipment.

TABLE 7-6
EXCAVATION EQUIPMENT USED FOR SLURRY TRENCH CONSTRUCTION
(Case, 1982; D'Appolonia, 1980; Guertin and McTigue, 1982; Shallard, 1983)

Type	Trench Width (feet)	Trench Depth (feet)	Comments
Standard backhoe	1-5	50	Most rapid and least costly excavation method
Modified backhoe	2-5	80	Uses an extended dipper stick, modified engine & counter-weighted frame; is also rapid and relatively low cost
Clamshell	1-5	>150	Attached to a kelly bar or crane; needs $\geq$ 18 ton crane; can be mechanical or hydraulic
Dragline	4-10	>120	Primarily used for wide, deep soil-bentonite trenches
Rotary drill, percussion drill or large chisel	-	-	Used to break up boulders and to key into hard rock aquicludes. Can slow construction and result in irregular trench walls

When determining the sources of personnel for use at slurry trench construction site, two choices face the construction firm. The firm can send their own equipment and personnel to the construction site or they can rent equipment and hire personnel locally. Most firms will use varying combinations of each approach. For larger jobs and critical small jobs, equipment and personnel are more frequently sent directly from the construction firm. Small jobs can often be handled effectively by using only specialized company-owned equipment, such as an extended backhoe arm, accompanied by supervisory personnel. Other equipment such as bulldozers, cranes, clamshells, and large backhoes can be rented near the job site. Laborers and certain equipment operators can be hired locally for the specific job. However, there are not set rules and each construction contractor will tailor his approach on a site-by-site basis.

Slurry trench construction contractors are best able to judge appropriate equipment and personnel needed at the job site. However, site owners and their representatives should be aware of the approach to be taken by the construction contractor and should be satisfied that appropriate equipment and personnel are available at the job site.

7.1.3.2.2 Pre-excavation Site Preparation

Once site planning has been completed, necessary permits and clearances have been obtained, and required utility, water, and other services have been arranged, preparation of the construction site can proceed. The work site can then be cleared if necessary, security fences erected, utility and water hook-ups made, equipment and facilities brought in and set-up, and construction materials delivered. At this time, work can proceed to move or remove obstructions if necessary.

7.1.3.2.3 Slurry Preparation and Control

Before excavation begins, the slurry must be prepared. To do this, bentonite and water quality must be tested, hydration ponds must be constructed, lines laid, pumps placed, and the mixing area prepared. The slurry

is then mixed in a venturi or paddle mixer and allowed to hydrate fully prior to placement in the trench for soil-bentonite slurry trench cut-off construction or mixing with cement for cement-bentonite cut-off construction.

Bentonite quality is critical to the quality of the slurry. Bentonite is usually shipped to the job site accompanied by laboratory test results showing that it meets quality criteria. These criteria include physical and chemical purity, pH, gel strength, dry fineness (percent passing the number 200 sieve) and filtrate loss. At the job site, these criteria are checked frequently, such as by testing every truckload of bentonite delivered. Requiring field testing of delivered bentonite is important for the site owner because deliveries are occasionally rejected by field testing. At a minimum, testing of pH, viscosity, and fluid loss should be conducted in the field for bentonite delivered to the site.

Slurry quality decreases significantly if the quality of make-up water is poor. Make-up water should be relatively low in hardness, near neutral or slightly higher pH, and low in dissolved salts. Water suitable for drinking is not necessarily suitable for mixing with bentonite. Water of inadequate quality will result in higher bentonite consumption and a lumpy slurry that is difficult to mix and contains above average amounts of free water. In some instances, poor quality water can be chemically treated to make it suitable for mixing (Ryan, 1977).

Two types of mixing systems are most frequently used. These are batch mixing and flash mixing. In the batch system, specified quantities of water and bentonite are placed in a tank and mixed at high speeds with a circulation pump or paddle mixer. Mixing continues until hydration is complete and the batch is ready for use in the trench. Hydration is usually complete in a matter of minutes for the two- to five-cubic-yard batch produced by this system. Because of the low output of the batch system, its use is limited to small jobs.

The second type of mixing system is the flash or venturi mixer. For this system, bentonite is fed at a predetermined rate into a metered water stream

as it is forced through a nozzle at a constant rate. The slurry is subjected to high shear mixing for only a fraction of a second, which is not always adequate for hydration. Therefore, the slurry is often stored until hydration is complete. This is determined by periodically measuring the Marsh Funnel viscosity.

When Marsh Funnel viscosity readings stabilize, hydration is considered complete. Flash mixing is a process that can be operated at high production rates. Because a majority of cut-off walls require continuous production of large amounts of slurry, flash mixing is the more common of the two mixing methods (Ryan, 1977). The type of mixing system used has been found to affect the quality of the slurry produced. High shear (or high speed batch mixers) produce slurries with higher gel strengths (Xanthakos, 1979).

The grade of bentonite dictates the percent needed for a given slurry and hydration time. For example, a grade 90 (bbl/ton) bentonite may have to be mixed at a 6.3 percent concentration, while a grade 125 would require 4.5 percent concentration for an equivalent slurry. Hydration times for higher grades are likewise lower, which may result in higher slurry production rates for a given slurry preparation facility (Ryan, 1977). However, the higher cost of the higher grade bentonites requires that the selection of bentonite grade be made on a site-by-site basis. Frequently a mud balance test is run on slurry from the hydration pond as a quality control check of the bentonite content. Viscosity and pH are also checked frequently (Cavalli, 1982).

7.1.3.2.4 Slurry Placement

From the hydration pond, slurry is pumped on an as-needed basis to the excavated slurry trench. Slurry level in the trench must be maintained at least several feet above the water table and normally within a foot or two of ground level. This slurry level is maintained to provide the hydrostatic pressure necessary to hold open the trench.

Once a slurry trench installation is under way, backfill and excavation are being performed simultaneously, with a minimum amount of trench remaining

open under the slurry. Figure 7-13 illustrates the excavation and backfill placement operations. The amount of trench remaining open at any one time depends on the properties of the backfill material and the characteristics of the excavation equipment, which are discussed in the following sections. Samples removed from the trench for QC checks must be representative, that is, they should not be taken only from the top surface of the slurry but should be taken at various depths in the trench. The slurry, designed to keep the trench open during excavation and backfilling, must also allow displacement by the backfill material. That is the reason for stipulating maximum and minimum values for parameters such as density, viscosity, and sand content. Those requirements are listed in Table 7-7. During the excavation operation, some of the spoil becomes incorporated in the slurry. This increases slurry density and sand content. A high sand content indicates a high density and a likelihood of problems with eventual displacement of the slurry by the backfill (D'Appolonia and Ryan, 1979).

FIGURE 7-13.
CROSS-SECTION OF SLURRY TRENCH, SHOWING EXCAVATION AND BACKFILLING OPERATIONS

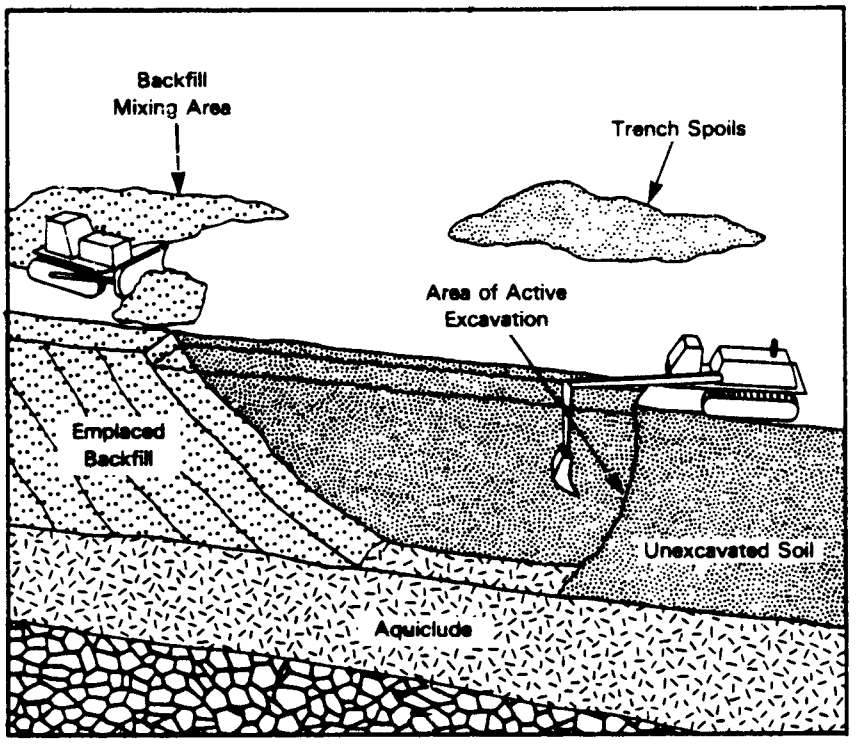


TABLE 7-7.
MATERIALS QUALITY CONTROL PROGRAM FOR SOIL-BENTONITE WALLS
(Federal Bentonite, 1981 as cited by Spooner et al., 1984)

Quality Control Item	Subject	Standard Name	Type of Test	Frequency	Specified Values
Materials	Water	-	-pH -Total hardness	Per water source or as changes occur	As required to properly hydrate bentonite with approved additives. Determined by slurry viscosity and gel strength tests
	Additives	-	Manufacturer certificate of compliance with stated characteristics		As approved by engineer
	Bentonite	API Std 13: Standard Procedure for Testing Drilling Fluids	Manufacturer certificate of compliance		Premium grade sodium cation montmorillonite
	Backfill soils	-	Selected soils obtained from a borrow area approved by the engineer		65 to 100% passing 3/8" Sieve 35 to 85% passing #20 Sieve 15 to 35% passing #200 Sieve
			Roll to 1/8" thread		
Slurry	Prepared for placement into the trench	API Std 13: Standard Procedure for Testing Drilling Fluids	- Unit Weight		Unit Weight = 1.03 gm/cc
			- Viscosity	1 set per shift or per batch (pond)	v = 15 centipose of 40 sec-Marsh @ 6°F
			- Filtrate Loss		Loss = 15 cc to 25 cc in 30 min @ 100 psi
			- pH		pH = 8
	In Trench	API Std 13B1: Standard Procedure for Testing Drilling Fluids	- Unit Weight	1 set per shift at point of trenching	unit weight = 1.03 to 1.36 gm/cc
			- Slump	1 set per 200 cu yds	Slump 2 to 6 inches
			- Gradation		65 to 100% passing 2/8" Sieve 35 to 85% passing #20 Sieve 15 to 35% passing #200 Sieve
Backfill Mix	At Trench	ASTM C 143 Slump Cone Test			

7.1.3.2.5 Trench Excavation

Excavation of a slurry trench parallels standard trench excavation procedure except that only the portion of the trench above slurry level can be visually inspected for continuity. Trench excavation is usually accomplished with backhoes with appropriate boom length and bucket capacity. Frequently, boom lengths are extended by construction contractors to meet the needs of the trench installation. Counterweights are often required to offset the movement created by the long boom lifting a full bucket from the trench. The backhoe is the favored means of excavating a slurry trench because it is much faster than other equipment, such as the crane and clamshell. However, boom lengths are currently limited to 70 to 90 feet. For greater depths, the crane and clamshell are normally used. Drag lines have been used in the past, but have been used rarely for recent installations (D'Appolonia, 1982).

Trench continuity is critical to a successful installation. For checking continuity of slurry trenches, several approaches have been used. All of them may be employed at a given site to ensure that the trenching is continuous from the ground surface to the aquiclude key-in.

The field inspector should have boring logs and a cross-sectional drawing of the trench so that visual inspection of the excavated material and the degree of extension of the backhoe boom will indicate approximate depth and whether the aquiclude has been reached. Soil boring data can also be used to quantify the aquiclude key-in. By watching excavated material for a change in color or texture, construction personnel can determine when the subsurface layer which is to be keyed into is reached.

Sounding of depth with a weighted line or a rod should be performed frequently to ensure an even trench bottom and to detect any irregularities. Finished trench depths should be recorded for preparation of a drawing showing trench cross section.

7.1.3.2.6 Backfill Preparation

Standard practice during backfill preparation for soil bentonite walls is to use excavated material mixed with slurry from the trench for backfill. In this case, the slurry provides moisture necessary for backfill mixing. When trench spoils are used, the material excavated from the trench is usually placed nearby, slurry is added, and a bulldozer is used to track and blade the material until it is thoroughly mixed.

A relatively level working surface is needed for backfill mixing. At sites that are too steep, backfill mixing areas can be excavated. These should be at least as wide as the width of the excavating equipment track. Where backfill mixing areas are not available, batch mixers or pugmills can be used, although these are slower than using a bulldozer for backfill mixing.

A number of quality control checks are necessary for backfill preparation activities. These include tests of the:

- Fines content
- Slump
- Wet density
- Presence of contaminants.

A key parameter in the design of a backfill is the sieve analysis and particularly the amount of fines. The content of fines in the backfill is directly related to the permeability of the finished soil-bentonite wall and its ability to withstand chemical attack. A standard practice is to perform permeability testing in the pre-construction phase to define the range of acceptable grain size distributions that will provide the design permeability. During construction, frequent grain size distributions and less frequent permeability testing is performed on the backfill material to ensure that the design permeability requirements will be met by the completed slurry trench. Grain size distributions can be performed in a field laboratory and are much less expensive to run than permeability tests, which are usually run in a laboratory.

Slump cone testing should be performed frequently on backfill material after mixing to make sure that backfill is wet enough to slump in the trench without trapping pockets of slurry, yet dry enough to displace the slurry easily. High slump also indicates a gentler slope of backfill in the trench, which would require keeping more of the trench open. A slump of 2 to 6 inches is adequate (D'Appolonia, 1980).

Mud balance testing should be performed frequently on the backfill before it is placed in the trench. Mud balance tests indicate the wet density of the backfill. This shows for certain that the backfill will or will not readily displace the slurry. The wet backfill sample used for the mud balance test can be dried and reweighed to determine the water content. A very high water content can result in excess water infiltration into the trench walls and excess settlement of the backfill. The separation of water from an excessively wet backfill can also dilute the slurry in the trench.

7.1.3.2.7 Backfill Placement

Once trench excavation has proceeded for a distance that will not result in backfill material being re-excavated, backfill placement can begin. First, samples of slurry at the base of the trench are collected and tested for wet density. The slurry should be at least 15 pcf less dense than the backfill mixture (D'Appolonia, 1980b). If the slurry is too dense, it will not be displaced properly during backfill placement. The dense slurry or coarse material on the trench bottom must be removed via airlift pumps, a clamshell bucket or other method. This slurry can be used for backfill mixing or be desanded via desanders, hydrocyclones, or screens, and returned to the trench.

To place the initial layer of backfill, a clamshell is often used. The backfill must not be allowed to drop freely through the slurry, as this may cause segregation of the backfill particles or entrapment of slurry pockets within the backfill. For this reason, the clamshell lowers the initial grabfull of backfill to the trench bottom. The next grabfull is placed on top of the first, and so on until the backfill is visible at the ground surface. Thereafter, the backfill is pushed into the trench by bulldozers or graders.

The point of trench backfilling progresses toward the area of active excavation (D'Appolonia and Ryan, 1979).

The slope of the emplaced backfill is normally 5:1 to 10:1. The distance to be maintained between the toe of the backfill and the area of active excavation varies greatly, depending on soil types and backfill slopes. Ideally this distance is kept to a minimum to avert trench stability problems (D'Appolonia, 1980b). Some specifications have set the distance at from 30 to 200 feet (U.S. Army Corps of Engineers, 1975; Ryan, 1976).

Soundings of the placed backfill should be taken to show the slope at which the backfill is coming to rest and to indicate possible problems with trench wall collapse and entrapment of pockets of slurry. Depths should be recorded and plotted on a cross-sectional drawing of the trench. Trench excavation and backfilling progress can also be recorded in this manner.

When a slurry wall is constructed to entirely surround a waste site, the excavation must end with enough overlap to ensure that all material designated for excavation is removed. The inspector must determine that the backhoe bucket is removing backfill materials from the full trench depth to verify that the trench is continuous for its entire length.

7.1.3.2.8 Capping

To protect the finished soil-bentonite wall, either a dessication cap or a traffic cap is generally applied to the slurry wall surface.

Once backfill has been completed, cracking is soon observed on the top of the slurry wall unless it stays wet. To complete the installation, the top 1 to 3 feet of wall is removed to eliminate the cracks and a high quality backfill material replaces it. The material used is usually required to have a high clay content and to be compacted in lifts over the trench. This forms a low permeability cap to protect the cut-off wall from excessive dessication (U.S. Army Corps of Engineers, 1976). This is followed by topsoil and seeding or a gravel layer to prevent water and wind erosion.

Where traffic over the wall is anticipated, a traffic cap can be constructed to reduce the load on the completed cut-off. To do this, lifts of compacted clay are interspersed with geotextile layers. Gravel can be used over the final geotextile layer at the surface. At one facility where a soil bentonite cut-off wall was close to a heavily traveled gravel road, the cap consisted of an 18-inch thick compacted clay lift topped by a geotextile sheet. This was overlain by another 18-inch thick compacted clay layer topped by another geotextile sheet. Several inches of gravel were placed over the final sheet of geotextile to distribute the weight and bear the load of the vehicles (Coneybear, 1982).

7.1.3.2.9 Cleanup Activities

After wall construction is complete, the excess slurry and mixed backfill must be disposed of in a manner that avoids erosion and disruption of sewer lines. The slurry should not be allowed to enter sanitary or storm sewer lines because of the potential for pipe blockage from the slurry. Excess slurry also should not be left as a thick layer on the soil surface, as this may result in excessive ponding of surface water. One method of slurry disposal would be to mix the slurry with dry, coarse soil to produce as dry a mixture as possible. This material could then be either buried or spread in a thin layer over disturbed areas, then fertilized and seeded. Any contaminated soil from the excavation must be disposed of in accordance with site requirements. All disturbed areas should be stabilized and site maintenance procedures should be instituted.

There are a number of differences in construction activities that vary with construction materials. The following is a brief summary of cement bentonite and diaphragm wall construction techniques.

7.1.3.3 Cement Bentonite Wall Construction

The discussion presented above is an outline of a soil-bentonite cut-off wall construction. Modifications to the construction specifications are

necessary when constructing a cement-bentonite cut-off wall. These modifications include the following:

- The requirement for backfill materials are eliminated
- A description of the standards for cement and cement storage is added in the materials section
- A description of the cement-bentonite slurry requirements and the cement/water (c/w) ratio to be maintained for desired compressive strength is added to the materials section
- The methods to be used for tie in of adjacent cement-bentonite panels (if used) are addressed in the performance section
- Under the Quality Control section, a requirement is added for the manufacturer's certification of the cement and the testing of cement/water ratio for each batch of mix is required.

Cement bentonite slurry walls involve the use of a slurry consisting of water, bentonite, and cement. The advantages of cement-bentonite walls are that backfill material is not needed and they exhibit some structural strength. In addition, by excavating one section (panel) at a time, a cement-bentonite wall can be installed on a site with more extreme topography.

Two types of cement-bentonite walls are being used. The in-place method involves simply excavating under a cement-bentonite slurry and leaving the slurry in place. The slurry eventually sets to provide some structural strength. For the replacement method, excavation takes place under a bentonite slurry. After excavation of the section of wall is complete, the bentonite slurry is pumped out of the trench and the cement-bentonite slurry is pumped in and allowed to set. The replacement method is used only when setting of the cement-bentonite slurry could possibly occur while excavation is being completed.

Cement-bentonite slurries begin to set within 2 to 3 hours after the cement and slurry are mixed. If the slurries are agitated for over 48 hours, they lose their ability to set (Jefferis, 1981b). For this reason, when it appears that the excavation of a single cement-bentonite panel will take longer than a day or so to complete, cement retarders are added to the slurry

or the replacement method is used. Examples of this situation would be very deep excavations, when rock is encountered in the excavation or when keying the trench into bedrock.

Quality control procedures for cement-bentonite walls are identical to those for soil bentonite walls. However, composition of the cement-bentonite slurry is more critical, therefore, care must be taken when weighing and mixing components of the slurry. This is because the calcium in the cement causes an irreversible decrease in slurry quality. Table 7-8 shows typical materials quality control standards for cement-bentonite cut-off walls.

7.1.3.4 Diaphragm Wall Construction

Construction of diaphragm walls also involves the use of bentonite or cement-bentonite slurries, although these walls are not normally used for pollution migration cut-offs, except where high strength is required. Diaphragm walls are composed of either precast concrete panels or cast-in-place concrete sections. Unlike soil-bentonite and cement-bentonite walls, these walls develop a great deal of strength over time and can be used as structural components. A brief description of the techniques used for construction of diaphragm walls is given in Section 7.1.2.2.3.

7.1.4 Completed Wall Costs

Contractors have provided average costs for completed slurry cut-off wall construction and installation. These estimates may vary widely, however, as they are based upon a number of site-specific factors. Some of these factors include the following:

- Type of backfill (either cement- or soil-bentonite)
- Distance materials must be transported
- Presence of contamination or high salt content in groundwater requiring special bentonite and excavation procedures
- Health and safety considerations

TABLE 7-8.
MATERIALS QUALITY CONTROL PROGRAM FOR CEMENT-BENTONITE WALLS
(Federal Bentonite, 1981 as cited by Spooner et al., 1984)

Subject		Standard	Type of Test	Frequency	Specified Values
Water		--	- pH - Total hardness (Ca & Mg)	Per water source or as changes occur	As required to properly hydrate bentonite with approved additives. Determined by slurry viscosity and gel strength tests.
Materials	Bentonite	API STD 13A	Manufacturer certificate of compliance		Unaltered sodium cation montmorillonite
	Cement	ASTM C 150	Manufacturer certificate of compliance		Portland, Type 1 (Type V or Type II for certain applications)
Bentonite Slurry	Prior to addition of cement	API STD 13B	- Viscosity	1 set per shift or per batch (pond)	v = 34 sec-Marsh @ 68°F pH = 8
Cement-Bentonite Slurry	Upon introduction in the trench	API STD 13B API STD 10B	- C/W ratio - Viscosity	Each batch 5 per shift	C/W = 0.20 v = 40 to 50 sec-Marsh

- Type of overburden being excavated
- Depth of excavation
- Presence of physical constraints upon working area (i.e., buildings or other structures which must be worked around)
- Suitability of native materials for use as backfill constituents.

Ressi di Cervia (1980) developed a chart (Table 7-9) which relates cut-off wall construction costs to type of backfill used, depth of excavation, and soil type present. Costs are given in terms of square foot of wall since the width of the excavation is generally determined by the excavation equipment.

7.2 GROUTING

Grouting is a process in which a fluid material is injected into a soil or rock mass in order to reduce water movement or to impart increased strength to the formation. Once in place within the formation voids, the fluids solidify or gel thus greatly reducing the permeability and imparting increased mechanical strength to the grouted mass (JRB, 1982; Herndon and Lenahan, 1976a & b).

Grout barriers can be many times as costly as slurry walls and are generally incapable of attaining truly low permeabilities in unconsolidated materials. Therefore, they are rarely used when groundwater control in unconsolidated materials is desired. The primary use of grouting is to seal voids in porous or fractured rock when other groundwater control methods are not practical. The installation of grout barriers has not been as common a practice for controlling leachate plume migration from hazardous waste sites as has slurry wall installation. Hence, there is not a great deal of information available on the application of grouting techniques at hazardous waste sites for controlling leachate plume migration.

Grouting is the most practical and efficient method for sealing fissures, solution channels, and other voids in rock. Where rock voids allow the passage of large volumes of water, a grout can often be formulated to set with

TABLE 7-9
RELATION OF SLURRY CUT-OFF WALL COSTS PER SQUARE FOOT
AS A FUNCTION OF MEDIUM AND DEPTH
(Ressi di Cervia, 1980 as cited by Spooner et al., 1984)

Geologic Material	Slurry Trench Prices in 1979 Dollars Soil Bentonite Backfill (Dollars/Square Foot)			Unreinforced Slurry Wall Prices in 1979 Dollars Cement Bentonite Backfill (Dollars/Square Foot)		
	Depth 30 Feet	Depth 30-75 Feet	Depth 75-120 Feet	Depth 60 Feet	Depth 60-150 Feet	Depth > 150 Feet
Soft to Medium Soil N 40	2-4	4-8	8-10	15-20	20-30	30-75
Hard Soil N = 40 - 200	4-7	5-10	10-20	25-30	30-40	40-95
Occasional Boulders	4-8	5-8	8-25	20-30	30-40	40-85
Soft to Medium Rock N 200 Sandstone, Shale	6-12	10-20	20-50	50-60	60-85	85-175
Boulder Strata	15-25	15-25	50-80	30-40	40-95	95-210
Hard Rock Granite, Gneiss, Schist*	---	---	---	95-140	140-175	175-235

Notes:
N is standard penetration value in number of blows of the hammer per foot of penetration (ASTM D1586-67)

*Normal Penetration Only

For standard reinforcement add $8.00 per sq. ft.
For construction in urban environment add 25% to 50% of price.
Costs do not reflect work in a contaminated environment.

sufficient speed to shut off the flow. In theory, placement of a grout curtain upgradient or downgradient from, or beneath, a hazardous waste site is possible. In practice, however, this can be a very difficult task to accomplish successfully.

As with slurry walls, placing a grout barrier upgradient from a waste site can redirect the flow so that groundwater does not flow through the wastes that are creating the hazardous leachate. Given normal groundwater chemistry, most grouts could be expected to function well in this capacity.

Placement of a grout barrier downgradient from or beneath a hazardous waste site is quite another matter. A variety of problems typically occur when attempting to grout in the presence of leachate or very impure groundwater. In many instances, controlling the set time, and consequently, ensuring a barrier of reliable integrity is difficult or impossible. Little information exists in the literature on the resistance of grouts to chemical attack. Should a case arise where a grout might contact leachate or very impure groundwater, extensive testing must be conducted to determine possible effects on the grout. Additional problems occur in attempting to grout a horizontal curtain or layer beneath a waste site. In order to inject grout in such a case, injection holes must be drilled either directionally from the site perimeter or through the wastes. The first situation can be very difficult and expensive and the second could be very dangerous. In either case, effective barrier placement is very difficult and virtually impossible to ensure (JRB, 1982).

7.2.1 Theory

Successful grouting depends on the selection of the proper grouting material for the specific area to be grouted. Thus, the physical and chemical characteristics, the geology and hydrology, and the groundwater chemistry of the site must be evaluated. The factors that will determine the groutability

of the site include (Bowen, 1981; Sommerer and Kitchens, 1980; Herndon and Lenahan, 1976a & b):

- Soil and rock physical characteristics: The permeability, porosity, particle distribution, and pore size distribution of the formation will control the physical properties, such as viscosity and particle size, that the grout must possess as well as the quantity of grout that will be needed.

- Groundwater characteristics: The hydrology and contaminants contained in the groundwater will determine the chemical properties such as set time and structure, that the grout must possess.

- Soil and rock chemical characteristics: The contaminants contained in the soil and rock structure will control the chemical properties that the grout must possess.

Based on these factors, an appropriate grout can be selected from (Guertin and McTigue, 1982; Bowen, 1981):

- Bituminous grouts: These grouts, either as emulsions or asphalts, can be used to waterproof soils or rock cavities.

- Suspension grouts: These are the most common types of grout and include coarse grouts which contain particles in suspension such as cement, clay and cement-clay. These materials are usually the more viscous of the available grouting materials as well as having the largest particle size. Thus, these grouts are best used in the grouting of rock or coarse material.

- Chemical grouts: These grouts rely on chemical reactions to form gels. They initially have low viscosities and thus can be used in finer-grained cohesionless soils and as a secondary treatment for grouting coarse soils and rock fissures. The common types of chemical grouts include silicates, acrylamides, and various polymers.

The following sections discuss the chemical composition and reaction theory of the major types of grout.

7.2.1.1 Bitumen Grouts

Bitumen or asphalt emulsions consist of bitumen, water, and an emulsifier. Bitumens are viscoelastic materials containing high molecular weight hydrocarbons (Kirk-Othmer, 1978a). When dispersed in water, bitumens yield an

emulsion with a low viscosity suitable for injection (Tallard and Caron, 1977a). The viscosity of the emulsion is controlled by adjusting the ratio of bitumen to water. Typical emulsions include bitumens; bitumen, soap, and casein; and bitumen with a filler such as clay (Bowen, 1981).

Emulsions are stabilized by the emulsifiers, which delay molecular aggregation and increase viscosity (Kirk-Othmer, 1978a). The emulsifiers are polar and determine whether the resulting emulsion is cationic or anionic (Bowen, 1981). For example, amine chains are often used as acid emulsifiers to yield cationic emulsions (Bowen, 1981; Kirk-Othmer, 1978a). Emulsifiers should be water soluble and have properly balanced hydrophilic and lipophilic properties (Koehmstedt, Hartley and Davis, 1977).

Bitumen emulsions break up upon contact with earth materials so that the more viscous emulsion components settle out and fill the pores and fissures of the earth material. The breakdown of bitumen emulsions can occur through direct breakdown, the addition of destabilizing agents, or the adsorption of emulsifying water. In direct breakdown, a stabilizing agent is eliminated through decomposition or absorption by fine soil material. Direct breakdown is difficult to control and often occurs too quickly or too slowly (Tallard and Caron, 1977a). A second method involves adding destabilizing agents (electrolytes or hydrolyzable esters) to the emulsion either before or after the emulsion is injected to promote breakdown and flocculation of the emulsion (Bowen, 1981; Tallard and Caron, 1977a). When destabilizing agents are added, a one-step method (addition of additives prior to injection) is preferable (Tallard and Caron, 1977a).

The set time of bitumen grout will vary depending on the method used to achieve a breakdown of the emulsion. Set time is further controlled by the emulsifier present in the grout (Koehmstedt, Hartley and Davis, 1977).

Bitumen grout is known to have long term stability (Tallard and Caron, 1977a). Oxidation and aqueous leaching of oxidation products are the primary causes of degradation. Oxidative processes include microbial action and sunlight. There is little evidence of anaerobic bacterial oxidation (Hartley,

et al., 1981). Upon aging, viscous bitumen material can develop an internal structure and the viscosity may increase (Kirk-Othmer, 1978a).

The bitumen used as grout typically consists of a coal tar or asphalt base (Tallard and Caron, 1977a). These materials consist of complex hydrocarbons which can be toxic if leached.

7.2.1.2 Cement Grouts

Cement grout consists of Portland cement and water. Several types of Portland cement are available. The types most often used in soil grouting include Type I (ordinary Portland cement), Type II (modified Portland cement, moderate sulfate resistance), and Type V (low alumina, sulfate resistant). Fillers such as clay, sand, or pozzolans, and additives such as chemical polymers may be added to the cement to vary the characteristics of the grout and improve its resistance to deleterious chemicals (Littlejohn, 1982; Bowen, 1981).

Portland cement is made from limestone, clay, and iron oxide and consists chiefly of tricalcium silicate (45 percent), dicalcium silicate (27 percent), tricalcium aluminate, tetracalcium alumino-ferrite, magnesium oxide, and other minor constituents (Littlejohn, 1982; Ingles and Metcalf, 1973). The calcium silicates are the major cementation compounds in cement (Neville, 1973).

Various materials (organic and inorganic) may be added to cement grout to achieve special characteristics or to control grout properties. Special additives include anti-bleed agents, accelerators, retarders, and expansion agents (Littlejohn, 1982). Latexes or water-soluble polymers may be added to achieve special properties (Kirk-Othmer, 1979). Calcium chloride may be added as an accelerator, however, calcium chloride may cause increased shrinkage upon drying (Littlejohn, 1982). Sand may be added so that cement grouts may be used to treat coarse materials (Bowen, 1981). Clays can be added to stabilize the cement while pozzolans or clay may be added to improve alkali resistance (Littlejohn, 1982; Bowen, 1981). Polyhydric alcohols can be added to provide acid-resistance (Farkas and Szwere, 1949). Colloids such as

gelatin, agar, and ammonium stearate may be added as stabilizers (Bowen, 1981).

Upon addition of water, the silicates and aluminates in cement form hydration products which have low solubilities in water (Neville, 1973). The calcium silicates form gels of mono- and di-calcium silicate hydrate (Ingles and Metcalf, 1973). The insoluble calcium silicate hydrate crystallizes to form a matrix within which the remaining hydration products form (Ingles and Metcalf, 1973; Neville, 1973). The resulting cement gel is considered to be a fine physical mixture of copolymers of hydrates (Neville, 1973).

The properties of fresh and cured concrete depend primarily upon the water-cement ratio and the degree of hydration of the cement (Kirk-Othmer, 1979). Excess water will result in poor durability, increased shrinkage, and bleeding (Littlejohn, 1982). High water-cement ratios also cause large numbers of capillary spaces in the matrix (Kirk-Othmer, 1979).

The viscosity of cement grouts depends on the amount of water added to the cement. As the water-cement ratio increases from 0.4 to 0.7, the viscosity decreases from 5,000 to 500 centipoise. The viscosity can also be reduced through the addition of certain organic admixtures (Kirk-Othmer, 1979).

The setting process occurs in two stages. First, fluidity of the grout decreases until it is not pumpable--this is called the set time. Second, the grout hardens and increases in strength--this is called the hardening time (Littlejohn, 1982). The various components of cement grouts set at different rates. These rates vary from a "flash set" (tricalcium aluminate) to several hours (tricalcium silicate and dicalcium silicate) (Littlejohn, 1982). Complete setting takes approximately six hours (Kirk-Othmer, 1979).

The set time may be increased by the action of a number of substances including: organic materials, silt, clay, coal, lignite, sulfates, sodium salts, metals, sugar, and tartaric acid (Fung, 1980; Littlejohn, 1982;

Thompson, Malone and Jones, 1980). Increasing the water-cement ratio will also increase the set time.

While concrete is durable under normal conditions, concrete is subject to deterioration as a result of deficiencies in grout quality, chemical attack, drying, and temperature fluctuations (Littlejohn, 1982). Increasing the water-to-cement ratio can greatly increase the permeability of the concrete through an increase in the number of capillary spaces which permit penetration of solutions (ACI Committee 515, 1979; Kirk-Othmer, 1979). Furthermore, all cement grouts expel bleed water which can lead to accumulations of water and a decrease in strength and lateral impermeability (Bowen, 1981). Cement may also shrink causing the formation of microfissures and cracks unless it is moist-cured (Littlejohn, 1982).

Concrete is vulnerable to chemical attack because of its alkalinity, reactivity, and permeability. Penetration of fluids may be accompanied by chemical reactions between the fluid and concrete constitutents (ACI Committee 515, 1979). Concrete will deteriorate because of sulfates, chemical wastes, and organic acids (Tomlinson, 1980). Furthermore, cement hydration compounds may leach from the matrix and this can also cause deterioration (ACI Committee 515, 1979).

The basic components of cement grouts are lime, silica, alumina, iron oxide and water, which are all nontoxic. However, substances such as chemical polymers (e.g., acrylamide) may be added to the grout to modify its properties, and these materials may be toxic.

