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PROCESS DESIGN MANUAL
FOR
LAND TREATMENT OF
MUNICIPAL WASTEWATER

SUPPLEMENT
ON
RAPID INFILTRATION
AND
OVERLAND FLOW

U. S. ENVIRONMENTAL PROTECTION AGENCY

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FOREWORD

This document is intended as a supplement to the 1981 Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater, (EPA 625/1-81-013). Throughout this document, the 1981 Process Design Manual will be referred to as the Manual. Part I in this text covers design of rapid infiltration systems and Part II discusses overland flow systems. A substantial amount of new information on both the concepts and their performance has been developed since 1980-81.

Part I on rapid infiltration provides additional guidance and detail on planning, design, construction, and operation of rapid infiltration systems to avoid problems which have been observed at a few recently constructed systems. The basic criteria in the 1981 Manual are still valid and are not repeated in this text.

Part II on overland flow supplements the 1981 design basis with information from operating municipal facilities, as well as additional results from research studies completed in the period December 1980 to March 1984. This information reinforces and strengthens the basis for this rapidly developing technology.

ACKNOWLEDGEMENTS FOR PART I

Preparation of Part I on rapid infiltration systems has involved the participation of a number of individuals. There were a team of authors, another group of individuals who deserve special recognition for their contributions and technical assistance, and a group of invited experts who reviewed the document and provided comments and suggestions for its improvement. Mr. Sherwood C. Reed of USACRREL was the principal author of Part I and coordinated the activities of the other authors and groups. Dr. James E. Smith Jr., EPA, CERI was Project Officer of this task.

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PART I. RAPID INFILTRATION

CHAPTER 1

INTRODUCTION

1.1 Background

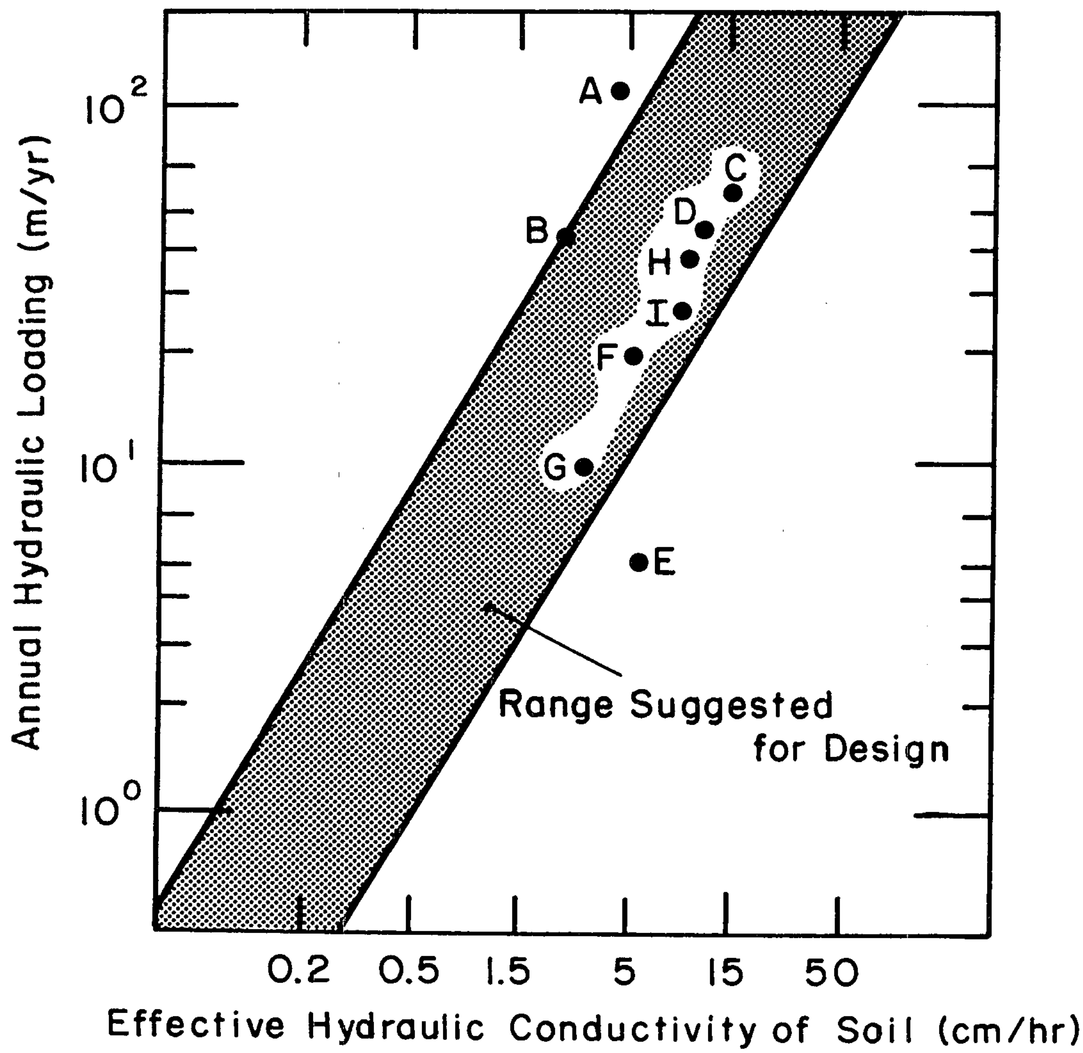
Rapid infiltration (RI) is a very successful and cost-effective method for wastewater management. RI systems can be designed for wastewater treatment; or as one component in a waste re-use plan involving recovery of the treated water and/or storage in an aquifer. Municipal RI systems have been in successful operation in the United States for up to 100 years. In 1981 there were about 320 municipal RI systems in the United States, either operating or under construction. Some 30 percent of these systems have been implemented since 1971.

The concept depends on a relatively high rate of wastewater infiltration into the soil followed by rapid percolation, either vertically or laterally, away from the application point. The best soils are relatively coarse textured, with moderate to rapid permeabilities. In practice, wastewater application rates have ranged from about 15 m/yr to 120 m/yr (50-400 ft/yr) for successful RI systems. For contrast, the typical loading on an agricultural system using wastewater might be 1 to 2 m/yr (3-6 ft/yr) and the maximum loading typically permitted for an on-site septic tank leachfield system is about 29 m/yr (95 ft/yr). Figure 1-1 compares the actual hydraulic loading at several successfully operating RI systems to the design range recommended by the 1981 Technology Transfer Process Design Manual [1].

The operational context for RI systems is typically earthen basins designed for a repetitive cycle of flooding, infiltration/percolation, and drying. The wet and dry periods designed for these cycles are a function of wastewater and soil characteristics, climatic conditions, and treatment goals. The percolate from cyclic RI systems is typically suited for indirect re-use for many purposes. If this percolate emerges in an adjacent surface water it should be of a high quality, equivalent to that obtained with complex and costly advanced waste treatment (AWT) systems.

1.2 Objective and Scope

The Manual [1] includes basic criteria and information on the design, construction and performance of municipal RI systems. A large number of successfully operating systems have been designed using the Manual [1] (and/or its first edition



Locations: A-Pilot Basins, Phoenix, AZ
B-Lake George, NY
C-Ft. Devens, MA
D-Boulder, CO
E-Hollister, CA
F-Corvallis, MT
G-East Glacier, MT
H-Jackson, WY
I-Eagle, ID

FIGURE 1-1
ACTUAL RI LOADING RATES VERSUS DESIGN REQUIREMENTS

TABLE 1-1
RAPID INFILTRATION - POTENTIAL PROBLEMS

Soils	Ground Water
• Layers or zones of less permeable soils not revealed during site investigation which impede water movement.	• Unexpected high seasonal ground water table which interferes with subsurface water movement.
• Field testing conducted at different location or different depth than the final system, so design may be based on inappropriate data.	• Inadequate capacity to move water away from the site laterally or vertically in the time allowed by design.
• Final surface layer in infiltration area contains significant clay or silt. These "fines" may segregate during flooding, resettle on the surface and impede future water movement.	• The subsurface flow from one basin influences the capacity of an adjacent basin.

Design Assumptions	Construction
• Less than design capacity for water movement because backfill operations have reduced soil permeability (see related note under Construction).	• Excess traffic and inadvertent compaction of final basin surfaces.
• Actual wastewater characteristics (algae, suspended solids) different than assumed.	• Failure to remove all of fine soils in surface layers or zones of unacceptable soils.
• Design based on improper use of criteria or inadequate infiltration measurements.	• Construction activity in the basin area when soil moisture is too high.
• Design ignores potential for freezing during winter operations in cold climates.	• Rainfall sorting of fines into layers of low permeability during fill operations.
	• Inadequate specifications for earthwork and infiltration surface preparation.

published in 1977). However, there are a few recently constructed systems, in a variety of locations in the United States, that are not meeting all design expectations, particularly with respect to the capability to infiltrate and then percolate wastewater at the design rates. A preliminary analysis of these systems indicated that the observed problems could be grouped as shown in Table 1-1.

The basic criteria in the Manual [1] are valid and if properly applied will result in successful performance. It is not the intent of this supplement to repeat all of that basic information. This supplement provides additional guidance and, where necessary, a stronger emphasis and more specific detail on critical aspects so that the problems listed in Table 1-1 can be avoided in future designs. In addition, this supplement presents information developed since 1981 on performance of RI systems with respect to nitrogen removal, organics removal, and the need for disinfection.

1.3 References

1. U. S. Environmental Protection Agency. Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater. EPA 625/1-81-013. U. S. EPA, Center for Environmental Research Information, Cincinnati, OH. October 1981.

CHAPTER 2

PLANNING AND SITE SELECTION

2.1 General

The basic procedures and criteria for planning and site selection are described in detail in Chapter 2 of the Manual [1] and can also be found in other sources [2, 3, 4, 5].

2.2 Preliminary Screening

The initial procedure to identify suitable sites can be a desk top analysis of existing information. Numerical rating procedures are described in references [1, 3, 4] and include consideration of soil and ground water conditions, grades, existing land use, and flood potential. However, the procedures in the Manual [1] do not take into account the economic factors related to pumping from the community to the RI site.

2.2.1 Land Area Requirements

At the planning stage it is necessary to make a preliminary estimate of the land area that will be required for the treatment portion of the RI system using Equation 2-1 (the basic equation is in the Manual [1], but is modified here for the operating period).

$$A = \frac{(1.9)(Q)}{(L)(P)} \qquad (2\text{-}1)$$

where: A = field area for treatment, ha
Q = design daily flow, m^3/d
L = annual design percolation rate, m/yr
P = period of operation, wk/yr

Since RI systems typically operate on a year-round basis, Equation 2-1 usually becomes:

$$A = \frac{(0.0365)(Q)}{(L)} \qquad (2\text{-}2)$$

Figure 2-3 in the Manual [1] relates the clean water permeability of the most restricting layer in the soil profile to the design wastewater percolation rate. That figure can be used to estimate the L value for use in equations 2-1 and 2-2 above. Figure 2-3 in the Manual [1] is only valid for these preliminary estimates and is not intended for final RI designs. The Manual [1] clearly states this precaution, but its mis-use for final designs has resulted in problems.

2.2.2 Economic Factors

Table 2-1 summarizes the economic rating factors [4] that can be used in conjunction with the technical factors already in Chapter 2 of the Manual [1] to evaluate the influence of distance and elevation on site suitability. Both pumping distance and elevation difference can have a significant influence on cost-effectiveness of site development. In general, a site within 10-12 km (6-7 mi) of the community may still be competitive with other alternatives.

TABLE 2-1
ECONOMIC RATING FACTORS FOR SITE SUITABILITY [4]

Characteristic	Rating Value
Distance from wastewater source, km	
0- 3	8
3- 8	6
8-16	3
> 16	2
Elevation difference, m	
< 0	6
0-15	5
15-60	3
> 60	1

km x 0.6214 = miles
m x 3.281 = ft

2.2.3 Regulatory Factors

Although the percolate from RI systems is of high quality, it may not in some cases satisfy all factors i state ground water quality requirements. In these situations, there are two options:

- Design for percolate recovery, via underdrains or wells, for subsequent re-use or surface discharge.
- Demonstrate via a hydrogeological survey and analysis that all of the percolate will emerge as base flow in an adjacent surface water or otherwise not adversely impact the *in situ* aquifer.

2.2.4 Site Characteristics

Special emphasis is necessary on site topography, and soil type and soil uniformity as indicated by the soils and topographic maps. Extensive cut and fill can significantly increase costs for RI construction. Thus, sites with significant and numerous changes in relief over a small area are not the best choice. As described in greater detail in Section 3.3.6, a significant clay fraction (>10%) in the soil would generally exclude RI construction on fill using such materials. Therefore, sites with that type of soil and a topography that would require fill construction are usually eliminated during the preliminary screening process. Extremely non-uniform soils over the site area do not absolutely preclude development for an RI system, but they significantly increase the cost and complexity of site investigation.

Most of the common field tests are only valid for a relatively small area or shallow depth. Where extreme soil variability is shown on the soils maps, it is generally best to consider other sites. The alternative may be to construct a large-scale pilot cell to define the hydraulic characteristics of the site. If the tests are successful, the pilot cell can then be incorporated into the full-scale system. The layout and construction of the remainder of the system then requires special care on the part of the designer and careful coordination with those responsible for construction.

2.3 Site Selection

It is costly to conduct the extensive field investigations required for a final design at multiple sites. It is appropriate to make a final screening and numerical rating of sites to verify feasibility for a single site, or to determine which of several sites should be the focus of detailed field investigations. Procedures discussed in Section 2.2 above can be used for this purpose.

2.4 References

1. U. S. Environmental Protection Agency. Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater. EPA 625/1-81-013. U. S. EPA, Center for Environmental Research Information, Cincinnati, OH. October 1981.

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CHAPTER 3

SITE INVESTIGATION

3.1 General

A basic requirement for all engineering designs is that the procedure will be used with valid data and information. For RI systems, that information comes from a properly conducted field investigation, coupled with the careful interpretation of the field data by the designer. Inadequacies in these two areas are the most common factors associated with problems in RI systems.

Problems that should not have arisen are listed on Table 1-1. These problems can be avoided, and it is the purpose of this section to provide additional guidance and detail to assist in RI site investigation. Several requirements deserve special emphasis:

- It is essential for the final field testing to be conducted on the actual site and at the actual depth in the soil profile intended for the RI system. A series of tests may, therefore, be required as the design is refined and the final basin configuration determined. <u>Extrapolation of data from nearby sites is not an acceptable basis for design.</u>

- Field investigation and testing can be expensive. Cost-effectiveness and reliable results are only insured if the investigative program is planned and conducted by persons familiar with soils and ground water testing, as well as having a complete understanding of the RI concept and the design expectations regarding water movement.

- Interpretation of field test results also requires competence in soils, hydrogeology, and a thorough understanding of the RI process. If suitable in-house expertise does not exist, outside assistance is necessary.

3.2 Site Evaluation

The first step in the site investigation involves confirmation of the feasibility of RI for the selected location. These procedures are in addition to the screening and selection guidance presented in Chapter 2. Several components are included in this more detailed phase of the evaluation:

1. Field examination of exposed soil profiles (on and near the site) to include road cuts, borrow pits, and plowed fields.

2. Field observation of ground water indicators: Wet spots, seepage areas, vegetation changes, ponds and streams, and general drainage characteristics such as standing water after rainfall.

3. Backhoe test pits, to a 3 m (10 ft) depth where soil conditions permit, in the major soil types on the site. Soil samples from critical layers (especially the layer being considered for the future basin surface) are collected and reserved for future testing. Ground water seepage into the pit is observed and the highest water level obtained recorded. Where ground water is encountered, a temporary water level observation tube {5 cm (2 in.) PVC pipe with diagonal hacksaw slots} can be installed as the pit is backfilled.

4. The water level in any on-site or adjacent wells is recorded, in addition to data from any observation tubes installed as part of step 3 above. The elevations for all of these observation points are obtained, as well as the elevations of adjacent surface waters, and a preliminary water table map prepared. The evaluation of these data includes consideration of potential seasonal high water levels.

5. The data obtained from steps 1-4 above allow preliminary definition of:

   a. General hydrological setting
   b. Soil descriptions and water table locations
   c. Proposed soil layer for RI basins
   d. Flow direction, depth, and discharge areas for ground water and the re-charge characteristics for the site
   e. Possible site modifications including fill or excavation, underdrains, or control of natural ground water flow

6. Evaluation of the data in step 5 results in one of three conclusions for the site:

   a. Site is suitable; proceed with further detailed field testing.

   b. Site may be suitable with modifications; proceed with additional field testing and analysis.

c. Site is unsuitable for RI due to factors not revealed in preliminary screening and site selection; no further testing or analysis is necessary.

7. Ground water samples are taken at a later stage of the detailed site investigation and analyzed for chemical and biological constituents to establish the background characteristics of the aquifer.

3.3 Soils Investigation

The soils investigation includes field and laboratory testing and observations based on borings and test pits. Table 3-1 summarizes the factors that are included in the field description of all soil samples obtained from test pits and borings.

TABLE 3-1
SOIL CHARACTERISTICS IN FIELD DESCRIPTION

	Characteristics	Significance
1.	Estimate of percent cobbles, gravel, sand, and fines	Influences permeability
2.	Plasticity of fines	Permeability and influence on cut or fill construction
3.	Major textural class	Permeability
4.	Soil color	Presence of minerals, indication of seasonal groundwater
5.	Wetness and consistency	Drainage characteristics
6.	Structural characteristics stratigraphy, and geologic origin	Ability to move water vertically and laterally.

3.3.1 Soil Texture

It is obvious from the list in Table 3-1 that some prior soils experience is essential for an effective and accurate field description of soil samples. Table 3-2 gives an indication of soil type and texture based on its feeling and appearance in the field.

A moderately coarse-textured soil is the best choice for RI. The field investigator still looks for layers or lenses in the soil profile that could restrict water movement. Fine-textured soils, and even sandy soils with a significant silt or clay content (>10%), are not desirable. Soils in this category tend to have relatively low in situ permeabilities. In addition, the remolding of clay during construction activities for either cuts or fills can reduce the permeability to an unacceptable level [3]. Figure 3-1, a typical density versus moisture content plot for a clayey soil, illustrates this loss of permeability.

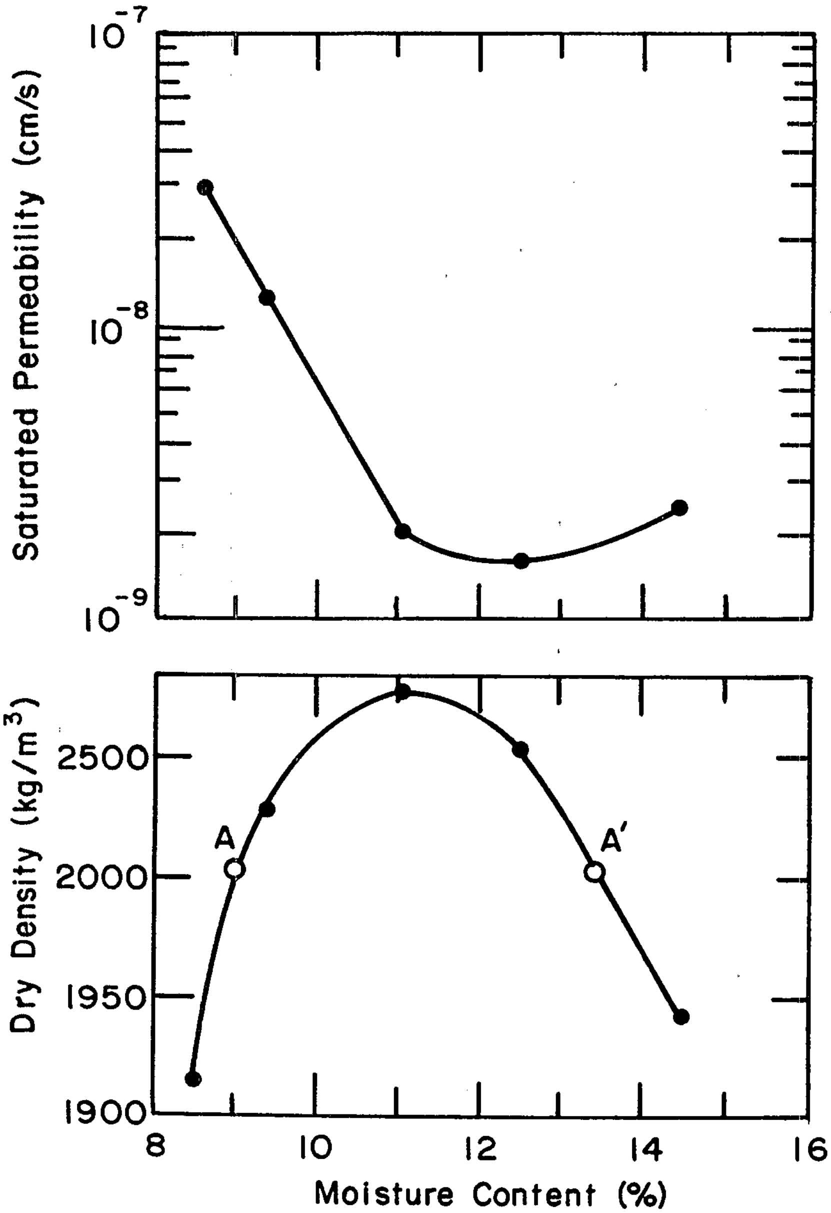
Point A on the lower curve might represent typical in situ characteristics in the soil profile. Point A' represents the possible results of construction activity when higher moisture contents might prevail. As shown by the figure, the soil density of A' is essentially the same as at point A, but the permeability has been reduced by an order of magnitude by the remolding of the clay. If the site topography requires placement of fill for infiltration surface construction, clayey soils are not recommended. Experience has shown that clayey sands and clays exceeding 10% were not successful when used in fill construction for the infiltration surfaces in RI basins [3]. Clayey material can also be a problem for basins entirely within cut sections and when used as fill material for dikes or embankments. These aspects are discussed in Chapters 4 and 5.

