CHAPTER 5

RAPID INFILTRATION PROCESS DESIGN

5.1 Introduction

The design procedure for rapid infiltration (RI) is diagrammed in Figure 5-1. As indicated by this figure, there are several major elements in the design process and the design approach is somewhat iterative. For example, the amount of land required for an RI system is a function of the loading rate, which is affected by the loading cycle and the level of preapplication treatment. If the engineer initially assumes a level of preapplication treatment and a loading cycle that result in a loading rate requiring more land than is available at the selected site, the level of preapplication treatment and loading cycle can be reevaluated to reduce the land area required.

5.1.1 RI Hydraulic Pathway

The engineer and the community must decide which hydraulic pathway (see Figure 1-2) is appropriate for their situation. This decision is based on the hydrogeologic characteristics of the selected site and regulatory agency decisions.

5.1.2 Site Work

For RI design, the results of the field investigations (Chapter 3) must be analyzed and interpreted. Backhoe pits and drill holes are needed to establish the depth and hydraulic conductivity of the permeable material and the depth to ground water. Sufficient subsurface information must be obtained in the Phase 2 planning process (Chapter 2) to allow the engineer to calculate:

1. Infiltration rate (Section 5.4)
2. Subsurface flow (Section 5.7)
   - Potential for mounding
   - Drainage (if needed)
   - Natural seepage (if adequate)
3. Mixing of percolate with ground water (if critical to meet performance requirements)

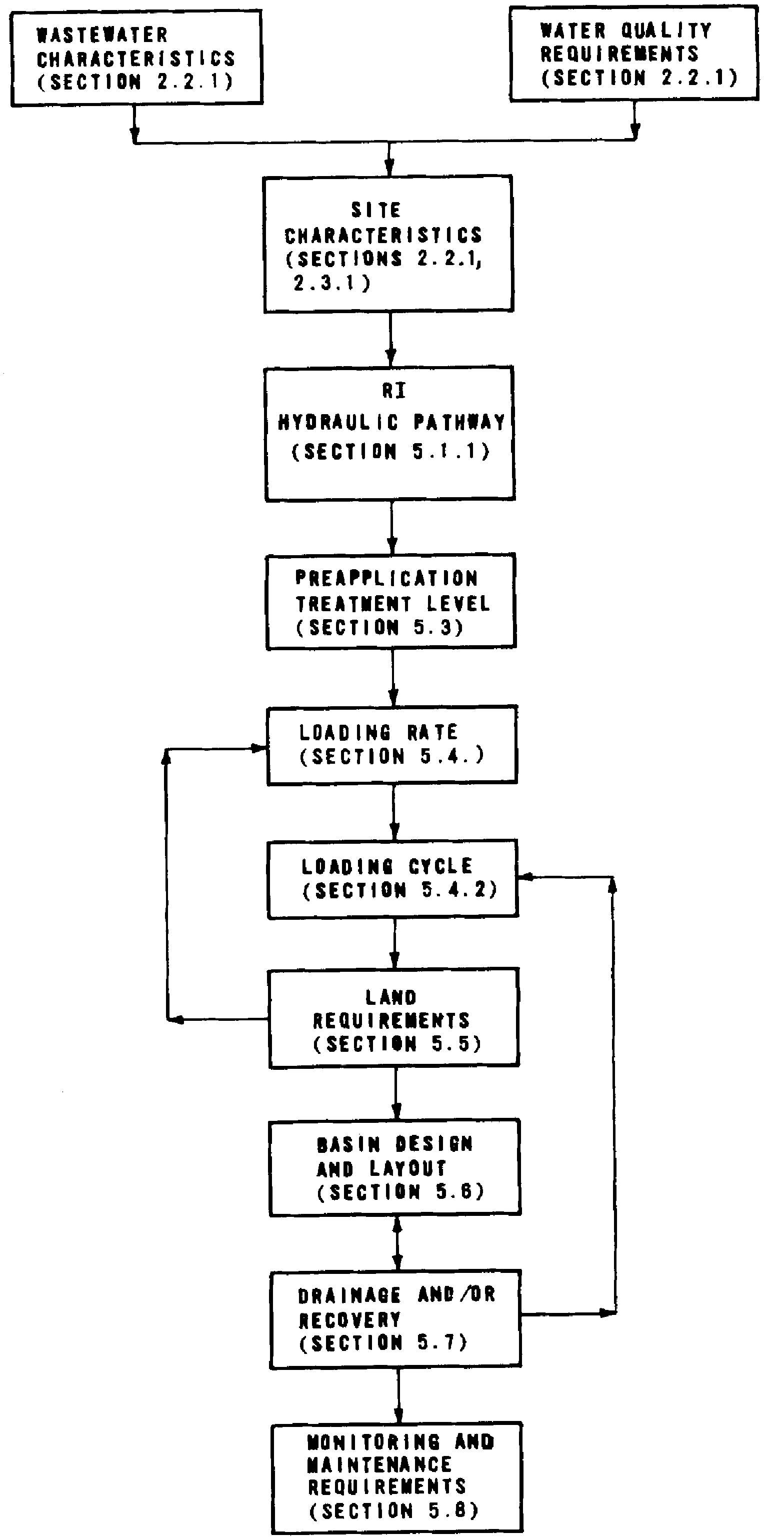


FIGURE 5-1
RAPID INFILTRATION DESIGN PROCEDURE

5.2 Process Performance

The RI mechanisms for removal of wastewater constituents such as BOD, suspended solids, nitrogen, phosphorus, trace elements, microorganisms, and trace organics are discussed briefly along with typical results from various operating systems. Chapter 9 contains discussions of the health and environmental effects of these constituents.

5.2.1 BOD and Suspended Solids

Particulate BOD and suspended solids are removed by filtration at or near the soil surface. Soluble BOD may be adsorbed by the soil or may be removed from the percolating wastewater by soil bacteria. Eventually, most BOD and suspended solids that are removed initially by filtration are degraded and consumed by soil bacteria. BOD and suspended solids removals are generally not affected by the level of preapplication treatment. However, high hydraulic loadings of wastewaters with high concentrations of BOD and suspended solids can cause clogging of the soil. <u>Typical BOD loadings (Table 2-3) are less than 130 kg/ha•d (115 lb/acre•d) for municipal wastewaters</u>. Removals achieved at selected RI systems are presented in Table 5—1. Some systems have been operated successfully at higher loadings.

5.2.2 Nitrogen

The primary nitrogen removal mechanism in RI systems is nitrification-denitrification. This mechanism involves two separate steps: the oxidation of ammonia nitrogen to nitrate (nitrification) and the subsequent conversion of nitrate to nitrogen gas (denitrification). Ammonium adsorption also plays an important intermediate role in nitrogen removal.

Both nitrification and denitrification are accomplished by soil bacteria. The optimum temperature for nitrogen removal is 30 °C to 35 °C (86 °F to 95 °F). Both processes proceed slowly between 2 °C and 5 °C (36 °F and 41 °F) and stop near the freezing point of water. Nitrification rates decline sharply in acid conditions and reach a limiting value at approximately pH 4.5. The denitrification reaction rate is reduced substantially at pH values below 5.5. Thus, both soil temperature and pH must be considered if nitrogen removal is important (Section 5.4.3.1). Furthermore, alternating aerobic and anaerobic conditions must be provided for significant nitrogen removal (Section 5.4.2). Because aerobic bacteria deplete soil oxygen during flooding periods, resting and flooding periods must be alternated to result in alternating aerobic and anaerobic soil conditions.

TABLE 5-1
BOD REMOVAL DATA FOR
SELECTED RI SYSTEMS [1-6]

Location	Preapplication treatment	Sampling depth, m	BOD: Average loading rate, kg/ha·d[a]	BOD: Treated water concentration, mg/L	BOD: Removal, %
Calumet, Michigan	Untreated	3.3	80	11[b]	86
Fort Devens, Massachusetts	Primary	20	87	12	86
Hollister, California	Primary	8	177	8[c]	95
Lake George, New York	Trickling filters	3	53	1.2	98
Milton, Wisconsin	Activated sludge	8-29	155	1.0-19.0	88-99
Phoenix, Arizona	Activated sludge	6-9	45	0-1	98-100
Vineland, New Jersey	Primary	2-14	48	6.5[c]	86

a. Total kg/ha·yr applied divided by the number of days in the operating season (365 days for these cases).

b. Soluble total organic carbon.

c. Average value from several wells.

Note: See Appendix G for metric conversions.

Organic carbon is needed in the applied wastewater to supply energy for the denitrification reaction. Approximately 2 mg/L of total organic carbon (TOC) is needed to denitrify 1 mg/L of nitrogen. Because the BOD concentration decreases as the level of preapplication treatment increases, preapplication treatment must be limited if denitrification is to occur in the soil. Thus, if the goal of RI is nitrogen removal, primary preapplication treatment is preferred.

Nitrogen removal efficiencies at various operating RI systems are shown in Table 5-2. As shown in this table, nitrogen removals of approximately 50% are typical. Greater amounts can be removed using special management procedures (Section 5.4.3.1).

TABLE 5-2
NITROGEN REMOVAL DATA FOR SELECTED RI SYSTEMS [1,2,4,6-9]

Location	Concentration in applied wastewater: total N, mg/L	Loading rate, m/yr	BOD:N ratio	Flooding to drying time ratio	Concentration in renovated water, mg/L		Removal, % of total N
					NO_3-N	Total N	
Boulder, Colorado	16.5	48.8	2.3:1	1:3	6-16	9-16	10-20
Brookings, South Dakota	10.9	12.2	2:1	1:2	5.3	6.2	43
Calumet, Michigan	24.4	17.1	3.6:1	1:2	3.4	7.1	71
Disney World, Florida	--	54.9	0.3:1	150:14	--	--	12
Fort Devens, Massachusetts	50	30.5	2.4:1	2:12	13.6	19.6	61
Hollister, California	40.2	15.2	5.5:1	1:14	0.9	2.8	93
Lake George, New York	11.5 12.0	58.0 58.0	2:1 2:1	1:4 1:4	-- --	7.70 7.50	33 38
Phoenix, Arizona	27.4	61.0	1:1	9:12	6.2	9.6.	65

At some sites the goal of RI may be only nitrification (for example, Boulder, Colorado). Generally, nitrification occurs if wastewater application periods are short enough that the upper soil layers remain aerobic. For this reason, if nitrification is the objective of RI, short application periods followed by somewhat longer drying periods are used. Because the nitrification rate decreases during winter months, reduced loading rates may be required in cold climates. Under favorable temperature and moisture conditions, up to 50 ppm ammonia nitrogen (as nitrogen) per day (soil basis) may be converted to nitrate [10]. Assuming that nitrification only occurs in the top 10 cm (4 in.) of soil, this corresponds to nitrification rates of up to 67 kg/ha•d (60 lb/acre•d). At the Boulder, Colorado, RI system, the percolate ammonia concentration remained below 1 mg/L on a year-round basis.

5.2.3 Phosphorus

The primary phosphorus removal mechanisms in RI systems are the same as described in Section 4.2.3 for SR. Phosphorus removals achieved at typical RI systems are provided in Table 5-3.

TABLE 5-3
PHOSPHORUS REMOVAL DATA FOR SELECTED
RI SYSTEMS [1, 2, 4-9]

Location	Average concentration in applied wastewater, mg/L	Distance of travel, m		Average concentration in renovated wastewater, mg/L	Removal, %
		Vertical	Horizontal		
Boulder, Colorado[a]	6.2	2.4-3.0	0	0.2-4.5	40-97
Brookings, South Dakota[b]	3.0	0.8	0	0.45	85
Calumet, Michigan[a]	3.5 3.5	3-9 --[c]	0-125 1,700[c]	0.1-0.4 0.03	89-97 99
Fort Devens, Massachusetts[b]	9.0	15	30	0.1	99
Hollister, California[b]	10.5	6.8	0	7.4	29
Lake George, New York[b]	2.1 2.1	3 --[c]	0 600[c]	<1 0.014	>52 99
Phoenix, Arizona[a]	8-11 7.9	9.1 6	0 30	2-5 0.51	40-80 94
Vineland, New Jersey[b]	4.8 4.8	2-18 4-16	0 260-530	1.54 0.27	68 94

a. Total phosphate measured.

b. Soluble phosphate measured.

c. Seepage.