7.2.1.3 Clay Grouts

Clay grouts are composed primarily of bentonite. The basic reaction theory and chemical composition of bentonite is described in section 7.1.2.1.

Bentonite grouts will start to set as soon as the injection pressure is decreased. Once this process starts, the viscosity and then the gel formation and strength will increase with time. The final strength of the gel will

depend on the setting time, colloid concentration, and the composition of the suspending fluid (Xanthakos, 1979). The final strength obtained, though, is very low compared to other grouts (Tallard and Caron, 1977b). The set time can be controlled through the addition of silicates to the bentonite grout, resulting in set times from a few minutes to five hours (Bowen, 1981).

The basic ingredient in this type of group, bentonite, is essentially nontoxic. The toxicity of most of the additives, such as sodium silicate, are also low, although agents used to gel the sodium silicate could pose a risk (Tallard and Caron, 1977b).

The presence of organic or inorganic compounds in the groundwater can have a detrimental effect on the ability of bentonite grouts to contain pollutants. These chemicals can affect the physical and chemical properties of the bentonite, result in flocculation, reduce swelling of the bentonite, or destroy the bentonite's crystalline structures.

If the bentonite is injected into groundwater which contains high concentrations of electrolytes, such as sodium, calcium, and heavy metals, the bentonite could flocculate. This will result in particles that can exceed 10 microns in diameter, thus hampering the grout's ability to penetrate into the soil structure (Tallard and Caron, 1977b; Matrecon, Inc., 1980; Alther, 1981b).

Various organic and inorganic compounds can cause a change in the amount of swelling that bentonite particles undergo (Alther, 1981b). This can lead to increased porosities and permeabilities. Strong organic and inorganic acids and bases can dissolve alumina and silica or alter the bentonite, and also lead to large permeability increases (D'Appolonia and Ryan, 1979; Alther, 1981b). Undiluted alcohols increase grout permeability by extracting water from the clay interlayers, thus reducing the amount of particle swelling.

7.2.1.4 Silicate Grouts

Silicate grouts consist of alkali silicates, water, a gelling or setting agent, and sometimes an accelerator. Typically, sodium silicate is used in the grout although potassium silicate may be used instead. The gelling or setting agent used varies depending on the desired properties of the gel. In general, acids, acid-forming compounds, polyvalent cations, and some organics may be used as setting or gelling agents (Kirk-Othmer, 1979; Hurley and Thornburn, 1971). Accelerators include chlorides, aluminates, or bicarbonates (Johnson, 1979). Other substances, such as organic esters, may be added to delay gelling time (Bowen, 1981). Grout compositions can vary considerably with the grouting method and are often proprietary (Herndon and Lenahan, 1976a).

The viscosity of sodium silicate grouts varies from 1.5 to 50 centipoise and may be as high as 260 centipoise (Sommerer and Kitchens, 1980; Tallard and Caron, 1977a). The viscosity of the grout depends on the ratio of SiO_2 to Na_2O. The higher the ratio of silica to sodium, the lower the viscosity. The set time of silicate grouts varies from less than a minute to several hours. Factors affecting the set time include silicate concentrations, setting agent concentrations, and temperature. Increasing any of these three factors will decrease the set time (Tallard and Caron, 1977a). The accelerator concentrations may also be varied to control set time. Soil conditions can also affect the set time with acid soils reducing gel time and alkaline soils potentially preventing gel formation (Office of the Chief of Engineers, 1973).

Tallard and Caron (1977a) report that silicate grout is quite durable. However, long-term strength and impermeability are of concern because silicate grouts are subject to deterioration via syneresis (water expulsion), shrinkage (dessication), and solution erosion by groundwater (Hurley and Thornburn, 1971).

Sodium silicate grouts are essentially nontoxic. The set grout has a toxicity (oral) of 15 g/kg, while sodium silicate has a toxicity (oral) of 1.1 g/kg. Amides frequently used in formulating the grout are skin irritants

(Tallard and Caron, 1977b). Other organic substances, such as formamide, are toxic, possibly carcinogenic, and require special precautions when preparing and injecting the grout (Karol, 1982a; Kirk-Othmer, 1979; Tallard and Caron, 1977a). Heavy metal salts, which may be used as gelling agents, are also toxic and could potentially be leached from the gel. In addition, sodium salts may be expelled from the grout and under certain circumstances they may be an environmental hazard (Karol, 1982a).

7.2.1.5 Organic Polymer Grouts

Organic polymer grouts represent only a small fraction of the grouts in use. These grouts consist of organic materials (monomers) that polymerize and cross-link to form an insoluble gel. This section addresses acrylamide, phenolic, urethane, urea-formaldehyde, epoxy, and polyester grouts and their properties.

7.2.1.5.1 Acrylamide Grouts

Acrylamide grouts consist of a base material (typically a monomer or mixture of monomers), a cross-linking agent, an initiator or catalyst, and an accelerator or activator. Persulfates or peroxides, typically ammonium persulfate, are used as an initiator. Accelerators or activators include dimethylaminopropionitrile, diethylaminopropionitrile, or triethanolamine (Tallard and Caron, 1977a). The substances used vary with the particular product. Gel time may be controlled through the addition of a reaction inhibitor, typically potassium ferricyanide (Karol, 1982a; Tallard and Caron, 1977a). Buffers may be required to maintain the pH of the grout solution around 8.

The set time of acrylamide grouts can vary from a few seconds to several days (Karol, 1982a; Cues, Inc., 1982; Tallard and Caron, 1977a; Office of the Chief of Engineers, 1973). The primary factors controlling set time are concentrations of reaction inhibitors, catalysts, and activators (Karol, 1982a; Kirk-Othmer, 1979; Tallard and Caron, 1977a). An acidic grout solution or acidic grouting conditions (groundwater or earth material) can lengthen the

set time and may even prevent gelation (Office of the Chief of Engineers, 1973). The set time may be shortened through the action of metals, triethanolamine, or ammonium persulfate by increasing the pH, or by increasing the dry matter in the grout (Clarke, 1982; Avanti International, 1982; Tallard and Caron, 1977a; Office of the Chief of Engineers, 1973).

Once an acrylamide grout is set, it is fairly stable chemically and is not subject to slow deterioration or syneresis (Karol, 1982a; Berry, 1982; Avanti International, 1981; Tallard and Caron, 1977a). Over time, however, the grout may shrink or some salts, unreacted materials, and hydrolysis products may leach out (Karol, 1982a; Tallard and Caron, 1977a).

7.2.1.5.2 Phenolic Grouts

Phenolic grouts, commonly referred to as phenoplasts, are polycondensates of phenols and aldehydes (Sommerer and Kitchens, 1980; Tallard and Caron, 1977a). A typical phenolic grout consists of a phenol, an aldehyde, water, and a catalyst. Formaldehyde is used exclusively because of its reactivity (Billmeyer, 1971). Sodium hydroxide or other alkaline materials (hydroxides, carbonates, phosphates) are typically used as catalysts, although acids can also be used.

Phenolic grouts may be mixed in a one or two solution system. The proportions of phenol, formaldehyde, and catalyst are fixed by reaction requirements so the only variable is the amount of water added (Tallard and Caron, 1977a). Polymerization begins as soon as the solutions or grout components are mixed. In general, polyvalent cations from the alkali catalyst initiate and promote polymerization (Chung, 1973). The polymerization (polycondensation) process results in the formation of a three dimensional network of polymer chains that are joined and cross-linked by formaldehyde (Karol, 1982a; Bowen, 1981; Tallard and Caron, 1977a). The catalyst is also attached to the polymer resin and may form secondary linkages within the resin network or bond with soil. The resulting resin is insoluble and retains all constituent materials (Tallard and Caron, 1977a).

Phenolic grouts made with resorcinol and formaldehyde have viscosities between 1.2 and 3.0 centipoise (Karol, 1982a; Tallard and Caron, 1977b). Commercial products consisting of tannin or polyphenols typically have higher viscosities: "Geoseal" ranges from 2 to 12 centipoise, "Terranier" ranges from 4 to 10 centipoise, "Rocagil" ranges from 5 to 10 centipoise (Bowen, 1981; Tallard and Caron, 1977a; Tallard and Caron, 1977b). The viscosity of resorcinol-formaldehyde grouts, like acrylamides, remains constant until gelation occurs. In tannin-based grouts, the viscosity gradually increases after the components are mixed (Karol, 1982a; Tallard and Caron, 1977b).

The gel or set time of phenolic grouts is proportional to the solution diluteness and may vary from several minutes to several hours (Bowen, 1981; Karol, 1982a; Tallard and Caron, 1977a). With very dilute solutions, the gel time increases so much that gelation never occurs and the grout becomes unusable (Tallard and Caron, 1977a). If all other factors remain constant, the choice of catalyst will affect the set time because different bases have different reactivities. Sodium hydroxide, the most common catalyst, provides approximately a 20 minute gel time as do some carbonates and calcium hydroxide. Other bases (hydroxides and carbonates) provide longer set times (Tallard and Caron, 1977b).

The length of set time affects the strength of the grout. Short set times give strong gels while long set times give weak gels. Strong grouts are not critical to waterproofing, but short set times are often important (Karol, 1982a). To decrease the set time, phenolic grouts may be combined with another grout such as a silicate that has a shorter set time. This grout will set first and provide a "false set." The phenolic grout is retained in the grout matrix and sets at its normal rate. The final properties of the grout mixtures are determined by the phenolic grout (Tallard and Caron, 1977a).

Wet cured phenolic grouts (under groundwater) are generally durable, however, there may be a slight weakening over time potentially caused by the gradual swelling of the resin (Tallard and Caron, 1977b). After setting is complete, the phenolic resin contains water that is not chemically bound in the matrix. Under dry conditions, this water can evaporate and the gel may

shrink and crack (Sommerer and Kitchens, 1980; Tallard and Caron, 1977a; Tallard and Caron, 1977b). Unlike acrylamides and silicates, this dehydration is irreversible and can lead to disintegration of the gel (Karol, 1982a; Tallard and Caron, 1977a).

Proper proportioning of phenol and formaldehyde in the grout will produce a complete reaction and the resulting gel is insoluble and inert (Tallard and Caron, 1977a; Tallard and Caron; 1977b). If the materials are not properly proportioned, excess materials may leach out of the resin. In either case, some of the catalyst may remain partially soluble and leach out of the matrix (Karol, 1982a; Tallard and Caron, 1977a; Tallard and Caron, 1977b).

7.2.1.5.3 Urethane Grouts

Urethane, or polyurethane, grout consists primarily of a polyisocyanate and a polyol or other hydroxy compound such as a polyether, polyester, or glycol (Karol, 1982a; Vinson and Mitchell, 1972). A diisocyanate is often used (Vinson and Mitchell, 1972). RokLok®, a polyurethane grout, consists of polymethylene polyphenyl isocyanate (containing diphenylmethane diisocyanate) and poly(oxyalkylene) polyether polyol resin (Mobay Chemical Corporation, 1982). Other substances such as catalysts, surfactants, dilution agents, plasticizers, and stabilizers may be added to control the reaction of the grout and its properties before and after setting.

Urethane grouts set through a polymerization process. The initial reaction occurs between excess isocyanate and the polyol compound to form a polyurethane prepolymer (Jiacai, et al., 1982; Karol, 1982a). The reaction may be stopped at this step and be completed later. To complete the reaction sequence, the prepolymer is reacted with water, carboxylic acid, or other hydroxyl-containing compounds to form polyurethane foam (Karol, 1982a; Vinson and Mitchell, 1972). This foam consists of cross-linked polyurethane chains with the cross-linking occurring through the formation of urea linkages and the generation of carbon dioxide gas (Vinson and Mitchell, 1972; Billmeyer, 1971).

Additional materials may be added to the grout. Catalysts such as tertiary amines (triethylamine, triethanolamine, triethylenediamine) or tin salt may be added to control the rate of gellation and foaming (Karol, 1982a; Jiacai, et al., 1982). Stabilizers and surfactants may be added to control the surface tension of the grout as well as the size of the bubbles (Karol, 1982a; Jiacai, et al., 1982). Since the prepolymer has a high viscosity, the grout may be diluted with acetone, xylene, ethylacetate, or dichloroethane. Plasticizers such as dibutylphthalate may also serve as dilution agents (Jiacai, et al., 1982).

The set time varies from several seconds to several minutes or hours (McCabe, 1982; Avanti International, 1982; Tallard and Caron, 1977a). The set time can be shortened by increasing the water content, moving from a primary to a secondary or tertiary alcohol, or increasing the catalyst (amine) content (Vinson and Mitchell, 1972; Tallard and Caron, 1977a). The set time can be lengthened by decreasing the size of the polyol or by adding acid (Sommerer and Kitchens, 1980; Vinson and Mitchell, 1972).

7.2.1.5.4 Urea-Formaldehyde Grouts

Urea-formaldehyde resin grouts consist of urea, formaldehyde, catalyst, and water. The urea-formaldehyde mixture may be in monomer or prepolymer form. The catalyst is an organic acid, inorganic acid, or acid salt.

The urea-formaldehyde resin is formed in a two-step reaction process. First, the urea and formaldehyde monomers react through methylolation or hydroxymethylation to form low molecular weight polymers. This reaction may be either acid or base catalyzed (Kirk-Othmer, 1978b). Second, further polymerization occurs through condensation of the polymers with water being generated. This reaction will only occur with an acid catalyst (Kirk-Othmer, 1978b; Billmeyer, 1971). The resulting resin is a stable network of cross-linked urea-formaldehyde polymers (Tallard and Caron, 1977a).

There are two mechanisms for achieving this reaction sequence. Where the grout mixture uses urea and formaldehyde monomers, the two reactions occur

rapidly and the overall reaction is difficult to control (Karol, 1982a). The second mechanism involves stopping the reaction sequence after methylolation. At this point, precondensates or prepolymers have formed that are soluble in water and are prevented from further reaction by the use of inhibitors or by pH control (Karol, 1982a; Tallard and Caron, 1977a). Commercially, the methylolation reaction is base catalyzed (Kirk-Othmer, 1978b; Tallard and Caron, 1977a). The second reaction, polymerization, may occur later by lowering the pH of the prepolymer solution. By introducing an intermediate stage in the reaction, better gel time control is achieved and sudden setting of the grout may be avoided (Karol, 1982a; Tallard and Caron, 1977a).

In either case, an acid medium is required for final polymerization to occur. Urea-formaldehyde resins cannot be used in alkaline media because the acid catalyst would react with the media and be destroyed before it could react with the grout (Sommerer and Kitchens, 1980; Tallard and Caron, 1977a; Rensvold, 1968). In addition to pH control, the urea-formaldehyde reaction is also controlled by the mole ratio of the reactants and the dilution of the mixture (Kirk-Othmer, 1978b; Tallard and Caron, 1977a).

Urea-formaldehyde grouts have a low viscosity. Urea solutions (unpolymerized) have viscosities similar to acrylamides and phenolics (Karol, 1982a). Solutions of urea-formaldehyde prepolymers are more viscous, having typical viscosities of 10 to 13 centipoise (Karol, 1982a; Sommerer and Kitchens, 1980).

The set time varies with the type of grout formulation and the type of catalyst. Monomer grouts have a very short set time because the reaction is abrupt. To achieve more control over the set time, prepolymer grouts are used, however, this increases the viscosity of the grout and makes the grout unsuitable for fine soils. Depending on the catalyst, the set time may vary from several minutes (hydrochloric acid) to almost an hour (sulfuric acid). The proportion of the catalyst and the dilution of the grout will also affect the set time (Tallard and Caron, 1977a).

For urea-formaldehyde grouts there is both a gel time and a cure time. The gel time refers to the time required to form a soft gel similar to acrylamide. Following gellation, the grout cures to a stiffer consistency. This occurs over a few hours to as long as a day depending on the gel time (Karol, 1982a). The setting of urea-formaldehyde grouts may be slowed by organic solvents and oils (Malone, Jones and Larson, 1980). Cement will prevent the setting of these grouts (Kirk-Othmer, 1979). In general, alkaline materials will inhibit the polymerization reaction through destruction of the acid catalyst.

7.2.1.5.5 Epoxy Grouts

Epoxy grouts are resins that consist of an epoxide, a hydroxy compound, and a hardener. The epoxide is typically epichlorohydrin while the hydroxy compound is typically bis-phenol-(2,2-bis(4-hydroxyphenol)propane (Modern Plastics Encyclopedia, 1981; Tallard and Caron, 1977a; Billmeyer, 1971).

Epoxies are generally cured through the addition of a hardener (Tallard and Caron, 1977a). These hardeners are cross-linking agents that react with epoxy and hydroxyl groups (Modern Plastics Encyclopedia, 1981). The resulting epoxy resins are polyethers (Billmeyer, 1971). A number of hardeners may be used, typically, amines, polycarboxylic anhydrides, or monocarboxylic acids. Each hardener reacts differently and imparts different properties to the resin. Different proportions between the resin and the hardener will also provide different types of resins. However, the resin-hardener ratio cannot be varied greatly (Tallard and Caron, 1977a).

As a result of the amine hardening process, amine-terminated polyamide resins are generated. These resins replace water on wet surfaces creating a water-free interface between the resin and the material covered (Engineering News-Record, 1965). For this reason, epoxy resins are useful in applications in wet areas or under water.

The viscosity of epoxy grouts varies with the molecular weight (Billmeyer, 1971). The most fluid of these resins has a viscosity of at least

400 centipoise. This viscosity may be lowered to 100 centipoise by adding a fluid hardener (Tallard and Caron, 1977a). Organic solvents and reactive dilution substances may also be used to change the viscosity (Sommerer and Kitchens, 1980; Tallard and Caron, 1977a). Ethers such as butyl glycidyl ether can decrease the viscosity of epoxy grouts to 20 centipoise (Tallard and Caron, 1977a).

The set time of epoxy grouts varies depending on the choice of hardener. In general, the set time is difficult to regulate (Tallard and Caron, 1977a).

7.2.1.5.6 Polyester Grouts

Polyester grouts consist of a resin base and a catalyst (Office of the Chief of Engineers, 1973). The resin is unsaturated and consists of a polyester produced from the reaction of a polyacid and a polyalcohol. Typically, this reaction involves the condensation of an unsaturated diacid (maleic acid or fumaric acid) with a dialcohol. In commercial products, a reticulant is included with the polyester resin. These products may contain 30 to 40 percent reticulant, typically styrene (Tallard and Caron, 1977a; Billmeyer, 1971).

Polymerization is achieved through the addition of the catalyst which is generally a peroxide. The catalyst causes the polyester resin to polymerize as well as copolymerize with the reticulant. A gel forms which eventually hardens to a solid material (Tallard and Caron, 1977a; Office of the Chief of Engineers, 1973). The hardening process is accompanied by shrinkage of the resin by as much as 10 percent (Office of the Chief of Engineers, 1973).

Accelerators may be added to speed up the setting by facilitating the decomposition of the catalyst into free radicals (Tallard and Caron, 1977a; Office of the Chief of Engineers, 1973). Accelerators include cobalt, manganese, or vanadium salts, mercaptans, tertiary amines, and quaternary ammonium salts (Tallard and Caron, 1977a).

The quantity and type of polyester, reticulant, and catalyst may each be varied. Each variation will produce different resins with different characteristics (Tallard and Caron, 1977a).

Commercial polyesters vary in viscosity from several hundred to several thousand centipoise. The minimum viscosity is between 200 to 250 centipoise, however, this is too high to grout fine earth materials such as sand. By adding reactive diluent, the viscosity may be reduced to 10 to 50 centipoise (Tallard and Caron, 1977a).

The set time of polyester grouts varies from a few minutes to several days. The resins may contain volatile compounds that make long set times uncertain (Tallard and Caron, 1977a). The set time is dependent on resin volume, ambient temperature, catalyst selection, and heat dissipation (polymerization is exothermic). In addition, excessive moisture may inhibit the setting of polyester grouts (Office of the Chief of Engineers, 1973).

7.2.2 Design and Construction

7.2.2.1 Types of Grouts

Different types of grouts are currently available including emulsion, polymers, and particle suspensions. These grouts are generally water solutions with low viscosity which allow for easy penetration of formation voids. There are five major grout types:

- Bitumen grouts
- Cement grouts
- Clay grouts
- Silicate grouts
- Organic polymer grouts.

The most common grouts in use are cement and clay, constituting approximately 95 percent of all grouts used. Silicates represent the majority of the remaining 5 percent. Bitumen and organic polymer grouts are of

relatively limited use in plume management. A brief description of these different grout types and their applications follows.

7.2.2.1.1 Bitumen Grouts

Bitumen or asphalt emulsions are direct emulsions in which water is the continuous phase. These emulsions have a variety of uses including surface and subsurface waterproofing applications. Emulsions are typically used for fine materials such as sand or in finely fissured materials. Hot bitumen grout has been used in coarse soil formations, however, it is difficult to handle (Tallard and Caron, 1977a). Bitumenous materials appear to have limited use as grouts.

Bitumen or asphalt is resistant to most chemicals. Inorganic chemicals (except concentrated acids), dilute acids, simple alcohols, glycols, and aldehydes will not affect bitumen (Puzinaurkas, 1982). ASPEMIX®, an asphalt-based slurry used in vibrating beam constructed slurry walls, has been found in the short term to be resistant to paint thinner and hazardous waste site leachate containing chemicals or brine (Slurry Systems, 1982). In particular, ASPEMIX appears to be a resistant coal tar, which contains dimethyl-naphthalene, methylnaphthalene, and pyrene (Drozda, 1981).

Asphalt is not compatible with concentrated mineral acids. Most polar and nonpolar solvents will dissolve asphalt as will chlorinated, aliphatic, and aromatic hydrocarbons. Ketones and phenols may also slowly degrade asphalt (Puzinaurkas, 1982). Salts and organic matter in the earth materials will prevent proper formation of a seal between bitumen or asphalt and soil. In addition, some salts will cause bitumen to effloresce (Ingles and Metcalf, 1973).

Some liners for hazardous waste sites consist of an emulsified asphalt membrane. These membranes are not compatible with nitric acid or oils. In general, materials reported to be detrimental to emulsified asphalt membranes include organic substances, highly ionic wastes, and waste containing salts, strong acids, or strong bases (Haxo, 1980).

The primary area of bitumen grout usage is for reducing the permeability of fine sands, fine soils (e.g., clayey sands), or finely fissured rock. This type of grout is not generally suitable for coarse soils because a poorly gelled grout may draw away from the coarse material resulting in an ineffective seal (Tallard and Caron, 1977a). Bitumen grout may be applied in combination with cement (Bowen, 1981).

7.2.2.1.2 Cement Grouts

Cement has probably been used longer than any other type for grouting applications (Bowen, 1981). Cement grouts utilize hydraulic cement which sets, hardens, and does not disintegrate in water (Kirk-Othmer, 1979). Because of their large particle size, cement grouts are more suitable for rock than for soil applications (Bowen, 1981). However, the addition of clay or chemical polymers can improve the range of usage. Cement grouts have been used for both soil consolidation and water cut-off applications, but their use is primarily restricted to more open soils. Typically, cement grouts cannot be used in fine-grained soils with cracks less than 0.1 millimeter wide (Bowen, 1981).

Cement grout may be applied to fractured rock (with voids of sufficient size to ensure penetration of the grout) for underpinning and constructing a variety of structures (Bowen, 1981; Kirk-Othmer, 1979). Type I Portland cement may be used for materials with large voids. Resin and gypsum cements are used by the oil well cementing industry for rapid water seal-off applications (Bowen, 1981).

Materials may be added to cement grouts to improve their applicability. Sand may be added to portland cement to create a grout suitable for coarse materials (Bowen, 1981). Bentonite may be added to improve the penetration of cement in alluvial soils (Solentanche, no date).

7.2.2.1.3 Clay Grouts

Clays have been widely used as grouts, either alone or in formulations because they are inexpensive (Guertin and McTigue, 1982). Only certain types of clay minerals, however, possess the physical and chemical characteristics favorable for use in grouting. These characteristics include the ability to swell in the presence of water and to form a gel structure at low solution concentrations. These properties are possessed most markedly by the montmorillonites. Other types of clay minerals, such as kaolinite and illite, can be used as fillers in grout formulations, such as clay-cement mixtures (Greenwood and Raffle, 1963).

Local deposits of clay will contain a mixture of clay minerals which, depending on their proportions, can be utilized as a grout material either directly or in mixtures. Many local clays require some type of treatment in order to remove large particles that would reduce the effective distance that the clay could penetrate into a formation. Instead of using locally derived material, pure montmorillonite or bentonite can be utilized.

Bentonite grouts alone can be used as void sealers in coarse sands with a permeability of more than 10^2 ft/day (10^{-1} cm/sec). Bentonite-chemical grouts can be used on medium to fine sands with a permeability between 10^2 ft/day (10^{-1} cm/sec) and 1 ft/day (10^{-3} cm/sec). Both of these grout types can also be utilized to seal small rock fissures (Guertin and McTigue, 1982). Because of their low gel strengths, bentonite grouts are not able to support structures and therefore can only be used as void sealers (Tallard and Caron, 1977b).

7.2.2.1.4 Silicate Grouts

Alkali silicates are the largest and most widely used type of chemical grouts. Sodium, potassium, and lithium silicates are available with sodium silicates being used most frequently. Chemical grouts (i.e., silicates and organic polymers) constitute less than 5 percent by volume of the grouts used in the United States although they represent almost 50 percent in Europe

(Kirk-Othmer, 1979). In addition to their use as a grout, sodium silicates may be used as additives to other grouts, such as Portland cement, to improve strength and durability (Hurley and Thornburn, 1971).

Silicate grouts are used for both soil consolidation and void sealing applications. These grouts are suitable for subsurface applications in soils with a permeability of less than 10 ft/day (10^{-2} cm/sec). Silicate grouts are not suitable for open fissures or highly permeable materials, because of syneresis, unless they are preceded by cement grouting (Karol, 1982a; Sommerer and Kitchens, 1980). Furthermore, tests conducted by the Waterways Experiment Station found silicate grouts to be ineffective in waterproofing fine-grained soils (Hurley and Thronburn, 1971).

Silicate grouts are resistant to moderate amounts of acids or alkalies (Kirk-Othmer, 1979). These grouts are also resistant to high concentrations of chromic, nitric, and sulfuric acid (Boova, 1977). Organic esters have little effect on silicate grout (Bowen, 1981). Silicate mortar is similar to silicate grout but contains fillers such as silica, quartz, or ganister. This mortar is resistant to most acids (except hydrofluoric acid) as well as neutral salt solutions (ASTM, 1982).

Silicate grouts gel through the action of acids or acid salts. Hurley and Thornburn (1971) and Karol (1982a) report that the setting time of silicate grouts may be significantly decreased in the presence of soils or groundwater with appreciable salt contents. These effects may be mitigated by using the groundwater to mix the grout. Acidic soils may also decrease the gel time (Karol, 1982a).

Silicate grouts are not compatible with a number of materials. Gel time and grout strength may be affected by large amounts of acid or alkali (Kirk-Othmer, 1979). Additionally, organic materials and high concentrations of some metals will slow the setting time (Malone, et al., 1980).

7.2.2.1.5 Organic Polymer Grouts

Organic polymer grouts represent only a small fraction of the grouts in use. These grouts consist of organic materials (monomers) that polymerize and cross-link to form an insoluble gel. The organic polymer grouts include:

- Acrylamide grouts
- Phenolic grouts
- Urethane grouts
- Urea-formaldehyde grouts
- Epoxy grouts
- Polyester grouts.

Acrylamide Grouts--Acrylamide grouts have been in use for about 30 years, and were the first of the organic chemical polymer grouts to be developed. Acrylamide grouts have the largest use among the organic polymer grouts and are the second most widely used chemical grouts (Karol, 1982a). They may be used alone or in combination with other grouts such as silicates, bitumens, clay, or cement (Tallard and Caron, 1977a).

AM-9® was the first acrylamide grout developed, however, it was removed from the market in 1978 because of the toxicity of one of its components (Karol, 1982a). There are only a few acrylamide or acrylamide-based grout products available and a number of them are imported. These grouts include acrylates, polyacrylamides, and acrylamide derivates. Some grouts, such as Rocagil 1295®, are not used in the United States (Karol, 1982a). Although AM-9 was removed from the market, it has been used as recently as 1980 in water cut-off applications (Berry, 1982). A number of acrylamide grouts that were similar to AM-9, have been imported and marketed after AM-9's removal (Karol, 1982a). AM-9 is probably the most studied of the acrylamide grouts and because of the similarity of the materials and the reaction mechanisms, much of the AM-9 data is valid to other acrylamide grouts (Karol, 1982a).

The use of acrylamide and acrylamide-based grouts is greater in the United States than in Europe where phenolic grouts are more common. Acrylic

and polyacrylamide grouts are typically used in ground surface treatment, ground treatment for oil well drilling, and subsurface applications (e.g., waterproof concrete structures). Acrylate grouts are more commonly used for ground surface treatment than for soil injection where acrylamide grouts are more frequently used. Acrylamide applications include structural support and seepage control for mines, soil consolidation for foundations of structures and dams, and water control and soil consolidation for tunnels, wells, and mines (Tallard and Caron, 1977a). Specific applications include grout curtains, loose sand stabilization, artesian flow shut-off, and water seepage control in jointed and fissured rock (Office of the Chief of Engineers, 1973). Based on AM-9 applications, acrylamide grouts may be used in a variety of soil materials such as fine gravel; coarse, medium, or fine sand; coarse silt; and some clays (Herndon and Lenahan, 1976b).

All of the acrylamide grout formulations contain substances that are toxic and require special handling (Berry, 1982; Geochemical Corporation, 1982; Tallard and Caron, 1977a). While the gelled grouts or polymerized acrylamides are reported to be non-toxic, unreacted toxic monomer or other substances can leach from the grout matrix if the polymerization process is not complete (Tallard and Caron, 1977a).

Many groundwater contaminants (e.g., acids, alkalies, salts) will have little effect on acrylamide grouts, particularly if dilute and the groundwater is used to mix the grout (Clarke, 1982; Avanti International, 1981; Kirk-Othmer, 1979). Acrylamide grouts are impermeable to gases and hydrocarbon solvents such as kerosenes (Office of the Chief of Engineers, 1973). Toluene, heptane, and dilute hydrochloric acid (2 percent) also do not have effect on acrylamide grouts (Berry, 1982). They are also unaffected by 10 percent solutions of alcohols, ketones, hydrocarbons, acids, and metal salts (Clarke, 1982).

Some salts and pH will affect the setting time of acrylamide grouts (Caron, 1963). Low pH conditions (less than 6.5) can prevent acrylamide grouts from setting. Polymerization inhibitors such as sodium nitrates and metallic salts can also delay gelation (Avanti International, 1981; Office of

the Chief of Engineers, 1973). Alkaline conditions or metal ions, such as iron, copper, zinc, or tin can shorten the gelation time (Avanti International, 1981; Tallard and Caron, 1977a). Other gel time accelerators include hydrogen sulfide and soluble salts such as NaCl, $CaCl_2$, sulfates, and phosphates (Karol, 1982a; Office of the Chief of Engineers, 1973).

The durability of acrylamide grout is affected by highly alkaline media. Such media can promote saponification of the grout, particularly the monomer (Sommerer and Kitchens, 1980; Tallard and Caron, 1977a), which will affect the performance of the grout (Tallard and Caron, 1977a). Herndon and Lenahan (1976b) report that sulfides affect AM-9 grout, but the effect is not specified. Strong or concentrated hydrating and dehydrating agents will have the greatest impact on acrylamide grouts. For example, acrylamide grouts will swell in the presence of sulfuric acid, sodium chloride, sodium sulfite, sodium hydroxide, and laundry detergent. Alcohols and glycols will cause the grout to shrink by drawing out the water (Berry, 1982).

Phenolic Grouts--The use of phenolic resin grouts in underground and foundation construction began in the 1960's (Kirk-Othmer, 1979; Tallard and Caron, 1977a). These grouts may be used in fine soils and sands for a variety of water control and ground treatment applications. However, phenolic grouts are not widely used alone but are typically used in conjunction with other grouts (Tallard and Caron, 1977a). Phenolic resin mortars are recommended for use with organic acids, wet gases (reducing), nonoxidizing and nonreducing gases, and nonoxidizing mineral acids except hydrofluoric acid and highly concentrated sulfuric acid. These recommendations are based on immersion of the mortar (ASTM, 1982).

Phenolic resins are not resistant to alkali (Boova, 1977). Both strong acids and bases will attack phenol-formaldehyde resins (Billmeyer, 1971). Phenolic resin mortars are not recommended for use with bleaches or wet gases (oxidizing), but they are recommended for limited use with oxidizing mineral acids, inorganic alkali, and organic solvents (ASTM, 1982). Phenol-formaldehyde resins are resistant to most organic solvents (Billmeyer, 1971).

Phenolic grouts are used in soils of high permeability such as fine gravel and coarse, medium, and fine sand (Sommerer and Kitchens, 1980; Herndon and Lenahan, 1976b). These grouts have been used to stop leaks in railway tunnels and for ground surface treatment (Flatau, Brockett and Brown, 1972; Tallard and Caron, 1977a). Phenolic grouts find their greatest use in combination with silicate grouts in treating fine sands and silts (Tallard and Caron, 1977a).

Different phenolic grout products have been formulated to meet different needs. For example, Geoseal MQ-4® was designed for resistance to saline groundwater, Geoseal MQ-14 was designed for treating low permeability materials, and Terranier C® was designed for silts (Bowen, 1981).

Phenolic grouts are limited however in that they contain toxic and caustic materials which require special handling (Karol, 1982a; Tallard and Caron, 1977b).

Urethane Grouts--Urethane grouts are the second most commonly used type of organic polymer grout (Jacques, 1981). Urethane grouts were developed in Germany for consolidation applications and are now used in Europe, South Africa, Australia, and Japan (Sommerer and Kitchens, 1980). These grouts are used for water and soil applications and can penetrate finely fissured material.

If properly formulated, urethane grouts resist most chemical and other degradative processes. Urethane grouts are reported to have good resistance to oxidation, shrinkage from drying, and biological agents, although some shrinkage of the grout may occur in response to water table fluctuations (Avanti International, 1982; Billmeyer, 1971).

While the prepolymer used in urethane grouts is flammable, this type of grout is relatively nontoxic (Berry, 1982; Avanti International, 1982; Mobay Chemical Corporation, 1982). Most of the grouting formulations contain some free toluene diisocyanate, hence, special handling and protective equipment

are needed when working with the grout materials because they are eye, skin, and respiratory irritants (Berry, 1982; Avanti International, 1982).

Urethane grouts have been used for a number of water and soils applications. Some formulations have been used for consolidation in coal mines and railroad tunnels (Mobay Chemical Corporation, 1982). Other formulations are primarily used for sewer grouting and pipe sealing (Avanti International, 1982). CR-250® has been used for potable water applications as well as for soil sealing and waterproofing applications (Jacques, 1981).

TACSS®, a urethane grout produced in Japan, has been used for treating rivulets in karstic materials and for consolidating and waterproofing ground through which large volumes of water circulate (Tallard and Caron, 1977a). Because of the high viscosity of this formulation (and by analogy, other urethane grouts), TACSS cannot be used to treat fine-grained soils (Karol, 1982a).