3.3.2 Soil Structure

Soil structure refers to the aggregation of soil particles into clusters of particles called peds. Well structured soils with large voids between peds will transmit water more rapidly than structureless soils of the same texture. Fine-textured soils which are well structured, with strong peds, can be used for RI systems. Field investigation of such sites require testing to insure that the soil structure will not be destroyed by application of wastewater. Structure is best observed and evaluated in the sidewalls of a test pit.

TABLE 3-2
TEXTURAL PROPERTIES OF MINERAL SOILS [2]

Soil Class	Feeling and Appearance	
	Dry Soil	Moist Soil
Sand	Loose, single grains which feel gritty. Squeezed in the hand, the soil mass falls apart when the pressure is released.	Squeezed in the hand, it forms a cast which crumbles when touched. Does not form a ribbon between thumb and forefinger.
Sandy Loam	Aggregates easily crushed; very faint velvety feeling initially, but with continued rubbing the gritty feeling of sand soon dominates.	Forms a cast which bears careful handling without breaking. Does not form a ribbon between thumb and forefinger.
Loam	Aggregates are crushed under moderate pressure. Clods can be quite firm. When pulverized, loam has velvety feel that becomes gritty with continued rubbing. Casts bear careful handling.	Cast can be handled quite freely without breaking. Very slight tendency to ribbon between thumb and forefinger. Rubbed surface is rough.
Silt Loam	Aggregates are firm, but may be crushed under moderate pressure. Clods are firm to hard. Smooth, flour-like feel dominates when soil is pulverized.	Cast can be freely handled without breaking. Slight tendency to ribbon between thumb and forefinger. Rubbed surface has a broken or rippled appearance.
Clay Loam	Very firm aggregates and hard clods that strongly resist crushing by hand. When pulverized, the soil takes on a somewhat gritty feeling due to the harshness of the very small aggregates which persist.	Case can bear much handling without breaking. Pinched between the thumb and forefinger, it forms a ribbon whose surface tends to feel slightly gritty when dampened and rubbed. Soil is plastic, sticky, and puddles easily.
Clay	Aggregates are hard. Clods are extremely hard and strongly resist crushing by hand. When pulverized, it has a grit-like texture due to the harshness of numerous very small aggregates which persist.	Casts can bear considerable handling without breaking. Forms a flexible ribbon between thumb and forefinger and retains its plasticity when elongated. Rubbed surface has a very smooth, satin feeling. Sticky when wet and easily puddled.



Point A: Density=2000 kg/m^3, Moisture=9%, k=2 x 10^{-8} cm/sec
Point A′: Density=2000 kg/m^3, Moisture=13.5%, k=2 x 10^{-9} cm/sec

FIGURE 3-1
MOISTURE CONTENT/COMPACTION VERUS PERMEABILITY FOR A CLAY SOIL (3)

3.3.3 Soil Color

The color and color patterns in a soil profile are a good indicator of the drainage characteristics of the soil [4]. Reds, yellows, and yellow browns are representative of well oxidized conditions; an indication of well-aerated, non-saturated soils. More subdued shades and grays and blues predominate if insufficient oxygen is present when saturated conditions prevail. Imperfectly drained or seasonally saturated soils show alternate streaks or pockets of oxidized and reduced soil elements. This condition is termed soil mottling. The observation of mottled soils in the test pit walls is evidence of saturated conditions at that level at some time in the past.

Some color photographs of soil mottling are included in reference [2]. Experience is still required for accurate interpretation since some mottling may have occurred in geological time and may have no reference to the current status of the soil profile, or mineral or organic stains may be mistaken for mottles. However, any mottles are suspect and should be taken as a preliminary indicator of seasonal high ground water conditions.

3.3.4 Field Procedures

Field procedures for soils investigations include borings and test pits. The practical depth for most test pits is about 3 m (10 ft), so borings are required for deeper exploration and for confirmation in the areas between test pits. Samples are obtained from borings for soil identification and undisturbed samplings when necessary for physical and mechanical testing.

3.3.4.1 Test Borings

Locating test borings in the areas of the site that are actually to be used for RI basin construction is essential. At least one boring is recommended in every major soil type on the site. With generally uniform conditions, one boring for every 1 to 2 ha (2-5 ac) for continuous areas of up to 20 ha (50 ac) would suffice for large scale systems. Small scale systems (<5 ha) should consider 4-6 shallow borings spaced over the entire site. All borings typically penetrate to below the water table if it is within 10 to 15 m (30-50 ft) of the ground surface. A few borings should extend all the way through the saturated zone, if possible, to define the thickness of the aquifer for use in subsurface flow calculations. The drilling methods most commonly used are hollow stem augers and wet rotary drills. Other methods include jetting and churn drilling. Soil samples are obtained from all borings using a 5 cm (2 in.) split-spoon or similar device. Recommended sampling intervals are:

- Borings for monitoring wells; spoon samples at 1.5 m (5 ft) intervals

- Borings for soils investigations; spoon samples, 2 each per 1.5 m (5 ft), down to 6 m (20 ft) then one per 1.5 m (5 ft) thereafter.

The blow counts to drive the sampling spoon are recorded for each sample. The count for the final 30 cm (12 in.) is termed the standard penetration test and is used to evaluate soil density. Visual soil descriptions of each sample are made in the field and samples preserved in plastic bags for chemical and mineralogical analysis. If a dry drilling technique is used, the position of the water table is observed prior to closing the hole. Where soils are relatively uniform in depth, it is possible to obtain spoon samples and undisturbed samples alternately from the same boring. When the profile changes significantly with depth, a second boring, within 3 m (10 ft) of the first, is recommended for undisturbed samples to insure sample continunity. One set of undisturbed samples for every 10 ha (25 ac) of RI basin area is usually sufficient.

An 8 cm (3 in.) thin wall Shelby tube is pushed at least 30 cm (12 in.) into each major soil unit identified for undisturbed samples. At least one tube per boring is recommended if the soil is uniform (isotropic and continuous) and a maximum of one tube every 1.5 m (5 ft) above the water table. The ends of the tube are sealed and transported in an upright position to the laboratory for analysis. It may not be possible to obtain tube samples from gravelly sands, and loose, dry, coarse sands. In these cases, a test pit will be necessary.

3.3.4.2 Test Pits

Test pits are strongly recommended for all RI site investigations, since, unlike borings, they permit the direct observation of a relatively large portion of the subsurface profile. Two or three test pits would be considered the minimum on even the smallest site. The various soil layers can be visually identified and the presence of fractured near-surface rock, hardpan, mottling, layers or lens of gravel or clay, or other anomalies can be recorded. If the pit extends into the ground water, it can also be used for in situ measurement of hydraulic conductivity (see Section 3.5).

Soil texture is described for each layer using the procedures described in Section 3.3.1. Soil structure can be examined using a pick, or similar device, to expose the natural cleavages and planes of weakness in the profile. Color and mottling can also be observed in the pit wall. One end of the backhoe pit can be excavated on a gentle incline. Following identification of the soil layers, benches can be excavated by hand at appropriate levels to expose undisturbed soils for _in situ_ testing. These tests include in-place dry density and field or oven-dried moisture content and, as described in Section 3.5, may include infiltration testing. Tests are made at the depth proposed for the RI basin surface and at least once again, 1 m (3 ft) deeper if the soil is uniform. Samples from each layer are preserved in plastic bags for laboratory analysis and a bag sample of about 40 kg (90 lb) of soil at the proposed infiltration surface collected for possible re-compacted permeability testing in the laboratory.

3.3.5 Laboratory Procedures

Physical, chemical, and mineralogical tests in the laboratory are conducted on the split-spoon and test pit samples described in the previous section. The minimum that should be tested consists of the soil unit that is planned for the final RI basin infiltration surface and the soil unit immediately beneath it. Composite samples can be assembled from the same soil unit in different borings or from samples of the same soil unit, if encountered at different depths. One set of chemical and mineralogical data is obtained for each of the soil units tested. That would be a minimum of two chemical and two mineralogical tests (if needed) for each 10 ha (25 ac) of area intended for use as RI basins.

The physical testing of these samples include gradation and related procedures to determine the relative percentage of sand, silt, and clay. The chemical tests may include: percent organic matter, phosphorus, iron, manganese, potassium, magnesium, calcium and sodium (exchangeable), base saturation, pH, cation exchange capacity, and electrical conductivity (EC).

Mineralogical tests are only necessary when the physical testing indicates the presence of clay in the soil sample. These tests are performed on the less-than-two-micron size fraction to identify the clay minerals. The analysis is typically done by x-ray diffraction.

Additional testing for phosphorus adsorption capacity may be necessary in special situations. Phosphorus may be a parameter of concern for RI in those cases where the hydrogeological analysis shows that percolate will emerge in an adjacent surface water which has stringent phosphorus limitations. The Manual [1] (Section 5.4.2.3) contains procedures for estimating phosphorus removal at RI systems. The design equation in the Manual [1] is based on absolutely-worst-case conditions and, as such, is unduly conservative. The rate constant is the lowest possible value, based on a neutral pH. Most soils are either slightly acidic or slightly alkaline, and would have a higher rate. The equation is based on saturated flow conditions and, therefore, predicts the shortest possible residence time for flow from the basin to the ground water table. The actual residence time through this unsaturated zone will be much longer and, therefore, P removal will be more effective than predicted by the equation.

The design example in the Manual [1] is also too conservative with respect to phosphorus removal during lateral flow. The example is based on a most conservative (but impossible) assumption that the hydraulic gradient is equal to 1 for lateral flow, which again results in the shortest possible residence time. For example, if a 10% gradient is assumed, the 0.2 mg/L P concentration could be achieved in a lateral distance of 20 m (65 ft) instead of the 200 m (650 ft) indicated by the Manual [1] (p.5-21). For the conditions specified in that example, a travel distance of 36 m (118 ft) would be required for the phosphorus concentration to approach "background" levels (assumed at 0.015 mg/L for this case).

If the calculated concentration at the distance of concern exceeds specified limits, phosphorus adsorption tests are performed on the soils through which the percolate will flow. Section 3.7.2 in the Manual [1] describes these tests; other useful information can be found in references [6, 7, 8, 9]. It is recommended that the results of the laboratory adsorption tests be multiplied by a factor of 5 to account for the slow precipitation of phosphorus that occurs over time in the soil profile [10].

The undisturbed soil samples obtained with the Shelby tubes are tested for dry unit weight, moisture content, and textural gradation. Then, assuming a specific gravity of 2.69 (valid for most soils) the porosity, void ratio, and degree of saturation can be calculated using standard procedures [4, 11]. The textural gradations are plotted on the standard semilogarithmic paper, and the effective size and uniformity coefficients derived [4, 11]. The same calculations can also be performed with results from field density measurements in the test pits.

If needed, laboratory permeability tests are conducted using the undisturbed samples. The basic procedures are described in Soil Science Society of America Journal 46(4):866-880 and ASTM Standards.

The results of laboratory permeability tests are plotted versus void ratio on semilogarithmic paper as shown in Figure 3-2. A design permeability for each hydraulic unit is selected near the lowest end of the void ratio range, as shown on the example Figure 3-2 and Table 3-3.

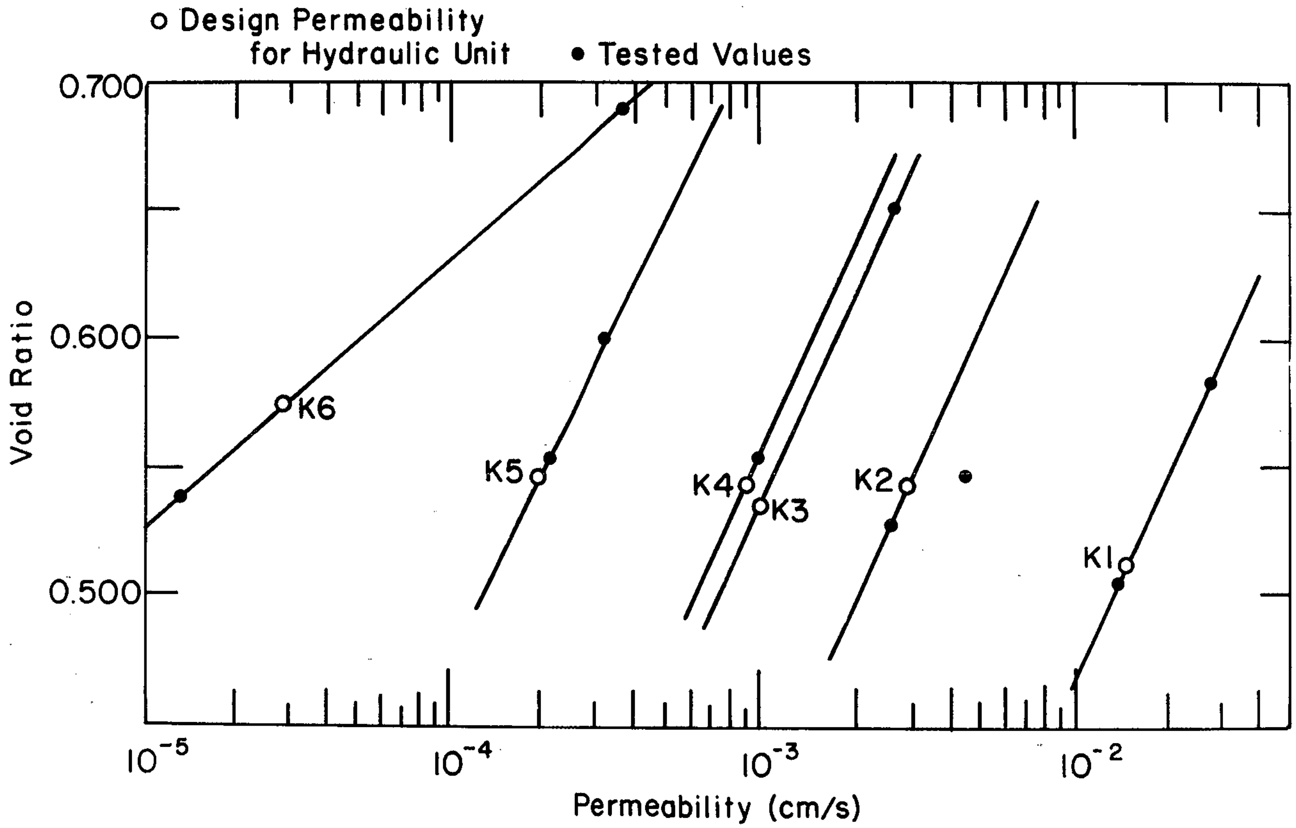


FIGURE 3-2
PERMEABILITY VS. VOID RATIO

TABLE 3-3
VERTICAL SATURATED PERMEABILITIES FROM FIGURE 3-2*

Hydraulic Unit	Permeability cm/sec	in./h	Void Ratio Range	Other Physical Properties
K_1	1.5×10^{-2}	21.26	0.46-0.62	Medium to coarse sand, loose, D_{10} 0.2-0.3
K_2	3.0×10^{-3}	4.25	0.47-0.66	Fine to medium sand, loose, D_{10} 0.1-0.1, less than 9% fines
K_3	1.0×10^{-3}	1.42	0.48-0.67	Fine to medium sand, loose, D_{10} 0.1-0.2, greater than 9% fines
K_4	9.0×10^{-4}	1.27	0.49-0.68	Fine to medium sand, dense, D_{10} 0.1-0.2
K_5	2.0×10^{-4}	0.283	0.49-0.69	Silty sand, D_{10} less than 0.1
K_6	3.0×10^{-5}	0.043	0.52-0.70	Clayey sand

*These are not general factors, but are only valid for the example in Figure 3-2

D_{10} is the 10% soil fraction

3.3.6 Evaluation of Soil Test Results

The results of all tests are reviewed and evaluated by an experienced engineer, geologist, or soil scientist. This is particularly important for the chemical and mineralogical data at sites where construction or operation may alter soil reactions or mineral structure (chemical exchange and leaching, cementation, swelling, etc.).

If test results indicate that a high percentage of the clay is montmorillonite, the relatively high cation exchange capacity may result in infiltration problems caused by swelling, even if the total amount of clay present is small. Any combination of either vermiculities or montmorillonites can be a problem, if the total clay content exceeds 10%. Soils with greater than 10% clay are increasingly subject to physical changes and chemical leaching, and may require soil amendments for use as RI application basins in a cut section. As previously indicated, soils with that much clay are not recommended as fill material for infiltration areas.

The results of the field observations and the laboratory tests for physical characteristics are combined and plotted on a site map and on several cross-sections through the site to provide a basis for final design decisions regarding basin location, water movement, etc. Figure 3-3 illustrates a typical profile.

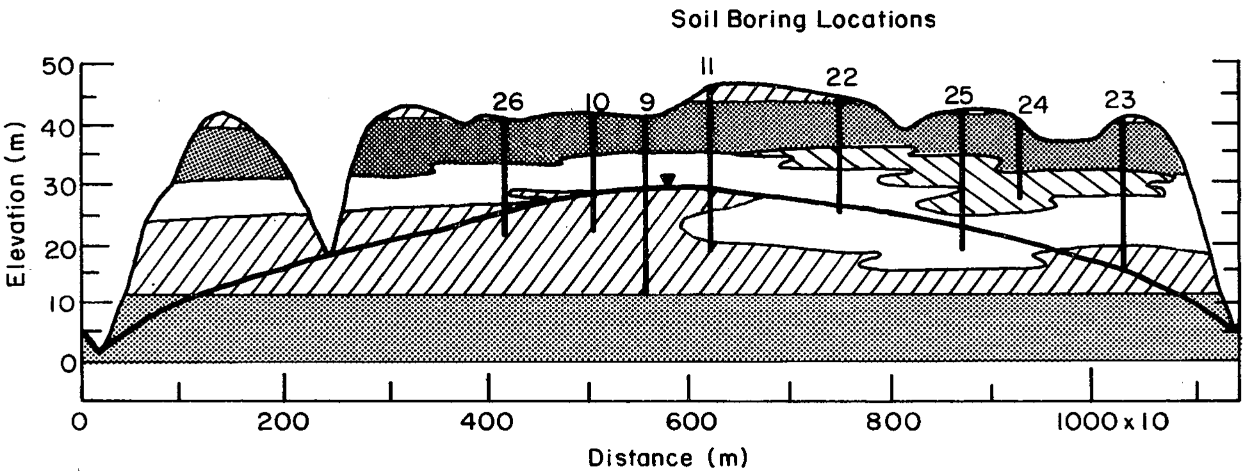


FIGURE 3-3
TYPICAL SUBSURFACE PROFILE

3.4 Ground Water Investigations

Ground water investigations are necessary to determine the depth to the ground water table, the direction of subsurface flow, and, in some cases, to determine water quality parameters for design purposes. The rate of ground water flow is discussed in Section 3.5. Installation of at least three wells is recommended during the site investigation. If the preliminary study has identified the general ground water flow direction and a tentative location for the RI basins, then one well should be up-gradient, one in the basin area, and one down-gradient on the ground water flow path near the planned system boundary. These wells can also be used later for operational water quality monitoring. If little is known about the area, the three wells are usually spaced in a triangle over the potential site. One well per 6-8 ha (15-20 ac) is usually sufficient for sites up to 40 ha (100 ac). The well bottom is usually between 3 and 10 m (10-30 ft) below the water table. The lower 1.5 to 2 m (5-6 ft) of the well consists of machine slotted well screen. A 5 cm (2 in.) inner diameter well screen with 0.25 mm (0.01 in.) slots is usually satisfactory. The bottom of the hole contains a medium to coarse sand-pack to about 1 m (3 ft) above the well screen with a bentonite seal above the sand pack. Figure 3-13 in reference [2] illustrates a typical shallow monitoring well.