5.2.4 Trace Elements

Trace element removal involves essentially the same mechanisms discussed in Section 4.2.4 for SR systems. The results presented in Table 5-4 compare trace element concentrations in wastewater at Hollister, California, to drinking water and irrigation requirements.

At RI sites, trace elements accumulate in the upper soil layers. Data from Cape Cod, Massachusetts, reflect this phenomenon and are presented in Table 5-5. As indicated in this table, the percent retention of most of the metals is quite high. For example, 85% of the copper applied over 33 years was retained in the top 0.52 m (1.7 ft). The distribution of the retained metals is also shown in Table 5-5.

TABLE 5-4
COMPARISON OF TRACE ELEMENT LEVELS TO IRRIGATION AND DRINKING WATER LIMITS [6]
mg/L

Element	Recommended maximum in irrigation waters	Maximum concentration in drinking waters	Hollister, California, average wastewater concentration
Ag (silver)	--[a]	0.05	<0.008
As (arsenic)	0.1	0.05	<0.01
Ba (barium)	--[a]	1.0	<0.13
Cd (cadmium)	0.01	0.010	<0.004
Co (cobalt)	0.1	--[a]	<0.008
Cr (chromium)	0.05	0.05	<0.014
Cu (copper)	0.2	--[a]	0.034
Fe (iron)	5.0	--[a]	0.39
Hg (mercury)	--[a]	0.002	<0.001
Mn (manganese)	0.2	--[a]	0.070
Ni (nickel)	0.2	--[a]	0.051
Pb (lead)	5.0	0.05	0.054
Se (selenium)	0.02	0.01	<0.001
Zn (zinc)	2.0	--[a]	0.048

a. None set.

TABLE 5-5
HEAVY METAL RETENTION IN AN INFILTRATION BASIN[a]

Percent

Depth, m	Cadmium	Chromium	Copper	Lead	Zinc
0-0.04	84	87	76	88	82
0.04-0.06	12	10	23	12	13
0.14-0.16	1	0	0.4	0	1
0.24-0.26	1	2	0.4	0	2
0.29-0.31	1	0	0.1	0	0.8
0.44-0.46	0.5	1	0.1	0	1.2
0.50-0.52	0.5	0	0.0	0	0
Total	100	100	100	100	100
Percent retention of 33 year loads					
0-0.52	113	62	85	129	49

a. Adapted from reference [11].

5.2.5 Microorganisms

Removal mechanisms for microorganisms are discussed in Section 4.2.5.

Fecal coliform removal efficiencies obtained at selected RI sites are given in Table 5-6. As shown in this table, effective removal of fecal coliforms can be achieved with adequate travel distance.

TABLE 5-6
FECAL COLIFORM REMOVAL DATA FOR
SELECTED RI SYSTEMS [1, 3–6, 12]

		Fecal coliforms, MPN/100 mL		
Location	Soil type	Applied wastewater	Renovated water	Distance of travel, m
Hemet, California	Sand	60,000	11	2
Hollister, California	Sandy loam	12,400,000	171,000	7
Lake George, New York	Sand	359,000 359,000	72 0	2 7
Landis, New Jersey	Sand and gravel	TNTC[a]	16	1-2
Milton, Wisconsin	Gravelly sands	TNTC[a]	0	8-17
Phoenix, Arizona	Sand	244,071 244,071	104 0	30 90
Santee, California	Gravelly sands	130,000 130,000	580 <2	61 762
Vineland, New Jersey	Sand and gravel	TNTC[a]	0	6-7

a. At least one sample too numerous to count.

The primary removal mechanism for viruses is adsorption. Because of their small size, viruses are not removed by filtration at the soil surface, but instead, travel into the soil profile. Only a limited number of studies have been conducted to determine the efficiency of virus removal. At Phoenix, Arizona, results indicate that 90 to 99% of the applied virus is removed within 10 cm (4 in.) of travel when either primary or secondary effluent is applied [13, 14] and that 99.99% removal is achieved during travel through 9 m (30 ft) of soil following the application of secondary effluent [15].

The only RI sites at which viruses have been detected in ground water, and the distances traveled by the virus prior to detection are listed in Table 5-7. As noted in the table, all four of these sites are located on coarse sand and gravel type soils. Infiltration rates on these soils are relatively high, allowing constituents in the applied wastewater to travel greater distances than normally expected. Thus, the coarser the soil is, the higher the loading rate, and the higher the virus concentration, the greater the risk of virus migration.

TABLE 5-7
REPORTED ISOLATIONS OF VIRUS AT RI SITES [16]

		Distance of migration, m	
Location	Soil type	Vertical	Horizontal
East Meadows, New York	Sands and gravel	11.3	3
Fort Devens, Massachusetts[a]	Sands and gravel	18.3	183
Holbrook, New York	Sands and gravel	6.1	45.7
Vineland, New Jersey[a]	Sands and gravel	16.8	250

a. Application of unchlorinated primary effluent.

5.2.6 Trace Organics

Trace organics can be removed by volatilization, sorption, and degradation. Degradation may be either chemical or biological; trace organic removal from the soil is primarily the result of biological degradation.

Studies to determine trace organic removal efficiencies during RI were conducted at the Vineland and Milton sites [3, 5]. At these two systems, applied effluent and ground water were analyzed for six pesticides and the results of the studies are summarized in Table 5-8. At both locations, the concentrations of 2,4-D, 2,4,5-TP silvex, and lindane were well below the maximum concentrations for domestic water supplies established in the National Primary Drinking Water Regulations.

If local industries contribute large concentrations of synthetic organic chemicals and the RI system overlies a potable aquifer, industrial pretreatment should be considered. Further, since chlorination prior to land application causes formation of chlorinated trace organics that may be more difficult to remove, chlorination before application should be avoided whenever possible.

TABLE 5-8
RECORDED TRACE ORGANIC CONCENTRATIONS
AT SELECTED RI SITES [3,5]
ng/L

	Vineland, New Jersey[a]			Milton, Wisconsin			
Pesticide	Applied	Shallow ground water	Control ground water	Applied	Shallow ground water[b]	Down-gradient[c]	Control ground water
Endrin	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03	<0.03
Lindane	2,830-1,227	453-1,172	21.3	41	157.6	3.9	7.4
Methoxychlor	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Toxaphene	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1
2,4-D	9.5-10.5	16.4-13.0	10.4	53.8	92.4	23.6	31.0
2,4,5-TP silvex	72	26.8-120	185	16.2	41.2	38.7	76.8

a. If two values are listed, the first is for the Vineland site and the second is for the Landis site (see reference [5]). If one value is listed, results were the same at both sites.

b. Shallow ground water was sampled directly below infiltration basins.

c. Ground water sampled approximately 45 m (148 ft) downgradient from the infiltration basins.

5.3 Determination of Preapplication Treatment Level

The first step in designing an RI system is to determine the appropriate level of preapplication treatment. This section describes the factors that should be considered as well as the levels of preapplication treatment that should be used to meet various treatment objectives.

5.3.1 EPA Guidance

EPA has issued guidelines suggesting the following levels of preapplication treatment for RI systems [17]:

- Primary treatment in isolated locations that have restricted public access
- Biological treatment by lagoons or in–plant processes at urban sites that have controlled public access

5.3.2 Water Quality Requirements and Treatment Goals

Preapplication treatment is used to reduce soil clogging and to reduce the potential for nuisance conditions (particularly odors) developing during temporary storage at the application site. If surface discharge is required and ammonia discharge requirements are stringent, the treatment objective should be to maximize nitrification. In all other cases, system design is based on achieving the maximum, cost—effective loading rate that provides the required level of overall treatment.

For all systems, the equivalent of primary treatment is the minimum recommended preapplication treatment. This level of treatment reduces wear on the distribution system, prevents unmanageable soils clogging, reduces the potential for nuisance conditions, and allows the potential for maximum nitrogen removal.

Nitrification may be achieved using either primary or secondary preapplication treatment. For this reason, the selection of a preapplication treatment level to maximize nitrification at a specific site is based on the same factors that influence the selection of a preapplication treatment level for maximizing infiltration rates.

In mild climates, ponds can be used if land is relatively plentiful and not expensive. In areas that experience cold winter weather, it may not be possible to operate RI systems that use ponds for preapplication treatment. Also, if ponds are used prior to infiltration, algae carryover may increase the potential for soil clogging. Ponds can also be used to reduce the nitrogen loading (Section 4.4.1).

Recommended levels of preapplication treatment are summarized in Table 5-9. This table should be used only as a guide; the designer should select preapplication treatment facilities that reflect local conditions, including local preapplication treatment requirements and existing wastewater treatment facilities.

TABLE 5-9
SUGGESTED PREAPPLICATION TREATMENT LEVELS

RI system objective	Preapplication treatment level
Maximize infiltration rates or nitrification	
General case	Primary
Limited land	Secondary
High quality effluent polishing	Secondary or higher
Maximize nitrogen removal	
General case	Primary

5.4 Determination of Hydraulic Loading Rate

Selection of a hydraulic loading rate is the most important and, at the same time, the most difficult step in the design procedure. The loading rate is a function of the site--specific hydraulic capacity, the loading cycle, the quality of the applied wastewater, and the treatment requirements.

5.4.1 Measured Hydraulic Capacity

Hydraulic capacity varies from site to site and is a difficult parameter to measure. For design purposes, infiltration tests are usually used to estimate hydraulic capacity. The most commonly employed measurement for RI design is the basin infiltration test; cylinder infiltrometers are used when basin testing is not feasible. Both methods are described in Section 3.4.

Saturated vertical hydraulic conductivity (also called permeability) is sometimes measured. However, saturated vertical hydraulic conductivity is a constant with time, whereas infiltration rates decrease as wastewater solids clog the soil surface. Thus, vertical conductivity measurements overestimate the wastewater infiltration rates that can be maintained over long periods of time. For this reason, and to allow adequate time for drying periods and for proper basin management, annual hydraulic loading rates should be limited to between 4 and 10% of the measured clear water permeability of the most restrictive soil layer.

Although basin infiltration tests are more accurate than soil hydraulic conductivity measurements and are the preferred method, the small areas usually used allow a larger fraction of the wastewater to flow horizontally through the soil from the test site than from an operating basin. The result is that infiltration rates at the test sites are higher than rates operating systems would achieve. Thus, design annual hydraulic loading rates should be no greater than 10 to 15% of measured basin infiltration rates.

Cylinder infiltrometers greatly overestimate operating infiltration rates. When cylinder infiltrometer measurements are used, annual hydraulic loading rates should be no greater than 2 to 4% of the minimum measured infiltration rates. Annual hydraulic loading rates based on air entry permeameter test results should be in the same range. Annual loading rates and corresponding infiltration rates for several operating RI systems are presented in Table 5-10. Suggested loading rates are summarized in Table 5-11.

TABLE 5-10
TYPICAL HYDRAULIC LOADING RATES FOR RI SYSTEMS [1, 4-9]

Location	(1) Operating basin infiltration rate, cm/d	(2) Cylinder infiltrometer rate, cm/d	(3) Vertical hydraulic conductivity, cm/d	(4) Annual loading rate				
				m/yr	cm/d[a]	% of (1)	% of (2)	% of (3)
Boulder, Colorado	33.6-110	106-290	--	30.5-48.8[b]	8.4-13.4	10-38	4-10	--
Brookings, South Dakota	41.5	--	--	24-36[b]	6.6-9.9	16-24	--	--
Flushing Meadows, Arizona	60	--	120	122[b] 60[b]	33.4 16.4	56 27	-- --	28 14
Fort Devens, Massachusetts	62.4	401	--	29[c]	7.9	13	2	--
Hollister, California	17.7	140	--	15.4[c]	4.2	24	3	--
Lake George, New York	>15.2	--	61	43[b]	11.8	<78	--	19
Vineland, New Jersey	--	379	--	21.5[c]	5.9	--	1.6	--

a. Average annual loading rate divided by 365.

b. Secondary effluent.

c. Primary effluent.

TABLE 5-11
SUGGESTED ANNUAL HYDRAULIC LOADING RATES

Field measurement	Annual loading rate
Basin infiltration test	10-15% of minimum measured infiltration rate
Cylinder infiltrometer and air entry permeameter measurements	2-4% of minimum measured infiltration rate
Vertical hydraulic conductivity measurements	4-10% of conductivity of most restricting soil layer

The total hydraulic load includes both precipitation and wastewater. If the local precipitation is significant, wastewater loading rates should be adjusted accordingly.