Urea-Formaldehyde Grouts--Urea-formaldehyde resins are frequently referred to as aminoplasts. The idea for the use of these resins as grouts came from their use as glue in the oil industry (Tallard and Caron, 1977a). Although urea-formaldehyde grouts have been available since the 1960's, they have found limited usage (Karol, 1982b; Sommerer and Kitchens, 1980). These grouts can only set-up in an acid environment, therefore, they cannot be used in basic formations.

Urea-formaldehyde grouts are considered to have good stability (Karol, 1982a). If the grout is properly formulated and a good polycondensation reaction is achieved, the resulting resin should be inert and insoluble to most solvents although it will contain some free formaldehyde (Karol, 1982a; Tallard and Caron, 1977a). However, these grouts may quickly break down when subjected to cyclic wet and dry or freeze and thaw cycles (Karol, 1982a). Fung (1980) reports that some urea-formaldehyde resins are also biodegradable.

Solutions of urea-formaldehyde grout are both toxic and corrosive because they contain formaldehyde and an acid catalyst. However, solutions using

prepolymers have less free formaldehyde (Karol, 1982a; Tallard and Caron, 1977a). The cured resin has low toxicity and considered inert, but it does contain some unreacted formaldehyde (Kirk-Othmer, 1978b; Tallard and Caron, 1977a).

Because of the acidity requirements, urea-formaldehyde grouts have undergone little development (Tallard and Caron, 1977a). They have been used for ground stabilization and for sealing coal mines but long-term applications have not been reported (Karol, 1982a; Tallard and Caron, 1977a). These grouts can generally be used only in ground and groundwater with a pH less than 7. Their development has been further limited because of their toxic constituents and the production of ammonia during condensation (Sommerer and Kitchens, 1980).

Most of the descriptions of urea-formaldehyde grout applications come from Eastern Europe, the USSR, and Japan. Products containing prepolymers have been used in Poland and Hungary. Injection of an acid solution to destroy carbonates before grouting has been used in the USSR to minimize high pH media problems. However, this technique is not used widely because it is costly and increases the size of soil voids. Further, more soil imbalances may be created through the destruction of soil components (Tallard and Caron, 1977a).

Epoxy Grouts--Epoxy grouts and other glue-like grouts have been in use since 1960. These grouts have had limited use in soil grouting primarily because of their high cost (Tallard and Caron, 1977a). Most of the applications reported in the literature involve the use of epoxy resins in mortars and for sealing cracks. Epoxy resins can adhere to and seal submerged concrete, steel, or wood surfaces and are useful in water applications (Engineering News-Record, 1965). They have been used for grouting cracked concrete for structural repairs and grouting fractured rock to improve its strength (Office of the Chief of Engineers, 1973).

Epoxy grouts exhibit good durability. In the ground, their properties are similar to those of polyester grouts in that they may be subject to

hydrolysis (Tallard and Caron, 1977a). Epoxy mortars tend to have little shrinkage and limited water absorption (U.S. Grout Corporation, 1981; Boova, 1977).

Epoxy grouts consist of substances requiring special handling precautions. If the grouts are properly formulated, the set resin should incorporate all of the materials such that the toxicity of the gel or its components is minimized (Tallard and Caron, 1977a).

Polyester Grouts--Polyester grouts have been in use since the 1960's. They have been used in a variety of construction applications, principally to treat cracks in buildings and structures (Tallard and Caron, 1977a). These grouts have also been used in mines as well as to stabilize and strengthen porous and fissured rock (Tallard and Caron, 1977a; Office of the Chief of Engineers, 1973). Polyester grouts have been used infrequently to treat sand (Tallard and Caron, 1977a).

The long-term behavior of polyester grouts is reported to be good, but there is a long-term risk of hydrolysis particularly in alkaline media (Tallard and Caron, 1977a). Furthermore, these gels shrink as much as 10 percent during curing (Office of the Chief of Engineers, 1973).

The components of the polyester grouts are toxic and often require special handling during grout preparation. After polymerization, the risks are lower although unreacted grout constituents may leach out (Tallard and Caron, 1977a).

7.2.2.1.6 Other Grouts

There are several other grout types that have limited application in soil and rock. The two major types are lignochrome and furan grouts.

Lignochrome grouts are also referred to as lignosulfonate or chromelignin grouts. This type of grout consists of a lignin-containing material and a hexavalent chromium salt (Kirk-Othmer, 1979; Ingles and Metcalf, 1973).

Calcium lignosulfates provide better waterproofing and stability than sodium lignosulfates (Rogoshewski, et al., 1980). Ammonium lignosulfonate is also used occasionally (Sommerer and Kitchens, 1980). Potassium dichromate may be used as the hexavalent chrome salt (Ingles and Metcalf, 1973).

Lignochrome grouts typically have low viscosity (2 to 15 centipoise) and moderately short setting times (3 to 300 minutes). The durability of these grouts is questionable. Their strength generally decreases over time in water-saturated environments (Kirk-Othmer, 1979). Also, chromium can be leached from the set grout depending on the age of the grout, the chrome-lignin ratio, the acidity (pH), and the curing time (Sommerer and Kitchens, 1980; Office of the Chief of Engineers, 1973).

The application of lignochrome grouts is limited because the chromium salts used in lignochrome grouts are toxic and the lignin materials can cause skin irritation. Lignochrome grouts should not be used with Portland cement because the pH of the materials conflict (Kirk-Othmer, 1979). Furthermore, lignochrome grouts are not compatible with fly ash because the fly ash's alkalinity can cause trivalent chromium to precipitate from the dichromate catalyst (Chung, 1973).

Lignochrome grouts have been used primarily for water cut-off and consolidation of fine, granular soils (Office of Chief of Engineers, 1973). However, they can be used in sands with a permeability between 10^2 to 1 ft/day (10^{-1} to 10^{-3} cm/sec) (Sommerer and Kitchens, 1980). Although, lignochrome grouts had been used in water cut-off applications for dams, their use is rare and they are not manufactured in the U.S. (Engineering News Record, 1953).

Furan grouts consist of simple polymers of furfuryl alcohol dissolved in excess furfuryl alcohol. They are used primarily as mortars in which the liquid resin is mixed with an inert powder filler (usually carbon) containing an acid catalyst. This catalyst promotes further polymerization to form a cross-linked, infusible material (Boova, 1977). Furan mortar grouts have excellent resistance to a broad range of chemicals including organic and inorganic acids, alkalies, salts, greases, and solvents (Boova, 1977).

However, in general they are not recommended for use with oxidizing acids or with chromic acid (20%), nitric acid, sodium hypochlorite (10%), or concentrated sulfuric acid (Boova, 1977; ASTM, 1982).

Furan resin grouts are used primarily in applications where resistance to corrosive media is important. They are also used for polymer concrete, in which alkali-free aggregate (silica or quartz) is bonded with furan resins and cured with an acidic catalyst. This concrete is resistant to acids, salts, solvents, and bases (Modern Plastics Encyclopedia, 1981).

Several other types of grouts have been referenced with regard to soil applications, the majority of which are polymers. One such grout, Polythixon FRD®, is an oil-based unsaturated fatty acid polymer. Polythixon FRD has low viscosity (10 to 80 centipoise) and a gel time of 25 to 360 minutes. This grout is recommended for high strength consolidation rather than waterproofing applications (Neelands and James, 1963).

Another polymer grout is PWG® sealant, a polymerized cross-linked gel of an unspecified polymer. This grout has a very low viscosity (1.5 centipoise) and a very short set time (several seconds to a few minutes). The set gel is insoluble in water, kerosene, and oil, and impermeable to water, oil, and gas. If the gel dehydrates, it can rehydrate in the presence of water to regain its original size. In addition, this gel may undergo "wicking," i.e., if one face of the gel dehydrates, moisture can move from the hydrated face to the dehydrated face (Lenahan, 1973).

Aniline-furfural resins may be used for the stabilization of cohesionless sand. These resins are catalyzed by pentachlorophenol or ferric chloride (Bowen, 1981). Emulsions such as latex and salt water, styrene butadiene latex, and pitch polyurethane mixtures may also be used as grouts (Bowen, 1981). Base-cured materials have been investigated for use as grouts, however, these materials do not have low viscosities or other characteristics that lend them to pumping or injection at pressures (Rensvold, 1968).

7.2.2.2 Grouting Preparation

This section discusses some of the considerations that must be addressed prior to actual grout injection. Because grouting is seldom considered for restricting leachate flow in unconsolidated materials, particular emphasis is placed on preparation for rock grouting. Many of these concerns also apply to soil grouting.

As with other types of barrier construction, the ultimate success of a grouting project depends on thorough site characterization. The ability to seal water bearing voids or zones is dependent on being able to locate them. In many remedial grouting operations, only a small portion of the rock mass will transport water and must be sealed. Consequently, the exploratory investigation must be very thorough. Detailed geologic mapping of the site, aided by remote sensing techniques and extensive rock coring, is required. Even with extensive investigation, the complexity of groundwater flow in fractured and fissured bedrock can make a grouting project impossible to plan in advance.

Based on the background and exploratory data, the location for a pattern of primary injection holes is chosen and injection at one or more zones is identified. The first few primary holes are then drilled and pressure washed with water and air (Millet and Engelhardt, 1982). This step removes drill cuttings and other debris from the hole to allow better grout penetration. Each hole is then pressure tested, often using a non-setting fluid of the same viscosity as the grout to be used. These tests are used to determine the initial grout mixture and are often conducted using the grout plant and other equipment to be used for the actual grouting (Millet and Engelhardt, 1982; Karol, 1982).

Each zone within each primary hole is then injected with the grout mixture until a predetermined amount is pumped (grout take), or a predetermined flow rate at maximum allowable pressure is reached. Maximum allowable pressure is typically around 1 pound per square inch (psi) per foot of overburden (Millet and Engelhardt, 1982). Data from the drilling and

injection of the first primary holes is analyzed, and if necessary, the grout mixture or injection pressure modified before completing the remaining primary holes. Following completion of the primary hole grouting, the program is again analyzed, necessary changes made, and a pattern of more closely spaced secondary holes drilled and injected.

The analysis and evaluation of the completed grouting becomes, in essence, another pressure test. Close quality control during drilling and grouting identifies areas that require tertiary hole grouting to complete sealing. Such areas are identified by faster than expected drilling rates and higher than expected grout takes (Millet and Engelhardt, 1982). For a successful grouting program, each hole series (i.e., primary, secondary) will have lower grout takes than the previous one. Many projects will require that proof holes be drilled and injected. A very low grout take on tertiary or proof holes indicates that most voids are grout filled and the grouting program was successful.

A variety of methods are in use for actual grout injection, and many types of grout are in use. The following sections describe grout injection techniques and selection techniques.

7.2.2.3 Grout Injection

Grouting methods combine two processes--mixing and injection. Chemical grouts are prepared by either batch mixing or by continuous mixing systems based on metering or proportioning pumps. Continuous mixing systems permit better control over the injection process since short gel times can be used. Typical gel times used with batch systems are several hours, whereas gel times used with continuously mixed systems are usually 10 to 20 minutes. Thus, the formation of large pools of ungelled grout in the ground is avoided. Additionally, long gel time grouts can become diluted and wash away before gellation occurs. Batch mixing can probably be used without difficulty in most soil grouting projects, but greater control is afforded by the use of short gel times (Hayward Baker et al., 1980). Mixing methods are dependent upon the size of the project in relationship to the grouting materials used

and the equipment available. A larger project would most likely require a continuous mixing system as opposed to a batch system to ensure that correct grout specifications are met throughout the site in a relatively short period of time.

Construction of a grout barrier is accomplished by pressure injecting the grouting material through a pipe into the strata to be waterproofed. The injection points are usually arranged in a triple line of primary and secondary grout holes (Figure 7-14). A predetermined quantity of grout is pumped into the primary holes. After the grout in the primary holes has had time to gel, the secondary holes are injected. The secondary grout holes are intended to fill in any gaps left by the primary grout injection (Hayward Baker et al., 1980). The primary holes are typically spaced at 20- to 40-foot intervals (Guertin and McTigue, 1982).

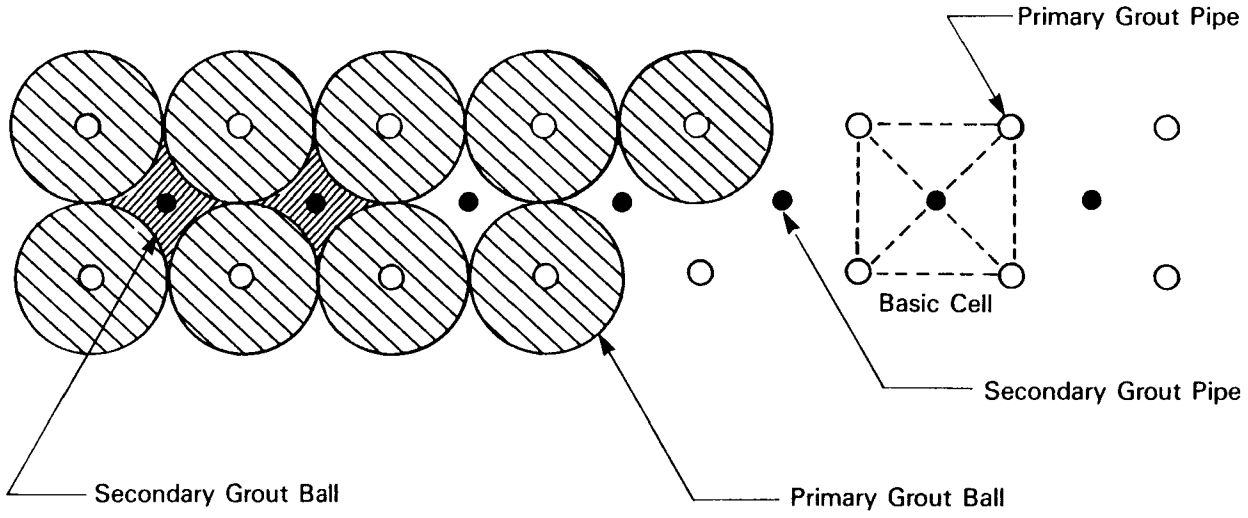
There are several basic techniques that are utilized to form the grout wall. These include (Hayward Baker et al., 1980; Guertin and McTigue, 1982):

- Stage-up method
- Stage-down method
- Grout-port method
- Vibrating beam method.

In the stage-up method, the borehole is drilled to the full depth of the wall prior to grout injection. The drill is withdrawn one "stage," leaving several feet of borehole exposed. Grout is then injected into this length of open borehole until the desired volume has been injected. When injection is complete the drill is withdrawn further and the next stage is injected (Hayward Baker et al., 1980).

Stage-down grouting differs from stage-up grouting in that the injections are made from the top down. Thus, the borehole is drilled through the first zone that is to be grouted, the drill is withdrawn, and the grout injected. Upon completion of the injection, the borehole is redrilled through the

FIGURE 7-14.
GROUTING PIPE LAYOUT FOR CONSTRUCTING A BARRIER WALL
(HAYWARD BAKER et al., 1980 AS CITED BY SPOONER et al, 1984b)



grouted layer into the next zone to be grouted and the process is repeated (Guertin and McTigue, 1982).

The grout-port method utilizes a slotted injection pipe that has been sealed into the borehole with a brittle Portland cement and clay mortar jacket. Rubber sleeves cover the outside of each slit (or port) permitting grout to flow only out of the pipe. The injection process begins by isolating the grout port in the zone to be injected using a double packer. A brief pulse of high pressure water is injected into the port to rupture the mortar jacket. Grout which is pumped between the double packers, passes through the ports in the pipe, under the rubber sleeve, and out through the cracked mortar jacket into the soil (Guertin and McTigue, 1982). This grouting process is illustrated in Figure 7-15 and a detailed diagram of the double packer is shown in Figure 7-16.

The vibrating beam method is not an injection technique as described above, but instead is a way of placing grout in such a way as to generate a wall. In this method, an I-beam is vibrated into the soil to the desired depth and then raised at a controlled rate. As the beam is raised, grout is pumped through a set of nozzles mounted in the beam's base filling the newly formed cavity. When the cavity is completely filled, the beam is moved along the direction of the wall, leaving a suitable overlap to ensure continuity (Harr, Diamond and Schmednecht, unpublished). Figure 7-17 shows the steps involved in forming a vibrating beam cut-off wall.

7.2.2.4 Grout Selection

The physical and chemical properties of grouts that determine the type that should be used at a specific site include (Herndon and Lenahan, 1976a):

- Viscosity
- Setting time
- Permeability
- Strength

FIGURE 7-15.
STAGE-UP GROUT-PORT INJECTION PROCESS
(HAYWARD BAKER et al., 1980)

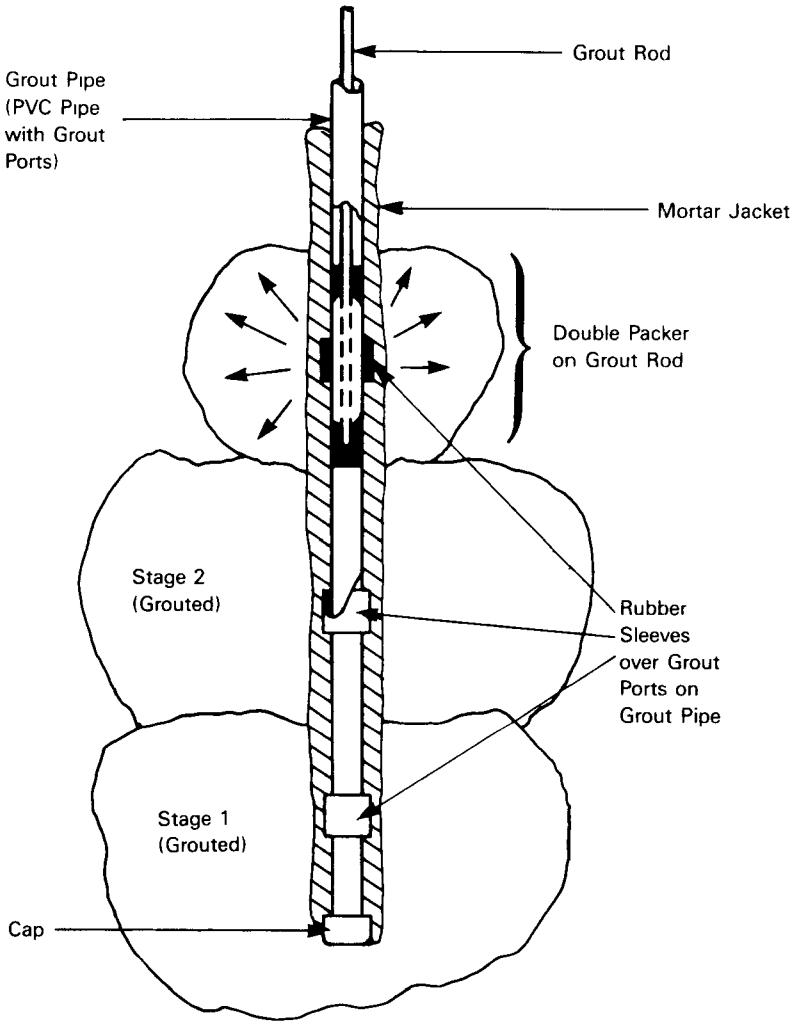


FIGURE 7-16.
DIAGRAM OF A DOUBLE PACKER USED IN GROUT PORT INJECTION
(GUERTIN AND McTIGUE, 1982)

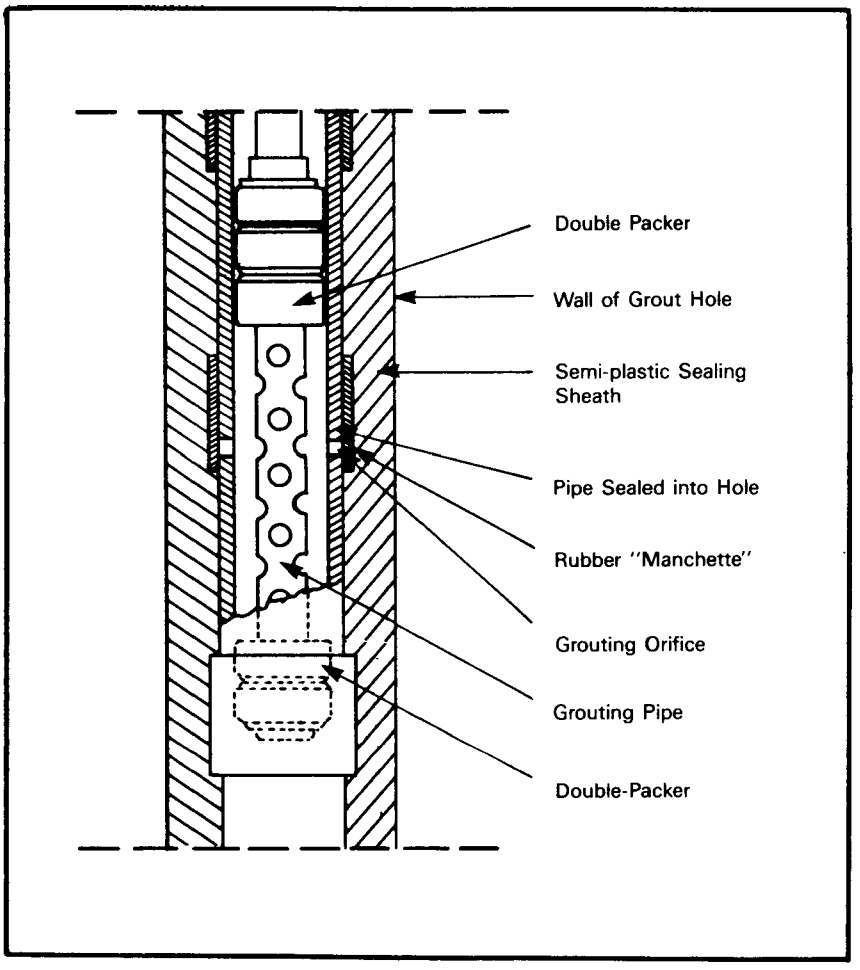
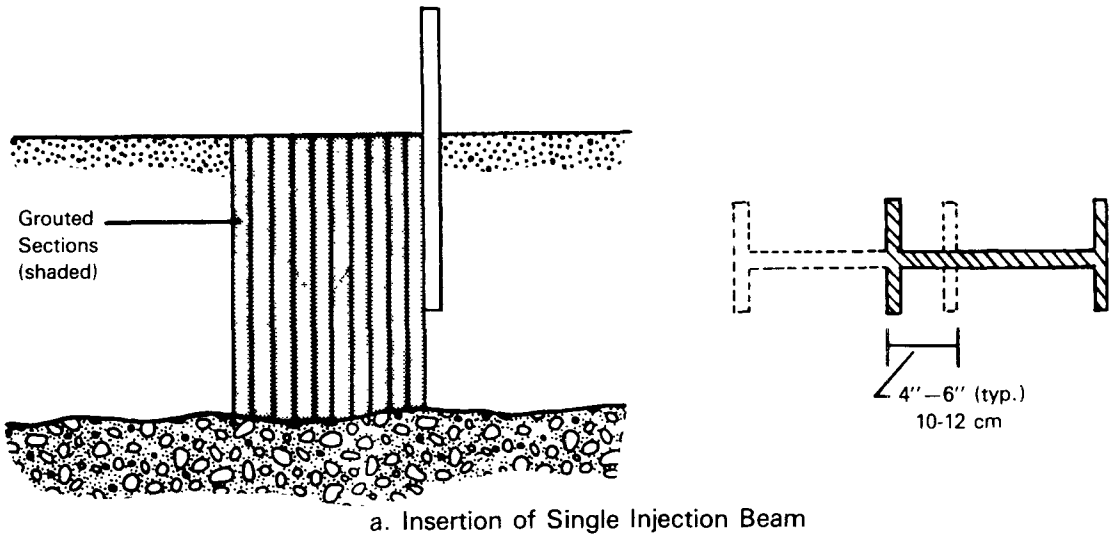
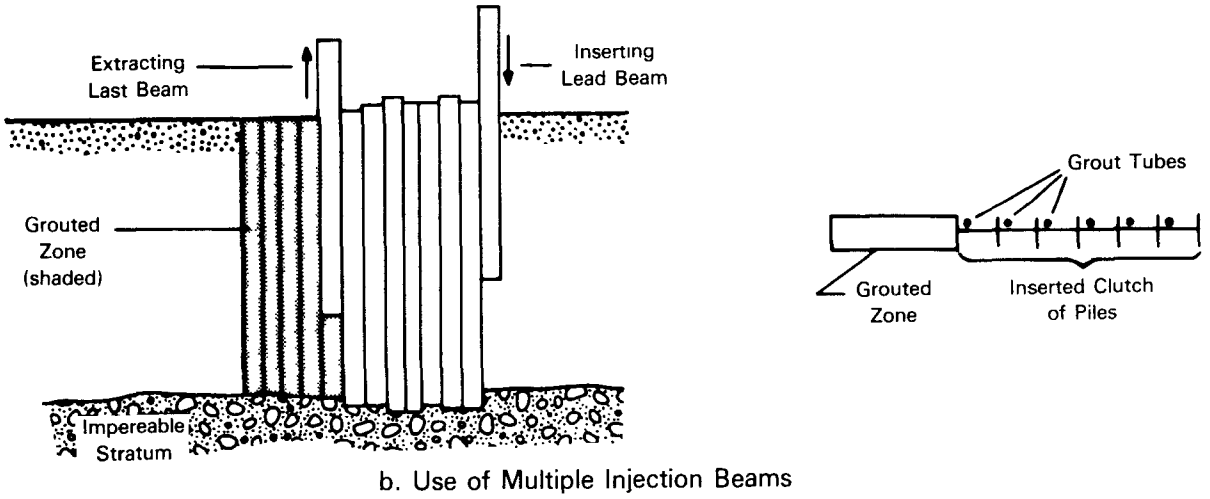


FIGURE 7-17.
VIBRATING BEAM GROUP PLACEMENT PROCESS
(BOYES, 1971, AS CITED BY GUERTIN AND McTIGUE, 1982)



a. Insertion of Single Injection Beam



b. Use of Multiple Injection Beams

- Stability
- Toxicity.

The viscosity of a grout will control its ability to penetrate the voids in the soil or rock structure (Tallard and Caron, 1977b). Grout viscosity can also contribute to the effectiveness of the groundwater cut-off wall. For example, if the initial viscosity is too low, all of the soil or rock voids may not be completely filled and holes in the wall may develop. Conversely, if the viscosity is too high, the grout may not be able to penetrate all of the strata, potentially resulting in wall openings (Tallard and Caron, 1977b). Low viscosity grouts, such as chemical grouts, can be used in fine soils having a permeability greater than 10^{-2} ft/day (10^{-5} cm/sec). High viscosity grouts can generally only be used in coarse-grained soils or in fractured rock having a coefficient of permeability greater than 10 ft/day (10^{-2} cm/sec) (Sommerer and Kitchens, 1980).

After a grout is mixed, its viscosity will generally increase with time until it gels or solidifies. This change in viscosity will control both the time period during which the grout can be pumped and the distance which the grout can travel through the soil or rock formation. If the setting time of the grout is too short, it will be too viscous to move far enough into the soil or rock structure. Conversely, if the grout has too long a setting time, the grout may filter down past the target zone (Tallard and Caron, 1977b). For suspension grouts, the setting time is a function of the water to particulate ratio and the temperature. As the water to particulate ratio is increased, the grout viscosity, set time, and pumpability will also increase. For chemical grouts, the set time will be a function of the ratio of chemicals used and the temperature (Sommerer and Kitchens, 1980).

Grout strength can determine the long term mechanical stability of a grout cut-off wall. The grouted soil or rock mass must have enough strength to resist hydrostatic forces (Tallard and Caron, 1977b). The long-term durability of the wall is critical if the groundwater barrier is to be maintained over extended periods of time. Any chemicals contained within the soil mass or in the groundwater must not affect the grout's set time or its

ability to set fully. If the set time is varied, the finished wall may not be continuous or will contain weak spots. Also, the grout must be able to withstand long-term contact with chemicals in the groundwater without any deterioration (Sommerer and Kitchens, 1980).

Since grouts used to form waterproof barriers come in contact with groundwater, the toxic properties of their ingredients should be considered. The toxicity of the grout has to be examined from two aspects--the toxicity of the grout's components and the toxicity of the hardened grout (Sommerer and Kitchens, 1980; Tallard and Caron, 1977b). If the compounds contained in the grout do not react fully, they can contaminate the groundwater. This happened in Japan in 1974 when groundwater was contaminated by acrylamide monomer from a chemical grout used in the construction of a sewer system. This contamination led to subacute poisoning of local inhabitants (Ando and Makita, 1977). Hardened grout can also release potentially toxic compounds through syneresis of the grout (Sommerer and Kitchens, 1980).

The selection of the proper grout will depend not only on the physical and chemical properties of the soil or rock strata but also on such factors as grout availability, costs, local experience, and groundwater flow and chemistry (Guertin and McTigue, 1982). Thus, grout selection is a very site specific process. In the selection of a grout to be used at hazardous waste disposal sites, the most important factor to be considered is its compatability with chemicals found in the groundwater and in the soil at the disposal site.

7.2.3 Grouting Costs

The drawbacks to grout usage stem from the fact that grouting is conducted by a limited number of firms in the United States and involves special techniques and equipment. In most cases, a substantial equipment mobilization fee must be paid. Equally important is the cost of characterization and testing that must be performed to ensure effective grouting. A final consideration is the cost of the grout itself. Approximate costs of grouts are found in Table 7-10.

TABLE 7-10
APPROXIMATE COSTS OF GROUTS (Spooner et al., 1984)

Grout type	Approximate cost $/gallon of solution (1979)
Portland cement	0.95
Bentonite	1.25
Silicate - 20%	1.25
- 30%	2.10
- 40%	2.75
Epoxy	$30.00
Acrylamide	6.65
Urea formaldehyde	5.70

Individual grout costs can show a wide variation, whereas, grout costs for a complete job show much less variation. This is because the cheaper, particulate grouts are used to seal large voids, thus using more grout, while the more expensive, chemical grouts are commonly used to seal small voids.

CHAPTER 8

INNOVATIVE TECHNOLOGIES

8.1 Introduction

The techniques presented in this chapter are relatively new or novel ideas that have seen limited use at hazardous waste sites with contaminated groundwaters, hence the use of the term "innovative technologies." The innovative technologies presented in this chapter include bioreclamation, in situ chemical treatment (e.g., soil flushing, polymerization, permeable treatment beds), and block displacement. Because this group of technologies has not been widely used or accepted, the amount of information available for each varies widely. Therefore, attempts have not been made to make the contents of this chapter parallel the other technology volumes (Chapters 5, 6, and 7). The reader is advised that because these technologies are relatively untested and unproven compared to the more established techniques presented previously, they will require more extensive research and development before they can be confidently implemented at hazardous waste sites.

Most of the techniques presented originated as methods to treat contaminated surface waters or spill sites, and have been modified for the in situ treatment of groundwater. In general, these technologies have many limitations associated with their use, and they may be most useful in the treatment of groundwater residuals and in cases where a prevalence of one type of contaminant exists, such as a biodegradable organic compound or a toxic metal.

Although bioreclamation techniques may be the furthest along in development and testing to fulfill this need, using any one of these innovative techniques as a sole method to control or mitigate the migration of a leachate

plume probably is not warranted at this time. Most of these technologies could be used with one of the more accepted technologies to enhance or accelerate contaminant recovery. The result of the combined technologies could be faster cleanups at lowered costs. Even within the group of innovative technologies, a combination of these technologies may prove to be more effective than a single technology alone. For example, oxidation (a chemical treatment) may be used to break down high molecular weight organic compounds so that they are more readily biodegraded.

This chapter is divided into three main sections: Bioreclamation, Chemical Treatment, and Block Displacement. Bioreclamation, presented first because it appears to have the greatest potential for treating contaminant plumes, is stirring the most interest and controversy, and is furthest along in development and testing.

The bioreclamation process involves the addition of nutrients and oxygen to the contaminated groundwater so that either the naturally occurring soil bacteria or introduced mutant species are provided with an optimized environment for the biodegradation of the contaminants in the plume. Bioreclamation techniques for cleaning up contaminated groundwaters originated with work performed to clean up gasoline. Since then, the technique has been used at numerous sites with gasoline contaminated groundwaters with varying degrees of success. With the discovery of naturally occurring microorganisms that are capable of degrading refractory chlorinated organics and the advent of genetically engineered microorganisms developed for specific compounds and chemical classes, bioreclamation is expected to see increasing applications to hazardous waste site cleanups and spills.

The second section of the chapter deals with those techniques that require the injection of a specific chemical or chemicals into the plume to either degrade, immobilize, or increase the mobility of the contaminants. The techniques reviewed include soil flushing (solution mining), oxidation/reduction, precipitation/polymerization, neutralization/hydrolysis, and permeable treatment beds.

Soil flushing or solution mining is a technique used to remove residual contamination from soils. The method involves the introduction of a solvent to the soil to solubilize substances adsorbed to the soil surface. Solvents that can be used are water, water plus a surfactant, dilute acids or bases, and complexing agents. Soil flushing has the potential to be used at hazardous waste sites in the removal of organics and metals from soils. Because soil flushing is used for removing residual contaminants from soils rather than from groundwater, it is not a true leachate plume control technique.

The remaining techniques--oxidation/reduction, precipitation/polymerization, neutralization/hydrolysis, and permeable treatment beds--involve injecting or placing specific chemicals in the groundwater to, in theory, destroy or otherwise render harmless contaminants in the plume. They are applicable only to specific chemical contaminants and have been proposed mainly for hazardous substance spill cleanup. Actual use of these chemical techniques at hazardous waste sites with contaminated groundwater has been rare.

Oxidation/reduction involves the injection of an oxidizing or reducing substance into the contaminated groundwater. The oxidizing agent may alter the oxidation state of a metal or decompose an organic to a less toxic, more soluble, or more biodegradable form. Reducing agents alter the oxidation state of metals to a less toxic or soluble form, and they may cause a precipitate to be formed.

Precipitation involves the introduction of a substance which will form an insoluble precipitate with the contaminant substance (usually a metal), thereby removing the substance from the groundwater flow regime. Similarly, polymerization involves injection of a catalyst into a groundwater plume to cause polymerization of an organic monomer. The polymerization reaction transforms the once fluid substance into a gel-like, non-mobile mass.

Neutralization/hydrolysis involves injecting acids or bases into the groundwater plume to adjust the pH of the groundwater. This method may be

useful as a preliminary treatment for bioreclamation, as a means to degrade organic chemicals, as a means to prevent toxic gas formation (such as cyanide), as a preparation for other chemical treatments (polymerization under acid conditions), or as a post-treatment technique to restore natural groundwater pH.

Permeable treatment beds are essentially excavated trenches placed perpendicular to groundwater flow and filled with an appropriate material to treat the plume as it flows through the material. Some of the materials that may be used in the treatment bed are limestone, crushed shell, activated carbon, glauconitic greensands, and synthetic ion exchange resins. Permeable treatment beds have the potential to reduce the quantities of contaminants present in leachate plumes.