The elevation of the top of each well is determined and the depth to ground water in each well periodically observed and plotted on maps and on profiles similar to Figure 3-3. In general, the ground water table tends to follow surface contours and the flow trends toward adjacent surface waters. This may assist in determining location of the monitoring wells.

It is strongly recommended that monitoring wells be installed prior to the normal "wet" period for the site area and observed during this period to detect seasonal high ground water. The use of test pits for detection of soil mottling (see Section 3.3.3 in this text) is also recommended.

3.5 Infiltration and Permeability Tests

Definition of infiltration capacity and subsurface permeability is essential for RI system design. The infiltration rate is defined as the rate at which water enters the soil from the surface. When the soil profile is saturated, with negligible ponding above the surface, the infiltration rate is comparable to the effective saturated hydraulic conductivity of the soil profile within the zone of influence for the tests. In both this text and in the Manual [1] the terms hydraulic conductivity and permeabilty are used interchangeably. Definition of both the vertical (K_v) and horizontal (K_h)

permeabilities in the soil profile are necessary for RI system design.

3.5.1 Infiltration Test-Procedures and Evaluation

A number of methods have been developed to measure infiltration rate or vertical hydraulic conductivity (K_v) in the field. A comparison of the various methods can be found in Table 3-2 in the Manual [1]. It is the intent of each of these methods to define essentially the same parameters, but their reliability varies according to individual test conditions.

3.5.1.1 Flooding Basin Tests

A flooding basin test, with the test area at least 7 m^2 (75 ft^2) is strongly recommended for all RI systems. Testing the larger area provides more reliable data than the smaller-scale cylinder infiltrometers or air entry permeameters (AEP). A much larger volume of water is obviously required for this test. The basic procedure is described in detail in Section 3.4.1 of the Manual [1]. For most cases a tensiometer at 15 cm (6 in.) and another at 30 cm (12 in.) are sufficient in lieu of the extensive instrumentation described in the Manual [1]. The use of picks and shovels (in lieu of special tools), and a bentonite seal around the perimeter are also suggested. The test basin is flooded several times to calibrate the instrumentation and to insure saturated conditions. The actual test run is conducted within 24 hours of the preliminary trials. For most soils with RI potential, this final test run may require 3 to 8 hours.

Since it is the basic purpose of the test to define the hydraulic conductivity of the near surface soil layers, the use of clean water (with about the same ionic composition as the expected wastewater) is acceptable in most cases. There are three major exceptions when the test utilizes actual or simulated wastewater and/or extends for a longer period.

1. If the wastewater expected in the actual RI system will have a high solids content, similar liquid is needed in the test. High solids concentrations might come from algae carryover from ponds or from specific industrial or commercial components in the wastewater (whey wastes, pulp and paper, food processing, etc.). Such solids clog the infiltration surface and clean water tests would be a misleading basis for design. In addition, the test also extends for a sufficient period (perhaps several weeks) to model the actual wet and dry cycles proposed for operation of these RI systems. In these situations, an appropriate number of standard flooding basin tests could be run with clean water to define the basic characteristics of the

site. Assuming that generally uniform conditions prevail, one longer term test can then be run to define the influence of unique wastewater types on infiltration. The subsurface flow characteristics are still controlled by the clean water K values in most situations.

2. If the system is to operate in the continuously flooded "seepage pond" mode, it is best for a test to simulate that condition for a sufficiently long period (perhaps several months) to insure that wastewater will percolate at expected rates. <u>The RI design procedures in the Manual [1] were not intended for permanently flooded seepage pond designs.</u> The wet and dry periods described in the Manual [1] are intended to restore the infiltration capacity of the surface soils to near maximum potential every cycle through bio-oxidation of the filtered organics. If the infiltration surface is continuously flooded, a different situation prevails and some other design approach is necessary and percolate water quality expectations modified.

3. If the borings and related site investigation reveal a heterogeneous mixture of soils, a large scale RI cell {0.5 to 3 ha (1 to 7 ac)} is constructed and operated as a pilot RI unit for development of design criteria. If test results are appropriate, the pilot unit can be incorporated into the full scale system.

A minimum of one basin infiltration test on each major soil type in the RI area is recommended. For large continuous areas, one test for up to 10 ha (25 ac) of usable land is typically sufficient. <u>The test is performed on the soil layer intended for use as the basin infiltration surface in the final operational system.</u> Such a requirement seems self evident, but it's disregard in the past has resulted in serious problems.

<u>Construction of RI basins on backfilled material is to be avoided whenever possible.</u> When construction on fill is absolutely necessary due to site topography, and if the soils are within acceptable limits for clay content, a basin infiltration test in a test fill area is recommended. The test fill is constructed using the same equipment and procedures that would be used for full scale construction. The test fill should be as deep as required by the site design or 1.5 m (5 ft), whichever is less. The top of the fill area should be at least 5 m (15 ft) wide and 5 m (15 ft) long to permit a standard flooding basin test near the center. This approach was developed, and used successfully to evaluate potential RI sites in Maryland [12].

3.5.1.2 Air-Entry Permeameter (AEP)

This device was developed by Dr. Herman Bouwer [13] to measure "point" hydraulic conductivity values in the absence of a water table. It has been used in site investigations for a number of land treatment systems and is described in Section 3.5.2 of the Manual [1]. The test is useful to verify site conditions in the areas between the larger scale flooding basin tests, and also for <u>in situ</u> tests in the end wall of a test pit as described in Section 3.3.4.2 of this text. An AEP unit, used in this manner can quickly determine the hydraulic conductivity of all of the soil layers in the profile. The device is not commercially available, but specifications and fabrication details can be obtained from:

U. S. Department of Agriculture
Water Conservation Laboratory
4332 East Broadway
Phoenix, AZ 85040

3.5.2 Permeability Tests

Definition of both vertical (K_v) and horizontal (K_h) permeability is necessary for RI design. The vertical component K_v can be inferred from the field tests described in the previous section or measured in the laboratory with undisturbed soil samples. <u>Field results are the primary basis for design in all cases.</u> Laboratory data on undisturbed samples are valuable for confirmation of test basin results and for interpolation for areas between the field tests. Laboratory values from deeper soil borings also provide data for ground water flow and mounding analysis.

In most soils, K_h will exceed K_v due to soil stratification and particle orientation. The relationship for some soils is given in Table 3-5 in the Manual [1]. The most conservative approach would be to assume that K_h equals K_v. The direct measurement of K_h in the field is suggested for any RI project where there is a concern regarding ground water mounding. A number of procedures are discussed in the Manual [1]. The auger hole test is recommended and complete details on procedure and calculations can be found in several references [1, 10, 13, 14]. A more recent development when the ground water is within 3 m (10 ft) of the surface using a test pit is described in references [15, 16].

3.6 References

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CHAPTER 4

DESIGN

4.1 General

The basic design procedures in the Manual [1] are still valid, so this Chapter only provides additional detail and a stronger emphasis on certain critical aspects. Significant advances have been made in computer sciences in recent years and most designers now have access to at least a microcomputer. Information on the continuing development of computer models to solve some of the problems encountered in RI system design can be obtained from:

Holcomb Research Institute
International Ground Water Modeling Center
Butler University
Indianapolis, Indiana 46208
317/283-9555

Determining the design annual hydraulic loading rate is one of the most critical aspects of RI system design. The operational cycle (wet/dry periods) is another important factor and is determined independently. Combining the annual loading and the wet/dry cycle then determines the unit wastewater application rate. Details on these factors are presented in this chapter. Some of the discussion may seem self-evident or overly simplistic, but a review of problem systems indicates that these factors and their inter-relationships may not always have been clearly understood in the past.

4.2 Hydraulic Loading Rate and Basin Area

The hydraulic loading rate is based directly upon the field and laboratory test results for infiltration, permeability, and hydraulic conductivity. Figure 2-3 in the Manual [1] indicates the general relationship between these factors. <u>That figure is not intended for final design of RI systems.</u>

If the site investigation reveals a specific layer of soil that will restrict flow, the design is based on the hydraulic conductivity of that layer regardless of its thickness. In many cases with a near surface deposit of silts or clays, it may be cost-effective to remove the restricting layer and locate the infiltration surfaces in the underlying soils. If there is not an obvious restricting layer, the "effective" hydraulic conductivity of the profile is the mean (arithmetic, harmonic, or geometric) of the values observed (see Section 3.5 in the Manual [1] for procedures). A small percentage of this "effective" hydraulic conductivity is then taken as the design loading rate. The percentages used are based on historically successful RI performance and allow for wastewater characteristics, soil reactions, and the need for cyclic operation. A further refinement was added in the Manual [1] to allow for a higher design percentage for the most reliable field test procedures. If a very large pilot scale cell is constructed and actual wastewater is used and applied for several months, then test results could be used directly for design with little or no modification. However, most of the commonly used test procedures require the adjustments (safety factors) summarized in Table 4-1.

TABLE 4-1
DESIGN FACTORS FOR HYDRAULIC LOADING

Test Procedure	Adjustment Factor for Annual Loading Rate
Basin flooding test	10-15% of "effective" rate observed
Air entry permeameter and cylinder infiltrometers	2- 4% of "effective" rate observed
Laboratory permeability measurements	4-10% of "effective" rate observed, or of restricting soil layer.

The hydraulic conductivity defines the amount of clean water that can move through a unit cross-section in the soil, at unit gradient and under saturated conditions. For example, a soil with an "effective" vertical conductivity of 5 cm/hr (2 in/hr) could transmit 438 m^3/yr (116,000 gal/yr) of water through every 1 m^2 (11 ft^2) of horizontal area:

K_v = 5 cm/hr

hydraulic gradient = 1, because of saturated vertical flow

Clean Water Rate, L_{cw}

$$L_{cw} = \frac{(5\ cm/h)(24\ h/d)(365\ d/yr)(1\ m^2)}{(100\ cm/m)}$$

$$= 438\ m^3/yr \quad (10{,}800\ gal/yr\ ft^2)$$

The loading can also be expressed in terms of a depth of water on a unit area because of the dimensions involved, so:

$$L_{cw} = \frac{438\ m^3/yr}{1\ m^2} = 438\ m/yr\ (1{,}437\ ft/yr)$$

An annual basis is used for the loading rate determination, since it is assumed that wastewater will be applied on some sort of regular schedule throughout the year, although the specific wet/dry cycle has not been determined at this point in the calculations.

Assuming for the example above, that basin flooding tests were used to determine K_v on a site with deep and uniform soils, it would be appropriate to adopt a factor of 10% from Table 4-1, to determine the design loading rate:

Annual Wastewater Loading, L_{ww}

$$L_{ww} = (0.10)(\text{clean water rate, } L_{cw})$$
$$= (0.10)(438)$$
$$= 44\ m/yr\ (144\ ft/yr)$$

It is, of course, possible to divide this annual loading by some other time unit and produce the apparent "average" weekly or daily loading. In earlier texts this was done to provide a basis for comparison with the loading rates for other land treatment concepts. <u>The actual unit application rate is not derived in this manner.</u> Confusion on this point has led to problems. The 10%, or other safety factors, are only used to determine the annual hydraulic loading. It is not appropriate to adjust either K_v or K_h with these factors for shorter time periods. Such adjustments <u>do not</u> define the unit rates for infiltration or subsurface flow (see Section 4.2.4 and 4.3).

Determination of the annual loading rate is in effect a definition of the capacity of the site to transmit wastewater if applied at some undefined, but regular schedule throughout the year. Most RI systems <u>do</u> operate on a year-round basis. However, if some winter storage is required by the regulatory authorities or there are seasonal constraints on operation, the annual loading is proportionally reduced to account for the non-operating period. System operation planned for nine months per year for the sample case would reduce the "annual" loading proportionately:

$$L_{ww} = (\frac{9}{12})(44 \text{ m/yr}) = 33 \text{ m/yr } (108 \text{ ft/yr}).$$

In some cases the hydraulic loading is limited by the capability to move water away from the application site due to shallow confining layers or a small hydraulic gradient (see Section 4.3.1 in this text and Section 5.7.1 in the Manual [1]).

4.2.1 Land Area Required

The required application area for RI systems can be determined with equations 2-1 and 2-2. The area calculated with these equations is the surface area that is needed for infiltration in the final system. There have been cases where the earthwork design has allowed encroachment for dike construction on this area. The completed system in these cases has significantly less than the required infiltration area.

4.2.2 Wet/Dry Ratio

A regular drying period is essential for the successful performance of RI systems. The period required for drying is a function of the solids and degradable organics in the wastewater and of the climatic influences on aerobic reactions. The ratio of loading to drying periods within a single cycle for successful RI systems varies, but is almost always less than 1. For primary effluent, the ratios are generally less than 0.2 to allow for adequate drying. The ratio varies for secondary effluents in relation to the treatment objective. Where maximum hydraulic loading and/or nitirification are the objective, the ratio might be 0.2 or less. When nitrogen removal is necessary (see Section 4.8.2) the ratio ranges from 0.5 up to 1.0 [2]. Table 5-13 in the Manual [1] presents suggested hydraulic loading/drying periods, related to wastewater type, treatment goal, and climatic season. In all cases, to avoid excessive soil clogging, the hydraulic loading period for primary, or similar, effluents does not exceed 1-2 days regardless of season or treatment goals. As discussed in Section 4.2.4, additional time may be needed for all of the wastewater to infiltrate.

4.2.3 Application Rate

The selected wet/dry cycle is combined with the "annual" hydraulic loading to determine the unit application rate. The procedure is more complex when different wet/dry cycles are selected for summer and winter operations; an example is given below:

Assume: design flow = 800 m^3/d; soil K_v = 5 cm/hr, use 9% adjustment factor; so: annual hydraulic loading = 39 m/yr (128 ft/yr) treatment goals with primary effluent are to maximize loading rates, so, from Table 5-13 in the Manual [1]:

	Wet (d)	Dry (d)	Total (d)
Summer period (April - Oct, 214 d)	2	7	9
Winter period (Nov - March, 151 d)	2	12	14

Summer period, each cycle = 9 d

$$\text{Cycles per season} = \frac{214}{9} = 24 \text{ cycles}$$

Winter period, each cycle = 14 d

$$\text{Cycles per season} = \frac{151}{14} = 11 \text{ cycles}$$

Total cycles per year = 35

Assuming the wastewater has similar characteristics all year, the amount applied per cycle is the same and the summer/winter drying periods relied on for restoring the infiltration rate.

If the wastewater has special seasonal characteristics (e.g., high solids for seasonal industries, etc.) it may be necessary to allow for extra drying time (or reduced loading) during those special periods.

Wastewater loading per cycle

$$= \frac{39 \text{ (m/yr)}}{35 \text{ (cycles/yr)}}$$

$$= 1.1 \text{ m/cycle (3.6 ft/cycle)}$$

Application rate (R) during 2 day wet period

$$R = \frac{1.1 \text{ m/cycle}}{2 \text{ d/cycle}} = 0.56 \text{ m/d (1.8 ft/d)}$$

Since in this example the wastewater flow is constant year-round, and no storage is provided, a greater land area will be needed during the winter months to maintain the same loading rate. Using a variation of equation 2-1:

Summer period,

$$A = \frac{(800\ m^3/d)(214\ d)}{(1.1\ m/cycle)(24\ cycles)(10{,}000\ m^2/ha)} = 0.65\ ha\ (1.6\ ac)$$

Winter period,

$$A = \frac{(800\ m^3/d)(151\ d)}{(1.1\ m/cycle)(11\ cycles)(10{,}000\ m^2/ha)} = 1.0\ ha\ (2.5\ ac)$$

The winter period would control the design in this case. The total area needed is divided into multiple basins. Table 5-14 in the Manual [1] suggests the number of basins required for various wet/dry ratios.

For this example, use 7 basins

$$\text{basin area} = \frac{(1.0\ ha)(10{,}000 m^2/ha)}{7}$$

$$= 1428\ m^2\ (15{,}370\ ft^2)$$

could use 38 m x 38 m square basins (125 x 125 ft)

During the summer months, four of the basins could be operated on an 8-day cycle, or five basins on a 10-day cycle to approximate the 9-day wet/dry, summer cycle recommended by the design. This would allow each one of the seven basins to be taken out of service for maintenance for an extended period each summer.

The unit application rate (R) derived for this example was 0.56 m/d (1.8 ft/d). That is compared to the effective steady state infiltration rate for soil (K_v) to insure successful performance. As a general "rule-of-thumb" the unit application rate is usually less than 50% of the K_v to allow for initial clogging by wastewater solids and organics, if optimization of hydraulic loading is the design goal. Even if the application rate (R) is higher, ponding will not typically commence until the later part of short flooding cycles. In either case, the depth of water remaining in the basin at the end of 1-2 day flooding periods will probably not exceed 0.3 m. The depth of ponding increases as the flooding period is extended and/or the concentration of solids in the wastewater increases.

4.2.4 Infiltration of Standing Water

The design approach is based on the assumption that all of the applied water will infiltrate during the very early part of the drying cycle so that most of that period is available for aerobic restoration of the upper soil profile. This might require from 0.5 to 2 days, depending upon the organic content and volume of the dose. When the restoration of aerobic conditions takes too long, it may be necessary to extend the drying period, perform maintenance on the basin bottoms, or in an extreme case, perhaps remove and replace a given depth of the surface soils.

It may occasionally be necessary to estimate either the depth of water remaining in the basins at some time after flooding has stopped or the time necessary to reduce the ponded depth to zero. There are several alternate approaches to this problem, depending upon whether a clogging layer has formed on the surface or not. If surface clogging does not exist, the equation proposed by Stefan [3] is applicable.

$$h = h_o - 2.22\ (1-n)^{0.35}(n)^{325}\ K_v t^{0.675} \qquad (4\text{-}1)$$

where:

h = depth of water in basin after time t, cm

h_o = depth at end of flooding period (at t=0),cm

n = soil porosity (decimal fraction)(see Table 4-2)

K_v = saturated vertical hydraulic conductivity, cm/h

t = time, h

In the application of this equation, it is important to remember that the properties n and K_v are values which apply to the wetting front, hence they will be affected by the presence of microbial growth in this (near surface) zone and are not equal to the values which existed before wastewater applications began. Typical <u>initial</u> porosity ranges for various soil textural classes are given in Table 4-2.

TABLE 4-2
POROSITY OF SELECTED SOILS [4]

Soil Type	Porosity (%)
Silt and Clay	50-60
Fine sand	40-50
Medium sand	35-40
Coarse sand	25-35
Gravel	20-30
Sand and gravel mix	10-30
Coarse glacial till	25-45

Based upon the studies of deVries [26], both porosity and hydraulic conductivity can be expected to decrease to the range of 30 to 60 percent of their initial values upon the establishment of biological growth in the near surface soil profile, the larger decreases corresponding to finer textured soils. Using n and K_v values equal to 50% of the original values will probably give reasonable results.

The basin is empty when h = 0. Substitution and rearrangement of equation 4-1 allows the estimation of time required to drain the basin.

$$t_d \simeq \frac{h_o}{K_v}\left(\frac{0.3}{(1-n)^{0.5}\ (n)^{0.5}}\right) \qquad (4-2)$$

where: t_d = time to drain basin, h

all other terms defined above.

For example, assuming a medium sand with a K_v = 8 cm/h (3 in./h) and a porosity of 35%, estimate the drainage time if 40 cm of effluent remain in the basin at the end of the flooding period.