Once the hydraulic capacity has been measured, the engineer must calculate an annual hydraulic loading rate. Experience in the United States with treatment systems using RI has been limited to annual loading rates of about 120 m (400 ft) or less.

For example, if the basin test infiltration rate is 3.6 cm/h (1.4 in./h), the annual hydraulic loading rate is calculated to equal:

$$3.6\ \text{cm/h} \times 24\ \text{h/d} \times 365\ \text{d/yr} \times 1\ \text{m/100 cm} \times (0.1\ \text{to}\ 0.15)$$
$$= 31.5\ \text{to}\ 47.3\ \text{in/yr}\ (103\ \text{to}\ 155\ \text{ft/yr})$$

It is necessary to ensure that BOD and suspended solids are within typical ranges (Sections 2.2.1.1 and 5.2.1) at the calculated annual loading rate. If the applied wastewater contains 150 mg/L BOD and 100 mg/L suspended solids, at a loading rate of 31 in/yr (102 ft/yr), the BOD and SS loadings would average 127 kg/had (114 lb/acre•d) and 85 kg/ha•d (76 lb/acre•d), respectively. These quantities are within the typical BOD range given in Table 2-3 and the suspended solids range discussed in Section 2.2.1.1.

5.4.2 Selection of Hydraulic Loading Cycle and Application Rate

Wastewater application is not continuous in RI, instead, application periods are alternated with drying periods. This improves wastewater treatment efficiency, maximizes long–term infiltration rates, and allows for periodic basin maintenance.

Loading cycles are selected to maximize either the infiltration rate, nitrogen removal, or nitrification. To maximize infiltration rates, the engineer should include drying periods that are long enough for soil reaeration and for drying and oxidation of filtered solids.

Loading cycles used to maximize nitrogen removal vary with the level of preapplication treatment and with the climate and season. In general, application periods must be long enough for soil bacteria to deplete soil oxygen, resulting in anaerobic conditions.

Nitrification requires short application periods followed by longer drying periods. Thus, hydraulic loading cycles used to achieve nitrification are essentially the same as the cycles used to maximize infiltration rates.

Hydraulic loading cycles at selected RI sites are presented in Table 5-12. Recommended cycles are summarized in Table 5-13. Generally, the shorter drying periods shown in Table 5-13 should be used only in mild climates; RI systems in cooler climates should use the longer drying periods. In areas that experience extremely cold weather, even longer drying periods than those presented in Table 5-13 may be necessary. The cycles suggested in Table 5-13 are presented only as guidelines; the actual cycle selected should be suitable and flexible enough for the community*s climate, flow, and treatment site characteristics.

Application rates can be calculated from the annual loading rate and the loading cycle. For example, the annual loading rate is 31 in/yr (102 ft/yr) and the loading cycle is 3 days of application followed by 11 days of drying.

! Total cycle time = 3 + 11 = 14 d

! Number of cycles per year = 365/14 = 26

! Loading per cycle = 31/26 = 1.19 in/cycle

! Application rate = (1.19 m/cycle)/(3 d)
= 0.4 m/d

The application rate can then be used to calculate the maximum depth of applied wastewater. For example, if the basin infiltration test rate of 3.6 cm/h (1.4 in./h) is maintained over the 3 day application period, the application rate of 0.4 m/d (1.3 ft/d) should not result in standing water at the end of 3 days:

(0.4 m/d x 100 cm/in) – (3.6 cm/h x 24 h/d)
= -46.4 cm (-18.3 in.)

TABLE 5-12
TYPICAL HYDRAULIC LOADING CYCLES [6, 9, 18, 19]

Location	Preapplication treatment	Cycle objective	Application period	Resting period	Bed surface
Boulder, Colorado	Trickling filters	Maximize nitrification and infiltration rates	<1 d	<3 1/2 d	Sand (disked), solids turned into soil
Calumet, Michigan	Untreated	Maximize infiltration rates	1-2 d	7-14 d	Sand (not cleaned)
Flushing Meadows, Arizona	Activated sludge				
Year-round		Maximize nitrification	2 d	5 d	Sand (cleaned)[a]
Summer		Maximize infiltration rates	2 wk	10 d	Sand (cleaned)[a]
Winter		Maximize infiltration rates	2 wk	20 d	Sand (cleaned)[a]
Year-round		Maximize nitrogen removal	9 d	12 d	Sand (cleaned)[a]
Fort Devens, Massachusetts	Primary				
Year-round		Maximize infiltration rates	2 d	14 d	Weeds (not cleaned)
Year-round		Maximize nitrogen removal	7 d[b]	14 d	Weeds (not cleaned)
Hollister, California	Primary				
Summer		Maximize infiltration rates	1 d	14-21 d	Sand
Winter		Maximize infiltration rates	1 d	10-16 d	Sand
Lake George, New York	Trickling filters				
Summer		Maximize infiltration rates	9 h	4-5 d	Sand (cleaned)[a]
Winter		Maximize infiltration rates	9 h	5-10 d	Sand (cleaned)[a]
Tel Aviv, Israel	Ponds, lime precipitation, and ammonia stripping	Maximize polishing	5-6 d	10-12 d	Sand[c]
Vineland, New Jersey	Primary	Maximize infiltration rates	1-2 d	7-10 d	Sand (disked) solids turned into soil
Westby, Wisconsin	Trickling filters	Maximize infiltration rates	2 wk	2 wk	Grassed
Whittier Narrows, California	Activated sludge with filtration[d]	Maximize infiltration rates	9 h	15 h	Pea gravel

a. Cleaning usually involved physical removal of surface solids.

b. Caused clogging and reduced long-term hydraulic capacity.

c. Maintenance of sand cover is unknown.

d. Treated wastewater blended with surface waters before application.

TABLE 5-13
SUGGESTED LOADING CYCLES

Loading cycle objective	Applied wastewater	Season	Application period, d[a]	Drying period, d
Maximize infiltration rates	Primary	Summer	1-2	5-7
		Winter	1-2	7-12
	Secondary	Summer	1-3	4-5
		Winter	1-3	5-10
Maximize nitrogen removal	Primary	Summer	1-2	10-14
		Winter	1-2	12-16
	Secondary	Summer	7-9	10-15
		Winter	9-12	12-16
Maximize nitrification	Primary	Summer	1-2	5-7
		Winter	1-2	7-12
	Secondary	Summer	1-3	4-5
		Winter	1-3	5-10

a. Regardless of season or cycle objective, application periods for primary effluent should be limited to 1-2 days to prevent excessive soil clogging.

If the calculated depth is a positive number, the maximum design wastewater depth should not exceed 46 cm (18 in.); a maximum depth of 30 cm (12 in.) is preferable because soil clogging and algae growth decrease as the loading depth and detention time decrease. If the calculated depth exceeds 46 cm (18 in.) either the application period must be lengthened or the loading rate decreased. From this example, it is clear that infiltration rates must be determined as accurately as possible. If the infiltration rate is overestimated, basin depth will be underestimated and difficulties will arise when system operation begins.

5.4.3 Other Considerations

The following three subsections describe other factors that can affect the loading cycle and loading rate and must be considered by the designer.

5.4.3.1 Nitrogen Removal

The amount of nitrogen that theoretically (under optimal conditions) can be removed by denitrification can be described by the equation [19].

$$\Delta N = \frac{TOC - K}{2} \tag{5-1}$$

where)N = change in total nitrogen concentration, mg/L

TOC = total organic carbon concentration in the applied wastewater, mg/L (see Table 2-1)

K = TOC remaining in percolate, assumed to equal 5 mg/L

The equation is based on experimental data that indicated 2 grams of wastewater carbon are needed to denitrify 1 gram of wastewater nitrogen [19].

Equation 5-1 can be used to determine whether a wastewater contains enough carbon to remove the desired amount of nitrogen. For example, if the applied wastewater contains 42 mg/L TOC and 25.8 mg/L total nitrogen, it is only possible to remove (42-5)/2 mg/L or 18.5 mg/L of nitrogen and to reduce the total nitrogen concentration from 25.8 mg/L to 7.3 mg/L. Thus, using this wastewater, complete nitrogen removal could not be achieved. If the applied wastewater contains 248 mg/L TOC and 40.2 mg/L total nitrogen, there is sufficient carbon to remove 121 mg/L of nitrogen. This means that, theoretically, under proper management, all of the nitrogen could be removed during RI (although total removal might never be achieved in practice). If nitrogen removal is important, the engineer should use Equation 5-1 to determine whether nitrogen removal is feasible using RI. If so, a loading cycle should be selected that maximizes nitrogen removal.

Nitrogen removal from secondary effluent is more difficult than nitrogen removal from a wastewater that contains high concentrations of organic carbon. Nitrogen removal is especially difficult when infiltration rates are high, because nitrates tend to pass through the soil profile before they can be converted to nitrogen gas. In fact, nitrogen removal from secondary effluent increases exponentially as the infiltration rate decreases [20]. This relationship is shown in Figure 5-2.

Although Figure 5-2 is based on data from soil column studies using loamy sand, data from operating systems in warm climates indicate that the figure can be used to obtain conservative estimates of a similar soil*s nitrogen removal potential. Thus, if secondary effluent infiltrates at a rate of 30 cm/d (12 in./d), using a loading cycle that promotes nitrogen removal, it should be possible to remove at least 30% of the applied nitrogen. To achieve 80% nitrogen removal, the soil column studies indicated maximum infiltration rates are:

- 20 cm/d (8 in./d) for primary preapplication treatment

- 15 cm/d (6 in./d) for secondary preapplication treatment

If nitrogen removal is important and these suggested rates are exceeded, soil column studies or pilot testing should be conducted to determine how much nitrogen can be removed. Also, infiltration rates can be reduced somewhat by decreasing the depth of the applied wastewater, or by compacting the soil surface.

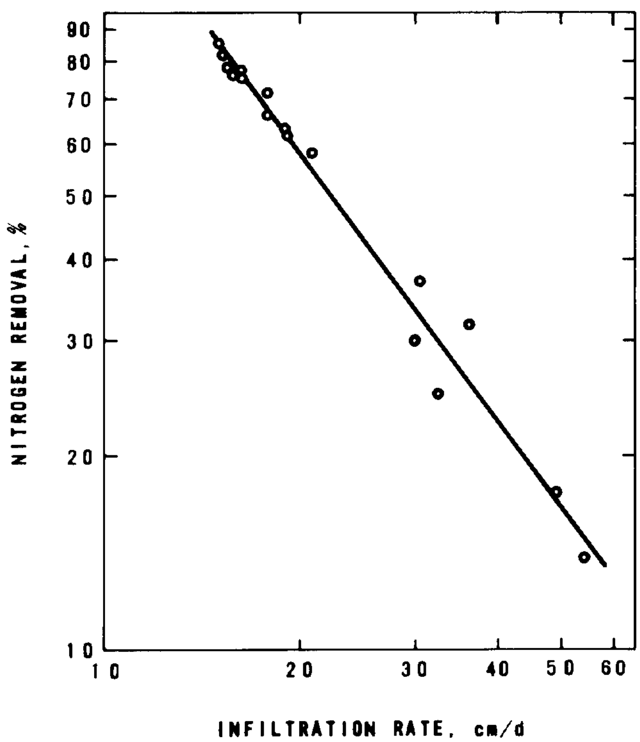


FIGURE 5-2
EFFECT OF INFILTRATION RATE ON NITROGEN REMOVAL [20]

5.4.3.2 phosphorus Removal

The amount of phosphorus that is removed during RI at neutral pH can be estimated from the following equation [19, 21]:

$$C_x = C_o e^{-kt} \qquad (5\text{-}2)$$

where C_x = total phosphorus concentration at a distance x along the percolate flow path, mg/L

C_0 = total phosphorus concentration in the applied wastewater, mg/L

k = instantaneous rate constant and equals 0.002 h^{-1} at neutral pH

t = detention time = X2/I, h

where x = distance along the flow path, cm

2 = volumetric water content, cm^3/cm^3, use 0.4

I = infiltration rate during system operation, cm/h (use basin test results, 20% of cylinder infiltration results, or horizontal conductivity for horizontal flow)

Because the minimum phosphorus precipitation rate occurs at neutral pH, this equation can be used to conservatively estimate phosphorus removal. If the calculated phosphorus concentration is an acceptable value, phosphorus concentrations from an operating RI system should be well within limits. However, if the calculated phosphorus concentration at a distance x exceeds acceptable values, a phosphorus adsorption test should be performed. This test measures the ability of a specific soil to remove phosphorus and is described in Section 3.7.2.