The final section in this chapter deals with a relatively new technique that had its origin in mining: block displacement. This technique involves vertically lifting a large mass of earth and isolating the mass completely from the groundwater flow regime using a physical barrier. The barrier is formed by pumping slurry into a series of notched injection holes which are physically connected underground. As the slurry is pumped in, the soil mass is displaced upward resulting in the block's displacement.

Prior to discussing the individual in situ treatment techniques, a general presentation of the types of wastes that are found at hazardous waste sites and the potentially useful in situ treatments that could be utilized may aid the reader. A survey of the 114 top priority Superfund sites (EPA, 1981) was performed by Ellis, et al. (1982) to determine the contaminants that are likely to be found and be amenable to in situ treatment. Table 8-1 presents the results of this survey. The occurrence of heavy metals, acids, alkalis, aromatics, phenols, PCB's, and other halogenated hydrocarbons is widespread among sites.

Ellis, et al. (1982) went further and suggested potential in situ treatments for each of the chemical groups based on available literature. This information is presented in Table 8-2. The table lists the five

TABLE 8-1.

HAZARDOUS SOIL CONTAMINANTS AT SUPERFUND SITES (Ellis, et al., 1982)

Contaminants	Number of Sites	Total Sites	Examples
Heavy Metal Wastes		47	
Chromium	9		
Arsenic	8		
Lead	7		
Zinc	5		
Cadmium	4		
Iron	3		
Copper	2		
Mercury	2		
Selenium	2		
Nickel	1		
Vanadium	1		
Fly ash	1		
Plating wastes	2		
Other Inorganics		26	
Cyanides	6		
Acids	7		sulfuric acid
Alkalis	6		lime, ammonia
Radioactive wastes	3		uranium mining and purification wastes, radium, tritium
Miscellaneous	4		beryllium, boron hydride, sulfides, asbestos
Hydrophobic Organics		38	
Polychlorinated biphenyls	15		
Oil, grease	11		
Volatile hydrocarbons	6		Varsol, hexane
Chlorinated hydrocarbon pesticides	5		endrin, lindane, DDT, 2,4,5-T, dieldrin
Polynuclear aromatics	1		

(Continued)

TABLE 8-1. (continued)

Contaminants	Number of Sites	Total Sites	Examples
Slightly Water Soluble Organics		64	
Aromatics			
Benzene	9		
Toluene	8		
Xylene	5		
Other aromatics	3		styrene, napthalene
Halogenated hydrocarbons			
Trichloroethylene	11		
Ethylene dichloride	6		
Vinyl chloride	4		
Methylene chloride	3		
Other halogenated hydrocarbons	15		chloroform, trichloroethane, tetrachloroethylene, trichlorofluoromethane
Hydrophilic Organics		20	
Alcohols	4		methyl, isopropyl, butyl
Phenols	12		picric acid, pentachlorophenol, creosote
Other hydrophilics	4		dioxane, bis(2-chloroethyl) ether, urethane, rocket fuel
Organic solvents (unspecified) and other organics		30	dioxin, dioxane, dyes, pigments, inks, paints, nitrobenzene

TABLE 8-2.

POTENTIAL USEFUL SOIL IN SITU TREATMENT PROCESSES (Ellis, et al., 1982)

Hazardous Waste	In Situ Treatment	Selected References
Heavy metals	• Sulfide precipitation	Pohland, et al. (1981, 1982) U.S. EPA (1979) Huibregtse and Kastman (1978)
	• Fixation with municipal refuse	Myers, et al. (1980) Phung, et al. (1982) Kinman, et al. (1982) Jones and Malone (1982)
	• Aqueous leaching to dissolve and/or flush with injection/recovery system	Epstein, et al. (1978) Fuller and Korte (1976) Huibregtse et al. (1978)
Hydrophobic organics	• Water injection/recovery system with:	
	Surfactants	Hill, et al. (1973)
	Oxidizing reactants (NaOCl, H_2O_2)	Chou, et al. (1982)
	Micellar-polymer, e.g., petroleum sulfonates and polyacrylamides	Klins, et al. (1976)
	Solvents	Anderson, et al. (1982) Griffin and Chou (1980)
	• Sodium polyethylene glycol reactant	Pytlewski, et al. (1980)
	• Clay immobilization: injection/ fixation process	

(continued)

TABLE 8-2. (continued)

Hazardous Waste	In Situ Treatment	Selected References
Hydrophobic organics (continued)	• Biostimulation	Wilkinson, et al. (1978) Kobayashi and Rittman (1982) Zitrides (1982)
Slightly water-soluble organics (e.g., benzene, toluene, trichloroethylene)	• Water injection/recovery system with surfactants	Texas Research Institute (1979, 1982b)
	• Chemical and aerobic oxidation	Dragun and Helling (1982) Texas Research Institute (1982a) Kinman, et al. (1982)
	• Anaerobic oxidation	
Hydrophilic organics (e.g., aniline, phenol)	• Water injection/recovery system with: pH adjustment (buffering)	Laguros and Robertson (1978)
	• Biostimulation	Wilkinson (1978), Kobayashi and Rittmann (1982), Zitrides (1982)

categories of hazardous waste materials previously identified as significant, several potential in situ treatment techniques for each hazardous waste category, and key citations to the source of information. These information sources should be referred to for elucidation of the processes and situations under which they were utilized.

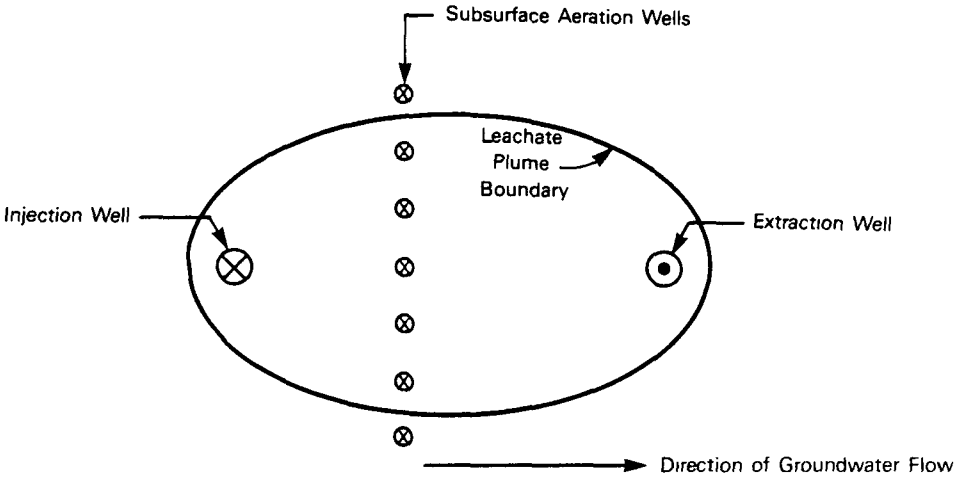
8.2 Bioreclamation

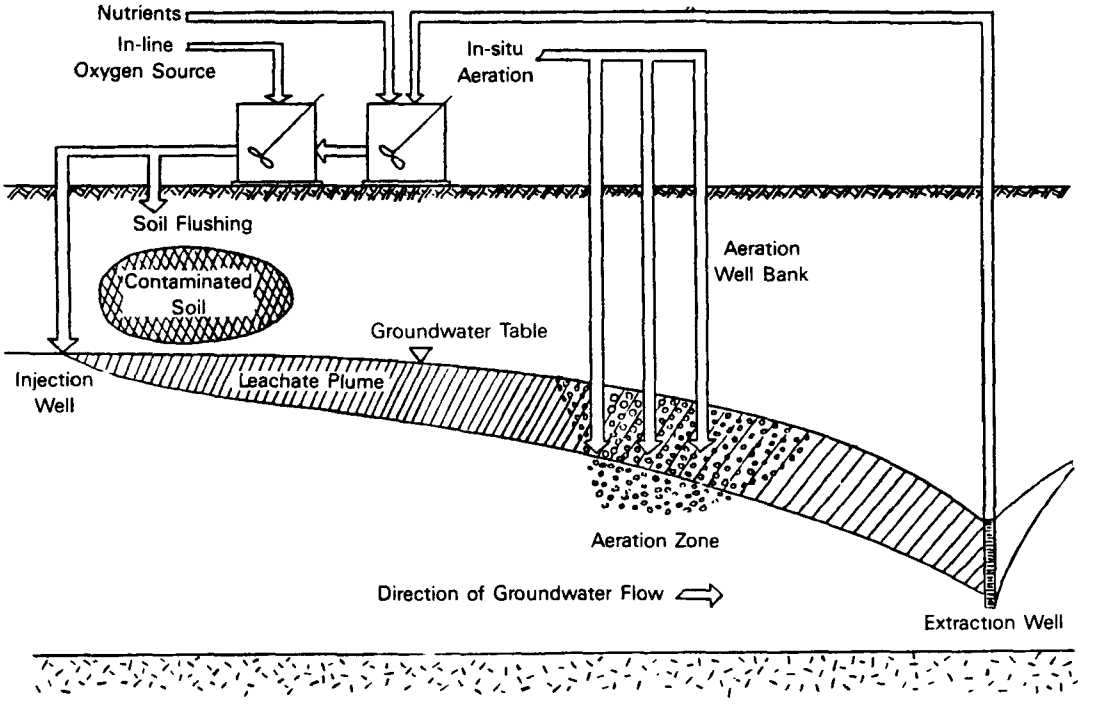
Bioreclamation is an in situ groundwater treatment technique based on the concept of enhancing microbial activity by altering the physical or chemical environment of the aquifer. Methods currently under investigation include aeration and the addition of nutrients to accelerate the biodegradation of groundwater contaminants. Typically, in bioreclamation, groundwater extraction wells are strategically placed downgradient to control the migration of the contaminant plume. Contaminated groundwater pumped to the surface is mixed with nutrients needed for organism growth, and the treated contaminated groundwater is reinjected upgradient. Specialized microorganisms may be injected along with the nutrients, or the indigenous soil bacteria may be stimulated with nutrients to adapt to the groundwater contaminants. The groundwater is aerated above ground by in-line injection or underground using a series of wells. A simplified view of the process is shown in Figure 8-1.

The bioreclamation process, as described above, results in aerobic decomposition of groundwater contaminants in the subsurface. Bioreclamation claims a number of advantages such as low capital costs, minimal worker safety and air pollution control requirements, and contaminant destruction rather than containment.

The bioreclamation technique has been used by a number of specialized firms to treat contaminated groundwater plumes resulting from underground gasoline and hydrocarbon leaks. The technique has not yet been demonstrated for groundwater treatment at an uncontrolled hazardous waste site. However, bioreclamation's potential to treat organics which are or can be made biodegradable in contaminated groundwaters establishes it as a viable technique.

FIGURE 8-1.
SIMPLIFIED VIEW OF GROUNDWATER BIORECLAMATION





8.2.1 Applications and Limitations

Bioreclamation techniques are applicable to leachate plumes which contain biodegradable organics. Relative biodegradabilities of selected compounds have been reported based on ratios of laboratory parameters associated with the oxygen requirement for decomposition. These include 5-day or 21-day biochemical oxygen demand (BOD_5, BOD_{21}), chemical oxygen demand (COD), and ultimate oxygen demand (UOD). Table 8-3 presents relative biodegradabilities by adapted sludge cultures of various compounds in terms of a BOD_5/COD ratio. A higher BOD_5/COD means a higher relative biodegradability. Table 8-4 presents relative biodegradabilities in terms of an alternate ratio, i.e., the BOD_{21}/UOD ratio, termed refractory index (RI). Again, the higher the RI, the more biodegradable. The data presented in Tables 8-3 and 8-4 provide a general guideline for determining the biodegradability of certain compounds, but do not preclude the need for a laboratory treatability study. The degradative capability of different microbial populations can vary considerably; and so, to accurately predict the treatability of a specific leachate plume, site-specific laboratory treatability studies are essential. Additionally, many compounds previously thought to be refractory have been found to be degraded by certain naturally-occurring bacteria as well as constructed strains.

In addition to the biodegradability of the leachate plume components, the applicability of bioreclamation also depends on groundwater temperature. The temperature range for optimal organism growth in aerobic biological wastewater treatment processes has been found to range from 68°F to 99°F (20°C to 37°C). Figure 8-2 gives groundwater temperatures throughout the United States. A comparison of Figure 8-2 to the optimal temperature ranges for aerobic biological wastewater treatment processes suggests that only the southern portion of the country is within the optimal range. In areas of the country where the groundwater temperatures are below optimum, slower rates of biodegradation should be expected. Studies of 2,4-D breakdown in surface water indicated that a 75 percent reduction in degradation rates occurred when temperature was reduced by 64°F (Lyman, et al., 1982). This reduction in degradation rates may be attributable to a drop in temperature from that which

TABLE 8-3. BOD_5/COD RATIOS FOR VARIOUS ORGANIC COMPOUNDS[1]
(Lyman, et al., 1982)

Compound	Ratio
Relatively Non-degradable	
Butane	0
Butylene	0
Carbon tetrachloride	0
Chloroform	0
1,4-Dioxane	0
Ethane	0
Heptane	0
Hexane	0
Isobutane	0
Isobutylene	0
Liquefied natural gas	0
Liquefied petroleum gas	0
Methane	0
Methyl bromide	0
Methyl chloride	0
Monochlorodifluoromethane	0
Nitrobenzene	0
Propane	0
Propylene	0
Propylene oxide	0
Tetrachloroethylene	0
Tetrahydronaphthalene	0
1-Pentene	<0.002
Ethylene dichloride	0.002
1-Octene	>0.003
Morpholine	<0.004
Ethylenediaminetetracetic acid	0.005
Triethanolamine	<0.006
o-Xylene	<0.008
m-Xylene	<0.008
Ethylbenzene	<0.009
Moderately degradable	
Ethyl ether	0.012
Sodium alkylbenxenesulfonates	0.017
Monoisopropanoalmine	<0.02
Gas oil (cracked)	0.02
Gasolines (various)	0.02
Mineral spirits	0.02
Cyclohexanol	0.03
Acrylonitrile	0.031
Nonanol	>0.033
Undecanol	<0.04
Methylethylpyridine	0.04-0.75
1-Hexane	<0.044
Methyl isobutylketone	<0.044
Diethanolamine	<0.049
Formic acid	0.05
Styrene	>0.06
Heptanol	<0.07
sec-Butyl acetate	0.07-0.23
n-Butyl acetate	0.07-0.24
Methyl alcohol	0.07-0.73
Acetonitrile	0.079
Ethylene glycol	0.081
Ethylene glycol monothyl ether	<0.09
Sodium cyanide	<0.09
Linear alcohols (12-15 carbons)	>0.09
Allyl alcohol	0.091
Dodecanol	0.097
Relatively Degradable	
Valeraldehyde	<0.10
n-Decyl alcohol	>0.10
p-Xylene	<0.11
Urea	0.11
Toluene	<0.12
Relatively Degradable (continued)	
Potassium cyanide	0.12
Isopropyl acetate	<0.13
Amyl acetate	0.13-0.34
Chlorobenzene	0.15
Jet fuels (various)	0.15
Kerosene	0.15
Range oil	0.15
Glycerine	<0.16
Adiponitrile	0.17
Furfural	0.17-0.46
2-Ethyl-3-propylacrolein	<0.19
Methylethylpyridine	<0.20
Vinyl acetate	<0.20
Diethylene glycol monomethyl ether	<0.20
Naphthalene (molten)	<0.20
Dibutyl phthalate	0.20
Hexanol	0.20
Soybean oil	0.20
Paraformaldehyde	0.20
n-Propyl alcohol	0.20-0.63
Methyl methacrylate	<0.24
Acrylic acid	0.26
Sodium alkyl sulfates	0.30
Triethylene glycol	0.31
Acetic acid	0.31-0.37
Acetic anhydride	>0.32
Ethylenediamine	<0.35
Formaldehyde solution	0.35
Ethyl acetate	<0.36
Octanol	0.37
Sorbitol	<0.38
Benzene	<0.39
n-Butyl alcohol	0.42-0.74
Propionaldehyde	<0.43
n-Butyraldehyde	<0.43
Ethyleneimine	0.46
Monoethanolamine	0.46
Pyridine	0.46-0.58
Dimethylformamide	0.48
Dextrose solution	0.50
Corn syrup	0.50
Maleic anhydride	>0.51
Propionic acid	0.52
Acetone	0.55
Aniline	0.56
Isopropyl alcohol	0.56
n-Amyl alcohol	0.57
Isoamyl alcohol	0.57
Cresols	0.57-0.68
Crotonaldehyde	<0.58
Phthalic anhydride	0.58
Benzaldehyde	0.62
Isobutyl alcohol	0.63
2,4-Dichlorophenol	0.78
Tallow	0.80
Phenol	0.81
Benzoic acid	0.84
Carbolic acid	0.84
Methyl ethyl ketone	0.88
Benzoyl chloride	0.94
Hydrazine	1.0
Oxalic acid	1.1

[1] BOD_5 values were not measured under the same conditions for all chemicals

TABLE 8-4.

REFRACTORY INDICES FOR VARIOUS ORGANIC COMPOUNDS
(Adapted from Lyman, et al., 1982)

Compound	RI
High Degradability	
Biphenyl	1.14
Antifreeze	1.12
Sevin	1.0
d-Glutamic acid	1.00
d-Glucose	0.93
l-Valine	0.93
Acetone	0.93, 0.71
Phenol	0.87
Sodium butyrate	0.84
l-Aspartic acid	0.81
Sodium propionate	0.80
Propylene glycol	0.78, 0.52
Ethylene glycol	0.76
Medium-High Degradability	
Potato Starch	0.72, 0.64
l-Arginine	0.65
Acetic acid	0.61
Aniline	0.58
Soluble starch	0.54
l-Histidine	0.52
l-Lysine	0.52
Hydroquinone	0.41
Low Degradability	
Benzene	0.23
Gasoline	0.21
Adenine	0.14, 0.12
Vinyl chloride	0
Carboxymethyl cellulose	0
Humics	0
DDT with carrier	0
p-Chlorophenol	0
Dichlorophenol	0
Bipyridine	0
Chloroform	0
Cyanuric Acid	0

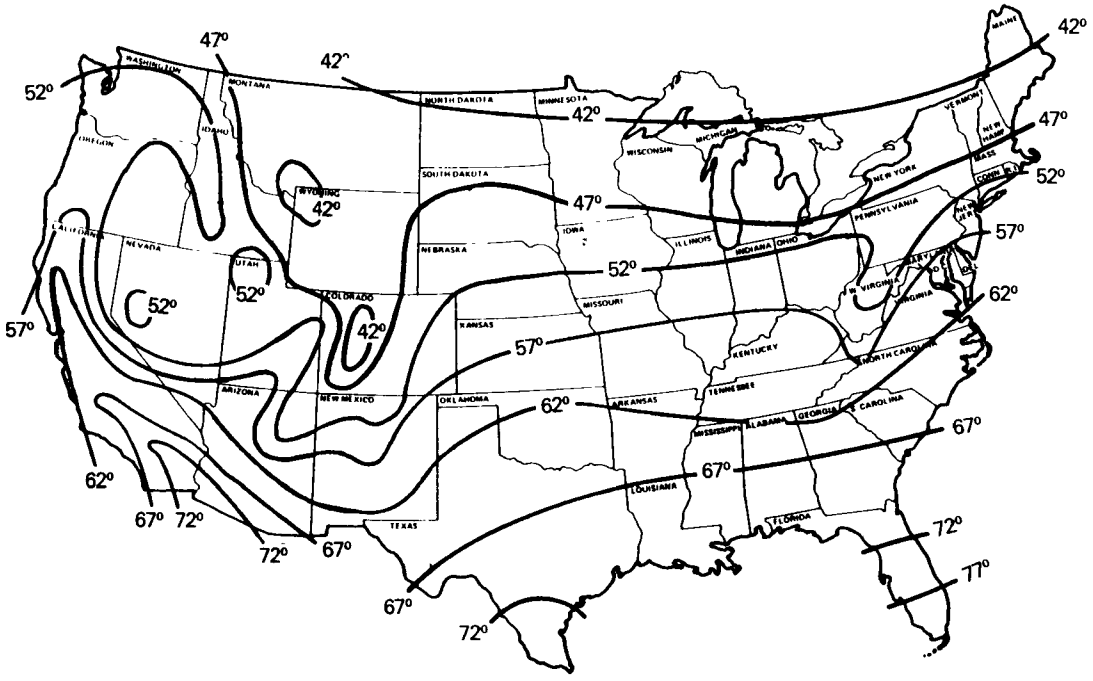


FIGURE 8-2.
TYPICAL GROUNDWATER TEMPERATURES (°F) AT 100 FT. DEPTH IN THE CONTERMINOUS UNITED STATES (JOHNSON DIVISION, UOP INC., 1975)

The organism is accustomed. Ladd, et al. (1982) reported high heterotrophic activities in groundwater samples from Alberta, Canada. Therefore, bioreclamation may not be as applicable to sites in the extreme north because of reduced biodegradation rates caused by low groundwater temperatures. However, organisms adapted to cold water will still grow and metabolize at appreciable rates if stimulated with oxygen and nutrients.

Site geology is also an important factor that may affect the feasibility of using the bioreclamation process. Optimum geologic conditions for bioreclamation require substrata with moderate to high hydraulic conductivities. Substrate materials with the above hydraulic conductivity characteristics may include gravels, coarse sands, sandstones, and highly fractured rocks.

Other factors may also limit the applicability of bioreclamation. The pH of the leachate plume is one such factor. Microbial growth can occur within a relatively wide pH range, about 5.0 to 9.0. However, optimum growth occurs within a pH range of about 6.0 to 8.0, with slightly alkaline conditions being more favorable. Leachate plumes having pH values outside this range may not provide suitable conditions for rapid biodegradation of contaminants. However, in some cases adjusting pH in the process may be possible.

The availability of oxygen and nutrients for aerobic decomposition enhances to the bioreclamation process, and if not present, would limit biodegradation. Introducing enough soluble oxygen into the groundwater has been the major challenge in applying bioreclamation to leachate plumes. This is because the demand for oxygen is high in this process and the solubility of oxygen in groundwater is relatively low. The problem is compounded by the difficulty of distributing the gas within the substrate. The design section of this chapter presents several approaches to solving the oxygen availability problem.

8.2.2 Theory

The bioreclamation process is based on the capability of many species of bacteria to degrade a wide variety of organic compounds. Species have been

isolated which are capable of degrading hydrocarbons, aromatics, and halogenated aliphatics and aromatics. Naturally occurring species of Pseudomonas and Arthrobacter have been associated with the biodegradation of gasoline contaminated groundwater (Raymond, et al., 1976). Other major genera known to attack organic contaminants include Achromobacterium, Flavobacterium, and Nocardia (Kincannon, 1972). In general, more rapid and more complete biodegradation is carried out by microorganisms in an aerobic (oxygenated) environment. However, for certain compounds, most notably the halogenated alkanes and certain pesticides such as DDT, more rapid breakdown occurs in anaerobic (oxygen-free) environments (Suflita, et al., 1982; Bouwer and McCarty, 1983).

In the aerobic decomposition of organic matter, bacteria respire oxygen while catabolizing the organic compound, producing metabolic by-products, cell biomass, carbon dioxide, and water. The generalized process is given by the following equation:

$$\text{Bacteria} + O_2 + \text{Organics} + \text{Nutrients} \longrightarrow CO_2 + H_2O + \text{Byproducts} + \text{Cell Biomass}$$

Approximately 5 to 50 percent of the organic material metabolized will be transformed into cell biomass. The more refractive a compound, the less carbon there is available for cell growth. Therefore, an increase in cell number is directly related to the biodegradability of the compound.

Inorganic nutrients such as nitrogen, phosphorus, and trace elements are required for proper cell respiration, growth, and reproduction. Oxygen serves as the terminal electron acceptor in aerobic cell respiration, and thus, must be present for aerobic biodegradation. The organic matter, known as the substrate, is the organic carbon containing energy for the microorganisms.

8.2.2.1 Microbial Populations

In bioreclamation, specialized microbial populations are promoted to degrade the organic materials contaminating the groundwater. The total microbial population may originate from naturally occurring species in the

environment, enriched strains, or genetically manipulated organisms developed in the laboratory. Supporting evidence is available that naturally occurring species of bacteria survive in the soil subsurface and that these can be stimulated to proliferate with the introduction of oxygen and a suitable substrate. Also, introduced strains, developed in the laboratory, have been shown to be able to survive in the subsurface environment.

Many types of bacteria have been isolated from soil cores, subsurface oil, and water from deep wells (Dunlap and McNabb, 1973). The data suggest that subsurface microbial colonization may occur down to several thousand feet, however, the studies did not take into account the possibility of contamination from the surface during sampling. A number of other studies identified microbial populations in the lower unsaturated zone and in upper regions of the groundwater table. Sulfate-reducing bacteria were found in concentrations of 10^1 to 10^4 organisms per milliliter in well waters in Montana (Dockins, et al., 1980). Nitrate-reducing bacteria were found to depths of 120 feet (40 meters) in the unsaturated zone of the Chalk Formation in Great Britain in a concentration of 10^6 to 10^7 organisms per gram of soil (Whitelaw and Edwards, 1980). In other studies of two water table aquifers in Oklahoma, a variety of bacteria were found with typical concentrations of 10^6 to 10^7 cells per gram of soil. The bacteria were relatively small, with both gram-negative and gram-positive forms being present. Many double cells were found suggesting that the bacteria were in the process of cell division (Ghiorse and Balkwell, 1981). The movement of bacteria through considerable distances of soil, particularly during saturated flow conditions, has been well established (Hagedorn, et al., 1981). Thus, there is strong evidence that the subsurface and water table aquifers are constantly inoculated with organisms from the surface during recharge events. Also, subsurface sedimentary formations may contain bacteria that were once surface organisms which have maintained significant populations.

Some species of these naturally occurring bacteria have the ability to feed on hydrocarbons and other organics, as evidenced by the studies of McKee (as cited by Litchfield and Clark, 1973), and Litchfield and Clark (1973).

McKee found that high bacterial populations of Pseudomonas and Arthrobacter were associated with gasoline contaminated groundwater, while low counts of bacteria were associated with odor-free water (Raymond, et al., 1976). Litchfield and Clark (1973) confirmed that significant populations of bacteria were present in groundwater contaminated with gasoline, fuel oil, and other petroleum products. They found that waters containing less than 10 parts per million (ppm) of hydrocarbons generally had populations of less than 10^3 organisms per milliliter, while hydrocarbon concentrations in excess of 10 ppm generally supported populations of 10^6 organisms per milliliter. Species were identified as belonging mostly to the genera Pseudomonas and Arthrobacter.

Firms specializing in the bioreclamation process have found that the naturally occurring organisms in the subsurface can, in fact, be stimulated by adding nutrients and oxygen to produce a thriving colony which uses the groundwater contaminants as substrates for growth (Williams, 1982). Alternatively, mutant strains developed for the treatment of industrial wastewaters have been used in the bioreclamation process. In this method, a thriving microbial colony, acclimated to the contaminants, is introduced rather than developed, and the time requirement for adaption of a microbial population is significantly reduced or eliminated (Thibault and Elliot, 1980).

The simplified selective adaptation and mutation process is illustrated in Figure 8-3. Original strains are collected from industrial or environmental sources, such as an oil well, coke oven, or contaminated aquifer, where natural populations have been exposed to the compound of interest. Adaptation can, sometimes, be accelerated by irradiation-induced mutation and growth on selective media. In the bioreclamation process, the mutant bacteria can be cultured with contaminated groundwater in a surface holding pond and then inoculated into the subsurface via spraying or injection wells.

8.2.2.2 Nutrient Requirements

Besides the carbon-containing substrate, inorganic nutrients are also required for proper cell growth and therefore are essential elements of the

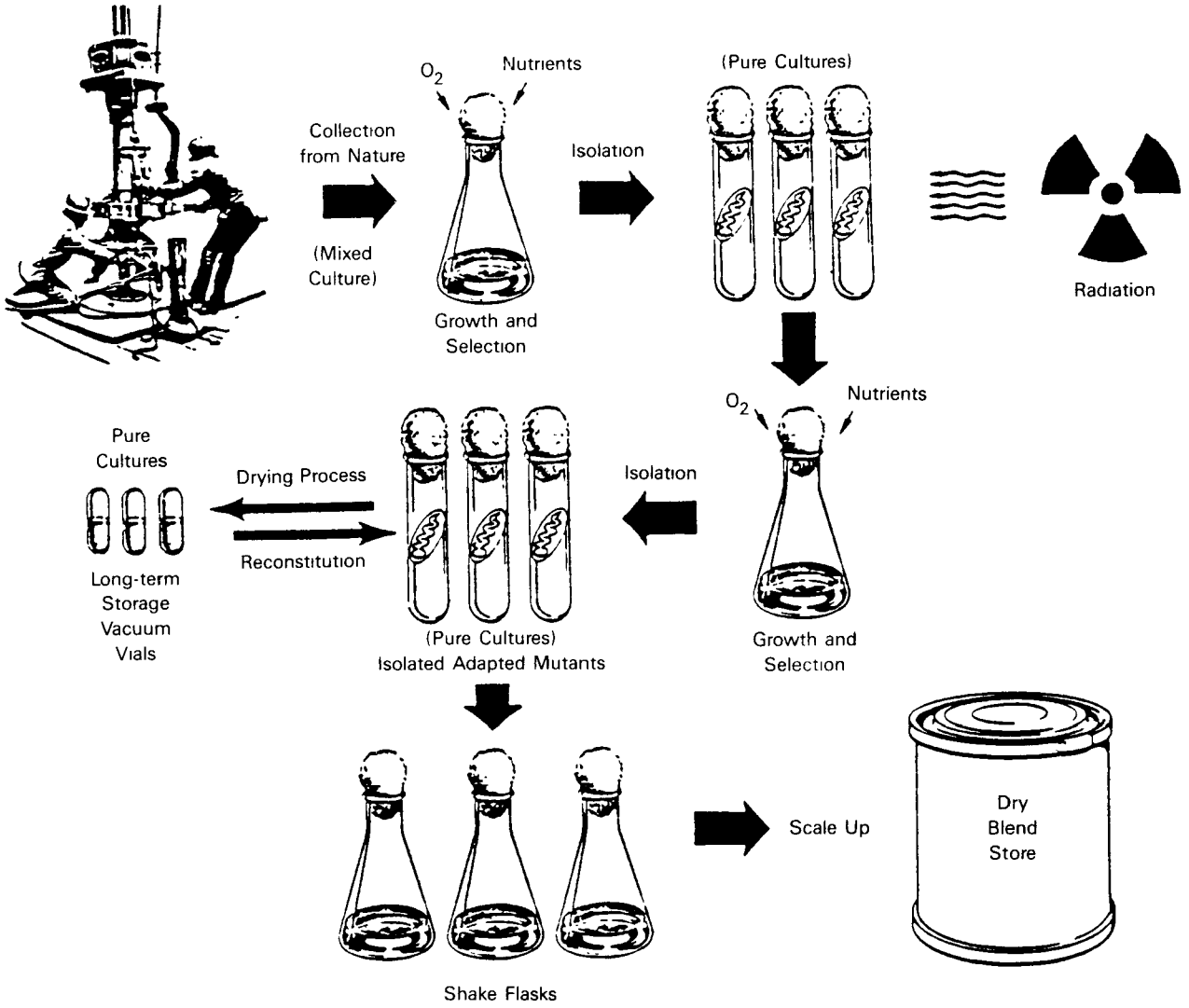


FIGURE 8-3.
SIMPLIFIED SELECTIVE ADAPTION/MUTATION PROCESS
(McDOWELL, et al., 1982)

bioreclamation process. Cell nutrients include nitrogen, phosphorus, and potassium. Trace elements are also required and include sulfur, sodium, calcium, magnesium, iron, and copper (Rogoshewski, 1977). Nitrogen and phosphorus are needed in the greatest quantities for cell growth and can limit cell growth if they are not present at sufficient levels. Trace elements usually are present in sufficient quantities in groundwater environments.

The quantity of phosphorus occurring naturally in groundwater usually is controlled by the presence and solubility of the mineral apatite. Phosphorus concentrations in solution rarely exceed 0.1 milligrams per liter (mg/l) (Bouwer, 1978). Bacteria contain about one percent phosphorus on a dry-weight basis (Doetsch and Cook, 1973), suggesting that groundwater containing 0.1 mg/l of phosphorus could support a maximum bacterial population of 10 mg/l. Assuming a 50 percent conversion efficiency, contaminant plumes of concentrations greater than 20 mg/l of substrate may become phosphorus deficient and require the addition of phosphate.

Similarly, bacteria were found to contain about 10 percent nitrogen on a dry-weight basis, while normal groundwater contains from 0.1 to 10.0 mg/l (Doetsch and Cook, 1973; Davis and DeWiest, 1966). These concentrations would permit degradation of 2 to 200 mg/l of substrate, assuming a 50 percent conversion of nitrogen-free organic matter to new cells. Chemical analysis of nitrogen in groundwater should provide a reasonable estimate of the amount of organic contamination that can be degraded according to the above relationships. This has been confirmed in a study by Kappler and Wuhrman (1978) in which 0.17 mg/l of bound nitrogen in groundwater permitted degradation of only 2.0 to 2.1 mg/l of water soluble organics. When ammonium chloride was added, further degradation took place. The relationships observed in this study indicate that organic concentrations in groundwater above 120 mg/l probably will require additional nitrogen. This addition of nutrients may cause problems by further contaminating the aquifer. Therefore, only the amount needed to sustain biological activity should be added.

8.2.2.3 Oxygen Requirements

Theoretical quantities for the amount of oxygen required to degrade organic chemicals can be determined from stoichiometric analysis. The major assumption in this approach is that all organic materials break down into carbon dioxide and water. Degradation of 1.0 mg of a simple organic acid, such as acetic acid, theoretically would require only 1.1 mg of oxygen, while a fully saturated aliphatic hydrocarbon would require about 3.4 mg. Thus, groundwater with an organic contaminant concentration of 100 mg/l may require oxygen concentrations as high as 340 mg/l if saturated organics are involved.

Dissolved oxygen concentrations in groundwater are generally considered to be quite low for aerobic degradation of compounds. This presumes that any dissolved oxygen in water that percolates through the soil column is used up by microbial oxidation of soil organics (Johnson Division, UOP Inc., 1975). Therefore, oxygen generally must be added to support biodegradation of organics in a contaminated groundwater plume. Different techniques of adding oxygen and the various advantages and disadvantages of each system are discussed in Section 8.2.3.3: Oxygen Supply.

8.2.2.4 Substrate Characteristics

The key to the success of bioreclamation of contaminated groundwaters is the ability of the microorganisms to use the pollutants as substrates or food for their metabolism and growth. Previously mentioned bioreclamation studies of groundwater contaminated with gasoline and fuel oil have shown that many hydrocarbons can be biodegraded. However, in a leachate plume associated with a hazardous waste site, many toxic compounds with different biodegradability rates may be present. In determining whether bioreclamation can be a viable process for treating a specific leachate plume, consideration must be given to the biodegradabilities of all substances present, their concentrations in the plume, and the interactive effects of the contaminants on the microbial community.