Use 50% reductions, so

$$K_v = (0.5)(8) = 4 \text{ cm/h}$$

$$n = (0.5)(0.36) = 0.18$$

$$t_d = \frac{40}{4}\left(\frac{0.3}{0.82^{0.5}\ 0.18^{0.5}}\right) = 7.8 \text{ h}$$

In the presence of a surface clogging layer (the usual case), equations 4-1 and 4-2 are not applicable as they depend upon a continuously downward moving saturated front which never occurs under a thin surface layer of higher hydraulic resistance than the soil beneath. An equation which roughly estimates the decrease in water depth with time is:

$$\ln\left(\frac{h+d}{h_0+d}\right) = -\frac{k_{vc}t}{d} \tag{4-3}$$

where: d = depth of clogged surface layer, cm

K_{vc} = hydraulic conductivity of clogging layer, cm/h

all other terms are previously defined.

The parameters d and K_{vc} are obviously functions of both wastewater suspended solids and operating conditions. As such, they are never known with certainty. There are several published studies which might be used to deduce probable values of these parameters [26, 27, 28]. Of these, the work of deVries [26] appears to simulate closely the operating conditions of interest. From this study it appears that K_{vc} will probably be about 0.6 to 1.0 cm/h (1.5-2.5 in./h) and d will probably be about 2 to 2.5 cm (0.8-1.0 in.). Note that when surface clogging controls, the properties of the upper soil profile do not enter into equation 4-3 in any way.

The procedures outlined above are particularly useful in conjunction with thermal calculations for RI systems in northern climates where winter freezing and ice formation may cause problems (see Section 4.5 for details).

4.3 Subsurface Flow and Ground Water Mounding

A proper design insures that the subsurface soils have the capacity to transmit the applied wastewater down and away from the basins at an acceptable rate to avoid failure. Ground water mounding beneath the basin occurs when there is insufficient gradient to move water away from beneath the basin in a lateral direction. Some temporary mounding is acceptable as long as it does not interfere with infiltration at the basin surface and dissipates quickly enough to allow for aerobic restoration of the near surface profile. Chapter 5 in the Manual [1] discusses these factors in detail and presents a graphical procedure for estimating mounding beneath an RI basin. Supplemental information and criteria are included in Section 4.3.3 in this text.

4.3.1 Subsurface Flow

Percolate flow in the unsaturated zone beneath an RI basin is essentially vertical and K_v controls flow. If a ground water table, impeding layer, or barrier exists at depth, a horizontal component is introduced and flow is then controlled by some combination of K_h and K_v within the ground water mound or perched mound. At the margins of the ground water mound, and beyond, the flow is typically lateral and K_h controls. Section 3.5.2 in this text, Chapter 3 in the Manual [1], and similar sources [3, 4] discuss K_h and K_v and techniques for their measurements.

The capability for lateral flow away from the application site controls the extent of mounding that will occur beneath the basins. The "space" available for lateral flow is the underlying aquifer and the zone between the existing ground water table or other barrier and whatever point is selected as an acceptable depth beneath the ground surface. The final basin locations and configurations should be plotted on the soils/ground water maps and an analysis conducted to insure that the adjacent profile has the capacity for lateral transmissions of the applied wastewater. The first zone of concern is the perimeter around the general basin area, since this may directly influence mounding. The analysis must consider the gradients and potential for flow in various directions away from the site, in addition to the loading schedules for various basins (see Section 4.3.2) to estimate what proportion of the applied wastewater will flow in a particular direction. The general analysis employs potential flow theory and is beyond the scope of this text. The use of this theory to construct flow lines in the presence of a uniform flow field of constant thickness represents a reasonable approximation to what takes place down-gradient of the mound. The calculations, if they are required by an approving agency, are described in the text by Bear [29], or the computer model by Daly [44].

In many cases the RI percolate emerges as base flow in adjacent surface water and it is necessary to predict the position of the ground water table between the RI basins and that point of emergence. This will reveal if seeps or springs are likely to develop in the intervening terrain. In addition, some state agencies may specify a residence time for the percolate in the soil to protect adjacent surface waters, so it may also be necessary to calculate the travel time from the basin to the surface water. Equation 4-4 can be used to estimate the saturated thickness of the water table at any point down-gradient of the basin area [5]. This value can be converted to an elevation and plotted on maps and profiles to identify potential problem areas.

$$h = \sqrt{(h_o)^2 - \frac{2Q_iD}{K_h}} \qquad (4\text{-}4)$$

where: h = saturated thickness of the unconfined aquifer at the point of concern, m

h_o = saturated thickness of the unconfined aquifer beneath the rapid infiltration area, m

Q_i = the lateral discharge from the unconfined flow system, per unit width of the flow system, m^3/d m

D = the lateral distance from the RI area to the point of concern, m

K_h = effective horizontal conductivity of the soil system, m/d (must have consistent units)

The quantity Q_i in equation 4-4 is determined with:

$$Q_i = \frac{K_h}{2D_i}(h_o^2 - h_i^2) \qquad (4\text{-}5)$$

where: D_i = distance to seepage face or outlet point, m

h_i = saturated thickness of the unconfined aquifer at point D_i

other terms defined above

Figures 5-10 and 5-11 in the Manual [1] and/or other references [6, 30, 31, 32] can also be used to estimate the saturated thickness of the water table at a point down-gradient of the basin area. The answers from all of these models are meaningless unless the soil and hydraulic parameters and boundary conditions of the site match reasonably well with those of the model selected.

The travel time for lateral flow is a function of the hydraulic gradient, the distance, the K_h, and the porosity of the soil. Equation 4-6 can be used for this purpose.

$$t_D = \frac{(n)(D)^2}{K_h(h_o - h_i)} \qquad (4\text{-}6)$$

where: t_D = travel time for lateral flow from basin area to emergence in surface waters, d

n = porosity of soils, % (as decimal; calculated from field data, see Table 4-2 this section for ranges)

D = travel distance, m

h_o = saturated thickness of aquifer at RI basin area, m

h_i = saturated thickness of aquifer at point of emergence

K_h = effective horizontal conductivity of the soil system, m/d (units must be consistent)

4.3.2 Ground Water Mounding

The material presented in the Manual [1] on mound height analysis was that developed by Glover [33] and summarized by Bianchi and Muckel [34]. The Manual [1] did not state nor imply that this simplified graphical analysis would solve all problems relating to ground water mounds. The curves were developed for basins of square or rectangular geometry, which lay above level, fairly thick, homogeneous, aquifers of (practically) infinite extent. Vertical re-charge was assumed uniform (or nearly so) and the height of the mound was limited to less than one-half the thickness of the original unconfined aquifer. Many potential RI sites will not conform well to these conditions and a certain amount of ingenuity is required on the part of the engineer who is analyzing such sites. In the simplest cases the original curves may be used with modifications. For example, a circular basin can be approximated by a square one of equal area, allowing the use of Figures 5-8 and 5-10 without introducing significant error [33]. Also, as the curves were developed using unsteady state equations, they necessarily show an ever-increasing mound height with time. It is obvious that during prolonged resting periods (e.g., during annual maintenance), the mound recedes to a certain extent. The figures are still applicable, but the principle of superposition in time must be employed. If dosing is stopped at time t, a uniform discharge (from the mound back to the basin) is assumed beginning at t, thus effectively cancelling the re-charge. The algebraic sum of the two mound heights in time approximates the mound shape after re-charge ends. More details on the analysis of mound decay can be found in references [35, 36, 37].

Other conditions encountered with reasonable frequency which may invalidate the use of the curves in the Manual [1] include the presence of sloped water tables [37], subsurface layers of reduced conductivity which give rise to perched mounds [38], and the presence of points of withdrawal (e.g., a stream [39] or a significant rise in mound height relative to the original saturated depth [37, 39]). Although there are analytical solutions and/or dimensionless plots from generalized solutions which satisfy some of these more complex boundary conditions, the designer would probably be better advised to use a computer for these complications, especially if the mound height analysis is thought to be critical to the success of the project. There are a number of existing programs available to perform these calculations, and improvements are continuously forthcoming. Since most engineering offices have access to at least a microcomputer, and some to larger systems, two applicable programs are discussed here and sources given.

An interactive program based upon Glover's analysis [33] has been written for the APPLE II+, 48K microcomputer [40]. In addition to calculating the mound height ordinates with time and distance (which also solves the problem discussed in Section 4.3.2) under continuous re-charge, the program can also solve the mound decay problem and can handle the boundary condition of re-charge to a stream. To do this an "image basin," discharging at the same rate as re-charge from the RI basin, is assumed to be located on the opposite side of the stream, equidistance from the real basin. The technique borrows from the theory of well hydraulics using image wells and ensures that the (mathematical) mound intersects the stream at its water level, a necessary physical condition.

The program can also calculate the hydrograph of stream re-charge, a computation occassionally required by regulatory agencies where water quality limiting stream segments are involved. The authors have designed the program to be "user-friendly," allowing several options, creating unambiguous displays, and allowing easy variation in parameters and variables. A floppy disk containing the program and all documentation can be purchased from the authors at:

Department of Civil Engineering
Colorado State University
Fort Collins, CO 80523

If more versatility than the above program can provide is required, or more diverse boundary conditions are encountered, another interactive program is available which seems to be admirably suited. It was developed by Dr. S. P. Neuman [41] and adapted for interactive mode on a FDP-11/23 minicomputer (256K) by Bloomsburg and Rinker [42]. The program called UNSAT, can be purchased from:

Dr. G. L. Bloomsburg
469 Paradise Drive
Moscow, ID 83843
208/882-5726

This program can treat non-uniform flow regions having irregular boundaries, arbitrary degrees of local anisotropy, evapotranspiration, and percolate recovery by fully- or partially-penetrating wells. Significant to some more difficult analyses, certain time-dependent boundary conditions can be handled, such as varying infiltration rates or stream levels. The major drawback to the use of this program is the necessity of knowing (or estimating), for each soil unit in the flow system, the following relationship:

1. Saturated hydraulic conductivity

2. Relative conductivity ($0 \le K_r \le 1$) as a function of moisture content (see Section 3.3.4 in the Manual)

3. Porosity

4. Pressure head as a function of moisture content

5. Initial values of pressure head (or moisture content) at each point in the system

6. Conditions to be imposed at each boundary

These requirements are not as intimidating as they may seem, because at steady state, the soil beneath a RI system is very nearly at constant moisture content; hence, a constant K_r value and pressure head value may be assumed without too much loss of accuracy. This is a very powerful model, but obviously should not be used unless one is relatively confortable with the principles of unsaturated flow and armed with more field data than is usually the case. Default functions for both relative conductivity and pressure head versus moisture relationships are available for several soil textural classifications [43] when field data are lacking. It should be apparent that where simpler models do not appear adequate, something at this level of sophistication can be considered.

4.4 RI Basin Configuration and Application Scheduling

When the availability of suitable land for an RI system is limited and preliminary mounding calculations indicate a problem, there is no alternative to the use of subsurface drainage to control the rise of the water table (see sections 5.7.2 to 5.7.4 in the Manual [1]). This requires underdrainage and a discharge permit unless all the recovered percolate is re-used. However, when land is not a limiting factor, it may often be possible to avoid underdrainage by "optimizing" the configuration and application scheduling of the RI basin system; for example, by arranging the basins in a strip configuration and staggering basin operation so that no two adjacent basins are ever dosed sequentially. This was discussed in Section 5.6.1 of the Manual [1]. Further improvements are possible if the basins are spread out on the site, although this loses the cost advantage of common dikes.

The mound height analysis, if performed correctly, usually points to the best approach. By way of illustration, assume a system of seven RI basins arranged as shown in Figure 4-1 in a hydrogologic setting as shown in Figure 4-2.

The average loading is 1,727 m^3/d (60,984 ft^3/d) on a bottom area of 2.83 ha (7 acres), an average loading of 0.061 m/d (0.2 ft/d). The horizontal hydraulic conductivity is 6.1 m/d (20 ft/d) and the specific yield is 0.20. It is planned to operate the basins on one day of dosing and six days rest using the sequence 1, 4, 7, 2, 6, 2, 5. Will mounding interfere with the operation of this system? One possible method of analysis is to assume that the intermittent loading to each basin is equivalent to a continuous dose, averaged over the entire system. Correcting for dike area, this loading is about 0.055 m/d (0.18 ft/d). Selecting one year (365 d) as a trial value of t and using Figure 4-9 in the Manual [1] with L/W = 2 and W = 132 m (432 ft.).

$$R = \frac{i}{V} = \frac{0.055}{0.2} = 0.275$$

$$= \frac{KD}{V} = \frac{(6.1)(15.2)}{0.2} = 465$$

$$\frac{W}{\sqrt{4\ t}} = \frac{132}{\sqrt{(4)(465)(365)}} = 0.16$$

$$h_o = (0.08)(0.275)(365) = 8.3 \text{ m } (26 \text{ ft})$$

Referring to Figure 4-2, this is clearly an unacceptable mound since it would have intersected the basin surfaces before 365 days had elapsed.

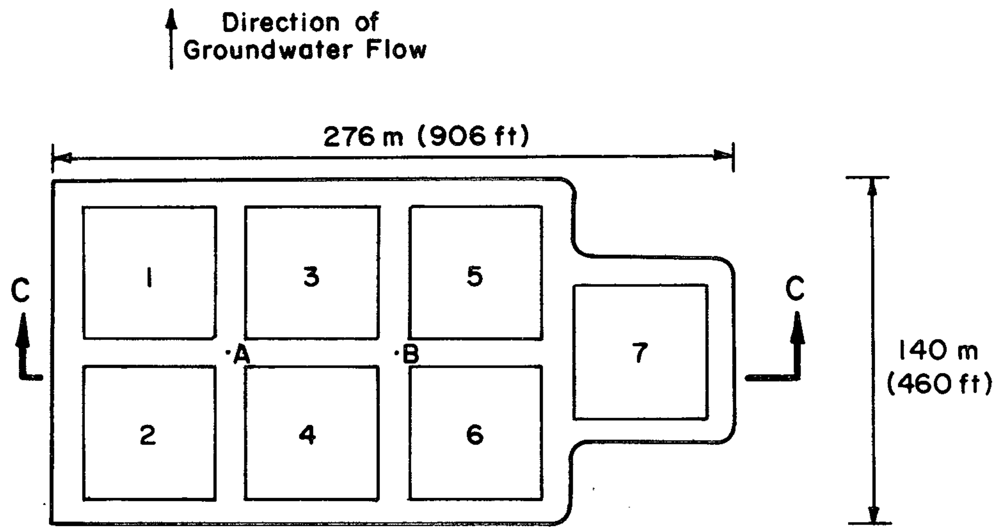

FIGURE 4-1
BASIN LAYOUT FOR EXAMPLE CALCULATION

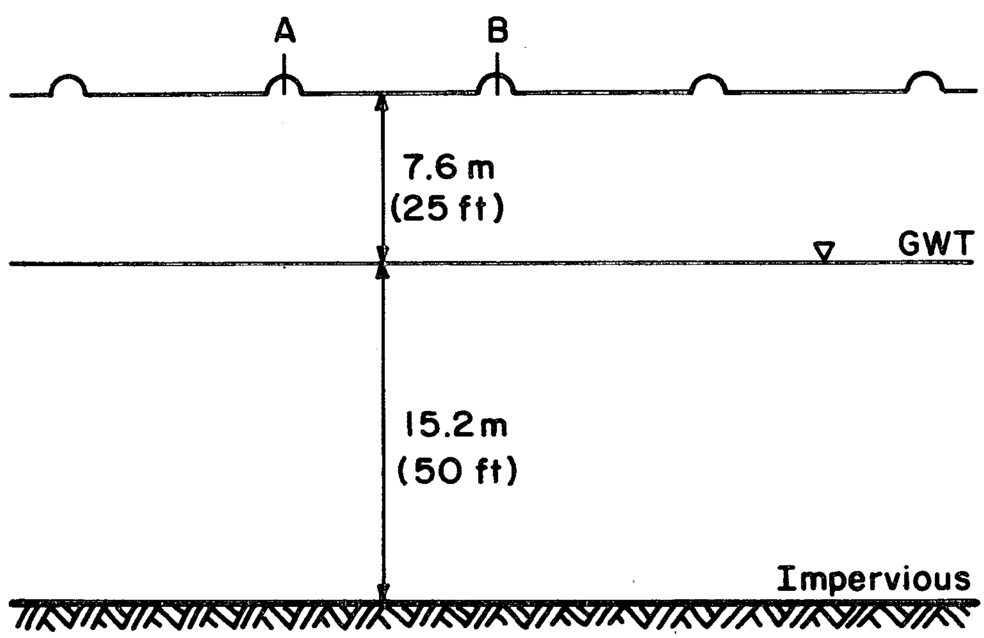

FIGURE 4-2
SUBSURFACE PROFILE SECTION C-C FOR EXAMPLE CALCULATION

However, the previous analysis does not reflect the actual operation. Each basin is loaded at 0.43 m/d (1.4 ft/d) for one day, then allowed to drain and rest for six days. At worst, only basins with touching corners (not sides) are dosed on consecutive days, thus, minimizing mound interference. An analysis based upon a single basin shows the maximum mound height is 1.60 m (5.3 ft) at the end of one day, but then recedes to less than 0.09 m (0.30 ft) by the end of the seventh day (using the principle of superposition in time discussed in Section 4.3.2). Using the principles of superposition in time and space together, the time history of mound heights at a few critical points within the space together, the time history of mound heights at a few critical points within the system; for example, points A or B can be calculated. Such a plot shows that mounding is not a serious problem with this basin arrangement and application scheduling. If the analysis indicated that mounding would likely be a problem, the design could be changed slightly to alleviate it. For example, the dikes could be made a little wider and/or the basins could be changed from square to rectangular (long axis perpendicular to ground water flow direction), or arranged in a strip.

4.5 Design Requirements for Winter Operation in Cold Climates

Rapid infiltration systems can operate successfully on a year-round basis in cold climates. Systems in Massachusetts, New York, Michigan, Wisconsin, South Dakota, Montana, and Idaho have all operated through the winter without significant difficulty. Proper thermal protection for the pipes, pump stations, valves and related plumbing is essential for wintertime operation. This is covered in other references [7]. The unique concern for RI is prevention of a permanent, impermeable ice barrier at, or in, the near surface soils in the basin. Two possible problems can develop:

- The standing water in the basin freezes into a layer of ice that, because of entrapped vegetation, cannot float to the surface. If the next increments of applied wastewater are not warm enough to melt through the ice layer, the basin can be in the failure mode for the rest of the winter.

- If at the end of the infiltration period, the soils drain too slowly, the water remaining in the soil pores may freeze, rendering the surface impermeable. If this frozen, saturated soil layer is not thawed, the basin is in the failure mode.

The successfully operating systems in northern climates tend to have relatively coarse, rapidly drained soils and operate with dilute raw sewage or relatively warm wastewater from conventional primary or secondary treatment systems. Suggested operating procedures to deal with some of these problems are discussed in Chapter 6. Design features which have been successful include:

1. Ridge and furrow configuration on the basin bottom combined with a floating ice sheet. The ice gives thermal protection, and rests on the ridge tops in the final infiltration stage.

2. Inducing snow drifting with snow fences in the basins and then flooding beneath the snow cover. The snow retards freezing, but the water equivalent of the melted snow is a negligible contribution to the hydraulic load in the basin.

3. By-pass some of the final cells in a long detention facultative lagoon to retain some of the heat originally present in the wastewater.

4. Design one or more of the RI cells for winter operation in the seepage pond mode with continuous flooding and a floating ice cover. These cells can be drained, dried, and restored each spring. Any special nitrogen limits or other water quality limits must be considered, since removal efficiencies are reduced at low temperatures.

5. Based on thermal and hydraulic calculations, adjust the wet/dry ratio during the critical periods so the near surface soil never irreversibly freezes (see discussion below and reference [8] for details).

Wastewater storage for winter conditions is not included in any of the alternatives listed above. Storage may be necessary to equalize flow variations, and storage may be provided for emergencies, but winter storage is not an absolutely necessary component in RI system design for the contiguous United States. A cost-effective design attempts to understand and adjust for the low temperature conditions rather than increasing costs by including long-term storage.

Storage may be needed during the winter in cold climate areas, however, if high nitrogen removal is a system requirement.