For example, consider a site where wastewater percolates through the soil to the ground water table, which is 15 m (49 ft) below the soil surface. The initial phosphorus concentration is 10 mg/L and the basin infiltration test rate is 40 cm/d (16 in./d). By the time the water reaches the ground water table, the phosphorus concentration should be less than:

$$(10\ \mathrm{mg/L})e^{-0.002h^{-1}\left(\frac{15\ \mathrm{m} \times 0.40}{0.4\ \mathrm{m/d}}\right)\left(\frac{24\ \mathrm{h}}{\mathrm{d}}\right)} = 4.9\ \mathrm{mg/L}$$

If the movement is then predominantly horizontal, with the renovated water seeping into a creek 200 m (650 ft) from the infiltration site, and the horizontal hydraulic conductivity is 120 cm/d (47 in./d), the phosphorus concentration in the seepage should be less than:

$$(4.9\ \mathrm{mg/L})e^{-0.002h^{-1}\left(\frac{200\ \mathrm{m} \times 0.40}{1.2\ \mathrm{m/d}}\right)\left(\frac{24\ \mathrm{h}}{\mathrm{d}}\right)} = 0.2\ \mathrm{mg/L}$$

5.4.3.3 Climate

In regions that experience cold weather, longer loading cycles may be necessary during winter months (Section 5.4.2). Nitrification, denitrification, oxidation (of accumulated organics), and drying rates all decrease during cold weather, particularly as the temperature of the applied wastewater decreases. Longer application periods are needed for denitrification so that the application rate can be reduced as the rate of nitrogen removal decreases. Similarly, longer resting periods are needed to compensate for reduced nitrification and drying rates.

Combined with the reduced hydraulic capacity experienced during cold weather, the need for longer loading cycles changes the allowable wastewater loading rate. Cold weather loading rates are somewhat lower than warm weather rates; therefore, more land is required during cold weather as long as winter and summer wastewater flows are equal. If loading rates must be reduced during cold weather, either the cold weather loading rate should be used to determine land requirements or cold weather storage should be included.

In communities that use ponds as preapplication treatment and experience cold winter weather, winter storage may be required. This is because the temperature of the wastewater becomes quite low prior to land treatment and makes the applied wastewater susceptible to long-term freezing in the basin. Alternatively, RI may be continued through cold weather if warmer wastewater from the first cell of the pond system (if possible) is applied. In such communities, the engineer must keep in mind that the annual loading rate actually applies only to the portion of the year when RI is used.

5.5 Land Requirements

An RI site must have adequate land for infiltration basins, buffer zones, and access roads. At some systems, land is also needed for preapplication treatment facilities, storage, or future expansion.

5.5.1 Infiltration Basin Area

If wastewater flow equalization is provided (including treatment ponds), the land area required for infiltration only (ignoring land required between and around basins) is simply the average annual wastewater flow divided by the annual wastewater loading rate. For example, if the annual average daily flow is 0.3 m^3/s (6.8 Mgal/d) and the wastewater loading rate is 25 in/yr (82 ft/yr), the area required for infiltration is:

$$\frac{(0.3\ m^3/s)(86{,}400\ s/d)(365\ d/yr)}{(25\ m/yr)(10^4\ m^2/ha)} = 37.8\ ha\ (93.5\ acres)$$

If the wastewater flow varies with season and seasonal flows are not equalized, the highest average seasonal flow should be used. An RI site must either have enough basins so that at least one basin can be dosed at all times or have adequate storage for equalization between application periods.

5.5.2 Preapplication Treatment Facilities

The communities that already have preapplication treatment facilities will, in general, only need additional land for facilities to convey wastewater to the RI site. In communities that are constructing a completely new treatment facility, land requirements for preapplication treatment will vary with the level and method of preapplication treatment.

5.5.3 Other Land Requirements

Additional land may be needed for buffer zones, access roads, storage or flow equalization (when provided), and future expansion. Buffer zones can be used to screen RI sites from public view. Preapplication treatment facilities, access roads, and storage or flow equalization may be included in the buffer area.

Access roads must be provided so that equipment and labor can reach the infiltration basins. Maintenance equipment must be able to enter each basin (for scarification or surface maintenance).

Typically, access roads should be 3 to 3.7 in (10 to 12 ft) wide. In any case, access roads should be wide enough for the selected maintenance equipment and curves should have large enough radii to allow maintenance equipment to turn safely.

Land requirements for flow equalization or storage vary with the type and amount of storage provided. This subject is discussed in greater detail in Section 5.6.2.

5.6 Infiltration System Design

Items that must be addressed during RI system design include wastewater distribution, basin layout and dimensions, basin surfaces, and flow equalization or storage. In areas that experience cold winter weather, cold weather system modifications should also be considered.

5.6.1 Distribution and Basin Layout

Although sprinklers may be used, wastewater distribution is usually by surface spreading. This distribution technique employs gravity flow from piping systems or ditches to flood the application area. To ensure uniform basin application, basin surfaces should be reasonably flat.

Overflow weirs may be used to regulate basin water depth. Water that flows over the weirs is either collected and conveyed to holding ponds for recirculation or distributed to other infiltration basins. If each basin is to receive equal flow, the distribution piping channels should be sized so that hydraulic losses between outlets to basins are insignificant. Design standards for distribution systems and for flow control and measurement techniques are published by the American Society of Agricultural Engineers (ASAE). Outlets used at currently operating systems include valved risers for underground piping systems and turnout gates from distribution ditches. An infiltration basin outlet and splash pad are shown in Figure 5-3. An adjustable weir used as an interbasin transfer structure is shown in Figure 5-4.

Basin layout and dimensions are controlled by topography, distribution system hydraulics, and loading rate. The number of basins is also affected by the selected loading cycle. As a minimum, the system should have enough basins

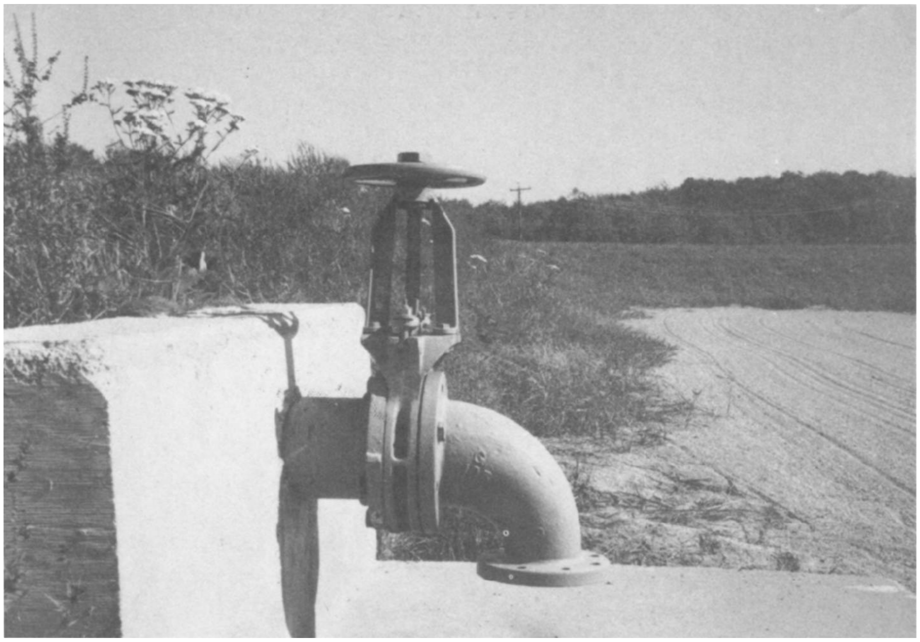


FIGURE 5-3
INFILTRATION BASIN OUTLET AND
SPLASH PAD

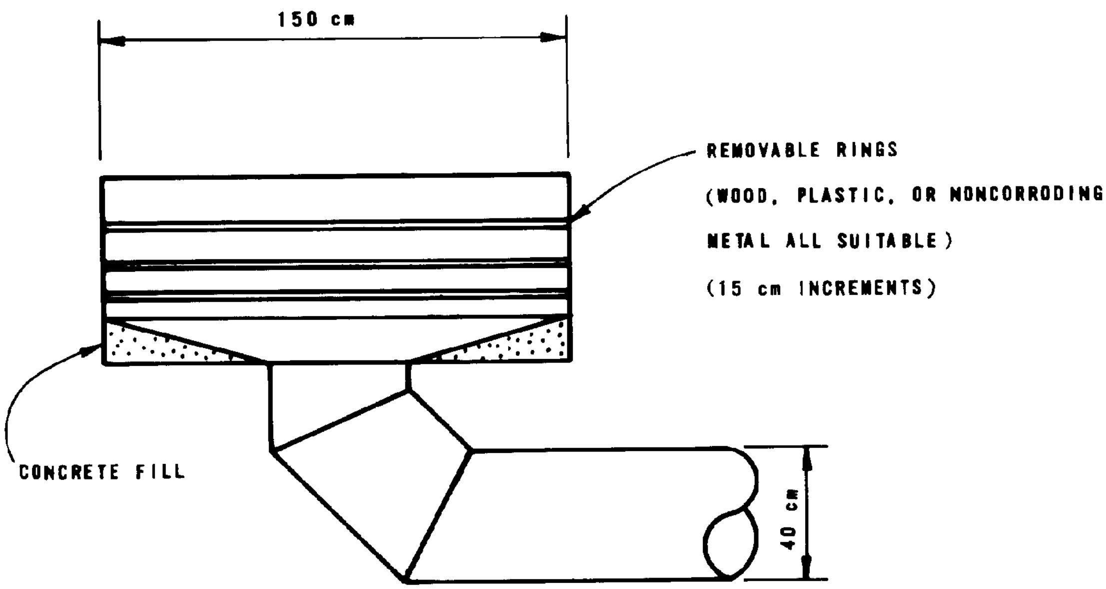


FIGURE 5-4
INTERBASIN TRANSFER STRUCTURE WITH ADJUSTABLE WEIR

so that at least one basin can be loaded at all times, unless storage is provided. The minimum number of basins required for continuous wastewater application is presented as a function of loading cycle in Table 5–14. The engineer should keep in mind that if the minimum number of basins is used, the resulting loading cycle may not be exactly as planned. For example, if the selected loading cycle is 2 application days followed by 6 days of drying and 4 basins are constructed, the resulting loading cycle will be the same as the selected loading cycle. However, if a cycle of 2 days of application followed by 9 days of drying is selected initially and 6 basins are constructed, the resulting loading cycle will actually be 2 days of application followed by 10 days of drying.

TABLE 5–14
MINIMUM NUMBER OF BASINS REQUIRED FOR CONTINUOUS WASTEWATER APPLICATION

Loading application period, d	Cycle drying period, d	Minimum number of infiltration basins
1	5-7	6-8
2	5-7	4-5
1	7-12	8-13
2	7-12	5-7
1	4-5	5-6
2	4-5	3-4
3	4-5	3
1	5-10	6-11
2	5-10	4-6
3	5-10	3-5
1	10-14	11-15
2	10-14	6-8
1	12-16	13-17
2	12-16	7-9
7	10-15	3-4
8	10-15	3
9	10-15	3
7	12-16	3-4
8	12-16	3
9	12-16	3

The number of basins also depends on the total area required for infiltration. Optimum basin size can range from 0.2 to 2 ha (0.5 to 5 acres) for small to medium sized systems to 2 to 8 ha (5 to 20 acres) for large systems. For a 25 ha (62 acre) system, if the selected loading cycle is 1 day of wastewater application alternated with 10 days of drying, a typical design would include 22 basins of 1.14 ha (2.8 acres) each. Using 22 basins, 2 basins would be flooded at a time and there would be ample time for basin maintenance before each flooding period.

At many sites, topography makes equal-sized basins impractical. Instead, basin size is limited to what will fit into areas having suitable slope and soil type (Section 2.3.1). Relatively uniform loading rates and loading cycles can be maintained if multiple basins are constructed. However, some sites will require that loading rates or cycles vary with individual basins.