The biodegradability of a specific substance can be related to its chemical structure. However, there is much disagreement centered around the development of generalized structure/biodegradability relationships for predictive purposes (JRB Associates, 1982). Empirical approaches have been developed in which the relative biodegradability of specific chemical substances is obtained from laboratory measurements. One such approach uses the ratio of the five day biochemical oxygen demand (BOD_5) to chemical oxygen demand (COD). Compounds with a BOD_5/COD ratio of less than 0.01 are defined as relatively nondegradable, whereas compounds with a BOD_5/COD ratio of 0.01 or greater are defined as relatively degradable. BOD_5/COD ratios for specific chemicals have been given previously in Table 8-3. A second empirical approach involves the use of a ratio of ultimate biochemical oxygen demand (BOD_u) to ultimate oxygen demand (UOD). This ratio is known as the refractory index (RI). An RI over 0.5 indicates that a compound biodegrades readily to carbon dioxide, water, and other associated mineralization products. Compounds with an RI value of 0.0 to 0.5 are considered to be of low to moderate biodegradability. Table 8-4 gives refractory indices for a number of organic substances.

The classification of a substance as having a moderate or low biodegradability does not mean it cannot be biodegraded. Gasoline, which is listed as moderately biodegradable in Table 8-3 and only slightly biodegradable in Table 8-4, has been the substance most frequently treated successfully by the bioreclamation process. Studies indicate that certain bacterial species can degrade compounds considered to be fairly refractory (Vandenbergh, et al., 1981). Also, pretreating a compound of low biodegradability may be possible with ozone for example, to partially oxidize it to a more biodegradable form.

In leachate plumes consisting of many components which are biodegradable, organisms may preferentially degrade one compound before another (SCS Engineers, 1979). The following generalizations apply concerning these preferences:

- Non-aromatics are biodegraded preferentially over aromatics
- Substances with unsaturated bonds are biodegraded preferentially over substances with saturated bonds

- Straight chain compounds are biodegraded preferentially over branched isomers and complex, polymeric substances
- Soluble compounds are usually more biodegradable than insoluble compounds
- The presence of various functional groups affects biodegradability: i.e., alcohols, aldehydes, acids, esters, amides, and amino acids are more biodegradable than corresponding alkanes, olefins, ketones, dicarboxylic acids, nitriles, and chloroalkanes (Lyman, et al., 1982)
- Halogen substituents generally make a compound more biorefractory.

Concentrations of substances present in the leachate plume can also affect biodegradability. When substances are present at levels below 0.1 mg/l, the assimilative processes of microorganisms are sometimes not stimulated, thus adaption to the particular substrate will not occur and the substance will generally not be degraded (SCS Engineers, 1979). Conversely, high concentrations of organic substances may cause inhibition of normal microbial processes because of their toxic nature, and thus may appear to be of low biodegradability. Table 8-5 presents a list of compounds that have been found to be problematic in this respect during industrial wastewater treatment. Problem concentrations are listed for two conditions: substrate limiting--in which the subject compound is the sole source of microorganism food; and non-substrate limiting--in which other carbon substrates are present. In a leachate plume of mixed organics, the non-substrate limiting condition would be expected to prevail. If the leachate plume contains inhibiting concentrations of organics, the bioreclamation process can be designed to treat a diluted portion of the contaminated groundwater to circumvent inhibitory effects.

8.2.3 Design and Operation

The design of an in situ groundwater bioreclamation system must take into account biodegradation kinetics, hydraulic design, oxygen supply, and the need for nutrient addition. In the past, bioreclamation projects have been conducted largely on a trial and error basis. Even though the basic scientific principles of the system may be fairly well understood, the application of these elements for cleanup at a hazardous waste disposal site is not yet fully

TABLE 8-5.
PROBLEM CONCENTRATIONS OF SELECTED CHEMICALS
(SCS Engineers, 1979)

Chemical	Problem Concentration (mg/l)	
	Substrate Limiting	Non-Substrate Limiting
n-Butanol	--	>1000
sec-Butanol	--	>1000
t-Butanol	>1000	>1000
Allyl alcohol	--	>1000
2-Ethyl-1-hexanol	500-1000	--
Formaldehyde	--	50-100
Crotonaldehyde	200	50-100
Acrolein	--	--
Acetone	--	>1000
Methyl isobutyl ketone	>1000	100-300
Isophorone	>1000	--
Diethylamine	--	300-1000
Ethylenediamine	--	100-300
Acrylonitrile	>1000	100
2-Methyl-5-ethylpyridine	>1000	100
N,N-dimethylaniline	>1000	--
phenol	>1000	300-1000
Ethyl benzene	>1000	--
Ethyl acrylate	600-1000	300-600
Sodium acrylate	--	>500
Dodecane	>1000	--
Dextrose	>1000	>1000
Ethyl acetate	>1000	--
Ethylene glycol	>1000	>900
Diethylene glycol	--	>1000
Tetralin	>1000	--
Kerosene	--	>500
Cobalt chloride	--	>1000

developed. Furthermore, most private companies specializing in bioreclamation of polluted groundwater have not always been willing to disclose information about bioreclamation technology for fear of revealing trade secrets. Nevertheless, enough design information for the bioreclamation process is available for adopting reliable bioreclamation treatment at hazardous waste sites. The following sections describe biodegradation kinetics, hydraulic design, oxygen supply, nutrient addition, and other design aspects.

8.2.3.1 Biodegradation Kinetics

In situ bioreclamation of highly contaminated groundwater (i.e., over 100 mg/l) generally involves aerobic microbial degradation at ambient temperatures. In this case, the rate equation describing biodegradation of organic material in the aqueous environment is based on Monod kinetics (Lyman, et al., 1982):

$$- d[C]/dt = (U_m)(C_m)(C)/Y_d(C_{u/2} + C)$$

where: $d[C]/dt$ = the rate of disappearance of substrate

U_m = maximum growth rate of the microorganism

C_m = the concentration of microorganisms

C = concentration of the substrate

$C_{u/2}$ = concentration of the substrate supporting a half-maximum growth rate ($U_{m/2}$); can be assumed to range from 0.1 to 10 mg/l for most substances

Y_d = the yield coefficient, which is $-dC_m/dC$, can be assumed to be 0.5 for non-dilute systems.

The equation is a variable order rate equation, however, it can be simplified to first and second order approximations for two conditions--when C is much greater than $C_{u/2}$ and when C is much less than $C_{u/2}$. At the start of bioreclamation of contaminated groundwater, the first condition will apply because substrate concentrations will be substantial. However, when the concentration of contaminating substance has been lowered by bioreclamation to a residual level, the second condition will apply.

When C is much greater than $C_{u/2}$, the expression $(C_{u/2}+C)$ is approximately equal to C. By substituting C for $(C_{u/2}+C)$ and the expression 0.5C+C for C_m (which is derived from integrating the yield coefficient equation $Y_d = 0.5 + -dC_m/dC$), the equation reduces via integration to a first order equation:

$$C_f = C_i e^{-(U_m t)}$$

where: C_f = final concentration

C_i = initial concentration

t = time

U_m = maximum growth rate of microorganisms; equal to the first order rate constant k

e = base of natural logarithms.

This equation is applicable only for large values of C in relation to $C_{u/2}$. If C is at least ten times $C_{u/2}$, less than a 10 percent error in the calculated final concentration can be expected.

In a residual concentration, when C is much less than $C_{u/2}$, the value $C_{u/2}+C$ is approximately equal to $C_{u/2}$ and the equation reduces to the second order expression:

$$- d[C]/dt = f(C_m)(C),$$

where: $f = U_m/Y_d(C_{u/2})$.

When the substrate concentration is of the same order as the $C_{u/2}$ value, the rate expression is a hybrid between first and second order.

Biodegradation rates in groundwater can be calculated more accurately if sorption/desorption effects of the earth material comprising the aquifer are taken into account. In this case, the rate order constant (k) can be related

to an apparent constant (k_{app}) by the partition coefficient of the substrate (k_p) and the earth-to-water mass ratio (p) as (Maki, et al., 1980):

$$k_{app} = k/(1+pk_p)$$

Unfortunately, much of the data compiled on biodegradation rate constants are not generally useful for assessing compound degradation in an in situ groundwater bioreclamation situation because the data have been developed for river die-away predictions based on concentrations of organics which are too dilute. Some rate data have been developed based on aerobic biodegradation of organic materials by an adapted activated sludge process, in which the rate is expressed in mg of COD degraded per mass of the microbial inoculum per unit time (Pitter, 1976). However, these rate data may not be directly applicable to assessing in situ biodegradation rates for a combination of organics because of synergistic and antagonistic effects such as preferential biodegradation and co-metabolism.

In properly assessing the applicability of bioreclamation as a means of treating contaminated groundwater, bench and pilot scale testing should be conducted to obtain the necessary values of the variables described in the above equations. Test results for in situ biodegradation studies may prove to be more accurate if they are conducted in a system mimicking the groundwater system to be treated (i.e., physical and chemical properties).

Degradation rates obtained from controlled bench and pilot scale studies can be extrapolated to ambient temperature rates by using the Arrhenius equation (Lyman, et al., 1982):

$$R_t = R_i e^{-(E_a/R_g T_a)}$$

where:

R_t = temperature-corrected reaction rate
R_i = initial rate of reaction
E_a = activation energy
R_g = gas constant
T_a = absolute temperature
e = base of natural logarithms.

This equation also demonstrates the relationship between temperature and reaction rate (i.e., lower temperatures result in lower reaction rates and higher temperatures result in higher reaction rates).

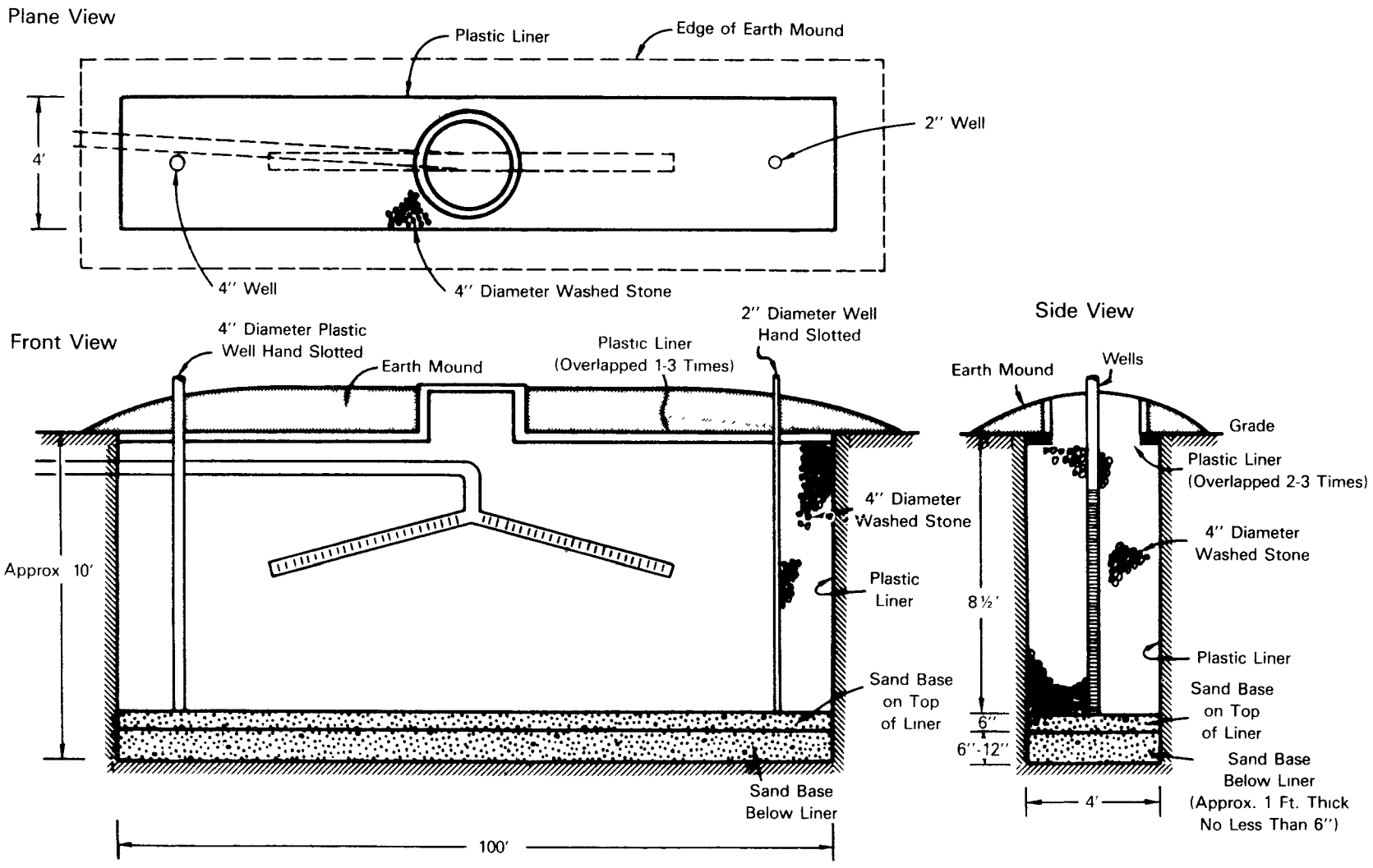
8.2.3.2 Hydraulic Design

Hydraulic design is an essential element of the bioreclamation process pertaining to the collection and containment of the groundwater plume. Proper design of groundwater injection systems is important for the prevention of pollutant migration away from treatment areas. In addition, hydraulic design relates to placement of the extraction system to collect contaminated groundwater.

Injection and extraction of contaminated groundwater may be accomplished by using either a pumping system (Chapter 5) or a subsurface drainage system (Chapter 6). In either case, the groundwater extraction and injection system must be properly designed to contain the groundwater plume and minimize backflow as a result of groundwater reinjection.

A subsurface trench was used to reinject treated groundwater during the cleanup of an underground spill in Waldwick, N.J. (Jhaveri and Mazzacca, 1983). In this case, the bioreclamation process was augmented by surface biological treatment. A diagram of the injection trench is shown in Figure 8-4. The trench is 10 feet deep by 4 feet wide by 100 feet long. The trench has a 15 mil plastic liner installed on the bottom, back, ends, and top such that reinjected water can only flow out of the front (downgradient) face of the trench. About 40 feet of slotted steel pipe is installed horizontally in the trench to carry reinjected water into the trench system. As water flows into the injection trench, the water is forced to exit only from the front face. Back flow is minimized by this design feature. Barriers can also be used behind the trench and extended to a point where backflow is further minimized. In extreme cases, total control of backflow and plume containment can be obtained by installing a circumferential barrier wall.

FIGURE 8-4.
CONFIGURATION OF REINJECTION TRENCHES
(JHAVERI AND MAZZACCA, 1983 AS CITED BY
COCHRAN, et al., 1984)



Note: Treated water exits only from one side of the trench.

8.2.3.3 Oxygen Supply

The supply of oxygen to subsurface microorganisms is essential to the effectiveness of the bioreclamation process. Oxygen can be supplied in a number of ways, including aeration, oxygenation, and the use of hydrogen peroxide and other oxygen containing compounds.

8.2.3.3.1 Aeration

Air has been used for many years in conventional wastewater treatment to provide the necessary oxygen for microbial activity and growth. In an in situ bioreclamation process, air can either be added to the extracted groundwater before reinjection or injected directly into the contaminated plume. The first method involves adding the air into the pipeline and mixing it with a static mixer (Figure 8-5). The aerated water can then be reinjected to the subsurface. Air injection can provide a maximum of about 10 mg/l of dissolved oxygen at a temperature sufficient to carry out bioreclamation (15°C). Table 8-6 shows saturated dissolved oxygen concentrations in equilibrium with air for other temperatures.

A higher oxygen concentration can be attained in a pressurized line or a confined aquifer. The equilibrium oxygen concentration in water increases with increased air pressure according to Henry's Law (Sawyer and McCarty, 1967):

$$C_L = \alpha P H_k$$

where:

C_L = concentration of oxygen in liquid (mg/l)

α = volume fraction (0.21 for O_2 in air)

P = air pressure (atm)

H_k = Henry's Law Constant for oxygen.

The value of Henry's Law constant is 43.8 mg/l-atmosphere at 68°F (20°C). Pressure increases with groundwater depth at the rate of 0.0294 atmospheres per foot.

FIGURE 8-5.
CONFIGURATION OF STATIC MIXER

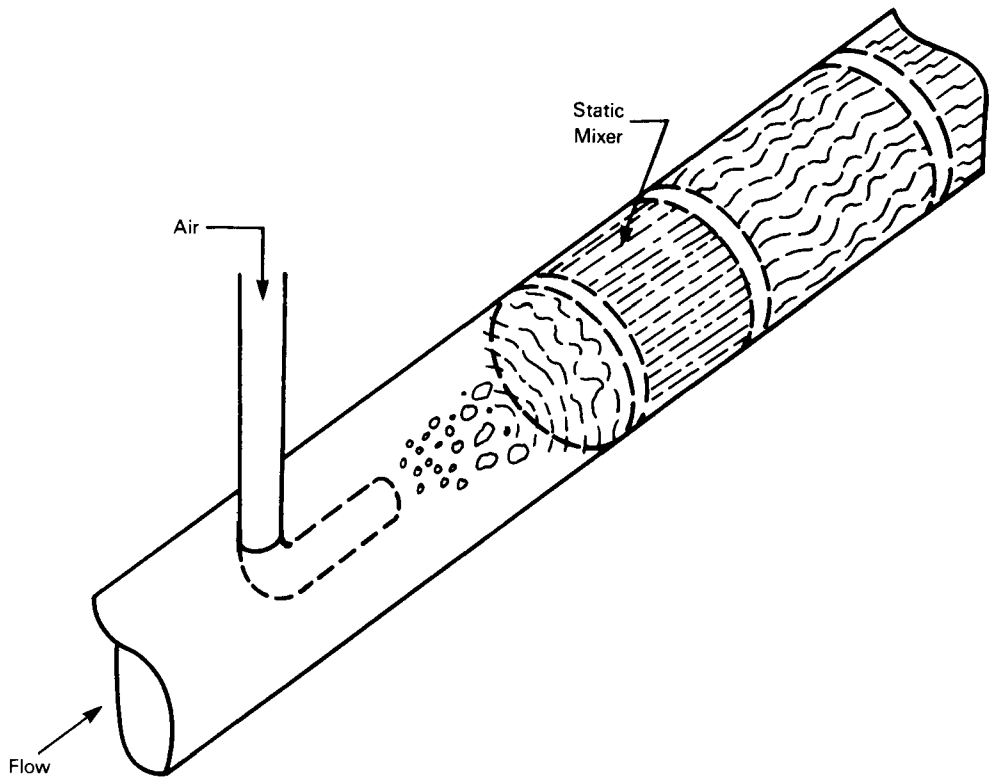


TABLE 8-6. SOLUBILITIES OF OXYGEN IN WATER AT SELECTED TEMPERATURES
(After Davis and DeWiest, 1966)

Temperature °F (°C)	Dissolved Oxygen Concentration (mg/l) in Equilibrium with the Air at 760 mm
32 (0)	15
50 (10)	11
68 (20)	9
96 (30)	8
122 (50)	6

Oxygen solubility in a non-polar organic compound, such as hexane, is approximately ten times higher than the solubility of oxygen in water (Texas Research Institute, 1982b). Therefore, in leachate plumes consisting of an aqueous phase and a non-aqueous organic phase, concentrations of dissolved oxygen may be higher than that attainable with ordinary water.

The theoretical amount of oxygen required to degrade 1 mg/l of hydrocarbon substrate can be calculated by performing a stoichiometric analysis for the given substance, as shown by the following equation:

$$C_xH_y + (x+(y/4))O_2 \longrightarrow x\ CO_2 + (y/2)H_2O$$

Usually, about 3 to 4 mg/l of oxygen is required to degrade 1 mg/l of a medium-length hydrocarbon compound. If 50 percent of the organic material is converted to bacterial cell matter and the other half oxidized to carbon dioxide and water, only 4 to 6 mg/l of organic material can be converted and oxidized under oxygen saturation conditions. Thus, for contaminated groundwaters having organic concentrations significantly higher than the above values, in-line aeration prior to injection is insufficient, because only about 10 mg/l dissolved oxygen can be attained on a single pass, and the reinjected groundwater will use up all available oxygen in a very short period of time.

The use of in situ aeration wells is a much more feasible approach to treating highly contaminated leachate plumes. A bank of aeration wells can be installed to provide a zone of continuous aeration through which the contaminated groundwater would flow. Oxygen saturation conditions can be maintained for degrading organics during the residence time of groundwater flow through the aerated zone. The required time for aeration can be derived from bench scale studies. Residence time (t_r) through the aerated zone can be calculated

from Darcy's equation (Freeze and Cherry, 1979) using groundwater elevations (i.e., head) and hydraulic conductivity as follows:

$$t_r = (L_a)^2/K(h_1-h_2)$$

where:

t_r = residence time

K = hydraulic conductivity

L_a = length of aerated zone

h_1 = groundwater elevation at beginning of aerated zone

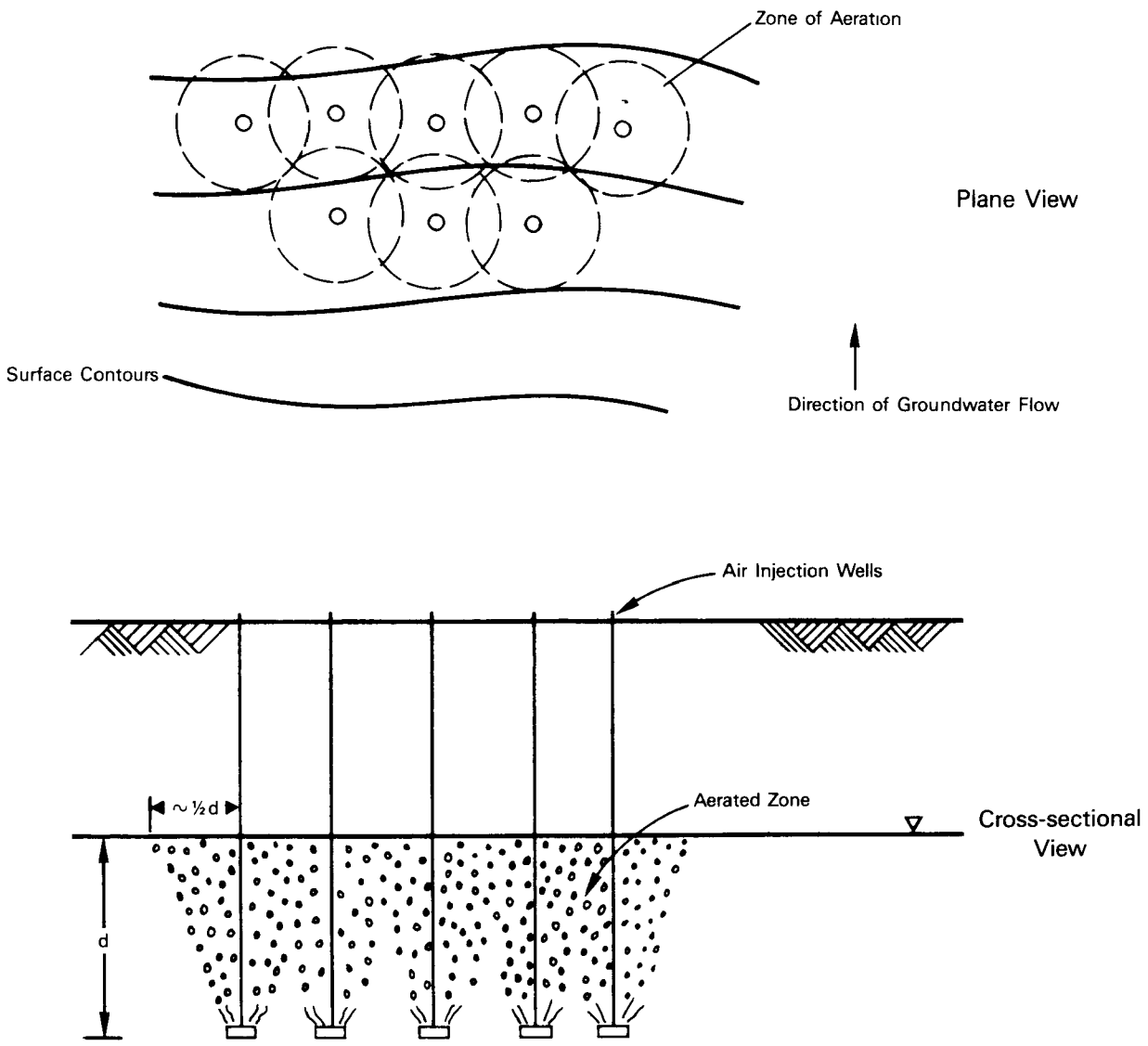
h_2 = groundwater elevation at end of aerated zone

In the design of an in situ aeration well zone system, the zone must be wide enough to allow the total plume to pass through. The flow of air must be sufficient to give a substantial radius of aeration while small enough not to cause an air barrier to the flow of groundwater.

Data based on de-icing of waste lagoons by bubbling has shown that a ratio of about 1:1 exists between the depth of aeration and the diameter of the zone of aeration (Rothman, 1983). Based on this logic, an aeration well 20 feet into pure water would give a radius of aeration of about 10 feet. However, the effect of aeration wells in a geological formation is not known and the effect on the zone of aeration is not well understood but would probably be less than in water. Pilot studies would be necessary to determine the relationship accurately.

At greater groundwater depths, greater pressure is present (2 atms) at 33 feet) and oxygen solubility and mass transfer is increased. A possible configuration for an aeration well bank is given in Figure 8-6. Much of the information presented needs further bench and pilot scale investigation to determine optimum design criteria.

FIGURE 8-6
POSSIBLE CONFIGURATION OF IN SITU AERATION WELL BANK



A blower can be used to provide the flow rate and pressure for aeration. At the groundwater bioreclamation project in Waldwick, N.J., 5 pounds per square inch pressure was maintained in nine 10-foot aeration wells, each with an air flow of 5 cubic feet per minute (Groundwater Decontamination Systems, Inc., 1983).

8.2.3.3.2 Oxygenation

Oxygenation systems, either in-line or in situ can also be installed to supply oxygen to the bioreclamation process. Their advantage over conventional aeration is that higher oxygen solubilities can be attained and hence, more efficient oxygen transfer to the microorganisms. Oxygen solubilities in water in equilibrium with gaseous oxygen at one atmosphere are given in Table 8-7.

TABLE 8-7. OXYGEN SOLUBILITY IN WATER IN EQUILIBRIUM WITH OXYGEN GAS AT ONE ATMOSPHERE (After Davis and DeWiest, 1966)

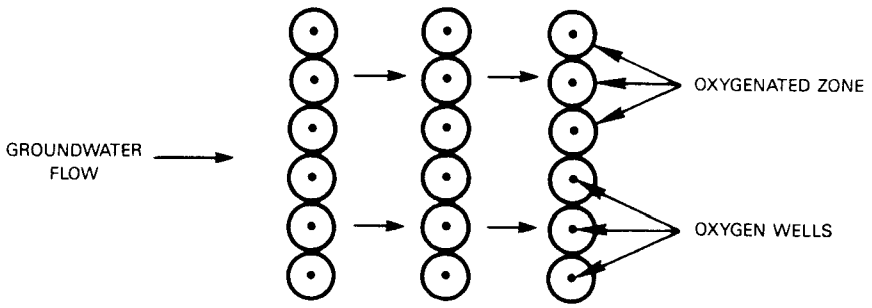
Temperature (°F) (°C)	Oxygen Solubility (mg/l)
32 (0)	70
50 (10)	54
68 (20)	44
86 (30)	37
122 (50)	27

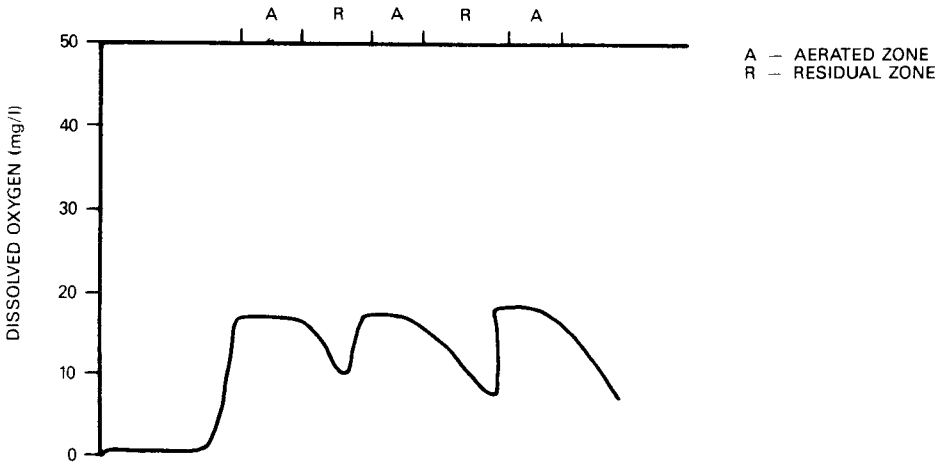
Solubilities of oxygen in various liquids are four to five times higher under pure oxygen systems than with conventional aeration. Therefore, in-line injection of pure oxygen will provide sufficient dissolved oxygen to degrade 20 to 30 mg/l of organic material, assuming 50 percent cell conversion. This value is still probably insufficient for treating contaminant plumes with higher levels of contamination, since many recyclings would be required. However, according to Henry's Law, oxygen solubilities will significantly

increase at groundwater depths greater than 30 feet or in pressurized aquifers.

As with aeration, in situ wells can be installed to provide pure oxygen to the groundwater. The higher oxygen solubilities may provide some flexibility in the design of cell banks, especially at greater pressures, since the oxygen may not be used up immediately, as with aeration. Well distances parallel to groundwater flow may then be designed on the basis of time for oxygen consumption if groundwater flow rates are compatible. Configuration of an in situ oxygenation well system may be similar to Figure 8-7.

FIGURE 8-7.
CONFIGURATION OF IN SITU OXYGENATION WELL SYSTEM AND DISSOLVED OXYGEN CONCENTRATIONS AS A FUNCTION OF DISTANCE FROM WELL





Pure oxygen systems are expensive. Two types of systems are available: stored liquid oxygen and on-site generation. Costs for the stored system are about $100,000 for a 45 ton storage installation and supply costs are about $100/ton within a 50 mile radius of the supplier (additional $16 for each 50 mile increment). An on-site oxygen generation plant costs about $2.5 million for a 30 ton per day (tpd) plant and about $1.5 million for a 10 tpd plant. Power and operation and maintenance (O&M) costs are about $55 to $75 per ton, respectively (Lamparella, 1983).

For a temporary installation such as bioreclamation, the stored system is recommended over the on-site generation system because of lower costs. A typical installation may be a bank of 50 aeration wells, supplying 5 cubic feet per minute at 1 bar or 250 cubic feet per minute. This converts to 12 tons of oxygen per day using a conversion factor of 24,000 cubic feet per ton of oxygen (Lamparella, 1983). Supply costs for oxygen would run about $1,200 per day.

8.2.3.3.3 Hydrogen Peroxide

Classically, hydrogen peroxide (H_2O_2) has been used as a bactericide in medicine and as a chemical oxidant in industrial wastewater treatment. However, in dilute concentrations, H_2O_2 may be feasible to use as a source of oxygen for the microorganisms associated with bioreclamation. Bacterial cells normally produce H_2O_2 during respiration, and though H_2O_2 is cytotoxic at higher concentrations (3%), bacteria have developed enzymatic defenses against H_2O_2 toxicity, known as hydroperoxidases (Texas Research Institute, 1982a). This phenomena suggests that a threshold concentration exists below which microorganisms can tolerate hydrogen peroxide. The cells may then be free to use the oxygen provided by the decomposition of H_2O_2 to aerobically degrade organic material.

Studies conducted in flasks indicate that toxicity threshold levels for hydrogen peroxide are dependent on cell populations. H_2O_2 concentrations higher than 1000 mg/l have not been found to elicit a toxic response from established microbial populations. In fact, maximum cell mass was measured at

concentrations of about 1000 mg/l H_2O_2 (Texas Research Institute, 1982a). A much lower toxicity threshold of 100 mg/l H_2O_2 was exhibited with non-established, smaller populations. Measurements were not conducted on hydrocarbon utilization rates.

A major concern in the use of hydrogen peroxide as an oxygen source is that it may be completely decomposed in a matter of hours, thereby releasing oxygen all at once instead of under more controlled conditions. Then, oxygen may bubble out of the system instead of being available for areas farther downgradient in the aquifer.

Decomposition of H_2O_2 can be catalyzed either biologically or chemically. Enzymatic decomposition by hydroperoxidases (catalases and peroxidases) is the defense mechanism of bacteria mentioned earlier. The reactions are as follows (Texas Research Institute, 1982a):

$$2H_2O_2 \xrightarrow{\text{catalase}} 2H_2O + O_2$$

$$H_2O_2 + XH_2 \xrightarrow{\text{peroxidase}} 2H_2O + X$$

where X is reduced nicotinamide adenine dinucleotide (NADH), glutathione or another biochemical reductant.

Enzymatic decomposition of H_2O_2 to oxygen would occur only with catalases and not with peroxidases. Decomposition of H_2O_2 to oxygen also is catalyzed by metal salts, particularly ferrous iron. The mixture of H_2O_2 and ferrous salts is called Fentons reagent, and has been widely used as a hydroxylation reagent (Texas Research Institute, 1982a). Reduced iron present in the substrata may catalyze the decomposition reaction and cause immediate decomposition and release of O_2. Similarly, an alkaline pH will also accelerate decomposition of the H_2O_2 molecule.

Decomposition of H_2O_2 may be slowed by the addition of mineral acid, dissolution in an acidic solution, or addition of a stabilizing agent, such as acetanilide or sodium pyrophosphate (Texas Research Institute, 1982a). However, the effect of stabilizing agents is limited to conditions of low metal contamination (FMC Corporation). Introduction of H_2O_2 to soil would cause contact with metals and thus probably catalyze the decomposition reaction. Other stabilizers, which slow the rate down considerably, may be developed . Studies to measure decomposition rates and dissolved oxygen retention are required to properly assess the feasibility of H_2O_2.

Hydrogen peroxide is available commercially for industrial uses in 35 percent, 50 percent, and 70 percent solutions (FMC Corporation). A highly stabilized product mix is recommended for bioreclamation; however, even this may not be adequate because of the high susceptibility to catalytic decomposition when in contact with metal-containing substances, such as soils. An H_2O_2 storage and metering facility is required for use. Drum or tank storage may be used. Drum storage has inherent labor costs associated with it, while a tank facility would incur large capital costs. Since H_2O_2 is a powerful oxidizer, only specific materials of construction can be used such as aluminium alloys, white chemical porcelain, pyrex, teflon, and Kel- F® 81 resin (FMC Corporation). Polyethylene, stainless steel, and polyvinyl chloride also can be used for limited contact application.