Ice starts to form as soon as the surface layer of water reaches $0^{\circ}C$ ($32^{\circ}F$) and the latent heat is withdrawn. The ice thickness that will develop can be predicted with equation 4-7, which is valid for ponds and for the standing water in a wet/dry RI operation.

$$I = C\sqrt{(\Delta T)(t)} \tag{4-7}$$

where: I = ice thickness, cm

C = coefficient, depending on environmental conditions

= 1.7 for snow cover on basin or snow on top of ice

= 3.0 for no snow.
(see reference 7 for other conditions)

$\Delta T = (0^{\circ}C - T_A)$

T_A = average air temperature over period t, $^{\circ}C$

t = time, days (this is the time, in wet/dry RI operations after the water at the surface reaches $0^{\circ}C$ ($32^{\circ}F$)

The factor $(\Delta T)(t)$ in equation 4-7 is called the freezing index and is an environmental characteristic for a particular location. It can be calculated from weather records or typical values can also found in reference [7] and similar sources. The design calculations are based on the coldest winter of record in 20 years, or even longer if data are available. Equations 4-2 or 4-3 and 4-7 can be solved for winter operating systems to determine how long it takes for infiltration to be complete and if any ice will form. Any ice formed has to be melted by the next increment of wastewater. The energy input required is given by equation 4-8.

$$E_m = \frac{(H_L)(I)(A)}{100} \tag{4-8}$$

where: E_m = energy required to melt the ice, cal

H_L = latent heat of ice, 80×10^6 cal/m^3

I = ice thickness, cm

A = surface area of ice, m^2

The energy available in the next increment of wastewater is dependent on the incoming liquid temperature. Only a fraction of the heat released in cooling the incoming water to $0^{\circ}C$ ($32^{\circ}F$) is available for ice melting, since there will also be significant heat losses to the atmosphere. The situation where both the soil and the water in the soil pores freeze is a more complex, two-phase problem and reference [8] should be consulted.

A design based on a floating ice layer is recommended for critical situations. Even under wet/dry operations when the ice is resting on the basin bottom, it provides insulation and retards freezing of the soil. Absence of the ice layer would require a more frequent flooding cycle to prevent soil freezing. Stefan [8] shows that the presence of a floating ice layer protects the soil from freezing for a period of over 27 days as compared to 2.5 days for freezing to commence in an unprotected soil under the same conditions. It is essential that the ice layer re-float at every flooding cycle.

4.6 Design of Seepage Ponds

The Manual [1] presents criteria for RI design that are based on the assumption that the system will be operated on a wet/dry cycle so that the near surface soils can be aerobically restored. <u>Those criteria were not intended for basins or infiltration ponds expected to be permanently flooded.</u> It should be understood that the percolate from seepage ponds will not be comparable in quality to a properly managed RI system designed for a regular wet/dry cycle. Seepage ponds in suitable soils can infiltrate large volumes of water where water quality is not an issue. Specific criteria for their design is not included in this text.

4.7 Design of Physical Elements

Most of the physical elements for RI basins are similar to wastewater lagoons. These include dikes, access ramps, inlet and transfer structures, and flow control devices. Chapter 4 in reference [11] provides details on some of these features, as well as Section 5.9 in the Manual [1].

4.7.1 Dikes

The dikes of RI basins intended for wet/dry cyclic operation do not need to be more than 1 m (3 ft) high and could be even less in many cases. Extra freeboard is not recommended for routine wastewater containment. If a basin does not infiltrate at the expected rate, restoration of the surface is necessary and extra freeboard in that basin does not solve the problem. Dikes that are higher than necessary can actually contribute to operating problems through the extra runoff and potential for erosion of soil fines. Some of the basins in a system, however, can be designed for emergency storage and these dikes can be higher. In these cases the dike includes sufficient freeboard and incorporates wave protection at the water line zone. All basins designed for flooding for more than a few weeks duration might include an outlet or other positive drainage features so the basin can be drained quickly when maintenance is necessary. This is because the time period required for natural drainage through the basin bottom may be unacceptably long.

The dikes are compacted to prevent seepage through them. The top of the dike often is a road for vehicle access and, if possible, should be pitched to the outside so that all drainage and runoff goes away from the basin. Erosion control for the inner dike surface is necessary, both during construction and during system operation. The washout of soil fines from this source has sealed the basin surfaces in a number of systems. Grass, or similar vegetation, may eventually stabilize the slope, but the use of a silt fence or similar porous barrier at the toe of the slope is suggested during construction and may still be necessary during operation. It is also necessary to provide a ramp or other easy access for maintenance equipment to enter the basins.

4.7.2 Basins in Fine-Textured Soils

If the soils in the basin bottom contain a significant fraction of fines (silts and/or clay >10%), then stabilization with grass may be necessary [12]. Flooding the bare surface of such soils may suspend some of the fine soil fraction in the water. The repetitive sorting and re-deposition of those fines on the surface can eventually reduce the infiltration rate significantly. A surface layer of gravel or similar material may hold the fines in place, but the interface may not dry properly and a clogging, biological layer may develop. Other mixed-in amendments (sand, gravel, lime, woodchips) have not been effective [12]. A grass cover is the only effective treatment for this condition known to date. These RI systems tend to be near the low end of the range of hydraulic loading and the grass cover may actually improve the infiltration capacity of the basin surface.

4.7.3 Inlet, Distribution and Transfer Structure

Small basins with low velocity, low volume wastewater flow may only require a simple splash block at the discharge pipe. Large basins may require a more complex arrrangement to dampen the entering flow to prevent erosion of the basin and/or the adjacent dike. In all cases, uniform wastewater application over the entire basin surface is necessary. This may automatically occur with a high application rate on finer textured soils, but may require some structural assistance on coarse soils to insure uniform distribution. The distribution system might range from a network of pipes and troughs to sprinklers in the extreme case.

In many cases, several basins or sets of basins may be flooded at the same time so interbasin transfer structures may be necessary. In general, these are similar to those used for lagoons [11]. Figure 5-4 in the Manual [1] illustrates a transfer structure with removable rings to control the depth of water in a basin. A device of this type gives the operator greater control and permits optimization of basin use.

4.7.4 Flow Control

Depending on the size of the system, the design can provide for a control system ranging from simple manual valves to a fully automated operation involving time switches, float switches, tensiometers, and automatic valves. In either case, it is necessary for regular visits to all of the basins by the operator to observe conditions and make necessary adjustments.

It is essential that the design provide sufficient flexibility so that adjustments in loading and application rate for a particular basin can be made by the operator. The adjustable transfer structure discussed above is one example. <u>It is also absolutely essential that particular attention be paid to the preparation of the O & M Manual for the system.</u> It is critical for the operator to understand clearly the RI concept, what controls and adjustments are available, and what the consequences of these adjustments might be.

4.8 RI System Performance

Basic information on the performance of RI systems can be found in Chapters 1, 2, and 5 of the Manual [1] and other sources [2, 4, 14, 20, 24, 25]. The percolate after a few meters of travel in the soil is of such a quality that recovery and re-use for unrestricted agricultural irrigation is acceptable. The system can be managed to satisfy nitrogen and other drinking water limitations in the percolate, but the direct on-site recovery and re-use as potable water is not recommended without extensive further treatment.

There has been additional research and study on the removal of toxic organics and nitrogen in RI systems since publication of the Manual [1] and the need for disinfection is still a concern for many. Further details on these three topics are given below.

4.8.1 Removal of Toxic Organics

The discussion on organics in the Manual [1] was limited to removal efficiencies for pesticides and BOD. The confidence in BOD removal efficiency is based on a reasonably large historical data base. A few researchers have since monitored specific organic chemicals in rapid infiltration systems [13, 14, 15, 16, 17] and have demonstrated significant reductions in many toxic organic chemicals. Analysis of organic chemicals is expensive and the analytical techniques are relatively new, resulting in a limited data base for specific compounds identified in municipal wastewaters. Therefore, "rule-of-thumb" removal efficiencies are not available and are not expected to be available in the foreseeable future. For this reason, the engineer needs to forecast the removal efficiency based on an understanding of the processes in a RI system.

A model has been developed [16] which separates a RI system into three computation compartments involving the standing water, the surficial soils, and the deeper soil layers. The model was developed in the laboratory, but not tested on a full scale RI system.

An interactive computer aided design code has been written in Fortran 77 along with a user's guide. The code can run on most 64K byte microcomputers having Fortran 77 compilers. Copies of the information are available in Simulation Waste Access to Groundwater: A User's Guide [19] which can be obtained from the International Groundwater Modeling Center, Butler University, Indianapolis, Indiana, 46208.

4.8.2 Nitrogen Management in Rapid Infiltration Systems

The discussion in Sections 5.2.2 and 5.4.3.1 in the Manual [1] describes the nitrification/denitrification mechanisms in rapid-infiltration systems for nitrogen removal. The importance of temperature, pH, and the necessity of organic carbon (as an energy source for the denitrifying bacteria) is discussed. Equation 5-1 estimates the amount of total nitrogen that may be removed based on the total organic carbon (TOC) in the applied wastewater. Nitrogen removal versus infiltration rate is shown in Figure 5-2 and hydraulic loading rates versus percent nitrogen removal is provided in Table 5-2 (all in the Manual [1]). Research on RI systems has combined new and former design and operaion techniques for improved nitrogen control. If high nitrates are not objectionable, the design develops the maximum hydraulic loading potential. However, if the objective is to maximize total nitrogen removal, maximizing hydraulic loading is sacrificed to design for optimum denitrification.

In systems designed for maximum hydraulic loading, high nitrate concentrations will be experienced in the percolate near the beginning of each loading period. These short pulses of high nitrate-nitrogen may peak up to an order of magnitude greater than the average. The pulse is a result of nitrification of the ammonium stored in the soil profile during the previous loading period. This high nitrate water is released as a slug when the capillary water is displaced during the next loading period. Four to five days drying has been shown to be an adequate time to provide ample soil re-oxygenation for nitrification of this applied ammonium. After the high nitrate pulse, the nitrate-nitrogen concentration drops to a much lower value (see Section 5.2.2 of the Manual [1]). The average total nitrogen concentration, including the high peaks, will likely range between 9 and 20 mg/L.

When the objective of design is to maximize total nitrogen removal, a more detailed procedure is used. Loading schedules directly affect ammonium adsorption during wetting, while drying schedules affect oxygen mass transfer and diffusion into soil. To achieve complete nitrification of the ammonium nitrogen, the amount of ammonium nitrogen applied during loading is balanced against the amount of oxygen entering the soil profile during drying. Otherwise, ammonium nitrogen accumulates in the soil until its adsorption capacity for ammonium nitrogen is saturated, and then ammonium nitrogen, as well as nitrate-nitrogen, appears in the reclaimed water.

Loading periods must be long enough to maximize ammonium adsorption and develop an anoxic environment to allow denitirification. The organic carbon source provided must be sufficient to support adequate denitrification. The small amount of available carbon in secondary effluent can be a limiting factor. Drying periods must be long enough to allow near ultimate soil re-oxygenation. This period is determined by the infiltration rate and the moisture release capability of the soil at the site. The design procedure is summarized in these six steps:

1. Determine the mass of ammonium that can be stored in the soil profile per unit area for the unsaturated depth using the published equations [21] to calculate the ammonium adsorption ratio and exchangeable ammonium percentage.

2. Calculate the length of the loading period required for maximum ammonium adsorption, using the infiltration measurement described in Section 5.4 of the Manual [1] and the known ammonium concentration of the applied effluent.

3. Estimate the mass flow of oxygen and the mass of diffused oxygen that can be accumulated in the soil profile for a specific drying period, using the published technique [23].

4. Balance the ammonium adsorption with the available oxygen to establish the length of loading and drying periods for optimum nitirification of the system.

Practical lengths of loading and drying periods might also be considered to fit the operation and maintenance schedule of the municipal system operators; i.e., five to nine days loading and two to five days drying, respectively.

5. Balance nitrate-nitrogen produced against the mass of organic carbon entering the soil.

6. Optimize denitrification by reducing the infiltration rate in the flooded basin.

Using these six steps, the designer can expect total nitrogen removal in excess of 60% and the discharge of total nitrogen to be below 10 mg/L, for typical municipal wastewaters during warm weather.

The infiltration rate can be fine-tuned to enhance denitrification by operating at reduced depth of ponding or by adding suspended solids. However, this will result in additional sacrifice of loading and will increase the land areas required. Selecting a site which naturally has low infiltration rates or compacting the soil surface can assure high total nitrogen removal rates, but either increases the land area needed. Reducing the infiltration rate, increases contact time between the soil micro-organisms, and nitrate-nitrogen, and more efficient denitrification can be accomplished. One of the most important factors for producing denitrification is to have near-saturated conditions in the soil media [20].

Thus, optimum total nitrogen removal in land treatment systems can be achieved by adjusting the loading period to insure complete nitrification of the ammonium nitrogen in the sewage and adjusting the infiltration rate to provide the needed level of denitrification. The use of primary effluent allows maximum total nitrogen removal at higher infiltration rates due to its higher organic carbon content. Nitrification reaction rates in the soil will be significantly reduced in the winter in cold climate areas. Winter storage, similar in duration to that described in Part II of this supplement, is necessary for those RI systems designed for maximum nitrogen removal.

4.8.3 Disinfection in RI Systems

The need for disinfection and the effectiveness of virus and pathogen removal in RI systems is discussed in the Manual [1] (Chapter 5) and in more recent texts [2, 24, 25]. Each one of these references discusses numerous projects, studies, and evaluations. In general, they show that RI system percolate can be of very high quality with respect to viral and bacterial content. There have been some studies where some viruses were detected in the percolate when high virus concentrations were present in the wastewater, rainfall during drying periods was heavy, and the wastewater was applied at high rates to very coarse soils. There has never been any evidence of any water-related disease problem related to the operation of any land treatment system in the United States.

Chlorination of wastewater can reduce the concentration of bacteria and virus significantly. Chlorination of wastewater can also significantly increase the number and concentration of refractory toxic organic compounds which can then reduce the effectiveness of RI treatment for their removal. Since the RI system itself will remove bacteria and virus effectively, there is no need for wastewater chlorination prior to application. In the general case, the ground water sources for down-gradient water supplies will be protected. On-site recovery of water for drinking purposes is not recommended without appropriate treatment. The worst case for down-gradient impacts might be high wastewater loading rates or seepage ponds on coarse soils with a shallow water table and nearby recovery for drinking water. These conditions are typically identified during the site investigation and design. In some cases, with such ground water use very close to the RI system, it may be prudent to provide standby disinfection for the recovered water. Disinfection at that point will be more effective and provide a greater degree of overall health protection than chlorination of wastewater prior to application on the RI basin.

4.9 References

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CHAPTER 5

CONSTRUCTION

5.1 General

The most critical elements during construction of RI systems are the infiltration surfaces, earthwork for dike construction, and control of surface and subsurface flow when necessary. Special attention to all these factors is required to insure a successful system.

5.2 Infiltration Surfaces in RI Basins

An appropriate design locates the basins in the "best" soil materials on the site. In some existing systems this principle was sacrificed to obtain gravity flow to and among the basins. Gravity flow is desirable, but is not the priority requirement for site development. The design, if at all possible, also avoids basin construction on backfilled materials. If fill is absolutely necessary, then a test fill is used as described in Chapter 3 and criteria developed from that experience for construction of the full scale system.

The construction of RI systems utilizes the standard equipment and machinery used for conventional earthwork. However, most construction contractors have little or no experience with RI basin construction. It may be necessary to overcome the traditional attitudes that "successful" earthwork requires maximum compaction, maximum soil density and structural stability of the soil. At a very early stage in the project it is necessary to insure that all construction personnel understand the RI concept and the need to avoid any action that will unnecessarily reduce the hydraulic conductivity of the basins. In addition, the contract specifications have to be very explicit regarding the procedures and limitations on earthwork. Any soil layers or zones to be removed and wasted are carefully delineated on plans and profiles, and then rigorously controlled during the actual excavation and disposal process.

5.2.1 Cut and Fill in Coarse Soils

The field density test is the normal quality control procedure for placement of fill and related earthwork for most construction. There have been examples where field density was the only control on placement of fill for RI basins. The specifications might require that the density of the fill in the basin area should not exceed some percentage (75-80%) of the optimum density. In one project with such specifications, the basins failed.

The field density test is not by itself sufficient to insure adequate quality control during RI construction. As shown in Figure 3-1 (this text), if the field moisture happened to be on the "wet" side of optimum for a particular soil it would be possible to achieve a specified density limit and at the same time reduce the permeability significantly. Placement of all fill in the infiltration area of a basin or excavation activities within 0.3 m (1 ft) of the final basin surface is only recommended when the soil moisture content is on the "dry" side of optimum. Since the construction contractor has no control over natural rainfall events, the contract is written to be sufficiently flexible to allow for this downtime. The grading plan includes measures to prevent surface runoff from entering the basin area and even sprinkling for dust control is avoided in the same area when equipment is operating at or near the final grade.

The final infiltration surface in the basin needs to be uniformly graded to allow distribution of wastewater and utilization of the entire soil profile for treatment. Small, apparently insignificant depressions need to be avoided, since these will be the last to drain, resulting in a local accumulation of solids leading to infiltration failure in those areas. Basins with a slight grade have been successful, and when positive drainage features are used, they are located at the lowest spot in the basin. The normal construction efforts to achieve the required uniform grades {$\pm$5 cm (2 in.) is typical} can, with many soils, result in some compaction of the surface layer. The following procedures are recommended:

Fill Areas

1. Bring fill to specified elevation.

2. Fine grade to specified tolerance.

3. Both 1 and 2 are only done when soil is on "dry" side of optimum moisture.

4. Rip the surface to a depth of at least 0.6 m (2 ft) and cross rip in a perpendicular direction.

5. Break up clods and other consolidated material thrown up by the ripper. Disk-harrow with light tractor or other vehicle with low ground pressure tires is usually suitable.

Cut Areas

1. Cut to specified elevation.

2. Fine grade to specified tolerance.

3. Steps 1 and 2 when soil moisture is on the "dry" side of optimum, for the last 0.3 m (1 ft) of cut.

4. Rip the surface to a depth of 0.6 m (2 ft).

5. Disk and harrow surface using low ground pressure vehicles.

5.2.2 Cut and Fill in Fine-Texture Soils

The same basic precautions given in the previous section also apply to fine-textured soils. The requirements for low moisture content during earthwork activities are even more important in this case. It is assumed that the design has avoided basin fills (see Chapter 4) when clay and/or silt content exceeds about 10%, so this section is only concerned with cut sections and fills of acceptable materials.

As described in Chapter 3 and 4, the presence of significant silt and clay in the surface layer of RI basins may eventually result in infiltration problems due to the sorting and deposition on the surface of these fine fractions. The only surface treatment that has been shown to be effective is the establishment of a grass cover on such soils [1]. A combination of Coastal Bermuda (11 kg/ha) (10 lb/ac) and Dallis grass (22 kg/ha) (20 lb/ac) was recommended for one project in the Southern United States. Local agronomic experts can be consulted for recommendations. Seeding is only done at an appropriate season for the locality, and temporary sprinklers may be needed to establish grass in dry areas. Wastewater flooding does not then start until the grass is well established.

5.2.3 Ridge and Furrow Construction

Ridge and furrow construction can offer advantages for both grass covered and bare soil basins. A ridge and furrow configuration provides some increase in the infiltration area available at the surface. It also allows for more rapid aeration of the soils, since the ridge tops are dry when the last increments of wastewater are infiltrating in the furrow. A ridge and furrow basin bottom can extend the time between maintenance operations, since solids will tend to accumulate in the furrow bottoms, leaving the ridges clean. A final situation where ridges and furrows are suggested are those locations in extreme cold climates where the design is based on maintaining a floating ice cover in the winter. The ice layer rests on the ridge tops and infiltration continues in the furrows.

Special equipment may be available for ridge and furrow construction if the site is in an area where surface irrigation is normally practiced. A cross-section with furrows at least 30 cm (12 in.) deep was recommended for one RI project in the Southern United States. Whatever configuration is constructed, it is necessary that it be easily reproduced by the system operator, since reconstruction of the ridges is required periodically.

5.3 Dike Construction

Basic construction procedures and controls used for embankments are suitable for RI dikes. Erosion control for dike soils is necessary during construction and the use of silt fences or other barriers is recommended.

5.4 Control of Water

The design and the grading plan includes consideration of surface runoff requirements during construction, and these elements (i.e., ditches, temporary berms, etc.) are typically installed at a very early stage of construction. Such temporary elements are then removed when construction is complete, if not incorporated in the final site drainage plan. The surface runoff from site roads and outer dike slopes is no different than normal precipitation runoff, so special measures for water quality control are not required.