In flat areas, basins should be adjoining and should be square or rectangular to maximize land use. In areas where ground water mounding is a potential problem (Section 5.7.2), less mounding occurs when long, narrow basins with their length normal to the prevailing ground water flow are used than when square or round basins are constructed. Basins should be at least 30 cm (12 in.) deeper than the maximum design wastewater depth, in case initial infiltration is slower than expected and for emergencies. Basin walls are normally compacted soil with slopes ranging from 1:1 to 1:2 (vertical distance to horizontal distance). In areas that experience severe winds or heavy rains, basin walls should be planted with grass or covered with riprap to prevent erosion.

If basin maintenance will be conducted from within the basins, entry ramps should be provided. These ramps are formed of compacted soil at grades of 10 to 20% and are from 3.0 to 3.7 m (10 to 12 ft) wide. Basin surface area for these ramps and for wall slopes should not be considered as part of the necessary infiltration area.

The basin surface may be bare or covered with vegetation. Vegetative covers tend to remove suspended solids by filtration and maintain infiltration rates. However, vegetation also limits the application depth to a value that avoids drowning of vegetation, increases basin maintenance needs, requires an increased application frequency to promote growth, and reduces the soil drying rate. At Lake George, New York, allowing grass to grow in the basins improved the infiltration rate when flooding depths exceeded 0.3 m (1 ft) but decreased the rate at shallower wastewater depths [1] Gravel covered basins are not recommended. The long-term infiltration capacity of gravel covered basins is lower than the capacity of sand covered basins, because sludge-like solids collect in the voids between gravel particles and because gravel prevents the underlying soil from drying [4]

5.6.2 Storage and Flow Equalization

Although RI systems usually are capable of operating during adverse climatic conditions, storage may be needed to regulate wastewater application rates or for emergencies. Flow equalization may be required if significant daily or seasonal flow peaking occurs. Equalization also may be necessary to store wastewater between application periods, particularly when only one or two infiltration basins are used and drying periods are much longer than application periods.

One example of flow equalization at an RI site occurs at the Milton, Wisconsin, system. Milton discharges secondary effluent to three lagoons. One of these lagoons is used as an infiltration basin; the other two lagoons are used for storage. In this way, Milton is able to maintain a continuous flow into the infiltration basin [3].

In contrast, the City of Hollister formerly equalized flow with an earthen reservoir that was ahead of the treatment plant headworks. In addition, one infiltration basin was kept in reserve for primary effluent during periods when wastewater flows were excessive [6].

Winter storage may be needed if the soil permeability is on the low end for RI. In such cases, the water may not drain from the profile fast enough to avoid freezing.

5.6.3 Cold Weather Modifications

Rapid infiltration systems that operate successfully during cold winter weather without any cold weather modifications can be found in Victor, Montana; Calumet, Michigan; and Fort Devens, Massachusetts. However, a few different basin modifications have been used to improve cold weather treatment in other communities. First, basin surfaces that are covered with grass or weeds should be mowed during fall. Mowing followed by disking should prevent ice from freezing to vegetation near the soil surface. Floating ice helps insulate the applied wastewater, whereas ice that freezes at the soil surface prevents infiltration. Problems with ice freezing to vegetation have been reported at Brookings, South Dakota, where basins were not mowed and ponds are used for preapplication treatment [7].

Another cold weather modification involves digging a ridge and furrow system in the basin surface. Following wastewater application, ice forms on the surface of the water and forms bridges between the ridges as the water level drops. Subsequent loadings are applied beneath the surface of the ice, which insulates the wastewater and the soil surface. For bridging to occur, a thick layer of ice must form before the wastewater surface drops below the top of the ridges. This modification has been used successfully in Boulder, Colorado, and Westby, Wisconsin.

The third type of basin modification involves the use of snow fencing or other materials to keep a snow cover over the infiltration basins. The snow insulates both applied wastewater and soil.

5.7 Drainage

Rapid infiltration systems require adequate drainage to maintain infiltration rates and treatment efficiencies. The infiltration rate may be limited by the horizontal hydraulic conductivity of the underlying aquifer. Also, if there is insufficient drainage, the soil will remain saturated with water and reaeration will be inadequate for oxidation of ammonia nitrogen to occur.

Renovated water may be isolated to protect either or both the ground water or the renovated water. In both cases, there must be some method of engineered drainage to keep renovated water from mixing with native ground water.

Natural drainage often involves subsurface flow to surface waters. If water rights are important, the engineer must determine whether the renovated water will drain to the correct watershed or whether wells or underdrains will be needed to convey the renovated water to the required surface water. <u>In all cases, the engineer needs to determine the direction of subsurface flow due to drainage from RI basins</u>.

5.7.1 Subsurface Drainage to Surface Waters

If natural subsurface drainage to surface water is planned, soil characteristics can be analyzed to determine if the renovated water will flow from the recharge site to the surface water. For subsurface discharge to a surface water to occur, the width of the infiltration area must be limited to values equal to or less than the width calculated in the following equation [22]:

$$W = KDH/dL \qquad (5\text{-}3)$$

where W = total width of infiltration area in direction of ground water flow, m(ft)

K = permeability of aquifer in direction of groundwater flow, m/d (ft/d)

D = average thickness of aquifer below the water table and perpendicular to the direction of flow, m (ft)

H = elevation difference between the water level of the water course and the maximum allowable water table below the spreading area, m (ft)

d = lateral flow distance from infiltration area to surface water, m (ft)

L = annual hydraulic loading rate (expressed as daily rate), m/d (ft/d)

Examples of these parameters are shown in Figure 5-5.

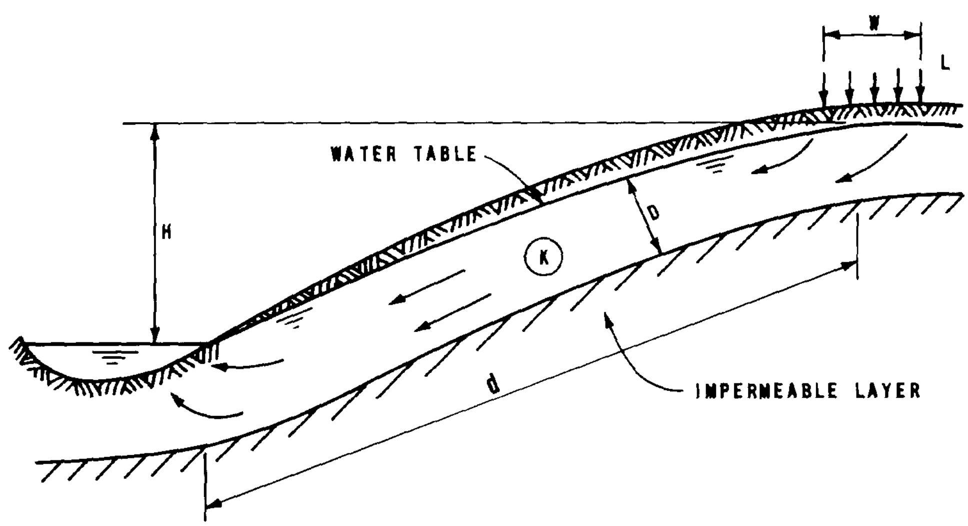


FIGURE 5-5
NATURAL DRAINAGE OF RENOVATED WATER INTO SURFACE WATER [22]

As an example, consider an infiltration site located above an aquifer whose permeability is 1.1 in/d (3.6 ft/d) and whose average thickness is 9 m (30 ft). The annual hydraulic loading rate is 30 in/yr or 0.082 m/d (98 ft/yr or 0.27 ft/d). The surface water elevation is 6 m (20 ft) below the infiltration site, and the water table should remain at least 1.5 m (5 ft) below the soil surface. The infiltration site is 25 in (82 ft) from the surface water. Thus,

$$W = \frac{(1.1 m/d)(9 m)(6 m - 1.5 m)}{(25 m)(0.082 m/d)} = 22 m (72 ft)$$

Under these conditions, either a single basin 22 m (72 ft) wide or multiple basins having a combined width of 22 m could be constructed. If more infiltration area is needed, additional basins could be built in the two directions perpendicular to the direction of ground water flow. Four basins oriented in this manner are illustrated in Figure 5–6.

If the calculated width is quite small (less than about 10 m or 33 ft), natural subsurface drainage to surface waters is not feasible and engineered drainage should be provided.

5.7.2 Ground Water Mounding

During RI, the applied wastewater travels initially downward to the ground water, resulting in a temporary ground water mound beneath the infiltration site. This condition is shown schematically in Figure 5-7. Mounds continue to rise during the flooding period and only recede during the resting period.

Excessive mounding will inhibit infiltration and reduce the effectiveness of treatment. For this reason, <u>the capillary fringe above the ground water mound should never be closer than 0.6 m (2 ft) to the bottom of the infiltration basin</u> [23]. This distance corresponds to a water table depth of about 1 to 2 m (3 to 7 ft), depending on the soil texture. The distance to ground water should be 1.5 to 3 m (5 to 10 ft) below the soil surface within 2 to 3 days following a wastewater application. The following paragraphs describe an analysis that can be used to estimate the mound height that will occur at various loading conditions. This method can be used to estimate whether a site has adequate natural drainage or whether mounding will exceed the recommended values without constructed drainage.