8.2.3.3.5 Other Oxygen Sources

Other oxygen containing compounds have been proposed, including potassium permanganate, barium and strontium peroxides, and urea-peroxide (Texas Research Institute, 1982b). All but the latter introduce undesirable metals into the system. Urea-peroxide has been used in conjunction with phosphate solutions to treat plants suffering from oxygen starvation in the root zone (Texas Research Institute, 1982b).

8.2.3.4 Pretreatment by Ozone of Other Chemical Oxidants

In some cases, improving the biodegradability of refractory substances in the leachate plume may be possible by pretreating with ozone or other chemical oxidants prior to nutrient addition, reinjection, and aeration. The role of an oxidative pretreatment step would be to partially oxidize the refractory compounds to a form more usable by microorganisms. Oxidative pretreatment, if conducted at excessively high oxidant concentrations, could destroy bacteria in the pumped leachate feed stream. Also, residual concentrations of oxidant could linger in the pretreatment effluent at levels toxic to the microorganisms. However, such deleterious effects are lessened by two factors:

- Much of the active bacterial population will remain in the substrata attached to or filtered by earth materials, and thus will not be carried off in the pumped leachate and destroyed by pretreatment
- Groundwater will dilute reinjected water containing oxidant residuals, and thus will help to bring down oxidant concentrations below toxic levels.

These factors suggest that oxidative pretreatment could be carried out in a bioreclamation scheme without significant adverse effects on the active microbial population. Any deleterious effects could be further attenuated by conducting pretreatment at subtoxic oxidant levels (to be determined by laboratory studies), providing a retention or aeration basin to remove residuals, or by pretreating at a lower flow rate, so that the dilution effect of groundwater is more pronounced.

Potential chemical oxidants include ozone, hydrogen peroxide (H_2O_2), ozone and H_2O_2 mixes and other free radical forming mixtures, potassium permanganate, and chlorine dioxide.

Classically, ozone has been used for disinfection and chemical oxidation of organics in water and wastewater treatment. In commercially available ozone-from-air generators, ozone is produced at a concentration of one to two percent in air (Nezgod, 1983). In bioreclamation, this ozone-in-air mixture could be contacted with pumped leachate using in-line injection and

static mixing or using a bubble contact tank. A dosage of 1 to 3 mg/l of ozone can be used to attain chemical oxidation (Nezgod, 1983). However, German research on ozone pretreatment of contaminated drinking waters indicates that the maximum ozone dosage should not be greater than 1 mg/l of ozone per mg/l total organic carbon. Higher concentrations may cause deleterious effects to microorganisms (Rice, 1983). Contact with the ozone-in-air mixture will increase the dissolved oxygen in the leachate stream, similar to in-line aeration. However, additional aeration may be required, depending on the concentration of organic material in the leachate (refer to Section 8.2.3.3.1). Partial oxidation should be noted as only causing an increase in biodegradability of a substance. Partial oxidation does not decrease the amount of substance present in the leachate.

Estimated costs for ozone pre-treatment, based on an ozone-from-air generator and an air preparation unit to remove moisture, are shown in Table 8-8 as a function of flow rate. Power requirements and average power costs for ozone-from-air systems are shown for different size units in Table 8-9.

TABLE 8-8. CAPITAL COSTS FOR OZONE TREATMENT
(Nezgod, 1983)

Gas Flow Rate		Cost ($)
scfm[1]	m^3/s[2]	
0.5	2×10^{-4}	7,090
7.0	3×10^{-3}	28,000
25	0.01	65,000
250	0.1	290,000

[1]scfm = standard cubic feet per minute
[2]m^3/s = cubic meter per second

TABLE 8-9.
POWER COSTS FOR OZONE GENERATOR SYSTEM (Nezgod, 1983)

Gas Flow Rate		Ozone Rate		Power Requirements		Cost
scfm	m^3/s	lbs/day	kg/day	KWhr/lb O_3	KWhr/kg O_3	Dollars per day at $0.08/KWhr
0.5	2×10^{-4}	1	2.2	27.5	12.5	2.2
7.0	3×10^{-3}	14	31	16.5	7.5	18.5
25	0.01	50	110	9.7	4.5	38.8
250	0.1	500	1,100	3.5	1.6	740

Hydrogen peroxide, previously discussed in section 8.2.3.3.3, can also be used in higher concentrations as a chemical oxidant. Hydrogen peroxide can partially oxidize a limited number of substances including alcohols, aldehydes, secondary amines, acryl chlorides, nitriles, dicarboxylic acids, polynuclear hydrocarbons, and various unsaturated compounds. A weight ratio of two to four units of H_2O_2 per unit of organic has been recommended for complete oxidation to carbon dioxide and water (FMC, 1978). Partial oxidation would most likely require a lesser concentration.

Hydrogen peroxide can also be used in combination with ozone or ferrous salts to produce hydroxyl (OH-) free radicals which will attack a greater number of substances (Rice, 1983). Incorporating these combined treatments to partially oxidize the more refractory compounds may be possible.

Other oxidants, such as potassium permanganate and chlorine dioxide, may also be considered for use in an oxidative pretreatment. However, the use of such chemicals introduces undesirable substances to the groundwater.

Potassium permanganate introduces manganese to the groundwater, which besides being a contaminant, may clog well screens and soil pores upon precipitation.

8.2.3.5 Nutrients and Other Additives

Nitrogen and phosphorus are usually the limiting nutrients in biodegradation processes and must be added if not present in sufficient quantities to support an active microbial population. Analysis of cell protoplasm indicate that the carbon: nitrogen: phosphorus weight ratio is 100:15:3 (Thibault and Elliot, 1980). The total amount of nitrogen and phosphorus required could be calculated if the following were known:

- The total volume of contaminated water to be treated
- The amount of elemental carbon available in the organic compounds present

- Nitrogen and phosphorus associated with the organic compounds present in the leachate plume.

Usually the above quantities are not known, therefore nutrient levels cannot be accurately calculated. Determining the optimum amount of these nutrients should be based on previous research from similar projects and treatability studies conducted in the laboratory, if available. Other nutrients required for sustaining microbial populations include potassium, sulfur, sodium, calcium, magnesium, iron, and manganese. With the exception of potassium, most of these substances are present in sufficient quantities in the groundwater and need not be supplemented. As with nitrogen and phosphorus, potassium supplements will most likely need to be added to the leachate plume.

In previous bioreclamation projects not utilizing in situ methods, required levels of nutrients have been derived empirically through plate counts and shaker studies. Growth studies conducted by Suntech Inc. on a gasoline-contaminated groundwater incorporated a basal salts medium. Composition of the medium is given in Table 8-10 (Jamison, et al., 1976).

Once microbial activity was established, Suntech, Inc. applied bulk quantities of ammonium sulfate [$(NH_4)_2SO_4$], disodium phosphate (Na_2HPO_4), and monosodium phosphate (NaH_2PO_4) to the groundwater in the form of a 30% concentrate in water which was metered into test wells. They determined that other nutrient substances were present in sufficient quantities in the aquifer and did not need to be added. The solution was made up in a 2200 gallon tank truck and contained 1.7 tons $(NH_4)_2SO_4$, 0.5 tons) Na_2HPO_4, and 0.4 tons NaH_2PO_4 (Raymond, et al., 1976). Suntech Inc., in flask studies, found that the forms of nitrogen and phosphorus were not critical. However, they did find from this initial investigation that diammonium phosphate could not be used because of excessive precipitation (Jamison, et al., 1976).

Groundwater Decontamination Systems (GDS) Inc. used a variation of the basal salt medium in a combined surface and in situ microbial degradation system. Concentrations of nutrients added to the surface biotreatment system are given in Table 8-11. GDS Inc. (1982) has also used the Davis medium,

TABLE 8-10. COMPOSITION OF BASAL SALTS MEDIUM
(Jamison, et al., 1976)

Salt Type	Concentration (mg/l)
KH_2PO_4	400
Na_2HPO_4	600
$(NH_4)NO_3$	10
$MgSO_4.7H_2O$	200
Na_2CO_3	100
$CaCl_2.2H_2O$	10
$MnSO_4.H_2O$	20
$FeSO_4.7H_2O$	5

TABLE 8-11. BASAL SALT MEDIUM USED BY GDS INC.
(Groundwater Decontamination Systems, Inc.,
1983 as cited Cochran, et al., 1984)

Salt	Concentration (mg/l)
NH_4Cl	500
KH_2PO_4	270
K_2HPO_4	410
$MgSO_4$	1.4
Na_2CO_3	9
$CaCl_2$	0.9
$MnSO_4$	1.8
$FeSO_4$	0.45

which contains citrate to hold trace metals in solution, for growth studies. Adding citrate if alkaline groundwater is encountered may be advisable when metals would tend to precipitate. Exact composition of the nutrient mix may be varied to accommodate pH adjustment and buffering capabilities.

Adding low concentrations of readily metabolized protein-type compounds (such as peptone, yeast extract, nicotinamide, riboflavin, pyridoxine, and thiamine) have been found to often promote biodegradation of other target organics. However, high concentrations of these substances may hinder degradation of the target hydrocarbons (Texas Research Institute, 1982b). Use of these protein sources may become important when low levels of target organics are reached and some substrate is required to sustain an active microbial population.

The use of biodegradable emulsifiers to "pseudo-solubilize" hydrocarbons also has been proposed to increase the availability of the nonsolubles to the microorganisms (Thibault and Elliot, 1980). However, emulsifiers have been found to cause foaming in aeration wells, which may present mechanical problems. This is especially true if surfactant emulsifiers are to be used (Raymond, et al., 1976). Non-toxic and biodegradable anti-foaming agents possibly may be added to control this.

As mentioned earlier, pH control can be attained by adjusting the nutrient salt composition to produce a specific pH (around 7) and serve as a buffer to maintain this pH. Also, mineral acids (preferably nitric or phosphoric) and alkaline substances (sodium hydroxide or lime solutions) can be used for pH control. Acid conditions (pH 6 to 7) should be maintained if hydrogen peroxide is being proposed as the oxygen supply to retard catalytic decomposition. Biological systems are vulnerable to sudden changes in environment and addition of nutrients, additives, and pH control reagents should be performed gradually. In operations, batch and slug doses should be avoided to prevent toxic, osmotic, and pH shock conditions, especially if the more susceptible engineered microorganisms are used. Since nutrient additions may be potentially contaminating the groundwater, consideration should be given to minimize the applied concentrations.

8.2.3.6 Other Design Aspects

Optimum extraction and injection flow rates will many times be predetermined by aquifer yield limits or hydraulic design for plume containment. The factors affecting aquifer flow rates are described in Chapter 5, Groundwater Pumping.

Aquifer flow rates should be sufficiently high so that the aquifer is flushed several times over the period of operation. Thus, if the cleanup occurs over a three-year period, flow rates between injection and extraction wells should be such that a residence time of one-half year or less occurs between the well pairs. This corresponds to six or more flushes. Several recycles would cause flushing of soils containing organics, preventing the clogging caused by microorganism build-up (because of increased flow rate); more even distribution of nutrients and organic concentration within the plume; and better and more controlled degradation. Flow rates and the number of recycles should not be high enough to incur excessive pumping costs, loss of hydraulic containment efficiency because of turbulent conditions, or cause corrosion, excessive deposition on system components, flooding, or well blow out. The operating period will depend on the field biodegradation rate of the contaminants in the plume and the number of recycles. If the period of operation is excessively long, for example more than 5 years, the operating costs of bioreclamation may outweigh the capital costs of another remedial alternative.

8.2.3.7 Operation and Maintenance

Operation and maintenance of a bioreclamation process involve aspects of the hydraulic system as well as the biological system. The hydraulic aspects relate to pumps, extraction wells, injection wells, and injection trenches, which have been discussed previously in Chapters 5 and 6.

In a biological system, pH should be maintained in a range between 6 and 8 and concentrations of both nutrients and organics should be kept as uniform as possible to protect against shock loading. Dissolved oxygen should be

maintained above the critical concentration for the promotion of aerobic activity, which ranges from 0.2 to 2.0 mg/l, with the most common being 0.5 mg/l (Hammer, 1975).

Clogging of the aquifer, injection wells or trenches, and extraction wells by microbiological sludge is a possibility. GDS Inc. installed two wells in each of their injection trenches in case flushing was ever required to remove sludge. After 1-1/2 years of operation clogging had not occurred (Groundwater Decontamination Systems Inc., 1982). Microorganisms growing on toxic organics have characteristically low growth rates compared to those growing on other substrate systems. Therefore, they are not expected to produce much sludge (Wilson, et al., 1982). The above suggests that clogging may not be a usual occurrence in bioreclamation of aquifers contaminated with toxic organics.

Maintenance of the bacterial population at their optimal levels is also important, especially for selective mutant organisms which tend to be more sensitive than naturally occurring species. There is a possibility that predators could destroy the cultured bacteria; however, this has not been reported in the literature. A continuous incubation facility operating at higher temperatures and under more controlled conditions could be used to maintain the microbial population. The high biomass containing stream-formed organisms from such a facility could then be reinjected via wells or trenches so as to reinoculate the subsurface continuously.

Aeration wells may be susceptible to operational problems. If injected gas fluidizes the surrounding material, soil substrata shifts can occur which may cause a well blowout (free passage of air to the surface). The cone of influence in a blown-out well will be greatly reduced, therefore, requiring the installation of a new well. The best method to prevent blowouts is to keep gas velocities below those necessary to cause fluidization or to place wells deep enough so that overburden pressure prevents excessive fluidization, or both (Sullivan, 1983). Suntech, Inc. stated that a number of aeration wells became inoperative becasue of blowout during their groundwater clean up in 1972 and had to be replaced (Raymond, et al., 1976). This suggests that

aeration well blowout could become a commonly encountered problem if attention is not paid to the design criteria.

8.3 Chemical Treatment

This section summarizes some of the basic information available on five types of in situ chemical treatments:

- Soil flushing/solution mining
- Oxidation/reduction
- Precipitation/polymerization
- Neutralization/hydrolysis
- Permeable treatment beds.

In general, the in situ chemical treatment technologies presented here are not as developed as other currently available technologies for restoring contaminated aquifers. This is true for a number of reasons. In situ chemical treatment is typically very pollutant or pollutant group specific. Also, a large number of possible chemical reactions can occur for each type of chemical treatment, making applied research and development of a technique a large and complex task. Nevertheless, a considerable amount of work is underway that could prove fruitful in the future and some successful demonstrations of in situ chemical treatment have been reported (Williams, 1982).

8.3.1 Soil Flushing/Solution Mining

Soil flushing or solution mining is the process of flooding contaminated soils with water or a water and chemical mixture (e.g., solvents, surfactants) and collecting the elutriated solution. During elutriation, contaminants from the sorbed phase are mobilized into solution by reason of solubility, formation of an emulsion, or by chemical reaction with flushing material.

Soil flushing techniques are typically applied to the latter stages of leachate removal as a method of accelerating the removal of sorbed residual contaminants. Injection treatments such as soil flushing are most applicable

to coarse-grained, permeable materials such as sands and gravels because the flushing material employed has a better chance of contacting the sorbed contaminants. Fine grained, less permeable soils such as clays and silts will not be as effectively treated because of the difficulty in having the solution come in contact with the contaminants.

Solutions with the greatest potential use for soil flushing fall in the following classes:

- Water
- Surfactants
- Acids-bases
- Complexing and chelating agents.

Water is the most feasible and cost effective flushing solution. In fact, the most basic soil flushing process is natural flushing in which clean groundwater is allowed to gradually flush the aquifer of contaminants. Water flushing can be very effective for organic compounds that are hydrophilic and for soluble inorganics.

Dilute aqueous solutions of acids or bases have been used widely to extract metal ions. Rogoshewski, et al., (1982) has suggested the use of sulfuric, hydrochloric, nitric, phosphoric, and carbonic acids to dissolve metal salts. Sulfuric acid is the most widely used acid in solution mining operations, but its use is restricted to metal cations that do not form insoluble sulfates. Solutions of hydrochloric acid can be used in conjunction with metal complexing agents, such as thiourea, to keep metals in solution. Carbon dioxide may be used with ammonia for the leaching of copper and nickel (Rogoshewski, et al., 1982). Also, acids may serve to flush some basic organics, such as amines, ethers and anilines, from soils (Weininger, 1972).

Sodium hydroxide solutions can be used to solubilize aluminum, zinc, tin, lead, and other metals (Rogoshewski, et al., 1982). Also, they may serve as a solvent for substances such as acidic sulfur compounds, phenols, organic acids, and cyanides (Huibregtse and Kastman, 1979).

Complexing and chelating agents also may be used to hold solubilized metals in solution. Commonly used substances include ammonia and ammonium salts, citric acid, ethylene diamine tetraacetic acid (EDTA), and thiourea (Rogoshewski and Carstea, 1980).

Surfactant solutions appear to be very promising for emulsifying non-soluble organics sorbed on the soil phase. The ability to mobilize organic compounds is desirable because much of the hazardous waste disposed of in landfills falls into this class. The use of surfactants also appears desirable at the present time because numerous environmentally safe and relatively inexpensive surfactants are commercially available. Because of the great potential for the use of surfactants in accelerating and enhancing contaminant recovery, the remainder of this section deals with surfactants.

Surfactants are a general class of chemicals whose amphipathetic molecular structures typically contain a hydrophobic structural group that has little affinity for the carrier solvent (water) and a hydrophilic structural group that is readily soluble in the solvent phase (Shaw, 1976). Because of their amphipathetic character, surfactants have a tendency to preferentially concentrate at phase interfaces (e.g., liquid-solid). This results in their unique ability to alter certain aqueous solution properties. In theory, therefore, surfactants can improve the effectiveness of contaminant removal by improving both the detergency of aqueous solutions applied and the efficiency by which organics may be transported by aqueous solutions (Envirosphere Company, 1983).

Theoretically, surfactants will increase the detergency of the flushing solution, and therefore its cleaning power, through the following processes (Envirosphere Company, 1983):

- Preferential Wetting--surfactants improve the wetting properties of an aqueous phase by decreasing the interfacial tension between the aqueous phase and the solid phase, e.g., water and a soil particle (Rosen, 1978).

- Solubilization--surfactants can enhance the ability of the aqueous solution to solubilize the contaminant by causing micelle formation, i.e., clustering of surfactant molecules within an aqueous phase such that surfactant hydrophobic groups are directed toward the interior of the micelle clusters and hydrophilic groups toward the solvent water molecules.

- Emulsification--surfactants can promote the dispersion of an insoluble organic phase within the aqueous phase (emulsification), resulting in enhanced detergency.

Besides improving the detergency of the flushing solution, surfactants can potentially increase the efficiency with which aqueous solution may entrain and transport contaminants through soils (Envirosphere Company, 1983). Contaminants which do not preferentially wet soil particles try to assume geometrical configurations with a minimum surface area and a minimum free surface energy, i.e., spherical droplets. In order to move these contaminants out of and through the soil, the geometry of the droplet must be distorted. This requires overcoming the interfacial energy barrier, or capillary pressure, which is directly related to the interfacial tension between the aqueous and contaminant phase. The addition of surfactants can lower the interfacial tension between phases and therefore increase contaminant mobility and recovery.

Use of surfactants to date has mainly been restricted to laboratory research. The major thrust of most of this research has been performed by the petroleum industry for tertiary oil recovery (Doe, et al., 1977; Wilson and Brandner, 1977; Cash, et al., 1977; Barakat, et al., 1983). Aqueous surfactants have been proposed for gasoline cleanup. In one study performed by the Texas Research Institute (1979) for the American Petroleum Institute, a surfactant mixture of anionic Richonate YLA and nonionic Hyponic PE-90 was shown to improve the effectiveness of contaminant recovery by up to 40 percent.

A similar laboratory study was performed by Ellis and Payne (1983) in which a contaminated sandy soil was packed into soil columns and washed with a mixture of 2 percent Adsee 799 and 2 percent Hyonic NP90 (both non-ionic surfactants). Soil in the columns was contaminated with either a Murban Crude Oil Distillation Fraction, PCB, or chlorinated phenol mixture (di-, tri-, and

pentachlorophenol). For the Murban Crude Oil contaminated soil, recovery efficiencies were increased from less than 1 percent with pure water to 86 percent with the surfactant mixture. PCB recovery using the surfactant increased from less than 1 percent with pure water to 68 percent. In the case of the chlorinated phenol mixture, water washing extracted 70 percent of the contaminants and futher washing with the surfactant had a negligible effect. Based on the study results, surfactant flushing of contaminated soils can greatly increase the efficiency of recovery of some contaminants over water flushing alone.

Because of the lack of data, pertaining to surfactant use in enhancing organic contaminant recovery, a listing of contaminants and usable surfactants for increasing extraction efficiency cannot be compiled at this time. However, guidelines for selecting an appropriate surfactant can be made. The factors that should be considered in the selection process are (Ellis and Payne, 1983):

- High solubility of the surfactant in water
- Ablility of the surfactant to disperse hydrophobic hydrocarbons
- Minimal suspension of fine soil particles
- Environmentally safe (i.e., low toxicity and easily biodegraded)
- Low cost.

Tables 8-12 and 8-13 list some of the characteristics of surfactants and their environmental and chemical properties. These tables can be used to aid in the preliminary selection of a surfactant. However, after the surfactant or mixture of surfactants has been chosen, laboratory testing of the surfactant in shaker table and column tests should be done to verify the properties of the surfactant.

8.3.2 Oxidation/Reduction

Chemical oxidation is the process by which the oxidaton state of a compound is increased by the loss of electrons. Conversely, reduction is the process by which the oxidation state of a compound is decreased by the

TABLE 8-12.

SURFACTANT CHARACTERISTICS (Envirosphere Company, 1983)

Surfactant Type		Selected Properties and Uses	Solubility	Reactivity
ANIONIC	1) Carboxylic Acid Salts	• Good Detergency	• Generally Water Soluble	• Electrolyte Tolerant
	2) Sulfuric Acid Ester Salts	• Good Wetting Agents		• Electrolyte Sensitive
	3) Phosphoric & Polyphosphoric Acid Esters	• Strong Surface Tension Reducers	• Soluble in Polar Organics	• Resistant to Biodegradation
	4) Perfluorinated Anionics			• High Chemical Stability
	5) Sulfonic Acid Salts	• Good Oil In Water Emulsifiers		• Resistant to Acid and Alkaline Hydrolysis
CATIONIC				
	1) Long Chain Amines	• Emulsifying Agents	• Low or Varying Water Solubility	• Acid Stable
	2) Diamines & Polyamines	• Corrosion Inhibitor		
	3) Quaternary Ammonium Salts		• Water Soluble	• Surface Adsorption to Silicaeous Materials
	4) Polyoxyethylenated Long Chain Amines			
NON IONIC	1) Polyoxyethylenated Alkylphenols Alkylphenol Ethoxylates	• Emulsifying Agents	• Generally Water Soluble	• Good Chemical Stability
	2) Polyoxyethylenated Straight Chain Alcohols and Alcohol Ethoxylates	• Detergents	Water Insoluble Formulations	• Resistant to Biodegradation
	3) Polyoxyethylenated Polyoxypropylene Glycols	• Wetting Agents		• Relatively Non-Toxic
	4) Polyoxyethylenated Mercaptans	• Dispersents		
	5) Long-Chain Carboxylic Acid Esters	• Foam Control		• Subject to Acid and Alkaline Hydrolysis
	6) Alkylolamine "Condensates", Alkanolamides			
	7) Tertiary Acetylenic Glycols			

(Continued)

TABLE 8-12. (CONTINUED)

Surfactant Type	Selected Properties and Uses	Solubility	Reactivity
AMPHOTERICS			
1) pH Sensitive	• Solublizing Agents	• Varied (pH dependent)	• Non-Toxic
2) pH Insensitive	• Wetting Agents		• Electrolyte Tolerant
			• Adsorption to Negatively Charged Surfaces

TABLE 8-13.

ENVIRONMENTAL CHEMICAL PROPERTIES OF SELECTED COMMERCIAL SURFACTANTS (Envirosphere Company, 1983)

SURFACTANT CLASS	EXAMPLE	WATER SOLUBILITY	ELECTROLYTE TOLERANCE (HARDNESS - PPM)	PH OF AQUEOUS SOLUTIONS[1]	BIODEGRADATION	TOXICITY[2]
Fluorocarbons	• Lodyne Series (CIBA-GEIGY)	• Soluble	• <300	5.0 - 8.5 (1%)	• Slow	• 3-10 (gm/kg) • Acute Dermal LD50 (Rabbit) 3-10 gm/kg
	• Zonyl Series	• >2gms/100gms			• Slow	• 1-25 (gm/kg)
Sulfonates (Anionics)	• Alkanol Series (DuPont)	• Soluble		7.5 - 10.0 (1%)	• Biodegraded	
Alcohol Ethoxylates (Nonionic)	• Merpol Series (DuPont)	• Generally >30%	• Electrolyte Tolerant	6.0-9.0 (1%)	• Biodegraded (20-60% in 20 days with acclimated bacteria)	• Acute Oral Toxicity for Fish 1-6 (mg/L)
Sulfosuccinates (Anionic)	• OT & Aerosol Series (Cyanamid)	• 1-60gms/100ml	• 500-2500	5.0 - 8.0	• 50-100% in <8 days (CSMA-Shake Culture Test)	• 1-10 (ml/kg)
Alkyl Sulfates (Anionic)	• Duponol Series (DuPont)	• Soluble	• Electrolyte Tolerant	7.5 - 11.0 (3%)	• Biodegraded (days to weeks)	• 2-20 (gm/kg) • Acute Oral Toxicity to Fish (5-20 mg/l)

[1] Parentheses indicate concentration of surfactant.

[2] Toxicity reported as acute oral for rats unless otherwise specified.

addition of electrons. Oxidizing or reducing agents can be injected into a leachate plume to detoxify or otherwise beneficially alter the groundwater contaminants (e.g., oxidizing or reducing contaminants to make them more readily soluble or biodegradable). Many hazardous substances including various organics, sulfites, soluble cyanide and arsenic containing compounds, hydroxylamine, and chromates may be able to be oxidized or reduced to forms which are removed more readily from the groundwater (Huibregtse and Kastman, 1979).

The choice of an oxidizing or reducing agent is dependent on the substance or substances to be detoxified. A single oxidation or reduction agent potentially can detoxify a number of different contaminants in a plume. However, the rate of oxidation or reduction will in all probability be different; therefore, detoxification of some chemicals may be rapid while others may not show signs of detoxification. As with the other chemical injection methodologies, oxidation or reduction techniques are most applicable in coarse-grained deposits and not as effective in fine-grained materials.

Chemical oxidation and reduction techniques have been widely used in treating aqueous waste streams and in wastewater treatments. In this application, undesirable chemical species are treated, rendering them harmless. These types of application are well documented in the literature. However, oxidation or reduction methods for the treatment of hazardous waste piles or contaminant plumes are scant. Therefore, most of the discussion that follows is based upon extrapolation of methods used in other treatment areas and is not based upon actual field applications.

Numerous oxidizing agents are available that can detoxify a variety of compounds. Of these agents, probably three--hydrogen peroxide, ozone, and hypochlorite--have potential for use in detoxifying contaminant plumes. These oxidizing agents appear to be environmentally safe (i.e., easily degraded and tending not to form toxic compounds or residuals) and are relatively inexpensive.

Hydrogen peroxide (H_2O_2) is a moderate strength chemical oxidant (i.e., compared to chlorine) that strongly reacts exothermically with high concentrations of some organic and inorganic wastes; e.g., amines, cyanides, formaldehyde, phenols, ferrous ions, and hypochlorite. Hydrogen peroxide is commercially available in aqueous solutions of a wide concentration range and is miscible with water at all concentrations. These properties make hydrogen peroxide ideally suited for chemical treatment of a contaminant plume using injection well technology.

The major chemical reaction of hydrogen peroxide is oxidation; however, some applications involve molecular additions, substitutions, and reductions as shown in the following expressions (Envirosphere Company, 1983):

- Oxidation -- $H_2O_2 + W \longrightarrow WO + H_2O$
- Molecular addition -- $H_2O_2 + Y \longrightarrow YH_2O_2$
- Substitution -- $H_2O_2 + RX \longrightarrow ROOH + HX$
- Reduction -- $H_2O_2 + Z \longrightarrow ZH_2 + O_2$

These reactions may occur directly after the hydrogen peroxide has ionized or dissociated into free radicals.

Hydrogen peroxide has been used to detoxify both municipal and industrial waste and wastewater. Hydrogen peroxide is used in municipal wastewater treatment to control hydrogen sulfide generation, reduce BOD and COD, and eliminate the need for identification and bulking in activated sludge plants (Envirosphere Company, 1983). For industrial wastewater treatment, hydrogen peroxide has been used to detoxify cyanide and organic pollutants including formaldehyde, phenol, acetic acid, lignin sugars, surfactants, amines and glycol ethers, aldehydes, dialkyl sulfides, dithionate, and certain nitrogen and sulfur compounds (Envirosphere Company, 1983; FMC Corp., 1979). Table 8-14 indicates chemical compound classes that may be degraded using hydrogen peroxide. However, this table is based upon wastewater treatment experience and available literature, not actual use for in situ treatment. Laboratory or pilot scale testing, or both, should be performed using the actual contaminant prior to instituting an in situ treatment program at a waste site.

TABLE 8-14.

WASTE CHEMICAL CLASSES
ABILITY TO REACT WITH HYDROGEN PEROXIDE (Envirosphere Company, 1983)

Chemical Compound	Yes	No	Unknown	Comments
Aliphatic Hydrocarbons	x	x		saturated alkanes unreactive; unsaturated compounds form epoxides and poly-hydroxy compounds
Alkyl Halides			x	
Ethers			x	
Halogenated Ethers and Epoxides			x	
Alcohols	x			require Fe^{+2} catalyst
Glycols, Epoxides	x			
Aldehydes, Ketones	x			may require catalyst
Carboxylic Acids	x			
Amides	x			
Esters			x	
Nitriles	x			
Amines	x			
Azo Compounds, Hydrazine Derivatives	x			
Nitrosamines	x			
Thiols	x			may require catalyst
Sulfides, Disulfides	x			may require catalyst
Sulfonic Acids, Sulfoxides			x	
Benzene and substituted Benzene	x			require Fe^{+2}
Halogenated Aromatic Compounds			x	
Nitrophenolic Compounds	x			
Fused Polycyclic Hydrocarbons	x			
Fused Non-Alterant Polycyclic Hydrocarbon	x			
Heterocyclic Nitrogen Compounds	x			
Heterocyclic Oxygen Compounds	x			
Heterocyclic Sulfur Compounds			x	
Organophosphorus Compounds			x	

Ozone is a strong oxidizing agent (gas) that is very unstable and extremely reactive. Because ozone is unstable and reactive, ozone cannot be shipped or stored; rather, it must be generated on-site immediately prior to application. Ozone decomposes back to oxygen rapidly in solutions containing impurities. Ozone's half life in distilled water at 68°F is 25 minutes, while in groundwater it drops to 18 minutes (Envirosphere Company, 1983). The solubility of ozone in water is dependent on the temperature of the water, where the solubility ranges from 0.64 liter/liter water at 32°F to 0.00 liter/liter water at 140°F.

Oxidation reactions with ozone can occur along three different pathways. These pathways are (Masschelein, 1982):

- Direct oxidation of the solute by ozone
- Oxidation of the solute by OH radicals formed from decomposed ozone
- Oxidation reaction induced by the solute.

Each of these oxidation pathways will result in different types of end products; therefore, the oxidation pathway should be known so that undesirable chemical species are not produced and detoxification of the pollutant occurs.

Ozone has been used in the treatment of drinking water, municipal waste water, and industrial waste. Ozonation has been successfully used on drinking water for the following applications (Rice and Netzen, 1982, as cited by Envirosphere Company, 1983):

- Disinfection and inactivation of bacteria and virus
- Oxidation of soluble iron and manganese, and organically bound manganese
- Removal of color, taste, and odor
- Removal of algae and organics (phenols, detergents, pesticides)
- Microflocculation of dissolved organics
- Oxidation of inorganics (cyanides, sulfides, nitrites)

- Removal of turbidity and suspended solids
- Pretreatment for biological processes (on sand, anthracite, and granular activated carbon).

Industrial wastewaters have been successfully treated by ozonation in the following areas (Rice and Browning, 1981 as cited by Envirosphere Company, 1983):

- Oxidation of cyanide from electroplating wastes
- Decolorization of dye stuffs
- Removal of phenolic compounds
- Recovery and reutilization of spent iron cyanide photoprocessing bleach waters
- Treatment of acid mine discharge
- Treatment of processing wastewaters of mixed ore
- Treatment of organic waste streams.

Based on the literature and past applications of ozone in wastewater treatment, Envirosphere Company (1983) developed a listing of chemical classes that have the potential to be treated by ozonation. These data are given in Table 8-15. Before ozonation can be used to treat chemicals in situ, thorough laboratory and pilot testing should be performed to determine (Envirosphere Company, 1983):

- The competition for ozone among the various compounds in the aqueous solution, so that detoxification rates are known
- The pathway ozonation will take for the various compounds present, so that oxidation by-products can be evaluated for toxicity.

Hypochlorites are a class of chemicals that act as strong oxidizing agents. They are almost always used in an aqueous solution, an ideal form for in situ chemical treatment use. Two forms of hypochlorite are generally available in commercial quantities; calcium hypochlorite ($CA(OCl)_2$) and sodium

TABLE 8-15.

WASTE CHEMICAL CLASSES ABILITY TO REACT WITH OZONE
(Envirosphere Company, 1983)

Chemical Compound	Yes	No	Unknown
Aliphatic Hydrocarbons			
Saturated		X	
Unsaturated	X		
Alkyl Halides			
Ethers	X		
Halogenated Ethers and Epoxides			X
Alcohols	X		
Glycols, Epoxides		X	
Aldehydes, Ketones	X		
Carboxylic Acids	X		
Amides			
Esters	X		
Nitriles			
Amines	X		
A_{ZO} Compounds, Hydrazine Derivatives			
Nitrosamines			
Thiols			
Sulfides, Disulfides	X		
Sulfonic Acids, Sulfoxides	X		
Benzene and Substituted Benzenes	X		
Halogenated Aromatic Compounds	X		
Aromatic Nitro Compounds	X		
Phenols	X		
Halogenated Phenolic Compounds	X		
Nitrophenolic Compounds	X		
Fused Polycyclic Hydrocarbon	X		
Fused Non-Alterate Polycyclic Hydrocarbon			
Heterocyclic Nitrogen Compounds			
Heterocyclic Oxygen Compounds			
Heterocyclic Sulfer Compounds			
Organophosphorus Compounds	X		

hypochlorite (NAOCl). Calcium hypochlorite is available in a form containing about 70 percent available Cl, and sodium hypochlorite is available in 5.25, 10 and 13.03 weight percent of NAOCl (Envirosphere Company, 1983).