The design may include trenches or underdrains to intercept and re-direct native ground water in the vicinity of the RI basins. These features typically require a surface outlet, but are not designed to intercept wastewater percolate. Therefore, a discharge permit is not necessary. These drains should also be installed at a very early stage of construction, as should monitoring wells where the final location will not interfere with construction activities. The routine observation of these wells and drainage features during construction may indicate if additional work of this nature is necessary. In one situation, even though some ground water diversion was installed, it was observed to be insufficient to prevent basin interference by a seasonally high ground water table.

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CHAPTER 6

OPERATION

6.1 General

In some cases, the design of existing RI systems did not provide sufficient flexibility for operational adjustments and basin optimization. This is often compounded by an inadequate O & M Manual so that the operator may not know how to make adjustments or understand their significance. There have been cases where all basins were loaded at the same time in what was supposed to be a cyclic operation, or where each basin in turn was loaded until it was "full" before flow was switched to the next. <u>It is absolutely essential that the operator be given sufficient information and resources so that design expectations can be realized</u>. The major concerns for RI systems are wastewater scheduling, maintenance of basin bottoms, and winter operations [1, 2]. The O & M procedures for pumps and other equipment, dikes, and similar features are not unique to RI systems and can be found elsewhere.

6.2 Wastewater Scheduling

In some cases, the design may specify different loadings for various basins in the system if their soils are distinctly different. In the typical case, however, the design is based on a uniform application for all basins as derived from the field and laboratory test results. It is not unusual for some of the basins to have a higher capacity than the design rate, or some less. The operator observes and records the volume of water applied to each basin and the time required for infiltration to be complete for every cycle. A staff gauge or a similar marker in each basin is also recommended. The operator can then observe the actual rate of infiltration in the final stages after flooding has been stopped. The operator also notes the location of wet spots and small areas of ponded water during the drying period so these can receive special attention at the next scheduled maintenance.

A routine evaluation of these data permits optimization of the basins by adjusting the application schedules and loadings on a regular basis. It is not often possible to alter the pump capacity to increase or decrease flow rate, so the adjustment requires a change in the flooding period for a particular basin. These data also give warning of when clogging is reaching an unacceptable level. Definition of a generally applicable rule is not possible, but a preliminary "rule-of-thumb" suggests that all standing water at the end of the flooding period should infiltrate within the first 1/3 of the drying period. If the final infiltration takes more than one-half of the drying period, then maintenance is necessary. In all but the coarsest soils, it is recommended that each basin in the system be allowed to dry and then the bottom scarified or the crust removed once a year during the warmest and driest season of the year, regardless of its infiltration capacity at the time.

In many systems the actual wastewater flow in the first few years of operation may be significantly less than the ultimate design capacity, and an appropriate operation program is needed. The best approach is to rotate operation among basins or sets of basins, using only the number needed for the current flow. These are loaded at the full design rates. This will also provide an early confirmation that all of the basins have the capability to function at design rates. Operational flexibility to make these types of adjustments requires multiple cell RI systems. A minimum of three cells is suggested for even the smallest RI system. The operator may also have to make adjustments, or install additions to any distribution network in the basin, if it appears that wastewater infiltration is not uniform.

6.3 Maintenance of Infiltration Surfaces

Regular maintenance of the infiltration surfaces in the RI basin is required and a useful O & M manual contains a tentative schedule and procedure. Once the system is operational, the evaluation of the routine observations discussed above will allow optimization of this maintenance schedule.

Equipment for routine maintenance typically consists of a tractor, or other towing vehicles, with low ground pressure tires and disk/harrow, or similar devices, to scarify the surface soil layer. The number and sizes of this equipment varies with the size of the RI system. Small RI systems may contract annually for these services unless there are some other municipal uses for the equipment. Since deep ripping is seldom required, it is not necessary to have such equipment dedicated for RI maintenance.

<u>These maintenance activities are only conducted when the basin is dry and the moisture content in the surface soil layer is on the dry side of optimum, as described in Chapter 5.</u> The first step is to remove any thick deposits of organic material that may have accumulated in the low spots and re-grade to eliminate the low spots. Any deposits of soil eroded from the dikes are also removed. The final step is scarification of the entire surface.

Maintenance of grass covered surfaces is similar, but instead of disking, the surface can be aerated when necessary with a spiked drum or similar device. The grass is cut and removed at least once a year. Experience may show, however, that more frequent mowing is necessary at a particular location.

Emergency maintenance should also be restricted to dry soils. If positive drainage elements do not exist in a basin, it is necessary to excavate a small sump with a backhoe or dragline, pump out any standing water, and allow the surface to dry prior to access with heavy equipment.

Deep ripping loosens a consolidated soil. However, if deposition of organics is the cause of reduced permeability, ripping cannot provide long-term benefits, since the ripping only re-distributes the material. When acceptable permeability cannot be restored, then removal and replacement of the affected soil layer is recommended. Fort Devens, Massachusetts, restored their basins in this manner after 20 years of operation [3]. Approxmately 1 m (3 ft) of the upper soil was removed and replaced with similar material from an adjacent borrow pit.

6.4 Monitoring.

The water quality monitoring requirements are established by the regulatory authorities. It is assumed that the design provided appropriate sampling points and the O & M Manual defined appropriate techniques. The water levels in all of the observation wells incorporated in the final system are routinely observed and recorded. The O & M Manual (or a similar document) explains what the operator should do with the data and how to interpret it.

A regular tour of the site and the general vicinity is recommended to insure the integrity of the dikes and to look for seeps or springs in unexpected locations. These may not occur for months or even years after system start-up, depending on soil conditions and topography. Small seeps can often be corrected with additional fill material. In some cases, cut-off trenches and/or subsurface drainage may be necessary. The appearance of a seep in an unexpected location is evidence that lateral flow is occurring in a direction not predicted by the design. Additional monitoring wells may, therefore, be required on such a flow path.

6.5 Winter Operations

The presence of weeds and other random vegetation is not typically a problem for many RI systems in either winter or summer. Ft. Devens, Massachusetts operates on a year-round schedule, but there is no special effort to manage or remove the vegetation. However, the Ft. Devens system receives relatively warm primary effluent and the soils are relatively coarse and infertile. Any ice that forms in the winter is melted by the next application and the basins drain rapidly. Any location in a cold region that does not have similar conditions needs to pay special attention to the vegetation, and either cut it close to the surface or burn it off in late fall. Basins in more fertile soils may require vegetation management during the warm season as well.

Those systems designed for a floating ice cover will require special operation early in each winter to develop the ice sheet, and then will have to rely on flow meters or other remote sensing devices for operation since the basin surface cannot then be seen beneath the ice layer.

Formation of the ice layer requires continuous flooding, after the onset of low temperature winter conditions, to maintain some standing water in the basin. The initial ice layer should be 8 to 10 cm (3-4 in.) thick for ridge and furrow basins, to bridge the gap between ridges. Equation 4-7 can be rearranged and solved for the time required to produce a specified ice thickness under particular temperature conditions, as shown in the example below.

$$t = \left(\frac{I}{C}\right)^2 \left(\frac{1}{\Delta T}\right)$$

(see equations 4-7 for definition of terms)

Assuming 10 cm (4 in.) of ice is required, and that the ambient air temperaure remains at about $-10^{\circ}C$ ($14^{\circ}F$) the time required to form the ice, when no snow cover is present (C = 3) would be:

$$t = \left(\frac{10}{3}\right)^2 \left(\frac{1}{10}\right) = 1.1 \text{ day}$$

The predicted time of 1.1 day assumes the water is already at $0^{\circ}C$ ($32^{\circ}F$). Since it is likely that warmer wastewater is used, there will be additional time required for cooling. Assuming for this example that the total time required would be about two days, it would be necessary to maintain a depth of standing water over the entire basin for that period. Since water is also infiltrating at a relatively high rate during the same period, maintaining several centimeters of standing water might be difficult if the system has rapidly permeable soils and a low application rate.

6.6 References

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PART II. OVERLAND FLOW

CHAPTER 1

INTRODUCTION

1.1 Background

When the second edition of the Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater [1] was published in October 1981, the overland flow (OF) process was just beginning to gain recognition as an effective wastewater treatment alternative. The experience gained and successes reported from these early OF projects have led to a substantial increase in the use of the OF process for smaller communities. As of March 1984, there were approximately 48 municipal OF systems in operation or in some stage of design or construction.

Overland flow treatment of municipal wastewater has finally moved from the research/demonstration stage to routine implementation. The guidance provided in the 1981 Manual [1] has been instrumental in giving the design engineers and owner/operators a sound basis upon which to design and operate OF systems. However, as with any new and/or unfamiliar process, the guidance can be improved as additional information is gained from construction and operational experience.

1.2 Objectives

The overall objective of this chapter is to update and refine the information contained in the Manual [1] with the more recent experiences gained during the design, construction, and operation of an increasing number of municipal OF systems. This supplement is not intended to replace the Manual [1] which continues to provide a firm base of understanding for the technology. Therefore, the Manual [1] will be referenced extensively in this document with existing sections repeated herein only when necessary to clarify a point.

The information provided in Part II is based principally on two sources of new information: (1) field investigations of six OF systems which were placed into operation in 1983 and 1984, ranging in size from 265 to 11,356 m^3/d (0.07 to 3.0 MGD) and three systems which were under construction in early 1984; and (2) recently available data from research/demonstration projects.

Although it is premature to assess adequately the effectiveness of the municipal OF facilities, it appears that most, if not all, of them will be capable of meeting their discharge criteria. The field investigations also uncovered a number of design, construction, and operational factors which could be modified to improve their performance and reliability. These factors are organized by subject matter in this text. As long term operating experience is gained from municipal OF facilities, it will be possible to develop further refinements and to optimize engineering criteria.

1.3 References

1. U. S. Environmental Protection Agency. Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater. EPA 625/1-81-013. U. S. EPA, Center for Environmental Research Information, Cincinnati, OH. October 1981.

CHAPTER 2

CURRENT STATUS

2.1 Introduction

There has been considerable experience gained regarding site selection, design, construction, and operation of municipal OF systems since publication of the Manual [1]. This is organized by subject matter into individual chapters within this text and briefly summarized in this chapter.

2.2 Site Selection

Experience has indicated two areas where the Manual [1] needs clarification. These are the use of the terms "slope" and "terrace," and the use of soil permeability data in selecting sites.

2.2.1 Process Description

The OF process consists of applying wastewater along the upper portions of sloping, grass covered fields and allowing it to flow over the vegetated surface to runoff collection ditches. Throughout this chapter, the uniformly sloped areas which receive wastewater for treatment will be called "terraces." The runoff collection ditches into which the treated effluent flows will be called "drainage channels." In the Manual [1], the term "slope" was used to describe a terrace, but the latter term is more appropriate and is more compatible with conventional agricultural engineering terminology.

The OF process was developed to overcome the limitations to land treatment which are created by soil types of low permeability. OF differs from the other two principal land treatment processes, slow rate and rapid infiltration, in that it is not dependent on infiltration and the treated effluent is discharged as a point source. As continued operating experience is gained from municipal OF systems, it is expected that the consideration and use of the process will increase. Furthermore, as already experienced in some states where OF is in use, both designers and regulatory officials are taking a less conservative attitude towards the process design criteria.

2.2.2 Soil Permeability

Although the OF process was developed to overcome the restrictions of low permeability soils, it can also be used on sites which do not contain heavy clay and silt soils [2, 3]. Table 2-9 of the Manual [1] suggests that the most suitable soils for OF are those with permeabilities of less than 0.5 cm/h (0.2 in./h). However, both designers and regulatory personnel have used that value as an inflexible limit. This can result in sites being eliminated from consideration for OF when they are, in fact, suitable.

OF can be designed successfully on sites where the surface soil permeability is greater than 0.5 cm/h. Consideration for potential ground water contamination (particularly nitrates) is necessary if significant percolation is likely to occur (see Section 4.5.2 of the Manual [1]). Although an artificial barrier can be constructed on sites where no natural barriers exist [2, 3], it is usually not cost-effective to do so.

2.3 Process Design

2.3.1 Design Criteria

The recently constructed municipal OF systems have utilized the design guidance provided in the Manual [1] in a conservative fashion. Although these facilities have not yet generated much performance data, observation of a number of them indicates they will be capable of performing according to their discharge criteria. Since the publication of the Manual [1], a number of independent research and demonstration projects have also been conducted to study the OF process. Most of these projects either have been completed or are in their final stages of work after up to ten years of evaluations. In general, these projects have verified and/or expanded the data base for treatment and the experiences from many of them were considered in the development of this document. It is worthy to note that some of the more recent studies contain initial results regarding topics not studied prior to the publication of the Manual [1], such as the removal of toxic organics [4, 5, 6]. Overall, these new sources of information support changes regarding preapplication treatment and storage.

2.3.2 Preapplication Treatment

The OF experience to date, as discussed in Sections 3 and 4, leads to the following conclusions:

- OF systems are capable of performing satisfactorily with only minimal levels (i.e., screening, comminution) of preapplication treatment [1, 7, 8, 9, 10].

- For certain treatment objectives (e.g., nitrogen removal), OF systems using minimal preapplication treatment perform better than those using higher levels of preapplication treatment [1, 7, 11, 12, 13].

- Algal solids are difficult to remove in OF systems [7, 10, 14, 15, 16] and preapplication treatment processes which encourage algal growth frequently result in greater discharge of suspended solids.

- Inadequate screening, resulting in clogging of distribution systems, is a common problem found in municipal OF systems.

2.3.3 Storage

OF systems often require less storage than slow rate land treatment systems for the same climatic conditions (see Chapter 5). In most cases, there is no need for storage during rainfall events. Because of the potential for algae production, storage cells should be designed as off-line, rather than on-line, components of the treatment system.

2.3.4 Distribution Systems

Most municipal OF systems designed to date use surface or low pressure distribution methods to minimize energy costs and aerosol generation. Although these methods can be used effectively, it is difficult to balance the hydraulics to achieve uniform flow to all terrace areas. Consequently, it is also difficult to achieve and maintain uniform sheet flow down the terraces. These factors do not lessen the advantages of using surface and low pressure systems for municipal wastewaters, provided they are considered during design and construction as discussed in Chapter 6.

2.4 Terrace Design and Construction

Observation of recently completed municipal OF facilities has shown that terrace construction has not always achieved the design goal of uniform sheet flow. In efforts to conserve costs, or because of a lack of understanding of the importance of grading, many OF terraces have not been designed and constructed in a manner and to the tolerances which will provide uniform sheet flow. Detailed recommendations for improving terrace design and construction practices are provided in Chapter 7.

2.5 Vegetation Selection and Establishment

The OF process requires water-tolerant turf grass. Thus, the options of vegetation selection are not nearly as flexible as for slow rate land treatment. The more intensive operating schedule of an OF system diminishes the significance and value of harvesting the grass crop. It is best to leave the cut grass on the terrace for the first year or two to build up a mulch layer, thereby accelerating system acclimation and providing better overall performance. Moreover, since the primary purpose of the OF system is wastewater treatment and not crop production, the cash value of the grass should not be given more importance than it deserves. An over-emphasis on cash value can lead to operating procedures which are counter-productive to good wastewater treatment performance. Recommended procedures for vegetation selection, establishment, and management are discussed in Chapter 8.

2.6 References

1. U. S. Environmental Protection Agency. Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater. EPA 625/1-81-013. U. S. EPA, Center for Environmental Research Information, Cincinnati, OH. October 1981.

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CHAPTER 3

DESIGN CRITERIA

3.1 Procedures

The Manual [1] describes two procedures for the design of an OF system. The most commonly used method is an empirical approach which was developed from successfully operating OF systems. The alternative presented in the Manual [1] is the preliminary development of a rational design procedure [2, 3, 4]. Advances in both of these methods are presented below.

3.2 Empirical Method

3.2.1 Revised Criteria

Most, if not all, of the recently constructed OF facilities were designed using the empirical method. Hydraulic loading rates, application rates, terrace lengths, and other variables were selected to be within the range of design criteria listed on Table 6-5 of the Manual [1]. Recent studies [5, 6, 7] have shown excellent performance with hydraulic loading rates greater than the upper limit of the ranges shown on Table 6-5 of the Manual [1], thereby implying that the ranges given in Table 6-5 [1] are conservative. However, these high rate application projects have been research/demonstration projects with near ideal construction and operational control. The use of high loading rates has not yet been observed on full-scale systems which are operated and controlled exclusively by a municipality. Most of the municipal systems were designed and are currently operating at the low end of the range given in Table 6-5 of the Manual [1]. Therefore, major modifications to Table 6-5 [1] cannot be supported by the results from current operating systems. However, on the basis of current knowledge, most municipal OF systems can operate successfully at loading rates nearer to the median of the design ranges. Conservative designs near the low end of the range may negate the advantage of using the OF process. Such conservatism can result in a non-discharging slow rate land treatment system which is unnecessarily costly due to the extensive preparation of terraces. It may also result in an intermittent overland flow system which produces a poorer quality effluent because the terraces often become too dry for good performance.

Table 3-1 is a revised version of Table 6-5 of the Manual [1]. Table 3-1 shows a range of values for each type of preapplication treatment based on successfully operating systems. The relationship between application rate and hydraulic loading rate is discussed in Section 6.4.2 of the Manual [1]. These ranges allow the designer to select specific

TABLE 3-1
SUGGESTED OVERLAND FLOW DESIGN RANGES

Preapplication Treatment	Application Rate m^3/h m	Hydraulic Loading Rate cm/d
Screening/Primary	0.07 - 0.12[a]	2.0 - 7.0[b]
Aerated Cell (1 day detention)	0.08 - 0.14	2.0 - 8.5
Wastewater Treatment pond[c]	0.09 - 0.15	2.5 - 9.0
Secondary[d]	0.11 - 0.17	3.0 -10.0

a. m^3/h m x 80.5 = gal/h ft

b. cm/d x 0.394 = in./d

c. Does not include removal of algae

d. Recommended only for upgrading existing secondary treatment.

design values on the basis of several inter-related factors such as effluent discharge requirements, climate, and the type of distribution system used. Rates near the upper end of the range may be used during warm months, while values near the low end are suggested if soil temperatures drop below approximately 10°C or if maximum removal efficiency is desired [1]. Slope lengths and application periods are discussed in Sections 3.2.2.3 and 3.2.2.4, respectively.

3.2.2 Use of Design Ranges

3.2.2.1 Selection of Hydraulic Loading Rate

The suggested design ranges shown on Table 3-1 are divided into four categories on the basis of the level of preapplication treatment provided. Within each of the four categories, the specific design criteria are shown as ranges, rather than as a single number. The range is provided to account for the variable conditions which may be encountered at different OF sites throughout the country. The principal variables are the climatic environment of the project and the effluent discharge limitations imposed on the system.

The first step in using Table 3-1 is to establish the level of preapplication treatment which will be provided. Next, the hydraulic loading rate can be selected on the basis of climate and effluent limitations. For use with Table 3-1, climatic conditions can be divided into three categories as follows:

<u>Cold climates</u> - Those which require the same number of storage days for OF as for slow rate systems. This is explained in more detail in Chapter 5. The geographical areas defined by this category are those at and above the 80 day storage line shown on Figure 5-1 of this document.

<u>Moderate climates</u> - Those locations between the 40 day and 80 day storage lines shown on Figure 5-1.

<u>Warm climates</u> - Those where the OF system will not be affected by short duration freezing temperatures. The warm climate zones are defined as those at and below the 40 day storage line shown on Figure 5-1.

Also for use with Table 3-1, the effluent discharge criteria can be divided into three categories as follows:

<u>Least stringent</u> - Defined as EPA's secondary treatment quality for trickling filters and wastewater treatment ponds.

Moderately stringent - Intermediate levels of BOD and suspended solids (between 10/10 and secondary limits defined above), but no nutrient limitations.

Most stringent - High levels of BOD and suspended solids removal (i.e., 10/10 or better) and/or the inclusion of nutrient limitations.

When the project location is in a warm climate zone (at or below the 40 day storage line on Figure 5-1) and the effluent discharge limitations fall into the least stringent category (as defined above), the highest application rate (and hydraulic loading rate) in a particular preapplication treatment category can be selected. Conversely, when the project is in a cold climate (at or above the 80 day storage line on Figure 5-1) and the effluent discharge limitations fall into the most restrictive category (as defined above), the lowest rates in a particular preapplication treatment category can be considered. If both variables (climate and discharge limitations) are in the moderate categories, rates near the median of a specific range are suggested. Using the same logic, all other combinations of the two principal variables (climate and discharge limitations) will fall somewhere within the range of application rates and hydraulic loading rates in Table 3-1.