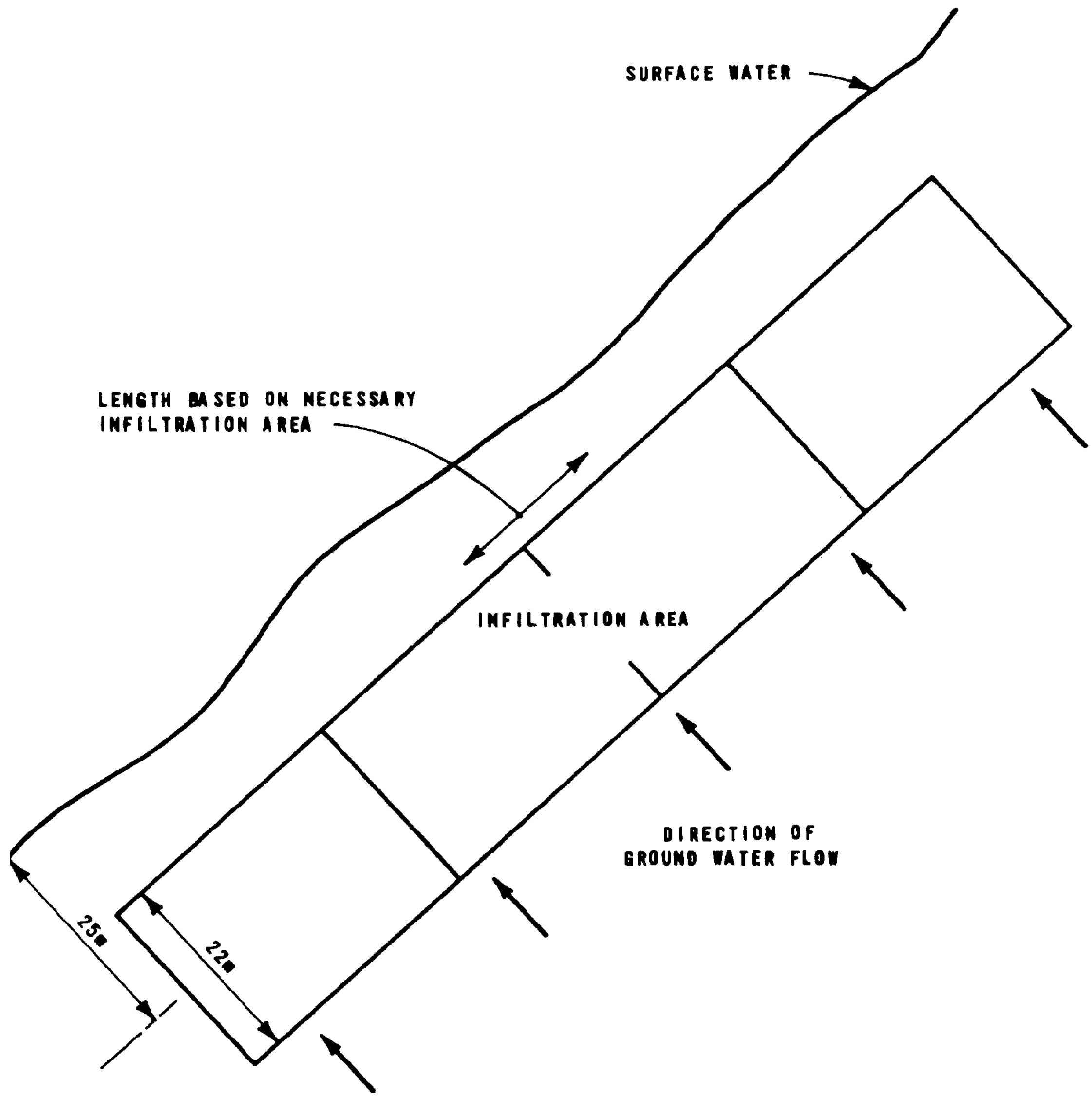


FIGURE 5-6
EXAMPLE DESIGN FOR SUBSURFACE FLOW TO SURFACE WATER

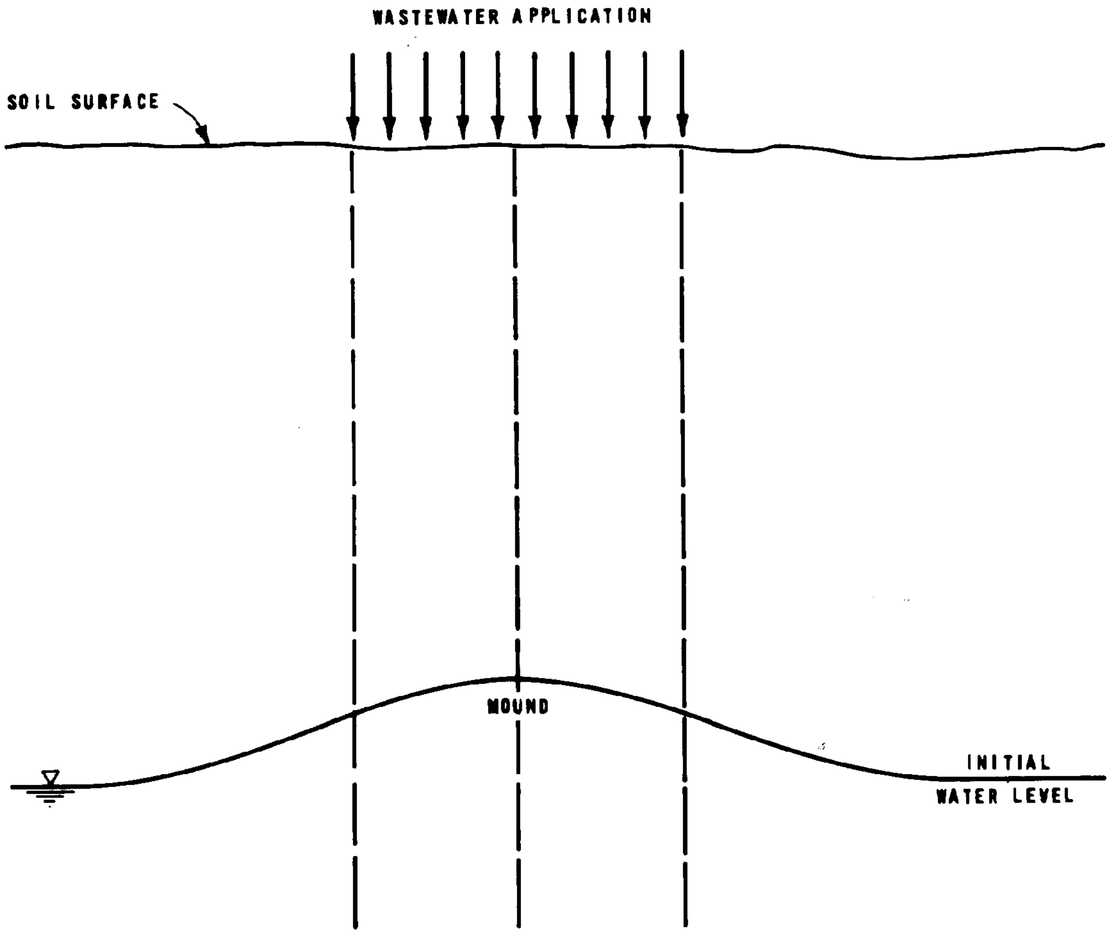


FIGURE 5-7
SCHEMATIC OF GROUND WATER MOUND

Ground water mounding can be estimated by applying heat-flow theory and the Dupuit–Forchheimer assumptions [24]. These assumptions are as follows:

1. Flow within ground water occurs along horizontal flow lines whose velocity is independent of depth.

2. The velocity along these horizontal streamlines is proportional to the slope of the free water surface.

Using these assumptions, heat–flow theory has been successfully compared to actual ground water depths at several existing RI sites.

To compute the height at the center of the ground water mound, one must calculate the values of $W/\sqrt{4\alpha t}$ and Rt,

where W = width of the recharge basin, m (ft)

" = KD/V, m^2/d (ft^2/d)

where K = aquifer (horizontal) hydraulic conductivity, m/d (ft/d)

D = saturated thickness of the aquifer, m (ft)

V = specific yield or fillable pore space of the soil, m^3/m^3 (ft^3/ft^3) (Figures 3-5 and 3–6)

t = length of wastewater application, d

R = I/V, m/d (ft/d)

where I = infiltration rate or volume of water per unit area qf soil surface, $m^3H_20/m^2 \bullet d$ ($ft^3H_20/ft^2 \bullet d$)

The parameters that can be shown schematically are illustrated in Figure 5–5.

Once the value of $W/\sqrt{4\alpha t}$ is obtained, one can use dimensionless plots of $W/\sqrt{4\alpha t}$ versus h_o/Rt, provided as Figures 5–8 (for square recharge areas) and 5–9 (for rectangular recharge areas), to obtain the value of h_o/Rt, where h_o is the rise at the center of the mound. Using the calculated value of Rt, one can solve for h_o.

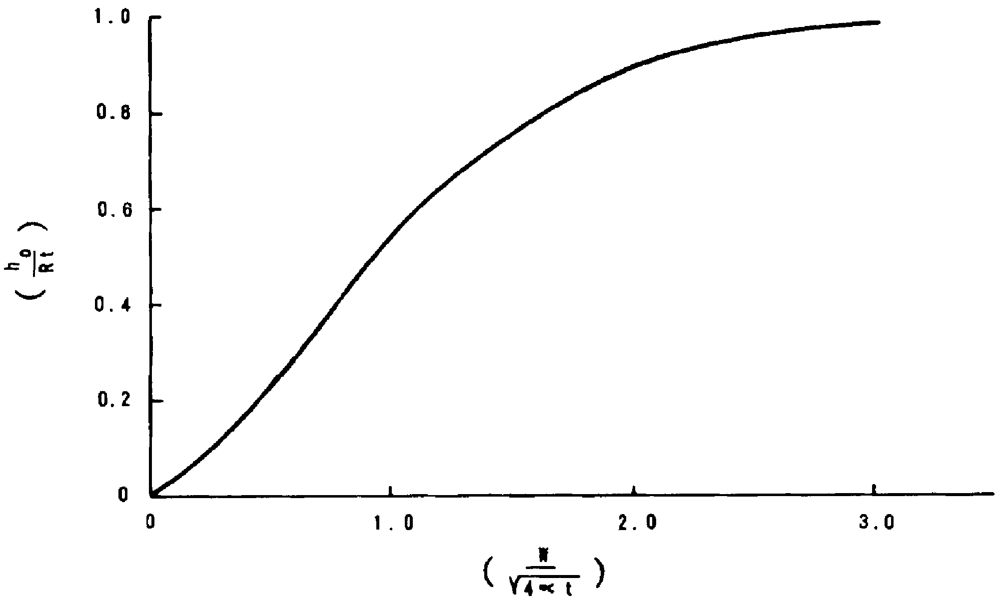


FIGURE 5-8
MOUNDING CURVE FOR CENTER OF A SQUARE RECHARGE AREA [24]

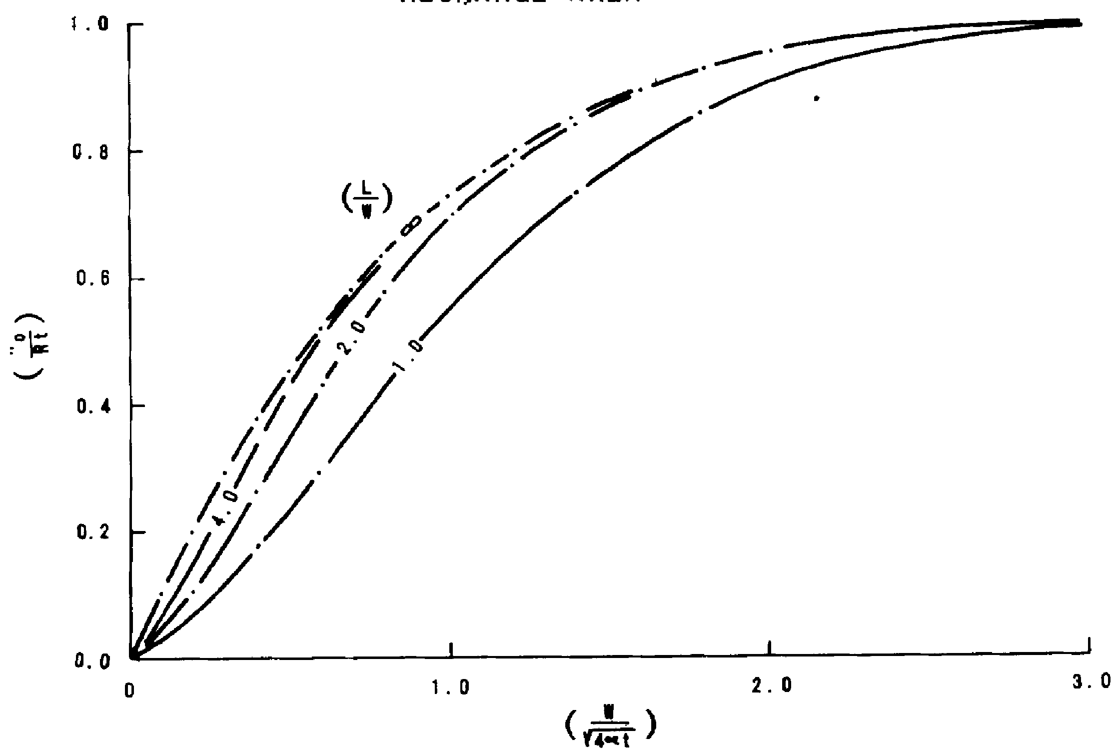


FIGURE 5-9
MOUNDING CURVE FOR CENTER OF A RECTANGULAR RECHARGE AREA AT DIFFERENT RATIOS OF LENGTH (L) TO WIDTH (W) [24]

For example, an RI system is planned above an aquifer that is 4 m (13 ft) thick. Auger hole measurements (Section 3.6.2.1) have indicated that the hydraulic conductivity is (5 m3/d)/ 4 m or 1.25 m/d (4.1 ft/d). Using Figure 3–6 with this hyraulic conductivity, the specific yield is 15%. The basins are to be 12 m (39 ft) wide and square; the basin infiltration rate is 0.20 m/d (7.9 in./d); and the application period will be 1 day long. Using these data, the following calculations are performed.

$$\alpha = \frac{(1.25\ \text{m/d})(4\ \text{m})}{0.15}$$

$$= 33.3\ \text{m}^2/\text{d}\ (360\ \text{ft}^2/\text{d})$$

$$R = \frac{0.20\ \text{m/d}}{0.15}$$
$$= 1.3\ \text{m/d}\ (4.3\ \text{ft/d})$$

$$Rt = (1.3\ \text{m/d})(1\ \text{d})$$
$$= 1.3\ \text{m}\ (4.3\ \text{ft})$$

$$W/\sqrt{4\alpha t} = \frac{12\ \text{m}}{[4(33.3\ \text{m}^2/\text{d})(1\ \text{d})]^{1/2}}$$
$$= 1.0$$

Using Figure 5–8, ho/Rt equals 0.53.