Hypochlorite can act as either an oxidizing or chlorinating agent depending on the pH of the organic solution treated. For weakly acidic solutions at moderate pH, the oxidation reaction prevails, while for strong solutions at low pH, the chlorination reaction prevails. Three basic reaction mechanisms exist between hypochlorite and organic compounds:

- Addition
- Substitution
- Oxidation.

Addition and substitution result in the production of chlorinated organic compounds which may or may not be toxic. Oxidation reactions are only effective on a limited number of organic compounds.

Hypochlorite has been used in treating drinking water, municipal wastewater, and industrial waste. Uses in the drinking water and municipal wastewater treatment include control of algae and biofouling organisms, and bleaching of textile and paper products. Industrial waste treatments include the oxidation of cyanide, ammonium sulfide,and ammonium sulfite (Huibregtse and Kastman, 1979). Sodium hypochlorite solutions at concentrations of 2500 mg/l have also been used to detoxify (by oxidation) cyanide contamination from indiscriminate dumping (Farb, 1978).

Potential uses of aqueous solutions of hypochlorite in treating organic compounds in situ are given in Table 8-16 (Envirosphere Company, 1983). Because the principal products from chlorination of organic contaminants are chlorinated organics which can be as much of a problem as the original compound, hypochlorite in situ treatment will be very limited. Based on this information, Envirosphere Company (1983) concluded that the greatest potential lied in the treatment of phenols and phenolic compounds with hypochlorite. However, as with the other two oxidation treatments presented, extensive

TABLE 8-16.

WASTE CHEMICAL CLASSES
ABILITY TO REACT WITH HYPOCHLORITES (Envirosphere Company, 1983)

Chemical Compound	Yes	No	Unknown	Comments
Aliphatic Hydrocarbons	x			Possible chlorination and formation of chloramines
Alkyl Halides	x			Chlorinated product possible
Ethers			x	
Halogenated Ethers and Eposides			x	
Alcohols	x			Forms alkylhypochlorites, hazardous and explosive
Glycols, Epoxides			x	Used in preparation of Epoxides and Glycols from Halohydrin
Aldehydes, Ketones	x			Reaction of acetaldehyde yielding Chloroform ($CHCl_3$)
Carboxylic Acids	x			Chlorinated byproducts possible
Amides	x			Forms chloramines, hydrolysis of C-N bond, possible NCl_3 formation
Esters		x		Will not react unless unsaturated bonds are available for chlorohydrin formation
Nitriles		x		Will not react unless unsaturated bonds are available for chlorohydrin formation
Amines	x			Forms chloramines
Azo Compounds, Hydrazine Derivatives	x			Forms chloramines
Nitrosamines	x			Forms chloramines
Thiols			x	

(Continued)

Table 8-16. (continued)

Chemical Compound	Yes	No	Unknown	Comments
Sulfides, Disulfides	x			Sulfides oxidize to sulfoxides without forming sulfones
Sulfonic Acids, Sulfoxides		x		
Benzene, Substituted Benzene	x			Forms chlorinated aromatic
Halogenated Aromatic Compounds	x			Forms chlorinated aromatic, possible oxidation
Aromatic Nitro Compounds	x			Forms chlorinated aromatic or chloramine
Phenols	x			Forms chlorinated phenols, oxidized to aliphatic acid
Halogenated Phenolic Compounds	x			Oxidized to aliphatic acid
Nitrophenolic Compounds	x			Chlorination of aromatic ring
Fused Polycyclic Hydrocarbons (PNA's)	x			Chlorinated and oxidized products (e.g., phenols and quinolines)
Fused Non-Alterant Polycyclic Hydrocarbons	x			
Heterocyclic Nitrogen Compounds	x			Chlorinated product formed
Heterocyclic Oxygen Compounds	x			Chlorinated product formed
Heterocyclic Sulfur Compounds	x			Chlorinated product formed
Organophosphorous Compounds	x			

laboratory and pilot scale tests would be required before actual field use of the method could be performed.

Reducing agents have been proposed to detoxify wastes and plumes, but their application does not appear to have the potential that oxidizing agents have. For example, sodium sulfites have been proposed to treat groundwater contaminated by sodium hypochlorite, i.e., the reverse of that proposed in the above section (Huibregtse and Kastman, 1979). Ferrous sulfate in conjunction with hydroxides has been proposed to detoxify and insolubilize hexavalent chromium (Tolman, et al., 1978; Metcalf and Eddy, Inc., 1972). Little work has been done in the use of reducing agents for organic wastes.

Major disadvantages are inherent (e.g., costs, control of chemical reaction) with the use of oxidizing and reducing agents for in situ treatment of contaminant plumes, making their use at a hazardous waste site rather unlikely. This technology is generally compound specific making treatment of heterogeneous mixtures of contaminants a difficult problem. Also, the introduction of chemicals into the groundwater system may create a pollution problem in itself. As with soil flushing, uncertainty exists with respect to obtaining adequate contact with the contaminants in the plume.

Costs for oxidation or reduction include the costs for the distribution and collection system, chemical and feed system, and fees for probing, excavation and drilling. Costs for laboratory and pilot scale studies would also have to be considered in performing this type of cleanup. Tolman, et al. (1978) estimated the cost (excluding preliminary studies) of cleaning-up soils at a hypothetical 10-acre disposal site that received a single load of cyanide salts in drums. Cost for the oxidation treatment using sodium hypochlorite may be representative of costs for other oxidation or reduction treatments as well as for other chemical injection technologies. These costs are shown in Table 8-17.

TABLE 8-17.

COSTS FOR IN SITU DETOXIFICATION OF CYANIDE
(Tolman, et al., 1978 as cited by Rogoshewski, et al., 1982)

Item	Cost ($)
Exploratory probing, excavation, and drilling	15,000
Development of water supply well, 90 ft; pump and piping	5,000
Installation of 45 well points	10,000
Cost of chemical feed pump	2,000
Cost of chemical (sodium hypochlorite)	5,400
Labor for chemical injection, raising of well points to flood successive elevations (assumed 4 wells handled simultaneously), and general labor (1,600 hours)	48,000
Power (assumed electrical supply available)	500
Total	$85,900

Assumed 10-acre landfill with a total of 1,566 lbs of cyanide distributed within a fill volume of 4.9 million cubic feet. Chemical application rate of 68 gallons of sodium hypochlorite per pound of cyanide.

8.3.3 Precipitation/Polymerization

Precipitation involves the injection of substances into the leachate plume which form insoluble products with the contaminants, thereby reducing the potential for migration in the groundwater. This technique is mainly applicable to dissolved metals, such as lead, cadmium, zinc, and iron. Some forms of arsenic, chromium, and mercury, and some organic fatty acids can also be treated by precipitation (Huibregtse and Kastman, 1979). The most common precipitation reagents include hydroxides, oxides, sulfides, and sulfates. Also, calcium salts have been suggested to precipitate free fluorides (Tolman, et al., 1978).

Sodium sulfide used in conjunction with sodium hydroxide has shown widespread applicability for precipitation of metals. Precipitation takes place at a neutral or slightly alkaline pH. Resolubilization of sulfides is low. Addition of sodium hydroxide minimized the formation of hydrogen sulfide gas by assuring an alkaline pH. Experiments with sulfide precipitation of zinc indicate that a high residual of unreacted sulfide may remain in solution.

As with other in situ techniques, precipitation is only applicable to sites with aquifers having high hydraulic conductivities. Major disadvantages include the application to a narrow, specific group of chemicals (mainly metals), the injection of a potential groundwater pollutant, formation of toxic gases (in the case of sulfide treatment), and the possibility of resolubilization of the precipitate. The design and economics are similar to other chemical treatment techniques.

In situ polymerization involves the injection of a polymerization catalyst into a non-aqueous organic phase of a leachate plume to cause polymerization. The resulting polymer is gel-like and non-mobile in the groundwater flow regime. Polymerization is a very specific technique that is applicable to organic monomers such as styrene, vinyl chloride, isoprene, methyl methacrylate, and acrylonitrile, for example (Huibregtse and Kastman, 1979). In a hazardous waste site situation where groundwater pollution has occurred over time, any organic monomers originally present would most likely

have already polymerized in containers or upon contact with the soil. Therefore, in situ polymerization is a technique most suited for groundwater cleanup following land spills or underground leaks of pure monomer. Applications to uncontrolled hazardous waste sites are unlikely. Major disadvantages include very limited application and difficulty of initiating sufficient contact of the catalyst with the dispersed monomer (Huibregtse and Kastman, 1979).

In situ polymerization was successfully performed to remedy an acrylate monomer leak, in which 4,200 gallons of acrylate monomer leaked from a corroded underground pipeline into a glacial sand and gravel layer. Soil borings indicated that as much as 90 percent of the monomer had been polymerized by injection of a catalyst, activator, and wetting agent (Williams, 1982).

The in situ treatment of a leachate plume using a precipitation or polymerization technique probably has very limited application. Problems associated with these techniques include:

- Chemicals must come in contact with the contaminants to ensure complete precipitation or polymerization, thus limiting the technique's applicability to coarse-grained aquifers
- Closely spaced injection wells are required, even in coarse-grained deposits, because the action of precipitation or polymerization will lower hydraulic conductivities near the injection wells, thus reducing effectiveness of treatment
- Contaminants are not removed from the aquifer and some chemical reactions can be reversed, allowing contaminants to again migrate with groundwater flow
- Additional contamination can be caused by injecting a treatment agent that is a potential groundwater pollutant or by the formation of toxic by-products.

Therefore, prior to the application of an in situ precipitation or polymerization technique at a hazardous waste site, thorough testing in the laboratory and on the pilot scale should be initiated to determine possible deleterious

affects and ensure complete precipitation or polymerization of the chemical compounds.

8.3.4 Neutralization/Hydrolysis

In situ neutralization is the treatment technique that involves injecting dilute acids or bases into a leachate plume for the purpose of adjusting the plume's pH. Neutralization has a number of applications as presented below:

- Pretreating the leachate plume where bioreclamation is to be implemented; bioreclamation requires a pH range from 6 to 8; acids or bases may also act as cell nutrients (ammonia, nitric acid, phosphoric acid)
- Preventing the formation of hydrogen sulfide and hydrogen cyanide during oxidation or reduction treatments or sulfide precipitation
- Adjusting pH of a plume to optimum conditions prior to in situ oxidation or reduction treatment
- Restoring groundwater to an acceptable pH after using another treatment technique, using neutralizing materials that do not seriously affect groundwater quality (e.g., carbonic acid, calcium hydroxide)
- Neutralizing plumes that are either acidic or basic in composition, and that do not require another type of treatment.

In theory, strong bases are the most economical solutions to treat strong acids and strong acids are the most economical to treat strong bases. However, because of the problems associated with handling strong acids or bases and the potential for the neutralization agent to move away from the affected area, the use of weaker acids and bases that will not cause the pH to move outside the range of 6 to 9 if an excess volume is used are recommended. Prior to trying to neutralize a plume with an acid or base, laboratory and bench scale testing should be performed to determine the effectiveness of reagents in neutralizing the chemicals present and ensure that unwanted toxic by-products are not generated.

Besides using acid and bases for adjusting the pH of a plume, they can be used to increase the hydrolysis rate of some organic chemicals. Hydrolysis is the chemical reaction of water with an organic molecule and can be expressed by the following displacement reaction:

$$RX + H_2O \longrightarrow ROH + HX$$

where R is the organic moiety and X is the leaving group in the hydrolysis reaction. The hydrolysis reaction can occur along a variety of pathways which can result in different by-products that can be more toxic than the parent compound. Therefore, before a hydrolysis reaction is promoted, the pathways for reactions should be determined to ensure that toxic by-products are not produced.

The rate of the hydrolysis reaction is dependent on:

- pH
- Temperature
- Solvent composition
- Catalysts.

Of the parameters listed above, changing the pH of the plume has the greatest potential for adjusting the rate of in situ hydrolysis, since this factor can be controlled by the injection of chemicals. Because the hydrolysis rate of a given compound may be the sum of the neutral, acid-catalyzed, and base-catalyzed processes, the effect of pH on the acid- or base-catalyzed hydrolysis rate can be pronounced (Envirosphere Company, 1984). Changes of up to one order of magnitude for a one standard unit of change in pH can be realized. Envirosphere Company (1984) classified numerous organic chemicals that could be expected to show increased hydrolysis rates, when the pH of the solution is adjusted. Tables 8-18 through 8-24 summarizes their work, listing only those chemicals with hydrolysis rates that can be affected by pH changes. Attempts have not been made to classify by-products produced during hydrolysis.

TABLE 8-18.

HYDROLYSIS OF ALKYL HALIDES (Envirosphere Company, 1984)

Class/Compound	k_A	k_N	k_B	t/1/2 at pH						
				5	6	7	8	9	10	11
Methyl Fluoride	-	7.44E-10	5.82E-7	30y	30y	30y	30y	30y	30y	15y
Methyl Iodide	-	7.28E-8	6.47E-5	110d	110d	110d	110d	110d	110d	55d
Methylene Chloride	-	3.2E-11	2.13E-8	704y	704y	704y	704y	704y	704y	350y
CH_2CHCH_2Cl	-	1.16E-7	6.24E-5	69d	69d	69d	69d	69d	69d	44.9d
$CHCl_3$	-	-	6.9E-5	350,000y	35,000y	3500y	350y	35y	3.5y	0.35y
$CHBrCl_2$	-	-	1.6E-3	13,700y	1370y	137y	13.7y	1.37y	50d	5d
$CHBr_2Cl$	-	-	8.0E-4	27,400y	2740y	274y	27.4y	2.74y	100d	10d
$CHBr_3$	-	-	3.2E-4	6,600y	6860y	686y	68.6y	6.86y	250d	25d
$CHICl_2$	-	-	8.0E-4	27,500y	2750y	275y	27.5y	2.8y	0.28y	10d
CHFICl	-	-	2.2E-1	-	-	1.0y	36.5d	3.7d	0.37d	0.037d

Note: All values reported for 25°C. Rate constants in sec-1.

k_A = acid-catalyzed hydrolysis constant

k_N = neutral hydrolysis constant

k_B = base-catalyzed hydrolysis constant

t/1/2 = half life time

y = year

d = day

h = hour

m = minute

s = second

TABLE 8-19.

HYDROLYSIS OF EPOXIDES (Envirosphere Company, 1984)

Compound	k_A	k_N	k_B	t/1/2 at pH						
				5	6	7	8	9	10	11
1,2-Epoxy ethane	1E-2	6.7E-7	-	12d	12d	12d	12d	12d	12d	11d
1,2 Epoxy-2-methyl propane	7.3	1.1E-6	-	0.1d	.95d	4.4d	6.8d	7.2d	7.3d	7.3d
1,2 Epoxy-3-hydroxy propane	2.5E-3	2.84E-7		26d	28d	28d	28d	28d	28d	28d
1,2-Epoxy-2-methyl-3-hydroxy propane	1.1E-2	4E-7	-	13d	16d	16d	16d	16d	16d	16d
1,2-Epoxy-2-methyl-3-chloropropane	1.84E-3	5E-7	-	15d	16d	16d	16d	16d	16d	16d
Trans-2,3 epoxy butane	1.2E-1	5E-7	-	4.7d	13d	15.7d	16d	16d	16d	16d
Cis-2,3-epoxy butane	2.4E-1	5E-7	-	2.8d	11d	15.3d	16d	16d	16d	16d

Note: All values reported at 25°C. Rate constants in sec-1

k_A = acid-catalyzed hydrolysis constant

k_N = neutral hydrolysis constant

k_B = base-catalyzed hydrolysis constant

t/1/2 = half life time
y = year
d = day
h = hour
m = minute
s = second

TABLE 8-20.

HYDROLYSIS OF ESTERS (Envirosphere Company, 1984)

Compound	k_A	k_N	k_B	t/1/2 at pH						
				5	6	7	8	9	10	11
Ethyl Acetate	1.1E-4	1.5E-10	1.1E-1	16y	16y	2.0y	0.2y	7.3d	0.73d	0.073d
Isopropyl Acetate	6.0E-5	-	2.6E-2	35y	68y	8.4y	308d	31d	3.1d	0.31d
Butyl Acetate	1.3E-4	-	1.5E-3	1.6y	78y	140y	1.5y	015y	5.5d	0.5d
Vinyl Acetate	1.4E-4	1.1E-7	1.0E+1	73d	67d	7.3d	0.8d	1.9h	12m	1.2m
Allyl Acetate	-	-	7.3E-1	30y	3.0y	110d	11d	1.1d	2.6h	0.26h
Benzyl Acetate	1.1E-4	-	2.0E-1	17y	10.4y	1.1y	40d	4.0d	9.6h	1h
O-acetyl phenol	7.8E-5	6.6E-8	1.4	119d	100d	38d	5.5d	0.57d	1h	5m
2,4-dinitrophenyl acetate	-	1.1E-5	9.4E+1	17h	16h	9.4h	1.8h	0.2h	1.2m	1+1m
$ClCH_2C(O)OCH_3$	8.5E-5	2.1E-7	1.4E+2	23d	5.0d	14h	1.4h	8.4	1m	1+1m
$Cl_2CHC(O)OCH_3$	2.3E-4	1.5E-5	2.8E+3	11h	4.5h	38m	4.1m	1+1m	1+1m	1+1m
$Cl_2CHC(O)OC_6H_5$	-	1.8E-3	1.3E+4	6.4m	6.1m	3.7m	1+1m	1+1m	1+1m	1+1m
$F_2CHC(O)OC_2H_5$	-	5.7E-5	4.5E+3	3.3h	1.9h	23m	2.6m	1+1m	1+1m	1+1m
$CH_3SCH_2C(O)OC_2H_5$	-	-	9.2E-1	24y	2.4y	87d	8.7d	0.87d	2.1h	13m
$CH_3S(O)CHC(O)C_2H_5$	-	-	1.3E+1	1.7y	62d	6.2d	0.62d	1.5h	9m	1m
$(CH_3)_2SCH_2C(O)$-OC_2H_5	-	-	2.0E+2	40d	96h	9.6h	1h	6m	1+1m	1+1m
$C_2H_5C(O)OC_2H_5$	3.3E-5	-	8.7E-2	52.3y	24.4y	2.5y	91d	9.1d	0.9d	2.2h
$C_3H_7C(O)OC_2H_5$	1.8E-5	-	3.8E-2	1000y	55y	5.8y	212d	21d	2.1d	5h
$(CH_3)_2CHC(O)OC_2H_5$	-	-	2.3E-2	960y	96y	9.6y	350d	35d	3.5d	0.35d
$CH_2CHC(O)OC_2H_5$	1.2E-6	-	7.8E-2	244y	35y	3.5y	128d	13d	1.3d	0.13d

(continued)

TABLE 8-20. (Continued)

Compound	k_A	k_N	k_B	t/1/2 at pH						
				5	6	7	8	9	10	11
trans-$CH_3CHCHC(O)$-OC_2H_5	6.3E-7	-	1.3E-2	1140y	160y	17y	1.7y	62d	6.2d	0.62d
$CHCC(O)OC_2H_5$	-	-	4.68	4.7y	170d	17d	1.7d	0.17d	24m	2.4m
$C_6H_5C(O)OCH_3$	4.0E-7	-	1.9E-3	3720y	1160y	118y	11.8y	1.2y	0.12y	4.4d
$C_6H_5C(O)OC_2H_5$	-	-	3.0E-2	730y	73y	7.3y	0.73y	27d	2.7d	0.27d
$C_6H_5C(O)OCH(CH_3)_2$	-	-	6.2E-3	3500y	350y	35y	3.5y	128d	12.8d	1.3d
$C_6H_5C(O)OCH_2C_6H_5$	-	-	8.0E-3	2700y	270y	27y	2.7y	99d	9.9d	1.0d
p-NO_2-$C_6H_4C(O)OCH_3$	4.3E-7	-	7.4E-2	282y	30y	3.0y	0.3y	11d	1.1d	2.6h
p-NO_2-$C_6H_4C(O)OCH_3$			6.4E-1	34y	3.4y	0.34y	12.4d	1.2d	2.6h	0.26h
p-NO_2-$C_6H_4C(O)$-OC_2H_5	1.4E-7	-	2.4E-1	92y	9.2y	0.92y	34d	3.4d	0.34d	0.8h
1-$C_5H_4NC(O)OC_2H_5$	-	-	5.4E-1	41y	4.1y	0.41y	15d	1.5d	3.6h	0.36h
o-$C_6H_4[C(O)OC_2H_5]_2$	-	-	1.0E-2	2200y	220y	22y	2.2y	80.3d	8.0d	0.8d
o-$C_6H_4[C(O)OCH_2$-$C_6H_5]_2$	-	-	1.7E-2	1300y	130y	13y	1.3y	47.5d	4.8d	0.48d
p-$C_6H_4(CCO)OCHH_3]_2$	-	-	2.5E-1	880y	88y	0.88y	32d	3.2d	7.7h	0.77h
p-$C_6H_4[C(O)OC_2H_5]_2$	-	-	6.9E-2	320y	32y	3.2y	117d	11.7d	1.2d	2.6h

Note: All values reported for 25°C. Rate constants in sec-1.

k_A = acid-catalyzed hydrolysis constant
k_N = neutral hydrolysis constant
k_B = base-catalyzed hydrolysis constant
t/1/2 = half life time
y = year
d = day
h = hour
m = minute
s = second

TABLE 8-21.

HYDROLYSIS OF AMIDES (Envirosphere Company, 1984)

Compound	k_A	k_N	k_B	t/1/2 at pH						
				5	6	7	8	9	10	11
Acetamide	8.36E-6	-	4.71E-5	262y	2490y	3950y	465y	46.5y	4.65y	0.47y
Valeramide	5.43E-6	-	1.41E-5	404y	3950y	11,300y	1560y	156y	15.6y	1.0y
Isobutyamide	4.63E-6	-	2.40E-5	470y	4500y	7700	915y	915y	9.2y	0.92y
Cyclopentanecarboxamide	2.34E-5	-	1.67E-5	93.9y	931y	5500y	1300y	1.32y	13.2y	132y
Methoxy acetamide	7.84E-6	-	3.95E-4	280y	1860y	500y	55.6y	5.6y	0.56y	20d
Chloroacetamide	1.1E-5	-	1.5E-1	84.5y	14.6y	1.46y	0.15y	5.5d	0.55d	1.3h
Dichloroacetamide	-	-	3.0E-1	73y	7.3y	0.73y	26.6d	2.7d	6.5h	0.6h
Trichloroacetamide	-	-	9.4E-1	23y	2.3y	0.23y	8.4d	0.84d	2.0h	12m
Bromoacetamide	-	-	1.03E-5	$2x10^6y$	$2x10^5y$	21,200y	2120y	212y	21.2y	2.1y
N-methylacetamide	3.2E-7	-	5.46E-6	6900y	58,600y	38,000h	4020y	402y	40y	4.0y
N-ethylacetamide	9.36E-8	-	3.10E-6	23,000y	$1.8x10_{5y}$	70,000y	7090y	709y	71y	7.1y

Note: All values at 25°C. Rate constants in sec-1.

k_A = acid-catalyzed hydrolysis constant

k_N = neutral hydrolysis constant

k_B = base-catalyzed hydrolysis constant

t/1/2 = half life time
y = year
d = day
h = hour
m = minute
s = second

TABLE 8-22.

HYDROLYSIS OF CARBAMATES (Envirosphere Company, 1984)

Compound	k_A	k_N	k_B	t/1/2 at pH						
				5	6	7	8	9	10	11
$Ch_3OC(O)N(H)C_6H_5$	-	-	5.5E-5	$4x10^5y$	40,000y	4,000y	400y	40y	4y	146d
$C_2H_5O(CO)N(CH_3)C_6H_5$	-		5.0E-6	4.4x106hr	$44.4x10^5y$	44,000y	4,400y	440y	44y	4.4y
$C_6H_5O(CO)N(H)C_6H_5$	-	-	5.42E1	150d	15d	1.5d	3.6h	21m	2m	13s
$C_6H_5O(CO)N(CH_3)C_6H_5$	-	-	4.2E-5	$5.2x10^5y$	52,000y	5200y	520y	52y	5.2y	191d
$p\text{-}CH_3OC_6H_4(O)N(H)\text{-}C_6H_5$	-	-	2.5E1	320d	32d	3.2d	7.7h	46m	4.6m	28s
$m\text{-}ClC_6H_4OC(O)N(H)\text{-}C_6H_5$	-	-	1.8E3	4.5d	11h	1.1h	6.4m	19s	3.9s	0.4s
$p\text{-}NO_2C_6H_4OC(O)N(H)\text{-}C_6H_5$	-	-	2.7E5	43m	4.3m	26s	2.5s	0.3s	0.03s	0.003s
$p\text{-}NO_2C_6H_4OC(O)N\text{-}(CH_3)C_6H_5$	-	-	8.-0E-4	27,500y	2750y	275y	27.5y	2.7y	100d	10d
$1\text{-}C_{10}H_9OC(O)N(H)CH_3$	-	-	9.4E0	2.3y	85d	8.5d	20h	2h	1.2m	74s
$(C_2H_5)_2NCH_2CH_2OC\text{-}(O)N(H)C_6H_5$	-	-	2.6E-5	$8.5x10^5y$	85,000y	8400y	850y	85y	8.5y	310d
$(C_2H_5)_2NCH_2CH_2OC\text{-}(O)N(H)C_6H_3(CH_3)_3$	-	-	9.4E-7	$2.3x10^7y$	$2.3x10^6y$	240,000y	23,000y	2300y	230y	23y
$(CH_3)_3NC_6H_4OC(O)N\text{-}(H)CH_3$	-	-	6.7E-1	33y	3.3y	120d	12d	1.2d	2.9h	1.7m
$(CH_3)_3NC_5H_4OC(O)N\text{-}(CH_3)_2$	-	-	2.8E-4	89,500y	7850y	785y	78.5y	7.9y	268d	29d
$ClCH_2CH_2OC(O)N(H)\text{-}C_6H_5$	-	-	1.6E-3	24,000y	1400y	140y	14y	1.4y	50d	5d
$Cl_2CHCH_2OC(O)NHC_6H_5$	-	-	5.0E-2	440y	44y	4.4y	160d	16d	1.6d	3.9h
$CCl_3CH_2OC(O)NIC_6H_5$	-	-	3.2E-1	69y	6.9y	252d	25d	2.5d	6h	36m
$CF_3CH_2OC(O)NHC_6H_5$	-	-	1.0E-1	220y	22y	2.2y	80d	8d	20h	

Note: All values at 25°C. Rate constants in sec-1.

k_A = acid-catalyzed hydrolysis constant

k_N = neutral hydrolysis constant

k_B = base-catalyzed hydrolysis constant

t/1/2 = half lift time
y = year
d = day
h = hour
m = minute
s = second

TABLE 8-23.

HYDROLYSIS OF PHOSPHORIC AND PHOSPHONIC ACID ESTERS (Envirosphere Company, 1984)

Compound	k_A	k_N	k_B	t/1/2 at pH						
				5	6	7	8	9	10	11
$CH_3P(O)(PCH_3)_2$	1.36E-9	-	2.5E-3	8700y	880y	88y	8.8y	321d	32d	3.2d
$CH_3P(O)(OC_2H_5)_2$	1.7E-9	-	2.2E-4	93,000y	9980y	990y	100y	10y	1y	3bd
$CH_3P(O)(OCH(CH_3)_2)_2$	6.4E-9	-	3.2E-9	$3.4x10^5y$	$2.3x10^6y$	663,000y	69,000y	6900y	690y	69y
$Ch_3P(O)(OC_2H_5)(O\text{-}p\text{-}C_6H_4NO_2)$	1.2E-7	-	4.0E-2	530y	55y	5.5y	200d	20d	2d	4.8h
$C_2H_5P(O)(OCH(CH_3)_2)_2$	3.2E-9	-	3.7E-8	$6.9x10^5y$	$6.2x10^6y$	$5.5x10^6y$	$5.9x10^5y$	59,000y	5900y	590y
$C_6H_5P(O)(OC_2H_5)_2$	1.1E-9	-	5.0E-4	43,000y	4400y	440y	44y	4.4y	160d	16d
$(CH_3O)_3PO$	-	1.8E-9	1.3E-4	1.2y	1.2y	1.22y	1.2y	1.1y	260d	54d
$(C_2H_5O)3PO$	-	4.0E-9	8.2E-6	5.5y	5.5y	5.5y	5.5y	5.4y	4.6y	1.8y
$(C_2H_5S)_3PO$	-	1.4E-9	1.2E-2	165	14y	8.5y	1.6y	66d	7d	16h
$(C_6H_5O)_3PO$	-	2.7E-11	1.7E-2	550y	112y	13y	1.3y	47d	5d	11h
$(C_2H_5))O(O)(\text{-}p\text{-}C_6H_4NO_2)$	-	3.3E-6	5.3E-1	2d	2d	2d	2d	9h	3h	22m
$(p\text{-}C_6H_4NO_2)_3PO$	-	1.0E-3	3.43E-1	12m	12m	11m	11m	9m	3m	20s
$(CH_3O)_2P(S)p\text{-}C_6H_4NO_2$	-	1.1E-7	5.95E-3	73d	73d	72d	69d	47d	11d	1.3d
$CH_3OP(S)SCHCH\text{-}(CO_2C_2H_5)_2$	-	-	4.3	5y	187d	18d	1.9d	4.5h	27m	3m
$(C_2H_5O)_2P(S)(p\text{-}C_6H_4NO_2)$	-	3.0E-9	2.2E-4	7y	7y	7y	7y	4y	321d	

Note: All valves reported for T=25°C. Rate constants in sec-1.

k_A = acid-catalyzed hydrolysis constant

k_N = neutral hydrolysis constant

k_B = base-catalyzed hydrolysis constant

t/1/2 = half life time
y = year
d = day
h = hour
m = minute
s = sec

TABLE 8-24.

HYDROLYSIS OF MISCELLANEOUS COMPOUNDS INCLUDING PESTICIDES (Envirosphere Company, 1984)

Compound	k_A	k_N	k_B	t/1/2 at pH						
				5	6	7	8	9	10	11
Dimethyl sulfate	-	1.66E-4	1.48E-2	1.2h	1.2h	1.2h	1.2h	1.2h	1.1h	1.1h
Methoxychlor	-	2.99E-9	3.64E-4	270d	270d	270d	267d	241d	121d	20d
Captan	-	1.87E-5	5.7E+2	10h	8h	3h	20m	2m	12s	1s
Malathion	4.8E-5	7.7E-9	5.5E0	1.6h	128d	14d	1.5d	3.5h	21m	2m
Parathion	-	4.5E-8	2.3E-2	178d	177d	170d	118d	29d	3.4d	8.4h
Paraoxon	-	4.1E-8	1.3E-1	195d	190d	149d	47d	6d	15h	1.5h
Diazinon	2.1E-2	4.3E-8	5.3E-3	32d	125d	176d	165d	14d	14d	1.5d
Diazoxon	6.4E-1	2.8E-7	7.6E-6	1.2	9d	23d	28d	29d	29d	29d
Chloropyrifos	-	1.0E-7	1.0E-1	80d	79d	73d	40d	7d	19h	2h
Sevin	-	-	7.7	2.9h	104d	10d	1d	2.5h	15m	1.5m
Sevin	-	-	3.4	6.5y	236d	24d	2.4d	5.7h	34m	3.4m
Baygon	-	-	4.6E-1	48y	4.8y	174d	17d	1.7d	4.2h	25m
Pyrolam	-	-	1.1E-2	2000y	200y	20y	2y	73d	7.3d	18h
Dimetilan	-	-	5.7E-5	3.9×10^5y	39,000y	3900y	390y	39y	3.9y	141d
P-Nitrophenyl-N-methyl carbamate	-	4.0E-5	3.0E+3	4.5h	2.8h	34m	3.8m	23s	2.3s	0.23s
2,4-D,m-butoxyethylester	2.0E-5	2.0E-5	3.02E+1	9.6h	9.5h	8.4h	4.4h	36m	4m	23s
Methoxychlor	-	2.8E-8	2.8E-4	286d	286d	286d	283d	252d	122d	20d
DDT	-	1.9E-9	9.9E-3	12y	11y	7.6y	1.9y	79d	8d	19h
2,4-D,methylester	-	-	1.7E+1	1.3y	47d	4.7d	11h	1.1h	6.8m	41s

Note: All values for 25°C. Rate constants in sec-1.

k_A = acid-catalyzed hydrolysis constant

k_N = neutral hydrolysis constant

k_B = base-catalyzed hydrolysis constant

t/1/2 = half life time
y = year
d = day
h = hour
m = minute
s = second

Based upon Envirosphere Company's work, the following classes of compounds have the potential for in situ degradation by hydrolysis:

- Esters
- Amides
- Carbamates
- Phosphoric and phosphonic acid esters
- Pesticides.

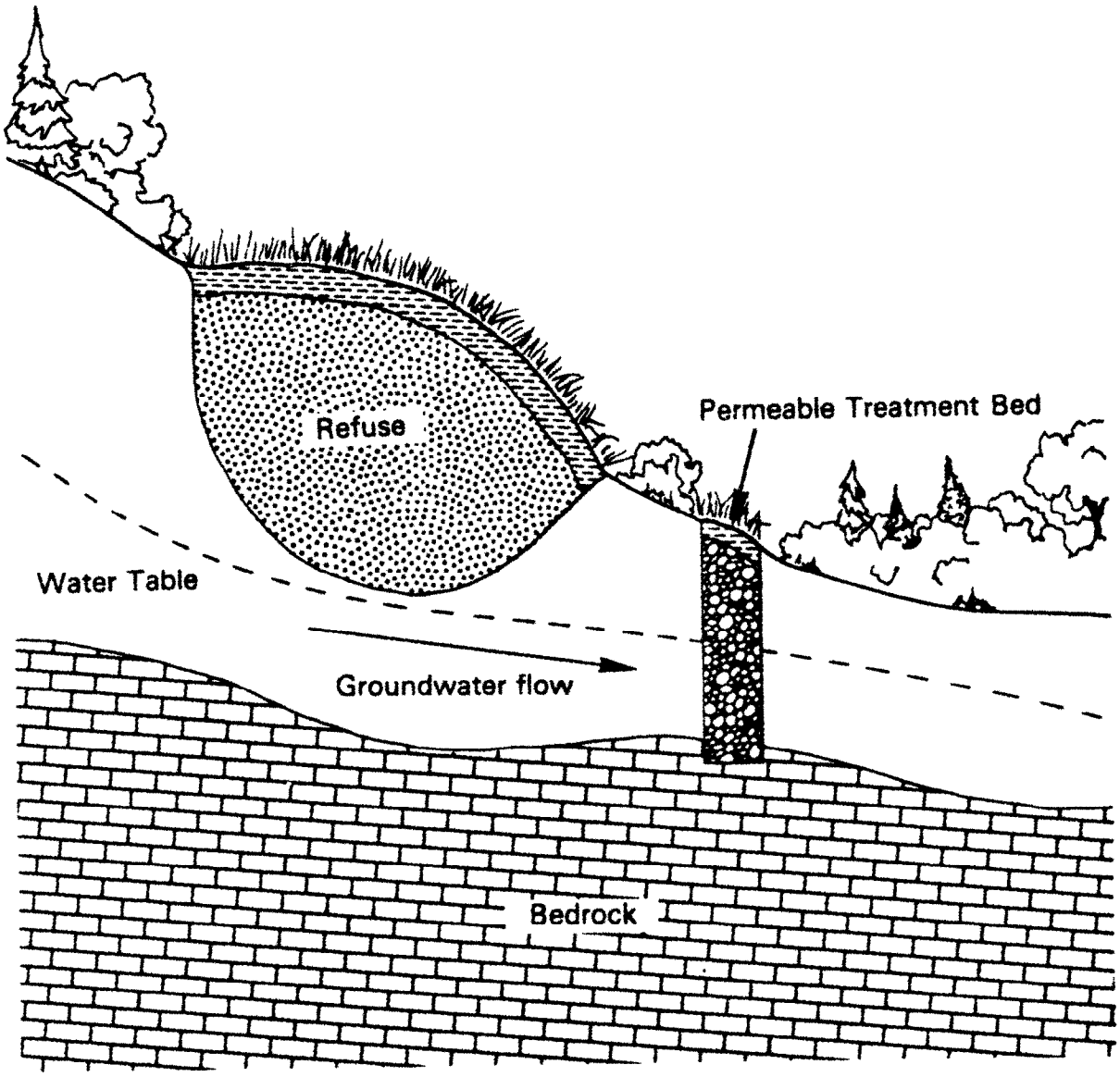
Because actual field application to plume treatment is not available, thorough laboratory and pilot-scale testing should be performed prior to using hydrolysis as an in situ treatment technique.