3.2.2.2 Determination of Land Area

The land area is determined by combining the facility design flow, the hydraulic loading from Table 3-1, and the number of operating days for the system. The equations in Section 6.4.8 in the Manual [1] can be used for this purpose. The operating days for a particular facility are based on the storage factors in Chapter 5 of this text and the designer's best judgement regarding any other non-operating periods. It should be noted that once the hydraulic loading rate is selected and the land area calculated, the designer still may have the option of selecting a surface, low pressure, or high pressure system, provided the application rates are maintained within the ranges presented in Table 3-1.

3.2.2.3 Selection of Terrace Length

The length of individual OF terraces is controlled by the choice of distribution system and the effluent discharge limitations. <u>Surface and low pressure distribution systems require terrace lengths of 20 to 30 meters, while high pressure sprinkler distribution systems require a terrace length of 10 to 20 meters greater than the diameter of the sprinkler pattern.</u> The three categories of discharge criteria listed in Section 3.2.2.1 also influence selection of terrace length. When the discharge limitations fall into the least stringent category, the shortest length can be considered, while the most stringent category will require the longest length and the lowest application rate.

3.2.2.4 Selection of Application Period

<u>Table 6-5 of the Manual [1] suggests that application periods can be from 8 to 12 hours per day, and that the application frequency can be from 5 to 7 days per week.</u> Although these remain as good guidelines for satisfactory treatment performance, they are not rigid boundary conditions. Of particular importance is the fact that the application periods (at the rates shown in Table 3-1) can be increased up to 24 h/d for periods of two to four weeks during warm weather with no adverse impact on treated effluent BOD and suspended solids concentrations. Continuous application (using 0.03 $m^3/h{\cdot}m$) at 24 h/d, 7 d/wk was successful for over a year using primary effluent at a pilot project in New York State [8]. At another project, with a higher application rate (0.18 $m^3/h{\cdot}m$), continuous operation for 24 h/d for extended periods of time (i.e., greater than two weeks), even during warm weather, was found to result in decreased performance, particularly with respect to nitrogen removal [2].

The capacity for accepting longer application periods for a few weeks allows portions of a system to be shut down completely for mowing and/or maintenance while the remainder of the system continues to treat the design volume of wastewater at hydraulic loading rates higher than the design average. It also allows stored wastewater to be added to the daily design flow and be satisfactorily treated without adding to the total land requirements.

Based on previous experiences, it follows that the application periods can be flexible, but they usually fall within a range of 6 to 12 hours, regardless of the climate and discharge categories used for selecting the design hydraulic loading rate. Selecting a specific application period between 6 and 12 h/d for individual terraces has not been found to be a critical part of the design procedure. However, there are certain advantages (explained in Section 6.5) to operating a system 24 hours a day, while still maintaining a 6-12 h/d application period for the individual terraces.

3.3 Rational Design Procedure

3.3.1 Introduction

A rational design procedure is presented in Section 6.11.2 of the Manual [1] as a model which describes BOD removal as a function of terrace length and application rate, where the application rate has the units $m^3/h{\cdot}m$ of slope width. This procedure was developed by Smith and Schroeder at the University of California-Davis [2] as an alternative to the empirical design approach.

3.3.2 Procedure

The rational design equation is presented in the following form [1, 2]:

$$\frac{C_z - c}{C_o} = A \exp^{-kz/q^n} \qquad (3\text{-}1)$$

where:

- C_o = concentration of BOD_5 in applied wastewater, mg/L
- C_z = concentration of BOD_5 at a distance (z) down the terrace, mg/L
- c = minimum achievable effluent concentration, mg/L (determined to be 5 mg/L BOD [2])
- A = empirically determined coefficient dependent on the value of q
- z = distance down terrace, m
- k = empirically determined rate constant
- q = application rate, m^3/h.m terrace width, the valid range for model shown on Figures 3-1 and 3-2
- n = empirically determined coefficient, exponent

The preceding equation has been developed further [4] as a family of curves of $(C_z - c/C_o)$ vs z for different values of q. These curves, as shown on Figures 3-1 and 3-2 for screened wastewater and primary effluent, respectively, can be used for selecting design application rates to achieve needed BOD reductions.

3.3.3 Example

The following example was developed by Smith and Schroeder [2]. It is presented here only to illustrate the design procedure just described.

Assume the following information is known:

1. Applied wastewater = screened raw municipal sewage

2. Flow (Q) = 3,000 m^3/day

3. Influent BOD_5 (C_o) = 150 mg/L

4. Required effluent BOD_5 (C) = 20 mg/L

The necessary design calculations are:

1. Compute the required removal ratio $C-5/C_o$

$$\frac{C-5}{C_o} = \frac{20-5}{150} = 0.10$$

2. Select application rate (q) in valid range of model.

Select $q = 0.37\ m^3/h \cdot m$

3. Determine required value of terrace length (z), referred to as distance down-slope in Figure 3-1.

$z = 41.5\ m$

4. Select application period (P).

$P = 12\ h/d$

5. Compute q for area calculation, applying a safety factor of 1.5 [2].

$$q = \frac{0.37\ m^3/h \cdot m}{1.5}$$

$$q = 0.25\ m^3/h \cdot m$$

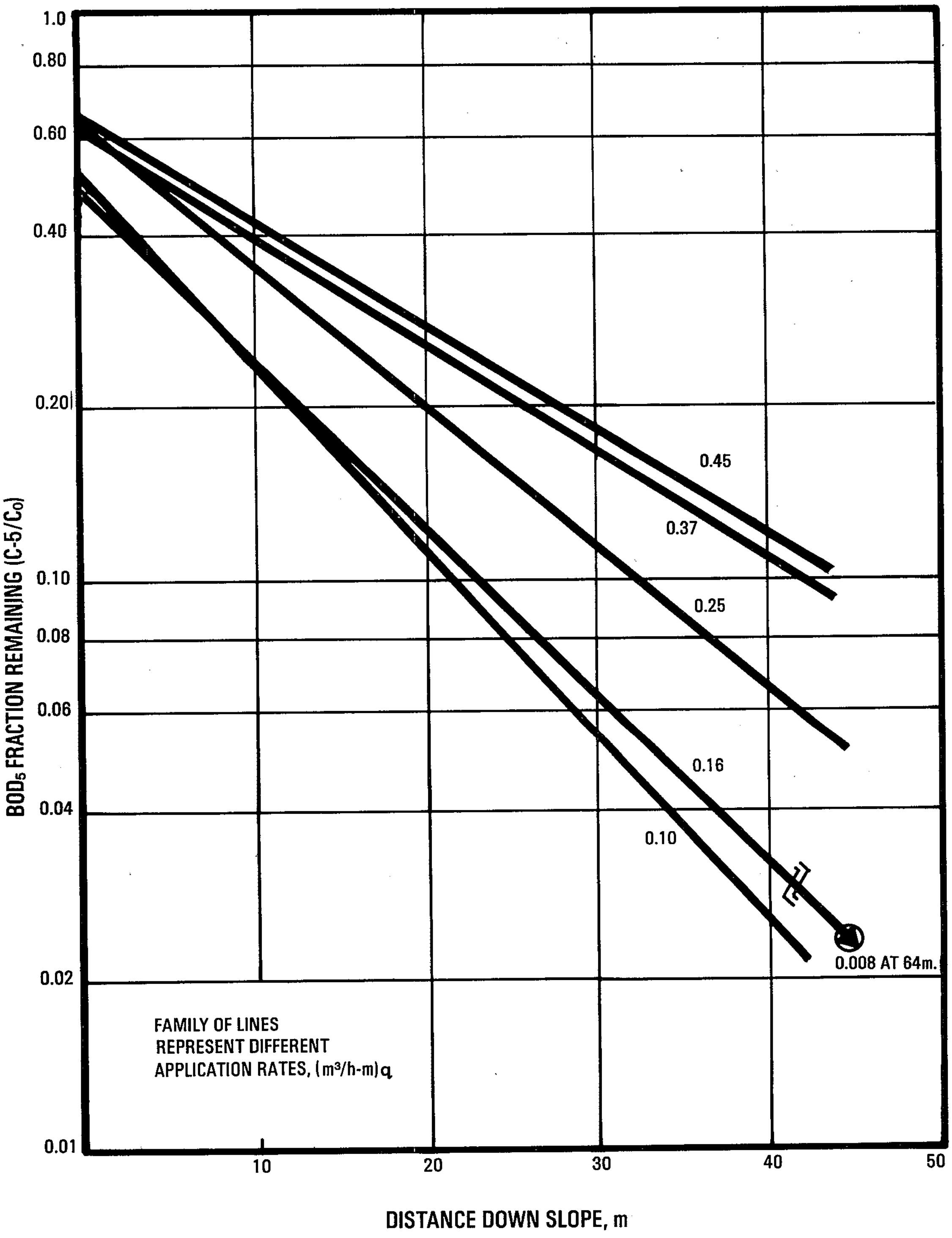

FIGURE 3-1

BOD_5 FRACTION REMAINING VS. DISTANCE DOWN SLOPE
WITH SCREENED RAW WASTEWATER (4)

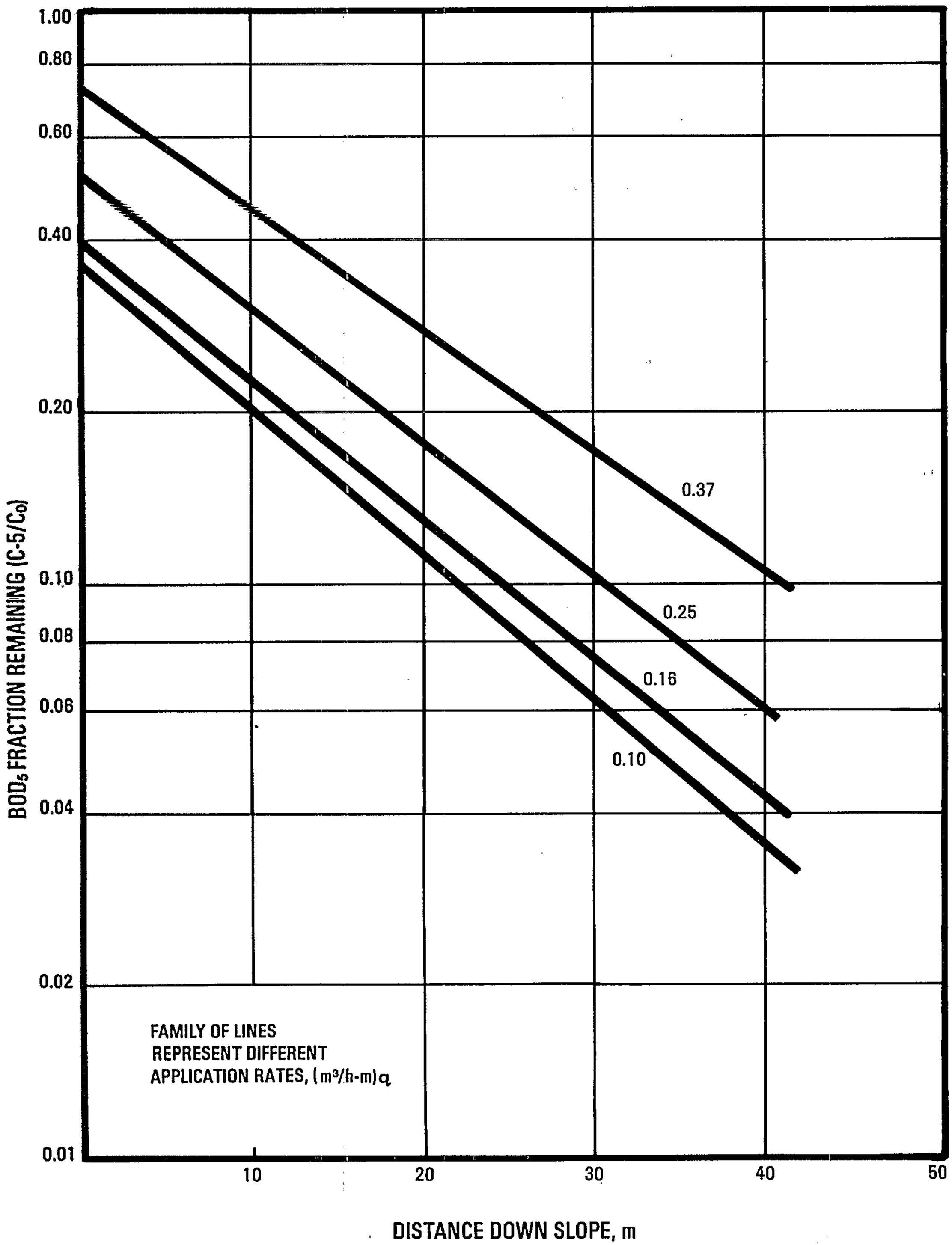

FIGURE 3-2
BOD_5 FRACTION REMAINING VS. DISTANCE DOWN SLOPE
WITH PRIMARY EFFLUENT (4)

6. Compute required total area. Assume 7 d/wk application frequency.

$$\text{Area} = \frac{(Q)(z)}{(q)(P)}$$

$$\text{Area} = \frac{(3{,}000\ m^3/d)(41.5\ m)}{(0.25\ m^3/h \cdot m)(12\ hr/d)}$$

$$\text{Area} = 41{,}500\ m^2 = 4.2\ ha\ (10.4\ ac)$$

The rational design procedure presented above has been tested on several OF research systems [2, 3, 9, 10]. Field investigations at Ada, OK [9] verified the design equation with respect to terrace length and expanded it to determine terrace length for desired concentrations of suspended solids and ammonia. Those field investigations [9] ran from July through October when maximum and minimum average monthly air temperatures were 35°C (95°F) and 9°C (48°F), respectively. Use of the equation for other than warm weather conditions was not tested. Field investigations at Easley, SC [10] partially validated the BOD removals defined by Equation 3-1.

As more experience is gained, this model, or some similar first order relationship, may become the basis for all designs. One of the major present benefits is the support that this procedure provides for the safety and conservatism of the criteria in Table 3-1, and the related empirical method described in this Chapter. It shows that the empirical design procedure (Table 3-1) has an adquate safety factor and <u>that additional safety factors are not needed when using the empirical method.</u>

3.4 References

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CHAPTER 4

PREAPPLICATION TREATMENT

4.1 Areas of Importance

Experience with OF technology continues to show several areas of preapplication treatment to be especially important to the cost-effectiveness and performance of the process. These are:

- The level of preapplication treatment
- Effective solids removal to prevent clogging of the distribution system
- The effect of algae on suspended solids removal

4.2 Level of Preapplication Treatment

Section 6.3 of the Manual [1] recommends that preapplication treatment consisting of screening and comminution is adequate for OF systems. Where the treatment site is not well isolated, aeration is also recommended to control odors during storage and/or application. Data from research studies [2, 3, 4, 5,] continue to confirm the ability of OF systems to produce an effluent superior to secondary effluent when applying screened raw municipal wastewater. <u>Thus, it is generally not cost-effective to provide high levels of treatment prior to OF.</u> In fact, in cases where nitrogen removal is a major consideration, high levels of preapplication treatment can actually interfere with this objective. Greater nitrogen removal can be achieved when the applied wastewater has a higher carbon to nitrogen ratio [6]. Thus, better nitrogen removal can be expected when applying screened raw wastewater than when applying either primary or secondary effluent.

Presently, many regulatory authorities require higher levels of preapplication treatment than screening or comminuting. A single-cell aeration pond with a detention time of approximately one day is an approach which is gaining acceptance. These short detention time aerated cells are in use in several recently constructed OF systems. Limited performance data are available from these systems. However, this method offers the following potential advantages:

> A level of preapplication treatment which often satisfies those state guidelines which require higher levels of preapplication treatment than screening or comminution.

- Positive odor control.

- Minimal sludge management (as compared to conventional primary and secondary treatment processes).

- Relatively simple and inexpensive construction and operation.

- Reduced potential for algal growth.

4.3 Solids Removal

A common problem with some newly constructed municipal OF systems has been solids clogging of the distribution system. The result is poor wastewater distribution onto the terraces and increased maintenance time for opening clogged orifices in the distribution system. Although this problem can never be completely eliminated, it can be substantially reduced by providing an adequate screening system as part of preapplication treatment.

4.4 Algal Interference

The Manual [1] indicates that algal solids have been difficult to remove from some stabilization pond effluents. Further experience [2, 5, 7, 8, 9] has confirmed that algal solids are not reliably removed by OF systems. Preapplication treatment that generates algae (i.e., stabilization ponds) increases the effluent suspended solids concentration from OF systems compared to preapplication treatment which does not encourage algal growth. Appropriate selection of preapplication treatment processes, coupled with appropriate design of storage facilities, is an important consideration when designing to minimize algal solids in system discharges.

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CHAPTER 5

STORAGE

5.1 Current Practice

Section 6.5.1 of the Manual [1] states that the EPA computer programs [2] EPA-1 and EPA-3 (storage programs based on freezing temperatures, rainfall, and snow accumulation constraints) may be used to estimate, conservatively, the winter storage requirements for OF systems. It further states that in areas of the country below the 40-day storage contour shown on Figure 2-5 of the Manual [1], OF systems can generally operate all year without storage. One example is a large industrial OF system in Texas [3] which has operated successfully for 20 years without any storage.

Using EPA-1 and EPA-3 produces a conservative design of storage facilities for municipal OF systems. This conservatism is fostered by several factors, including:

- EPA-1 and EPA-3 were developed primarily for slow rate land treatment systems.
- Lack of operating experience with OF operation in various geographical areas.
- Concern by state regulatory authorities over cold and wet weather performance.
- The use of storage facilities for other objectives (e.g., preapplication treatment, reduction of the length of the operating days, weekend shutdown, etc.) in addition to non-operation only during inclement weather.

A small amount of storage does provide considerable operational flexibility to a system. Even where climatic conditions are mild enough to avoid cold weather storage, two to five days of storage capacity will allow the convenience of weekday operation, if desired. It will also provide added flexibility for terrace maintenance and mowing. This flexibility is especially important for small to moderate size systems. However, a large percentage of recent designs are combining storage as an inseparable part of the preapplication treatment process (e.g., the storage volume is added to the volume of a stabilization pond). Such designs have limited operational flexibility to allow control of algal production or to remove the algae prior to application to the OF system.

5.2 Cold Weather Storage Requirements

Experience has been mixed regarding the ability of OF systems to operate during cold weather conditions. Studies in New Hampshire [4], Wyoming [5], and Indiana [6] have shown that OF, operated in the typical cyclic pattern, could not always meet secondary effluent requirements. On the other hand, an OF system treating primary effluent in New York State [7] has operated successfully through two winters without storage and met secondary effluent requirements by operating continuously for 24 hours a day, 7 days per week. That system operated under very severe climatic conditions, yet consistently met or exceeded secondary treatment quality.

General guidance for storage at overland flow systems is provided in Figure 5-1 which is a revised storage contour map similar to Figure 2-5 found in the Manual [1]. Figure 5-1 is only a general guide, based on the following conditions:

- Storage equivalent to that required for slow rate land treatment systems is shown for systems in moderate and cold climates (i.e., at and above the 40-day contour line on Figure 5-1).

- Storage for cold weather operation is not shown for systems in warm climates (i.e., those below the 40-day contour line on Figure 5-1).

- A minimum of two to five days storage is recommended for systems in warm climates to provide operational flexibility and convenience.

The designer and operator must recognize that, if storage is provided according to Figure 5-1, the storage facility must be reserved for those days which actually require storage. In moderate climate regions, there are frequently a significant number of favorable application days intermixed with unfavorable days during the winter. During these favorable days, it may not be possible to apply the full design loading rate, but it may very well be possible to apply at a reduced loading rate without violating the effluent discharge limitations. This procedure can be used to reduce the storage requirements for systems located between the 40- and 80-day contour lines. On the other hand, in the cold climate zones (those above the 80-day storage line), it may not be possible to use intermittent operation during the winter without violating the effluent discharge limitations.