Thus, ho equals (0.53)(l.3 m) or 0.7 m 2.3 ft). If the initial ground water depth is 6.0 m (20 ft), the depth after wastewater application is still 5.3 m (17 ft) and engineered drainage is unnecessary. Should the calculations indicate that the ground water table will rise to within less than 1 to 2 m (3.3 to 6.6 ft) below the basin, additional drainage will be needed.

Figures 5–10 (for square recharge areas) and 5–11 (for recharge areas that are twice as long as they are wide) can be used to estimate the depth to the mound at various distances from the center of the recharge basin. Again the values of $W/\sqrt{4\alpha t}$ and Rt must be determined first. Then, for a given value of x/W, where x equals the horizontal distance from the center of the recharge basin, one can obtain the value of ho/Rt from the correct plot. Multiplying this number by the calculated value of Rt results in the rise of the mound, h_o, at a distance x from the center of the recharge site. The depth to the mound from the soil surface is simply the difference between the distance to the ground water before recharge and the rise due to the mound.

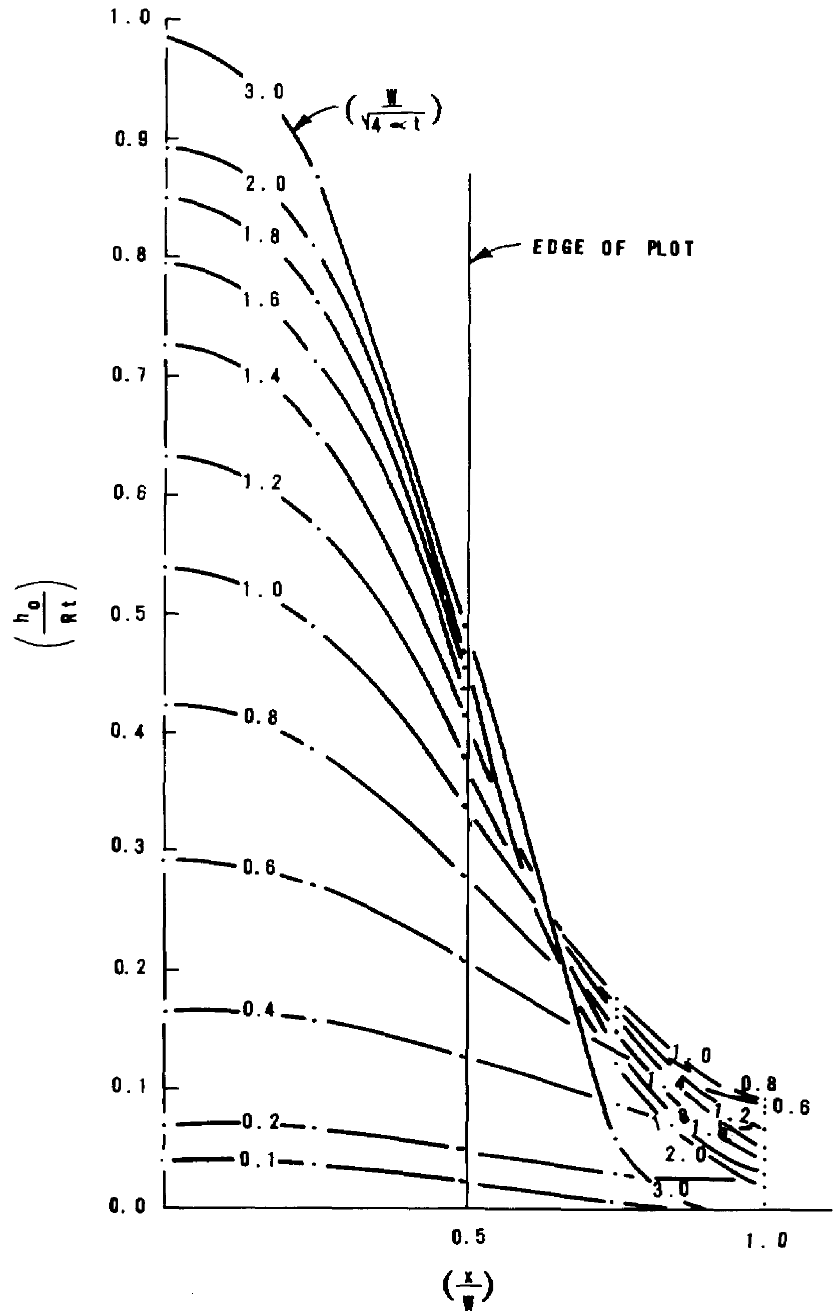


FIGURE 5-10
RISE AND HORIZONTAL SPREAD OF MOUND BELOW A SQUARE RECHARGE AREA [24]

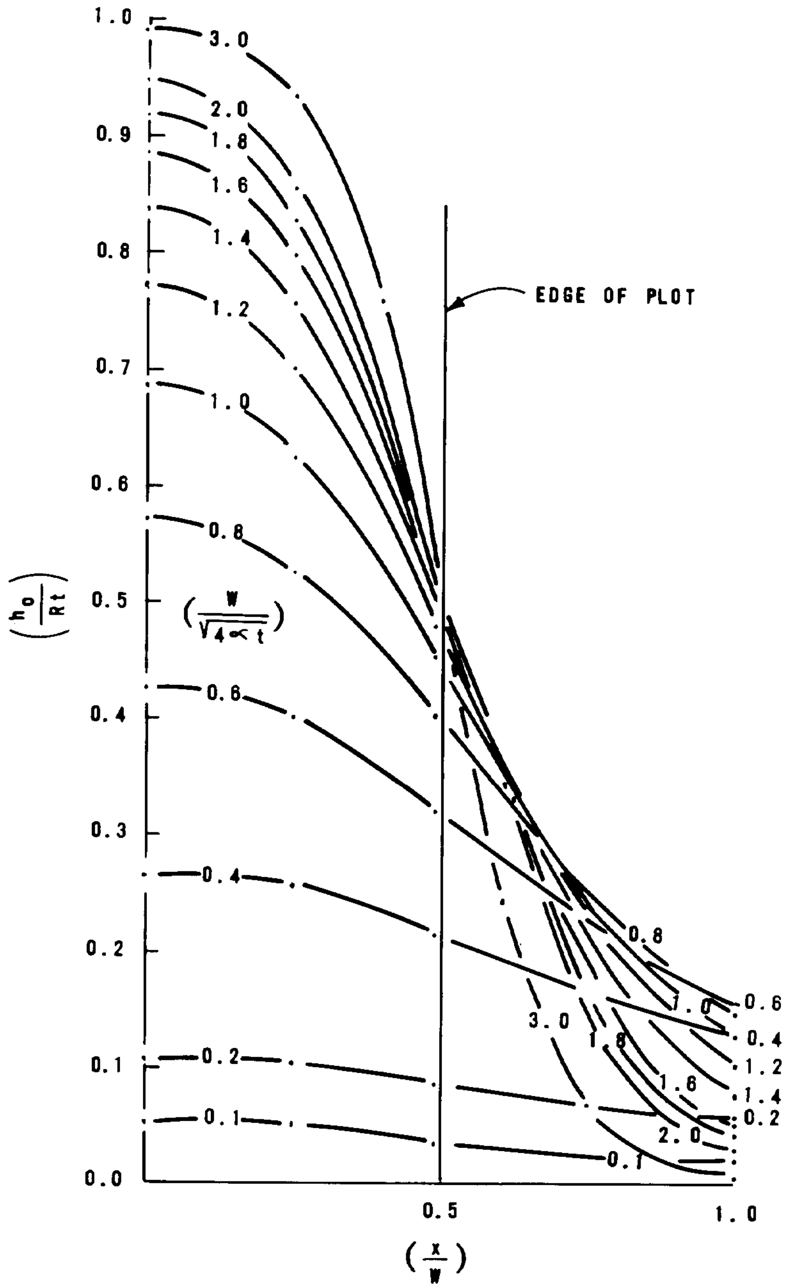


FIGURE 5-11
RISE AND HORIZONTAL SPREAD OF MOUND BELOW A RECTANGULAR RECHARGE AREA WHOSE LENGTH IS TWICE ITS WIDTH [24]

To evaluate mounding beneath adjacent basins, Figures 5–10 and 5–11 should be used to plot ground water table mounds as functions of distance from the center of the plot and time elapsed since initiation of wastewater application. Then, critical mounding times should be determined, such as when adjacent or relatively close basins are being flooded, and the mounding curves of each basin at these times should be superimposed. At sites where drainage is critical because of severe land limitations or extremely high ground water tables, the engineer should use the approach described in reference [25] to evaluate mounding.

In areas where both the water table and the impermeable layer underneath the aquifer are relatively close to the soil surface, it may be possible to avoid the complicated mounding analysis by using the following procedure:

1. Assume underdrains are needed and calculate the underdrain spacing (Section 5.7.3).

2. If the calculated underdrain spacing is relatively narrow, between 15 and 50 m (50 and 160 ft), underdrains will be required and there is no need to verify that the mound will reach the soil surface.
3. If the calculated spacing is less than about 10 m (30 ft), the loading rate may have to be reduced for the project to be economically feasible.

4. If the calculated spacing is greater than about 50 m (160 ft), mounding should be evaluated to determine if any underdrains will be necessary.

This procedure is not appropriate for unconfined or relatively deep aquifers. For such aquifers, mounding should always be evaluated.

5.7.3 Underdrains

For RI systems located in areas where both the water table and the impermeable layer underneath the aquifer are relatively close to the soil surface, renovated water can be collected by open or closed drains. In such areas, when drains can be installed at depths of 5 m (16 ft) or less, underdrains are more effective and less costly than wells for removing renovated water from the aquifer. Horizontal drains have been used to collect renovated river water from RI systems in western Holland, where polluted Rhine water is treated, and at Dortmund, Germany, where water from the Ruhr River is pretreated for a municipal water supply [23]. At

Santee, California, an open ditch was used to intercept reclaimed water [23].

Rapid infiltration systems using underdrains may consist of two parallel infiltration strips with a drain midway between the strips or a series of strips and drains. These two types of configurations are shown in Figures 5–12 and 5–13. In the first system, the drains are left open at all times during the loading cycle. If the second system is used, the drains below the strips receiving wastewater are closed and renovated water is collected from drains beneath the resting strips. When infiltration beds are rotated, the drains that were closed before are opened and those that were open are closed. This procedure allows maximum underground detention times and travel distance.

To determine drain placement, the following equation is useful [27]:

$$S = \left[\frac{4KH}{L_w + P}(2d + H)\right]^{1/2} \qquad (5\text{-}4)$$

where S = drain spacing, m (ft)

K = horizontal hydraulic conductivity of the soil, m/d (ft/d)

H = height of the ground water mound above the drains, m (ft)

L_w = annual wastewater loading rate, expressed as a daily rate, m/d (ft/d)

P = average annual precipitation rate, expressed as a daily rate, m/d (ft/d)

d = distance from drains to underlying impermeable layer, m (ft)

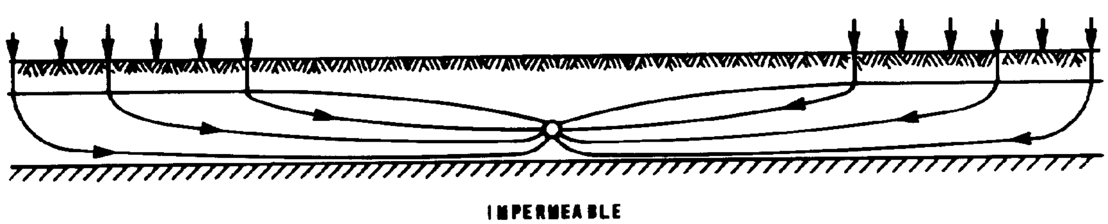


FIGURE 5-12
CENTRALLY LOCATED UNDERDRAIN [26]