Both neutralization and hydrolysis suffer from the same disadvantages as the other chemical treatment techniques, such as possible incomplete contact with the leachate plume, application limited to coarse-grained strata, and possible production of toxic by-products. However, these techniques are more applicable than some of the other in situ methods and can be applied to almost any leachate plume requiring pH adjustment.

8.3.5 Permeable Treatment Beds

Permeable treatment beds are a potential in situ treatment technique applicable to sites with relatively shallow groundwater tables containing a plume. In contrast to the conventional approaches of containing or removing a plume, the concept of a permeable treatment bed involves (Figure 8-8) excavating a trench, filling the trench with permeable treatment material, and allowing the plume to flow through the bed thus physically removing or chemically altering the contaminants. The conceptual function of a permeable treatment bed is to reduce the quantities of contaminants present in a leachate plume to acceptable levels. To date, permeable treatment beds have not been used at hazardous waste sites. However, bench- and pilot-scale testing has provided preliminary quantification of treatment bed effectiveness. Numerous potential problems exist in using a permeable treatment bed. These include saturation of bed material, plugging of bed with precipitates,

FIGURE 8-8.
INSTALLATION OF A PERMEABLE TREATMENT BED



and short life of the treatment material. Therefore, permeable treatment beds should probably be considered a temporary remedial action rather than a permanent one.

Selecting appropriate bed material to treat the contaminants present and designing the bed to function properly are two elements that partially determine the effectiveness of an installed permeable treatment bed. Types of currently available treatment bed fill material include limestone, crushed shell, activated carbon, glauconitic greensands, and synthetically produced ion exchange resins. Ensuring proper physical design of the treatment bed requires a knowledge of the hydrogeology of the site (e.g., groundwater flow rate and direction, hydraulic conductivities) and the chemical characteristics of the plume (i.e., types and concentrations of constituents).

8.3.5.1 Bed Fill Material

Relatively few fill materials exist that can feasibly be considered for permeable beds to control contaminated groundwater. These materials include:

- Limestone
- Activated carbon
- Glauconitic greensands
- Zeolites and synthetic ion exchange resins.

Knowledge of the type and concentration of contaminants targeted for treatment or removal by the bed is essential to selecting appropriate fill material. The four potentially useful bed materials listed are described in detail in the following sections.

8.3.5.1.1 Limestone

A crushed limestone bed may be applied in cases where neutralization of acidic groundwater flow is needed. Limestone beds have also been reported to be effective in removing certain metals such as cadmium, iron, and chromium (EPA, 1978). Crushed shell, which has similar chemical characteristics as

limestone, can also be used as a material for permeable treatment beds. In most coastal regions where the availability of shells is good, crushed shell can become a very useful material for permeable treatment beds, and may compete with limestone in the control of acidic groundwater by this method.

Fuller (EPA, 1978) studied the neutralization effect that a crushed limestone bed had on acidic water passed through the bed. Results of the study showed that the contact time required to change the acidic water 1 pH unit was in the range of 8 to 15 days. Extrapolating these results to a permeable treatment bed application would result in a permeable bed having either very low hydraulic conductivities or a large width to attain adequate contact time.

Fuller and other researchers (EPA, 1978) have discussed the use of crushed limestone as an effective low-cost landfill liner to aid in the attenuation of the migration of certain heavy metals from solid waste leachates. According to the authors, dolomitic limestone (containing significant amounts of magnesium carbonate) is less effective in removing ions than purer limestone containing little magnesium carbonate. Therefore, in the design of a limestone treatment bed, limestone with high calcium content is recommended to remove heavy metals and to neutralize contaminated groundwater.

Little information is presently available on determining the contact time needed for the optimum removal of heavy metals. According to Fuller (EPA, 1978), the efficiency of limestone in removing heavy metals from leachate depends heavily on contact time, leachate concentration, and leachate pH. Limestone has been shown to be very effective in attenuating the migration rate of chromate, but an explanation for the mechanism of removal of this metallic salt is not available. Removal of metallic cations such as beryllium, cadmium, nickel, and zinc from leachate were also studied, but conclusive results were not obtained. Thus, more studies are needed to determine the effectiveness of limestone in removing heavy metals from landfill leachate and the optimum contact time for the removal process.

In regard to designing vertical permeable treatment beds, the particle size of limestone used should be selected dependent on the type of soil in which the groundwater flows (i.e., which controls flow rates) and the level of contamination. In general, a mixture of gravel size and sand size limestone should be used to minimize settling through dissolution. Where excessive channelling through the bed by rapid groundwater movement is expected or where improved contact time between the contaminated groundwater and the treatment bed is required, a higher percentage of sand size particles is more appropriate.

Tests to quantify conclusively the effectiveness of limestone as a permeable treatment bed material for plume control have not been performed. However, advantages in the use of limestone as a treatment bed material can be identified. Generally, these advantages include the ability to neutralize acidic groundwater, the ability to remove certain heavy metals (good potential for removal of chromate anions), and the ready availability and low expense associated with the use of limestone. Limitations to the effectiveness of limestone as a permeable treatment bed are its potential for cementation, plugging, and channelization, and its inability to remove organic contaminants.

8.3.5.1.2 Activated Carbon

Activated carbon has been considered as a possible treatment bed material because of its ability to effectively remove nonpolar organic compounds such as PCB and CCl_4. The mechanism of removal associated with these nonpolar compounds is adsorbtion resulting from van der Waals forces and other chemical attractive forces. Activated carbon also can remove certain heavy metals, but is not very practical for actual use. Polar organic compounds, such as alcohols and ketones, do not appear to be effectively removed by activated carbon because of their electrical charges. This limits the effective use of activated carbon to removal of nonpolar organic compounds in a plume.

Although activated carbon is readily available, easy to install and handle, and has significant potential to remove nonpolar organics, many disadvantages are associated with its use as a permeable treatment bed material. These disadvantages include:

- Plugging of the bed
- Presence of other chemicals in the groundwater which may reduce the effectiveness of bed adsorption
- Desorption of hazardous adsorbed materials may occur, resulting in recontamination
- Life of the bed may be very short in the presence of complex organic compounds
- Removal and disposal of spent activated carbon is difficult and hazardous
- Not very effective in removal of polar organic compounds
- Cost of the material is very high.

The numerous disadvantages associated with the use of activated carbon as a treatment bed material have limited interest in and testing of this material. However, where nonpolar organics are principal contaminants, activated carbon may eventually become a suitable bed material.

8.3.5.1.3 Glauconitic Greensands

Glauconitic greensand deposits of the Atlantic Coastal Plain have high potential for the removal of a number of heavy metals from contaminated waters. Glauconite is a hydrous aluminosilicate clay mineral, which is rich in ferric iron and contains significant amounts of potassium (Spoljaric and Crawford, 1979). Glauconite occurs as dark, light, or yellowish-green pellets, as casts of fossil shells, as coatings on other grains, and as a clayey matrix in coarse-grained sediments.

Studies by Spoljaric and Crawford (1979) indicated that glauconitic greensand had good retention of heavy metal cations in bench-scale studies with leachate from the Pigeon Point Landfill in northern Delaware. Highest removal efficiencies were reported for copper, mercury, nickel, arsenic, and cadmium, as shown in Table 8-25. The chemical composition of glauconites from the Delaware Coastal Plain that were used in the study is given in Table 8-26. The authors reported increased efficiencies with increased contact time. Contact time for bench-scale testing was estimated at two minutes, suggesting that with contact times used in the field being in the order of days, metal removal efficiencies may be extremely high for many of the metals listed (Spoljaric, 1980). The glauconitic sand treatment was also found to reduce odors, suggesting that adsorption of volatile organics may occur as well.

Only minute amounts of metal cations were released from the charged greensand upon flushing with distilled water and solutions of pH 2 and pH 12 (Spoljaric and Crawford, 1979). The results of these experiments suggest that greensands have a high capacity for heavy metal cation retention, and thus seem very applicable as a material for permeable treatment beds (Spoljaric, 1980). Saturation points for heavy metal adsorption by glauconitic sands have not been determined and sorptive capacity has yet to be assessed through further experimentation. Also, the applicability of greensands in treating higher concentrations of heavy metals has not been determined. While they appear promising, more experimental work is required before glauconitic greensands can be thoroughly assessed as a permeable treatment bed material.

Ross (1980) conducted an in situ experiment near Alfred's Hill in Uffington, England, involving leachate interactions in the Lower Greensand (Cretaceous Age). The composition of the Lower Greensand is described as containing quartz as the dominant mineral, with 15 to 20 percent calcium montmorillonite, 8 percent calcite, and minor amounts of aragonite, pyrite, glauconite, and goethite. Synthetic leachate solutions were introduced into the Lower Greensand with known concentrations of heavy metals. The emphasis of the study was to determine the sequential attenuation of heavy metals by the Lower Greensand.

TABLE 8-25.

RESULTS OF CHEMICAL ANALYSES OF GREENSAND FILTRATION OF PIGEON POINT LANDFILL LEACHATE[1]
(Spoljaric and Crawford, 1979 as cited by Rogoshewski, et al., 1982)

Cation	Cation concentration (ppm)		Percent retained by greensand
	Before filtration	After greensand	
Aluminum	0.68	0.17	75
Arsenic	0.0022	0.0002	91
Cadmium	0.006	0.001	83
Calcium	129	48	63
Chromium	0.03	0.01	66
Cobalt	0.015	0.003	80
Copper	0.38	ND	100
Iron	8.1	1.1	86
Lead	0.13	ND	100
Magnesium	62	20	67
Manganese	4.1	0.48	88
Mercury	0.0087	ND	100
Nickel	0.074	0.003	96
Potassium	122	74	39
Silver	0.0014	0.0007	50
Sodium	275	175	36
Zinc	0.49	0.16	67
pH	7.65	6.29	

[1]Flow rate: 100 ml/min.
ND = Not detected.

TABLE 8-26.

CHEMICAL COMPOSITION OF GLAUCONITES FROM THE DELAWARE COASTAL PLAIN (Spoljaric and Crawford, 1979)

Oxide	Sample (wt/1/2)		
	1	2	3
SiO_2	49.6	47.3	47.3
TiO_2	0.84	0.28	0.27
Al_2O_3	9.6	7.1	7.2
Fe_2O_3	14.0	17.6	17.5
FeO	2.26	1.89	2.30
MnO	0.04	0.03	0.03
MgO	3.1	3.7	3.6
CaO	Trace	2.1	2.2
Na_2O	0.11	0.11	0.05
K_2O	6.2	8.0	8.0
Cr_2O_3	0.045	0.034	0.056
H_2O total	10.13	8.99	8.2
CO_2	--	1.0	1.1
Total	95.9	98.1	97.8
Fe_{total} as Fe_2O_3	16.5	19.9	20.1

On the basis of the concentration of heavy metals extracted by lysimeters from the sediments, the order of retention of the metals was found to be Ni<Cd<Zn<Cu<Cr<Pb, or inversely proportional to their order of mobility. Results of the tests also showed volumetric retention capabilities of the Lower Greensand of 300 g/m^3 nickel, 500 g/m^3 cadmium, 700 g/m^3 zinc, 1100 g/m^3 chromium and copper, and up to 1600 g/m^3 lead. Although this study does not directly indicate the contaminant treatment capabilities of the glauconitic Lower Greensand in relationship to constructed permeable treatment beds, it does reflect the potential use of glauconitic greensands as treatment bed material.

Advantages of glauconitic treatment beds, based on studies to date, include good permeability, abundance in the Atlantic Coastal Plain (i.e., New Jersey, Delaware, and Maryland), effectiveness in removal and retention of many heavy metals, and good retention time characteristics for efficient treatment. Among the disadvantages of using glauconitic treatment beds are unknown saturation characteristics and potential for plugging over time, potential reduction in pH, limited areas of applications to natural occurrence such as the Mid-Atlantic region, and possibility of land purchase requirements as it is not commercially mined.

8.3.5.1.4 Zeolites and Synthetic Ion Exchange Resins

These materials are very effective in the removal of heavy metal contaminants, but are economically and practically infeasible for use as permeable treatment bed material. Disadvantages contributing to the limited application of zeolites and synthetic ion exchange resins include high cost, short life, and reactivation problems. These materials are not recommended for use except in specific cases where engineering and economic evaluations prove their desirability.

8.3.5.2 Design and Construction

The construction of a permeable treatment bed is essentially the same for all the previously mentioned bed fill materials. Bed construction consists of

excavating a trench to intercept the flow of the contaminated groundwater, filling the trench with the appropriate materials, and capping the trench (refer to Chapter 6).

An important consideration in the design of a permeable treatment bed is the size and shape of the trench as it relates to the specific problem of groundwater contamination. The trench should be long enough to contain the plume and deep enough to stop the groundwater from flowing underneath the bed. For practical design purposes, the depth of the bed is the distance between the ground surface and the bedrock or other impermeable layer (such as a clay layer). The width of the trench is determined by the velocity of the groundwater flow, the permeability of the material used for treatment, and the contact time required for effective treatment.

Trench excavation will typically require the use of conventional shoring such as sheet piling, and bracers and struts. Because the trench will intersect the water table, dewatering will be required during excavation. Groundwater pumped from the trench will likely be contaminated and, therefore, will probably require treatment.

Prior to designing the permeable treatment bed, the residence time, or contact time, must be estimated. Residence time is a function of the level of contamination of groundwater and the treatment characteristics of the material used for the decontamination process. The residence time of the contaminated groundwater flow through the bed must be sufficient to ensure optimum treatment conditions. Thus, the determination of the optimum contact time, t_c, requires a knowledge of the chemistry of the contaminated groundwater and of its reaction to treatment by the material that will be present in the bed. Once the residence time is estimated, the bed width required for treatment can be calculated.

To properly design a permeable treatment bed, a working knowledge of the mechanisms of groundwater flow is essential. The flow of water through the ground or its filtration through sand may be determined by using Darcy's Law:

$$V = KI/n$$

where V is the pore velocity of water through a unit cross-sectional area (ft/day), I is the gradient or loss of elevation head per unit of length in the direction of the flow, K is the hydraulic conductivity (ft/day), and n is the effective porosity of the material. Thus, by knowing the hydraulic conductivity of the soil through which the groundwater flows and the difference in hydraulic head between two points, flow velocity can be calculated for the aquifer using estimates of appropriate effective porosity (refer to Chapter 3 for detailed discussions).

To ensure that disturbance of the groundwater flow by the treatment bed is minimized, flow velocities within the treatment bed (V_b) should be equal to or greater than flow velocities in the undisturbed soil (V_s). Assuming the hydraulic gradient at the bed and soil are the same, the ratios of hydraulic conductivity to effective porosity must be similar between the two media. Hydraulic conductivity can be determined either in the field (for the aquifer) or in the laboratory (for the aquifer and bed material) using constant or falling head methods. Effective porosity can be estimated from published data or calculated from laboratory tests.

Consequently, the particle size distribution of the bed material can be selected according to the desired hydraulic conductivity and porosity. Once the bed material is selected, a series of bench scale tests can be performed, varying particle size distribution to determine what sorting of particle sizes provides an appropriate K/n ratio.

Once the flow velocity of water through the bed (V_b) is calculated for the appropriate treatment bed fill material and the residence time (t_c) estimated, the thickness (W_b) of the permeable treatment bed can be determined from:

$$W_b = V_b t_c$$

where the units are: W_b in feet, V_b in feet per day, and t_c in days.

To summarize, in the actual design of a treatment bed, the hydraulic conductivity and porosity of the soil and the hydraulic gradients in the field must be studied in order to estimate the actual velocities of the groundwater flow. Then, the highest groundwater flow velocity is chosen as the minimum design velocity of the flow through the bed to ensure optimum operational conditions. To obtain the same velocity of groundwater through the bed as through the aquifer, the K/n ratio of the bed must be the same as the K/n ratio of the soil. Consequently, the particle size of the material used in the bed must be chosen to achieve the desired K/n ratio of the bed. This can be accomplished by conducting a series of bench-scale tests with bed materials of different particle size distribution. To design an effective treatment bed, an optimum contact time must be determined on the basis of expected rates of interaction between the groundwater and the bed material. When the velocity of the flow through the bed and the optimum contact time are estimated, the bed thickness can be determined.

Unit costs for the installation of a permeable treatment bed were estimated by the Rogoshewski, et al. (1982) and are shown in Table 8-27. Costs can be assumed to have increased marginally since estimates included in the report are somewhat dated.

8.4 BLOCK DISPLACEMENT

The Block Displacement (BD) method is a technique that was developed to completely isolate a large mass of contaminated soil containing a waste site or plume, or both. The BD concept relies on producing a fixed underground

TABLE 8-27.

ESTIMATED COSTS FOR INSTALLATION OF A PERMEABLE TREATMENT BED[1]
(Rogoshewski, et al., 1982)

Item	Cost ($) Unit	Cost ($) Total
Trench excavation costs (approximately 3,000 cubic yards)	3,000	
Cover spreading costs	1,980	
Well point dewatering system	75,000	
Sheet piling costs	365,000	
Water, connection and structure costs	6,100	
Total cost of trench construction		451,080
Limestone/materials and installation costs	34,500	
Total cost using limestone		485,580
Activated carbon/materials and installation costs	4,080,000	
Total cost using activated carbon		4,531,080
Glauconitic Greensand cost	Not provided[2]	
Zeolite/Synthetic ion exchange resins cost	Not provided[3]	

[1]Costs based on trench dimensions of 20' deep, 4' wide, 1000' long, costs for contractors and materials compiled during 1980

[2]Could entail possible land purchase, borrow excavation, and hauling to disposal site

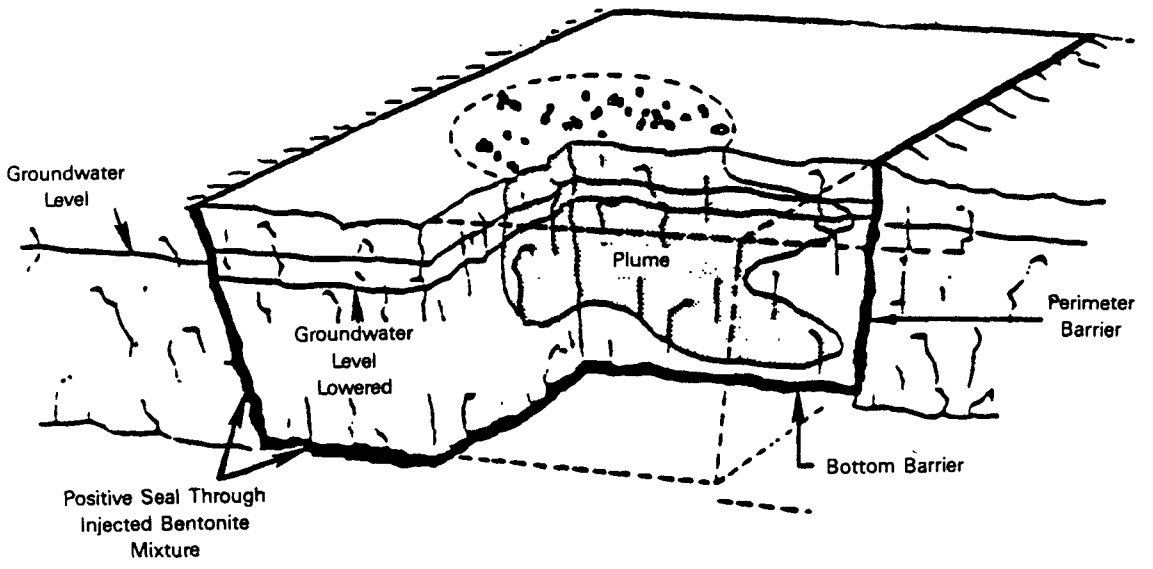
[3]Considered economically infeasible

barrier around and beneath the contaminated zone to essentially encapsulate a block of earth (Brunsing, 1983). The method involves the construction of two separate barriers, a perimeter barrier and a bottom barrier, which are interconnected to allow block displacement.

The perimeter barrier can be constructed using various conventional means including slurry wall, vibrating beam, or jet grouting techniques (refer to Chapter 7). The perimeter barrier can be constructed in conjunction with the bottom barrier either prior to, during, or following bottom barrier construction. If constructed prior to bottom barrier construction, the perimeter barrier can be used to aid in bottom separation.

The bottom barrier is formed by pumping slurry composed of local soil, bentonite, and water, into a series of notched injection holes. Continued pumping of slurry under pressure theoretically produces an uplift force against the bottom of the block that is capable of creating upward vertical displacement of the block. A typical Block Displacement barrier is shown schematically in Figure 8-9.

FIGURE 8-9.
SCHEMATIC DIAGRAM OF BLOCK DISPLACEMENT
(BRUNSING, 1983)



The Block Displacement method was developed for applications where an unweathered bedrock or low permeability stratum does not occur at shallow enough depths below a contaminated zone for a perimeter (e.g., slurry wall) alone to provide cost-effective isolation of the contaminant plume. In situ isolation using a slurry wall or other conventional vertical barriers require keying the barrier into a naturally occurring impermeable stratum. Also, Block Displacement is conceptually designed to minimize the volume of soil or earth material to be isolated. Thus, Block Displacement would theoretically isolate a contaminated area through the creation of a man-made confining layer immediately surrounding the contaminated zone.

Construction principles of the Block Displacement method require the use of barrier slurry material that is hydraulically and chemically compatible with in situ soils and contaminated groundwater. Construction of the bottom barrier proceeds in four phases:

1. Formation of notches at the base of the injection holes
2. Initial bottom separation at the notched holes during initial slurry pumping
3. Coalescence of local separations at each injection point to form a single large bottom separation
4. Generation of a complete bottom barrier by controlled vertical displacement of the earth mass using low pressure slurry injection into the horizontal separation.

The perimeter barrier can be constructed by various means including a slurry wall, vibrating beam or jet grouting technique. If constructed prior to bottom separation, the perimeter wall can be used to ensure a favorable horizontal stress field for proper orientation of the propagating bottom separation. Also, the injection pressure required to achieve bottom separation is dependent not only on block weight and geometry, but the shear resistance in the previously constructed perimeter as well. This relationship

The perimeter barrier can be constructed by various means including a slurry wall, vibrating beam or jet grouting technique. If constructed prior to bottom separation, the perimeter wall can be used to ensure a favorable horizontal stress field for proper orientation of the propagating bottom separation. Also, the injection pressure required to achieve bottom separation is dependent not only on block weight and geometry, but the shear resistance in the previously constructed perimeter as well. This relationship is shown in the following equation (Brunsing, 1983), which also takes into account the slurry pressure drop across the bottom separation:

$$P_o = P_1 + \Delta P_B$$

where P_o = required injection pressure

P_1 = pressure in the bottom separation at its intersection with the perimeter

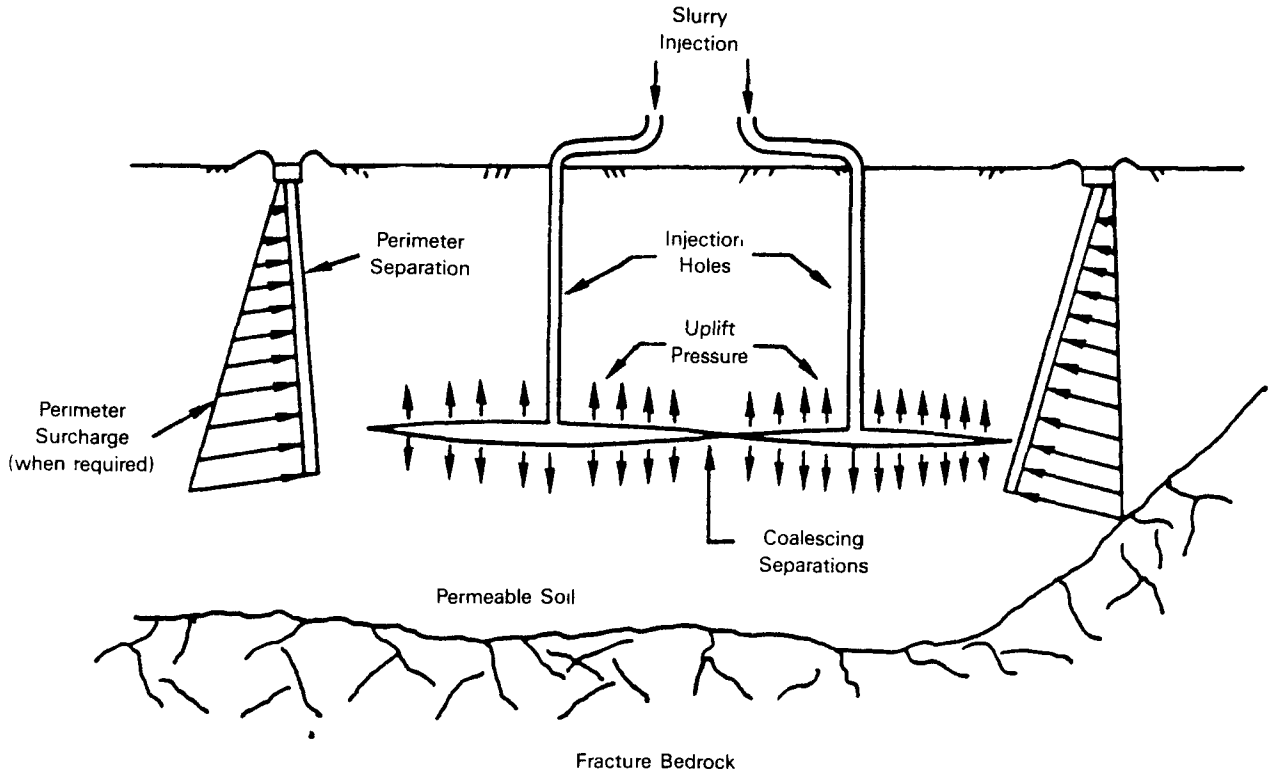
ΔP_B = the pressure drop across the bottom separation caused by flow resistance in the slurry.

The width of an initially thin perimeter barrier, such as might be constructed by vibrating beam or jet grouting techniques, can be increased during block lift or continued bottom separation. Upward displacement of the block (d) resulting from injection along the bottom barrier should increase the initial perimeter barrier thickness (W_o), to a desired barrier thickness (W) if the perimeter is constructed at a slight angle off vertical (Ø). This principle is expressed in the following equation offered by Brunsing (1983):

$$W = d \sin \emptyset + W_o$$

Hence, if perimeter barrier construction is performed prior to bottom separation and barrier construction, favorable horizontal stress fields may be ensured to help properly orient bottom separation propagation and thinly constructed perimeter barriers may be increased to a desired thickness. Figure 8-10, shows in cross-section, the schematic configuration of the perimeter barrier, injection wells, and hypothetically coalescing bottom separation.

FIGURE 8-10.
SCHEMATIC CROSS-SECTION OF BLOCK DISPLACEMENT SHOWING SEPARATING TO INDUCE DISPLACEMENT (BRUNSING, 1983)



The notching operation at the base of the injection hole requires a high-pressure rotating slurry jet (Figure 8-11). The jetting slurry should be designed to optimize notch erosion, remove cuttings, and minimize leakage into the soils. A large notch diameter is desired to reduce initiation slurry pumping pressure requirements during bottom barrier emplacement and to reduce the tendency of separations to propagate upward. The slurry injection pressure P_0 required to initiate bottom separation is defined by (Brunsing, 1983):

$$P_0 = \rho_r gh + \Delta P$$

where ρ_r = the average earth mass density

g = gravitational constant

h = depth to bottom separation

ΔP = pressure in excess of the overburden.

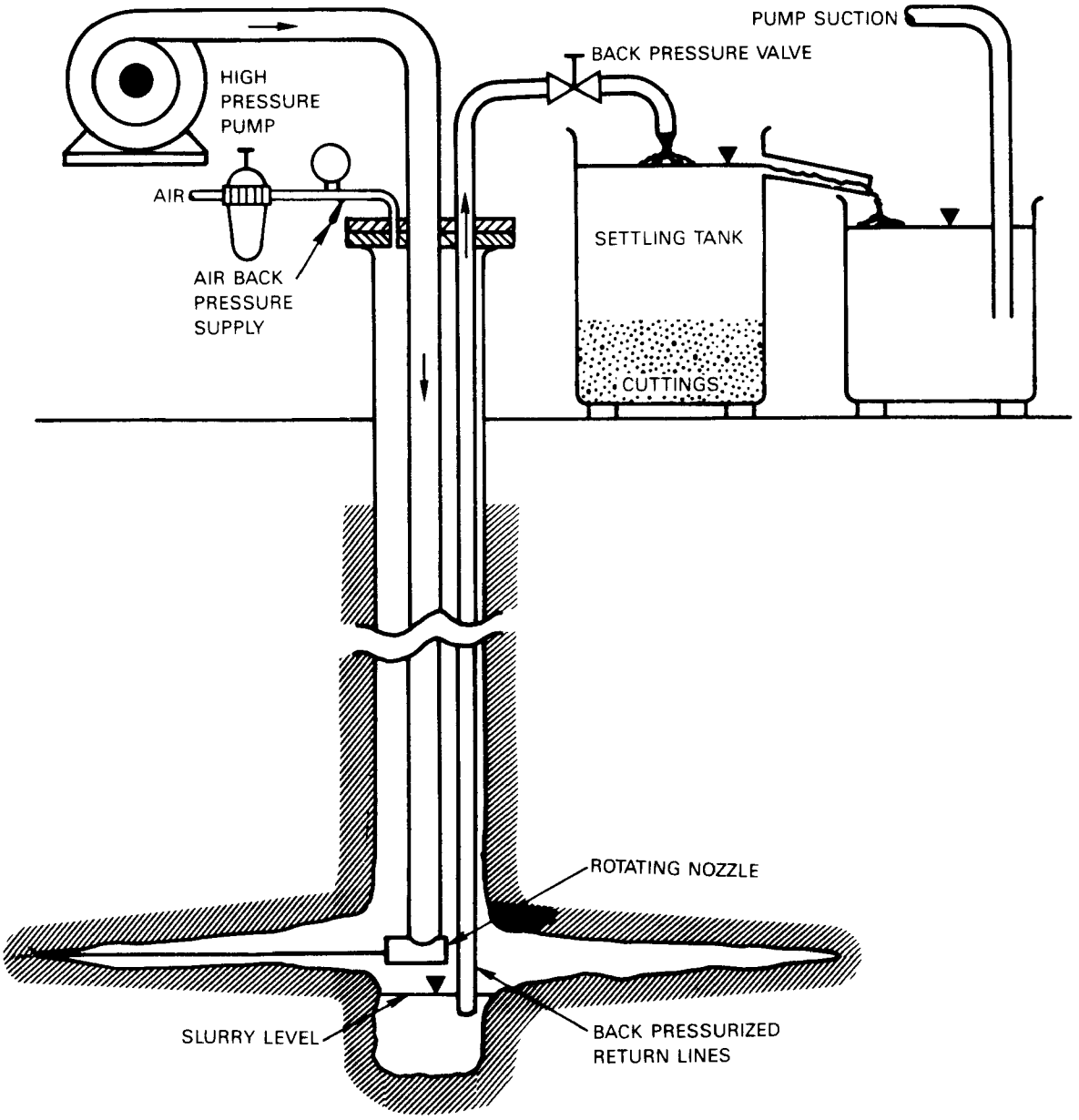
The pressure in excess of the overburden is a function of (Brunsing, 1983):

- Soil strength or strength of the material being fractured
- Notch diameter which affects the initial slurry pressure required to propagate fractures at the tip of the notch
- Slurry properties that should be designed to inhibit filter cake formation at the end of the notches, thus preventing deflected fracture propagation
- Slurry injection rate which should minimize the time available for filter cake formation.

Coalescence of the separations is then a function of adding slurry volume and by gradually increasing the gel strength and viscosity of the slurry. The increased area over which the slurry is in contact should decrease the slurry pressure required to propagate the horizontal separation.

The concept of block displacement for hazardous waste problems has not been substantially documented. However, a demonstration of the Block

FIGURE 8-11.
SLURRY JET IN AIR NOTCHING OPERATION
(BRUNSING, 1983)



Displacement method was conducted in Whitehouse, Florida, during 1982 (Brunsing and Grube, 1982). The demonstration block was approximately 60 feet in diameter and 23 feet in depth, and composed of unconsolidated marine sediments. Although the results of the test did not conclusively demonstrate complete block isolation, maximum uplift was measured to be 12 inches in portions of the block, and corings of the bottom barrier documented slurry thicknesses ranging from 1.5 to 4.75 inches. However, areas of the block were measured as not having undergone uplift and the varied thicknesses of the bottom barrier in the test corings did not match uplift measured directly at the surface. The varied thickness in the bottom barrier slurry also raises questions regarding complete lateral continuity of the bottom barrier beneath the block.

The subsurface geologic conditions to which BD would be an appropriate remedial action are restricted. Variable fluid flow characteristics of soils or soft sediments, and discontinuities within the vicinity of bottom barrier construction could inhibit effective slurry propagation and coalescence. In situ stresses control separation orientation, since separations beyond the notch diameter will propagate in a plane normal to the direction of minimum principal stress. Vertical stress, which is determined by overburden weight, is in many cases greater than horizontal stresses. In such cases, fractures initiated horizontally will have a tendency to turn upward. Also, block displacement would be considered infeasible in areas where faults, fractures, or solution cavities may be present.

Problems associated with the use of Block Displacement to encapsulate a contaminated block include not only difficulties in ensuring bottom barrier continuity, but also the health and safety, environmental, and construction risks of drilling injection holes through a contaminated zone, and potentially through hazardous materials. Even if injection holes could be constructed safely, they could potentially serve as conduits for increased rates of vertical contaminant migration prior to slurry injection.

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