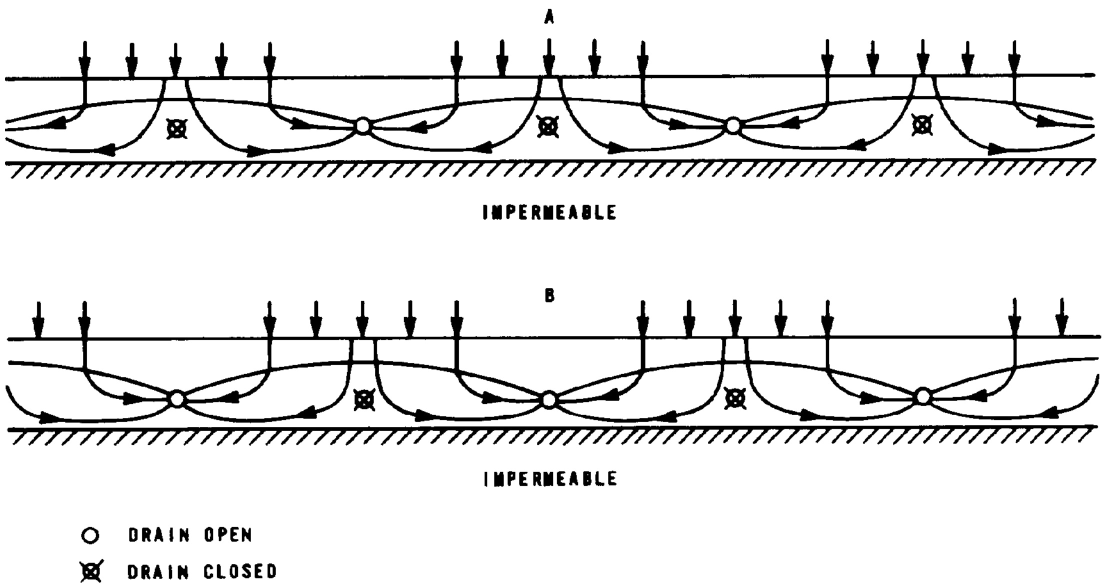


**FIGURE 5-13
UNDERDRAIN SYSTEM USING ALTERNATING
INFILTRATION AND DRYING STRIPS [26]**

For clarification, these parameters are shown in Figure 5–14. When L, P, K, and the maximum acceptable value of H are known, this equation can be used to determine S for various values of d. For example, consider an RI system loaded at an average rate of 44 m/yr or 0.12 m/d (144 ft/yr or 0.40 ft/d). Using Equation 5–4, the drain spacing can be calculated using the following data:

K = 12 m/d (39 ft/d)

H = 1 m (3.28 ft)

d = 0.6 m (2 ft)

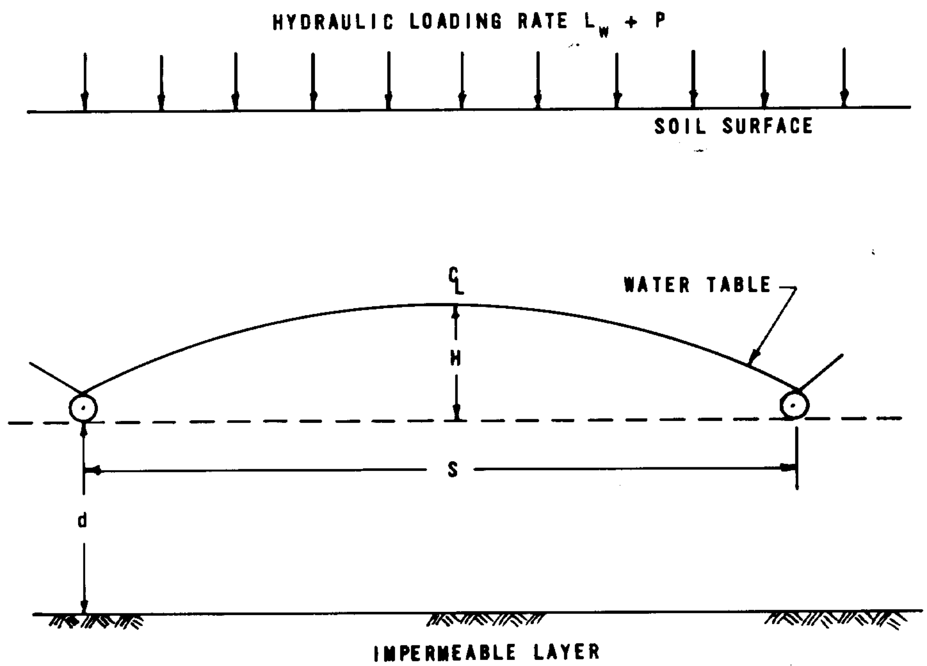


FIGURE 5-14
PARAMETERS USED IN DRAIN DESIGN [26]

The application rate must include precipitation as well as wastewater. Therefore, a design storm of 0.03 m/d (0.10 ft/d) is added to the 0.12 m/d (0.40 ft/d) wastewater load for a total of 0.15 m/d (0.50 ft/d). The drain spacing is calculated as:

$$S^2 = [4KH/L_w + P)]\ (2d + H)$$

$$= \frac{4(12\ m/d)(1\ m)}{0.12\ m/d + 0.03\ m/d}[2(0.6\ m) + 1\ m]$$

$$= 704\ m^2$$

$$S = 26\ m\ (85\ ft)$$

Generally, drains are spaced 15 m (50 ft) or more apart and are at depths of 2.5 to 5.0 m (8 to 16 ft). In soils with high lateral permeability, spacing may approach 150 m (500 ft). Although closer drain spacing allows more control over the depth of the ground water table, as drain spacing decreases the cost of providing underdrains increases. When designing a drainage system, different values of d should be selected and used to calculate S, so that the optimum combination of d, H, and S can be determined. Detailed information on drainage may be found in the U.S. Bureau of Reclamation Drainage Manual [28] and in the American Society of Agronomy manual, Drainage for Agriculture [29].

Once the drain spacing has been calculated, drain sizing should be determined, usually, 15 or 20 cm (6 in. or 8 in.) drainage laterals are used. The laterals connect to a collector main that must be sized to convey the expected drainage flows. Drainage laterals should be placed so that they will be free flowing; the engineer should check drainage hydraulics to determine necessary drain slopes.

5.7.4 Wells

Rapid infiltration systems that utilize unconfined and relatively deep aquifers should use wells to improve drainage or to remove renovated water. Wells are used to collect renovated water directly from the RI sites at both phoenix, Arizona, and Fresno, California. Wells are also involved in the reuse of recharged wastewater at Whittier Narrows, California; however, the wells pump ground water that happens to contain reclaimed water, rather than pumping specifically for renovated water.

The arrangement of wells and recharge areas varies; wells may be located midway between two recharge areas, may be placed on either side of a single recharge strip, or may surround a central infiltration area. These three configurations are illustrated in Figure 5-15. Well design is beyond the scope of this manual but is described in detail in reference [30].

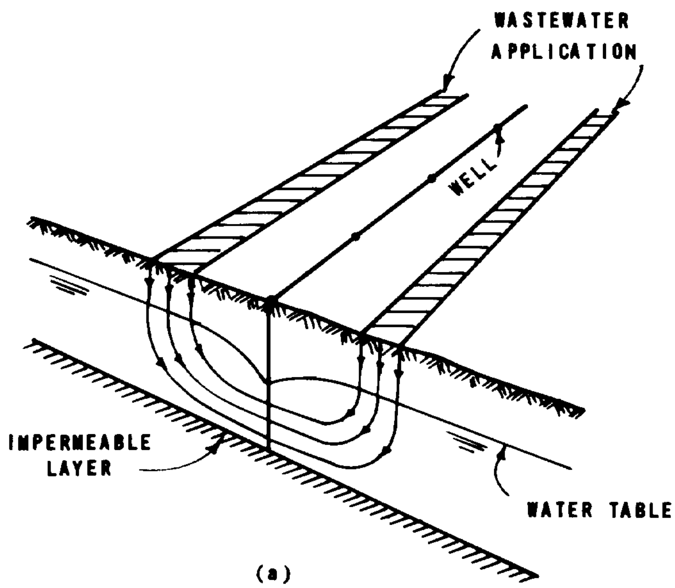
5.8 Monitoring and Maintenance Requirements

The purpose of discussing monitoring and maintenance requirements is to enable the engineer to determine labor and equipment needs. The engineer must know these needs to complete a thorough cost estimate and to ensure that the necessary labor and equipment are available.

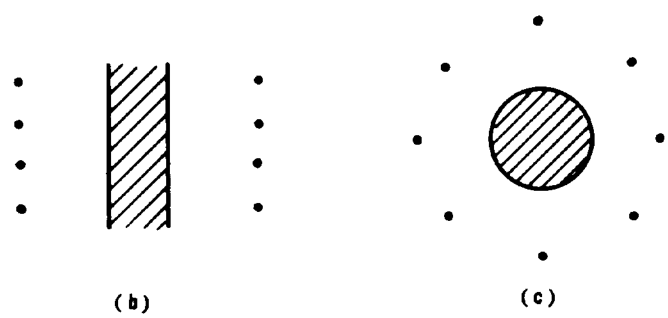
5.8.1 Monitoring

There are two distinct reasons for monitoring RI systems:

1. To document that the system meets any requirements established by appropriate regulatory agencies and to confirm that the design provides adequate treatment



a. WELLS MIDWAY BETWEEN TWO APPLICATION STRIPS



b. and c. WELLS (DOTS) SURROUNDING APPLICATION AREAS (HATCHED AREAS)

FIGURE 5-15
WELL CONFIGURATIONS [26]

2. To provide data needed to make management decisions

A monitoring program may include measurements of ground water quality, soil characteristics applied water quality, and, when appropriate, the quality of water removed from the aquifer for reuse. Representative measurements of ground water quality are difficult to obtain. Because constituent movement is slower than in surface water, a ground water sample can contain contributions from several years past that do not accurately reflect treatment occurring at the RI site. For this reason, it is important to place sampling wells in positions that minimize the time period between wastewater application and appearance of wastewater constituents in the observation wells. Techniques for monitoring well design and sampling procedures are included in references [31, 32]. Guidance in determining what parameters and site conditions to monitor can be obtained from federal, state, and local agencies.

Although soil monitoring is not required at many sites, it is periodically desirable. Below pH 6.5, soil retention of metals decreases substantially and the possibility of ground water contamination by heavy metals increases. Potential soil permeability problems may be indicated by either a high pH (above 8.5) or a high percent of sodium on the soil exchange complex (over 10 to 15%). High soil pH can indicate a high sodium content. This condition may be corrected by displacing the sodium with soluble calcium.

Both applied wastewater and any renovated water collected from the aquifer for reuse or discharge should be monitored. Applied wastewater analyses are necessary for process control to ensure that the design hydraulic loading is maintained. Renovated water that is recovered for any purpose must meet whatever water quality criteria have been established for those purposes.

5.8.2 Maintenance

Basic maintenance requirements are as follows:

- Periodic scarification or scraping of RI basin surfaces

- Periodic mowing of vegetated surfaces

As a result of bacterial activity and solids deposition, a mat forms on the surfaces of infiltration areas and reduces infiltration rates. Furthermore, wastewater applications may cause classification of the underlying soils, allowing the fines to migrate to the top and to seal the soil surface. Periodically, basin surfaces must be scarified (raked, harrowed, or disked) to break up the mat and loosen the soil surface. Alternatively, the mat may be scraped from the soil surface with a front-end loader [4] and landfilled or buried. These operations should be performed whenever regular drying fails to restore infiltration rates to acceptable levels. If scraping alone does not restore the initial infiltration rate, the soil surface should be loosened by disking or harrowing. Basin surfaces may be scarified following each drying period if time, labor, and equipment are available; basin scarification or scraping should be done at least once every 6 months to 1 year.

If grasses or other vegetation are grown on basin surfaces, the vegetation can be allowed to grow and die without maintenance. Heavy mechanical equipment that would compact the soil surface should not be operated on the infiltration basins. For aesthetic reasons, periodic mowing of the grass or harrowing of the soil surface may be desirable. In cold weather climates, vegetation should be mowed during late October or early November to prevent ice chunks from freezing to the vegetation and thereby cooling the applied wastewater.

5.9 Design and Construction Guidance

Some specific items that are unique to RI design and construction should be considered:

- Underdrains will operate only in saturated soil. If the water table does not rise, or is not already at the elevation of the drains, they will not recover any water.

- A filter sock can be used in place of a gravel envelope around plastic drain pipe in sandy soil. The filter sock will clog, however, with fines if used alone in silty clay soils.

- RI basins, when constructed, should be ripped to alleviate traffic compaction. After ripping, the surface should be smoothed and leveled, but never compacted.

- If soils at the RI site contain varying percentages of clay or silt, the heavier soils should be segregated and used for berms. Berms should be compacted, but infiltration surfaces should not be compacted.

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