5.3 Rainfall Considerations

There continues to be a common misconception that OF systems cannot be operated during rainfall events. However, since OF

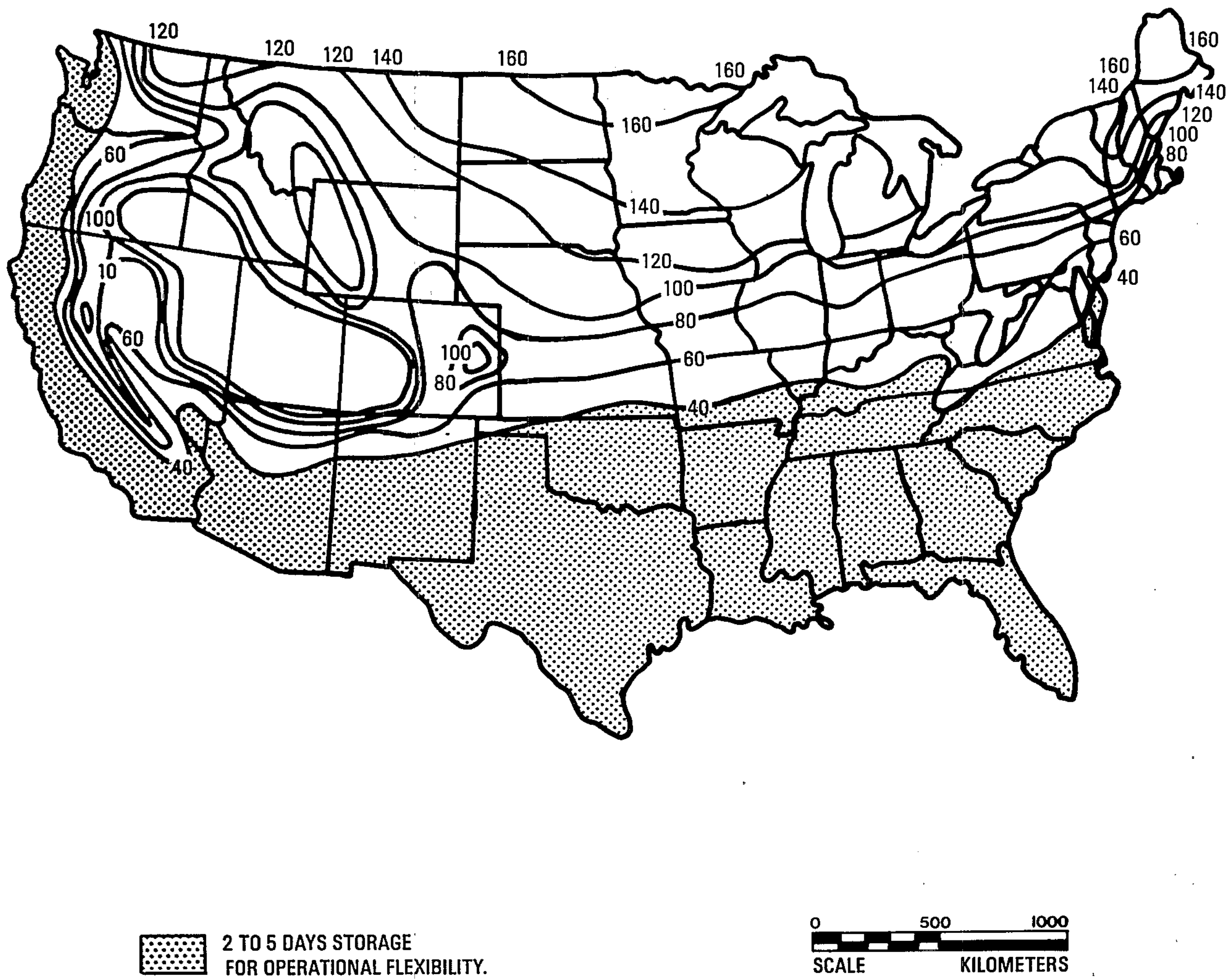

FIGURE 5-1
RECOMMENDED STORAGE DAYS
FOR OVERLAND FLOW SYSTEMS

systems are specifically designed for surface runoff, arbitrarily prohibiting wastewater application during rainfall will serve no purpose unless the discharge violates the conditions of the discharge permit and impairs water quality in the receiving stream.

5.3.1 Impact on Effluent BOD Concentration

Experience has shown that rainfall of any intensity has little effect on effluent BOD concentrations. Studies with screened raw and primary effluent municipal wastewaters showed effluent BOD concentrations during rainfall events to be only slightly elevated above normal operating values, but well below the 30 mg/L secondary treatment standard [8, 9]. Similar results have been found at several industrial OF systems treating food processing [10], textile [11], and pulp and paper [12] wastewaters. Most of the industrial experiences showed that the effluent BOD concentration increased slightly during moderate rainfall intensities and/or short duration storms, but actually decreased below normal operating values during high intensity and/or long duration rainfall events. This decrease can most likely be attributed to dilution.

5.3.2 Impact on Effluent Suspended Solids Concentration

The impact of rainfall on the suspended solids concentration of the treated effluent can be more significant than the influence on BOD. Increased concentrations of suspended solids have been found to consist principally of non-volatile solids resulting from varying degrees of soil erosion [8, 13]. Elevated levels of suspended solids are typically found in the rainfall runoff from newly constructed systems where the vegetation has not had sufficient time to provide thorough coverage of the terraces, where runoff channels have not been stabilized, or where the design/construction practices did not follow sound soil conservation and agricultural principles. Mature OF systems with stable soils and established vegetation show little or no increase in suspended solids concentrations. In one project where the impact of rainfall was studied [14], the suspended solids concentration in the discharge resulting solely from rainfall of approximately 1.5 cm was similar to the following day's discharge resulting solely from applied wastewater. In some cases, erosion protection (e.g., rip-rap, concrete lining, and other soil stabilization procedures) of drainage channels may be necessary to further reduce suspended solids discharges.

5.3.3 Mass Discharges

Even though the effluent BOD and suspended solids concentrations during rainfall are similar to dry weather conditions, the mass discharge of these constituents obviously increases proportionally to both the intensity and duration of the rainfall event. For systems with discharge limitations which are specified in mass units, heavy rainfall events can cause a violation of the mass discharge limitations during the actual event [14, 15]. However, it is unlikely that the 30-day average limits in the discharge permit will be violated.

5.3.4 Recommended Operating Practices

While rainfall increases the mass of most pollutants discharged from an OF system, it also results in increased stream flow and minimal impact on receiving water quality. Therefore, the operating permits for OF systems need not prohibit application of wastewater during rainfall events. Recognizing this, some state regulatory agencies and EPA regions have written permits based on flow which increase the mass discharge limits during rainfall and/or replace them with a concentration limit. The U. S. Army Engineer Waterways Experiment Station has recommended a detailed procedure for developing permits which take the above factors into consideration [9].

5.4 Storage Reservoir Design

To minimize the impact of algae on treatment performance, the storage reservoir should be designed as an off-line component of the system and used only as the need dictates. The storage reservoir should be emptied as soon as possible by blending the stored wastewater containing algae with pre-treated wastewater prior to its application to the OF system.

5.5 References

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CHAPTER 6

DISTRIBUTION SYSTEMS

6.1 Selection

OF achieves treatment primarily through contact between the applied wastewater and the soil medium. Other factors being equal, the distribution method which achieves the best sheet flow pattern will produce the best quality of effluent. As stated in Section 6.6 of the Manual [1], wastewater distribution on OF systems can be achieved by surface methods, low pressure sprays, and moderate to high pressure impact sprinklers. Observation of recently constructed systems has added considerable knowledge about the importance of selection and design. A summary of the advantages and limitations of various types of distribution systems is provided in Table 6-1.

6.2 Surface Methods

Surface distribution methods are favored by many regulatory authorities and engineers because they offer potentially lower operating costs and minimal aerosol generation. Because of the low aerosol potential, state regulations usually require less buffer zone area for surface systems, especially compared to high pressure sprinkler methods. Achieving and maintaining uniform flow onto and across the OF terraces can be a limitation of surface distribution systems. They require careful installation to achieve the leveling required for uniform flow through each orifice, and periodic maintenance thereafter. Hydraulically balancing and maintaining distribution becomes increasingly difficult as the elevation differences within the system increase. Uniform sheet flow over the terraces can generally be accomplished in the presence of a thick cover crop. Spreading can also be improved by applying the wastewater onto a gravel layer and/or splash blocks. The gravel layer is often underlain by a plastic membrane. Sufficient grade must be provided at the point of application to prevent backflow under the distribution pipes and resulting non-uniform distribution.

6.2.1 Gated Pipe

Gated pipe is probably the best choice of the surface application methods. However, because the gated pipe is located above ground, it has a potential for freezing and settling in addition to the limitations previously mentioned. Pipe-runs exceeding 100 m (300 ft), make it difficult to attain uniformity of discharge between gates without careful individual gate adjustment. Reasonably good results can be attained for longer runs by feeding from the high side of the

TABLE 6-1
SUMMARY OF OVERLAND FLOW DISTRIBUTION METHODS

Methods	Advantages	Limitations
Surface Methods (6.2)[a]		
General (6.2)	• Low energy costs • Minimize aerosols and wind drift • Small buffer zones	• Difficult to achieve uniform distribution • Moderate erosion potential
Gated Pipe (6.2.1)	• Same as General plus • Easy to clean • Easiest of surface methods to balance hydraulically	• Same as General plus • Potential for freezing and settling
Slotted or perforated pipe (6.2.2)	• Same as General	• Same as Gated Pipe plus • Small openings clog • Most difficult to balance hydraulically
Bubbling Orifices (6.2.3)	• Same as General plus • Not subject to freezing/settling • Only the orifice must be leveled	• Same as General plus • Difficult to clean when clogged
Low Pressure Sprays (6.3)	• Better distribution than surface methods • Less aerosols than sprinkler systems • Low energy costs	• Nozzles subject to clogging • More aerosols and wind drift than surface methods
Sprinklers (6.4)	• Most uniform distribution	• High energy costs • Aerosol and wind drift potential • Large buffer zones

a. Section where discussed

terrace and balancing the pressure gain from the elevation drop across the terrace against the pressure loss generated by the friction of the wastewater moving through the pipe. The advantages of gated pipe are that the movable gates allow relatively easy flushing of debris which tend to build up in the openings. The use of a surge flow wastewater feed method has been found to minimize the amount of gate plugging [2]. With careful adjustment, the gates can be used to attain reasonable uniformity of discharge along the pipe in spite of minor local height variations.

6.2.2 Slotted or Perforated Pipe

The grade of slotted or perforated pipe must be carefully established and regularly maintained since there is no other adjustment possible. Small slots and/or perforations are not as easily cleaned as the gated pipe openings, and it is often necessary to cut large openings in the top of the pipe for purposes of maintenance. Recommendations to use slotted or perforated pipe are limited to small systems having relatively short pipe-runs which are easy to adjust.

6.2.3 Bubbling Orifices

Bubbling orifices are essentially small riser pipes (often 1-inch diameter) connected to underground distribution laterals. Typically, they discharge onto some type of concrete dispersion pad. Bubbling orifices have the same limitations as other surface distribution systems except for pipe settlement and freeze damage, since only the orifice, not the pipeline, must be carefully leveled.

6.3 Low Pressure Sprays

Low pressure sprays {i.e., fixed nozzles <1.38 kg/cm^2 (20 psi)} are essentially a variation or extension of the bubbling orifice. They are available in many types and variations. The principal limitation of low pressure sprays is plugging from particles too large to pass the orifice and partial plugging from the buildup of smaller particles. Fine screening or double comminution before the distribution system or straining within the system can be used to prevent plugging by large particles. Flush outlets on the ends of the distribution laterals are helpful in clearing the lines of any large particles. It is usually necessary to open these outlets for only a few minutes, with the discharge applied to the terrace. The pulsation caused by the flushing also tends to clear the fixed spray nozzles of accumulated small debris that has built up and partially plugged them.

The principal advantages of low pressure spray systems over surface methods are better distribution of the wastewater onto the terrace and less sensitivity to local variations in elevation. While they produce less mist than high pressure sprinklers, they produce more mist than surface methods. Low pressure spray systems can usually provide adequate distribution of municipal wastewater on OF terraces. Splash blocks and/or gravel layers are often used to prevent erosion.

6.4 Sprinklers

Medium {1.38 to 3.45 kg/cm^2 (20 to 50 psi)} to high {>3.45 kg/cm^2 (50 psi)} pressure impact and gear driven agricultural type sprinklers have been used extensively to apply industrial wastewaters to OF systems, but only to a limited extent on municipal OF systems. The potential limitations of those sprinklers are non-uniform distribution during windy conditions, the risk of aerosol generation and the higher energy requirements associated with pumping. State regulations usually require a greater buffer zone area for sprinkler systems than for surface and low pressure distribution methods. Gear driven sprinklers produce less mist than impact sprinklers, but generally have a higher initial cost. They are also more subject to clogging if the wastewater contains stringy-type solids.

Sprinklers provide the most uniform distribution of wastewater onto the terraces, thereby making it easier to achieve uniform sheet flow with less maintenance. Minor variations in height have little effect on discharge and major variations in height between terraces can be easily overcome in design. Medium to high pressure sprinklers have less tendency to collect debris and become plugged, but will be plugged by particles too large to pass the nozzle. Screening should be used to reduce the particle size within the distribution system to one which will readily pass through the nozzles. As with low pressure sprays, flush outlets on the ends of the laterals are quite effective in clearing the pipe of any large particles.

Sprinkler distribution systems have not been shown to provide higher levels of performance than surface and low pressure spray methods when treating municipal wastewaters. However, on several industrial OF systems treating higher concentrations of BOD and suspended solids than typically found in municipal wastewater, high pressure sprinklers were found to be the only satisfactory distribution method [3].

Vertical distribution risers connected to the lateral pipeline through a flexible coupling allow easy cleanout of any riser stoppages and protect the buried lateral pipe from breakage due to vibration and impact. The piping configurations in Figure 6-1 show both the effective and non-effective methods for placing lateral lines and sprinklers on the terrace. The use of part-circle sprinklers as shown in Figures 6-1(b) and 6-1(f)

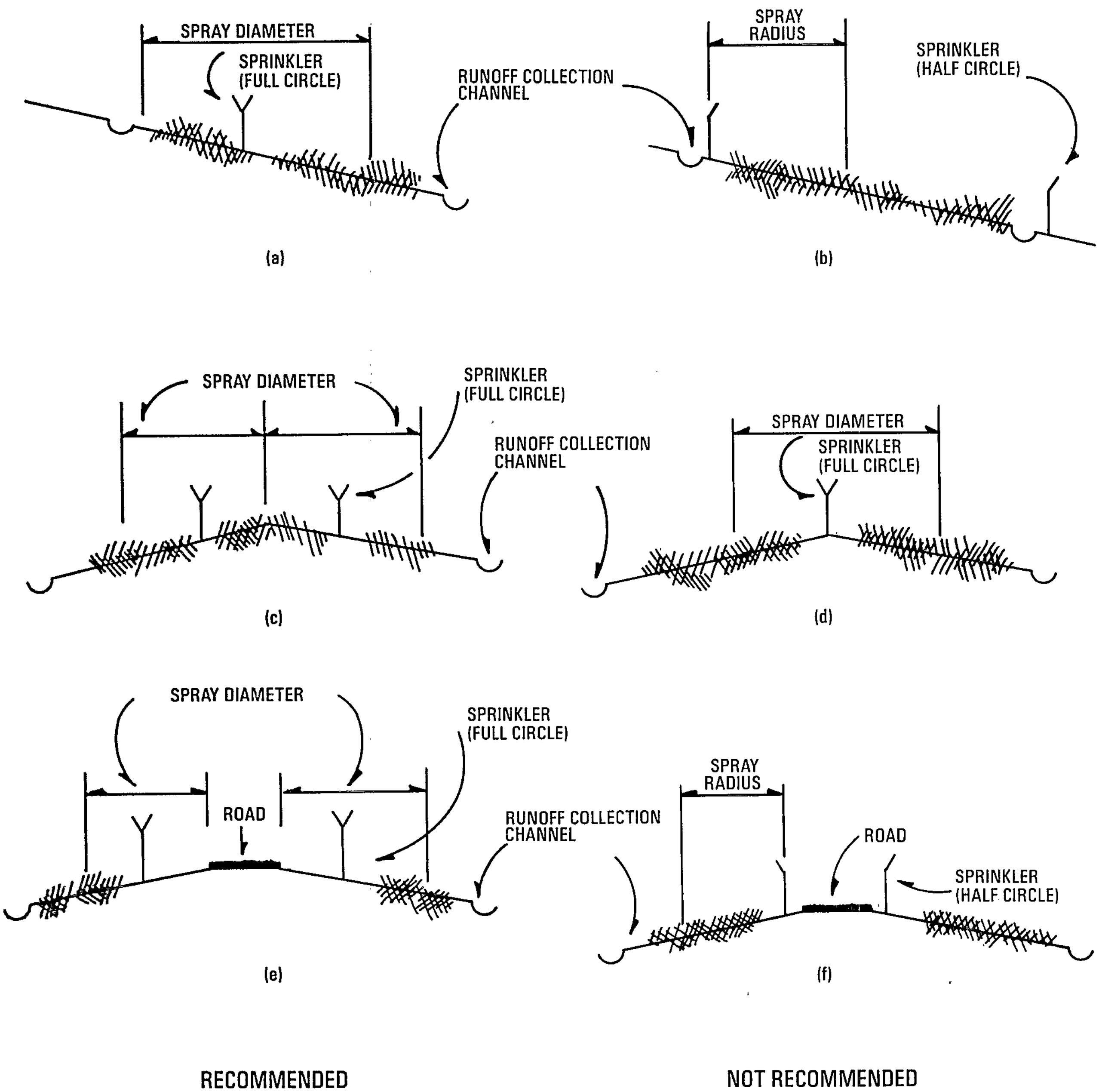

FIGURE 6-1
ALTERNATIVE SPRINKLER CONFIGURATIONS
FOR OVERLAND FLOW DISTRIBUTION

is not recommended because they will have the tendency to allow some wastewater to drift back, untreated, into the collection channel (b) or road (f) from the adjacent terrace. Also, part-circle sprinklers sometimes have a tendency to rotate out of their specific part-circle pattern. A very slight wind will cause the configuration shown in Figure 6-1(d) to spray most of the water on one terrace, thereby causing an overload of that terrace and little or no wetting of the other one.

6.5 Sizing of the Distribution System

A majority of the full-scale municipal overland flow systems have been sized to apply the full daily design flow to the system over a six to twelve hour period, five days a week. This mode of operation minimizes the labor required to operate the system, but it may increase both the capital and energy costs for the system. For example, if the 24-hour wastewater volume is applied to the system during an 8-hour period each day, the piping distribution system must be sized for three times the design flow, and greater yet if the system is only operated five days a week. Additionally, pumping capacity must also be increased by the same order of magnitude. Minimum capital and energy costs are usually achieved by designing for seven-day, 24-hour operation of the pumping and distribution system with no terrace receiving wastewater for more than twelve hours at any one time, followed by at least a twelve hour rest (see Section 3.2.2.4). The designer and the municipality should consider these factors during the design and planning stage and select the operating mode which is best for their particular situation.

There has been a tendency to design OF systems with large zones controlled by remotely operated valves. This increases the size of the distribution system compared to the use of smaller zones with additional, but smaller, remotely operated valves. Large zones can also limit operational flexibility.

6.6 Controls

Both manual and automatic controls have been used successfully for operating OF systems. The advantages of manual controls are their simplicity and the fact that the operator will have frequent contact with and opportunity for observation of the system. These advantages must be balanced against the need for more operator time and full dependency on the operator to control the application periods. Such demands on an operator's time frequently result in the overwatering of some terraces and the underwatering of others. Totally manual controls are not recommended for pumped distribution systems. As a minimum, a low level cutoff switch should be provided for pump protection.

The primary advantages of automation are better utilization of operator time, and in some cases, more accurate control of the application periods. Automation can range from a pump protection cutoff switch to a programmable control system capable of making decisions based on feedback from the operating system. The best system is a compromise which allows the operator to program any portion of the system to operate at any time for any pre-selected length of time. This flexibility is very helpful in providing the desirable short application and rest periods during start-up (see Section 8.4), while still maintaining the capability of providing longer application and rest periods during routine operation (see Section 3.2.2.4).

The controls should also provide minimum safety detection features to protect against damage. In addition, the operator should have the capability of manually bypassing the control system in case of failure and for maintenance purposes. The advantages of automatic controls increase with larger systems.

6.6.1 Automatic Valves

Both pneumatic and hydraulic remote controls have proven to be quite dependable for field installations. Pneumatically and hydraulically operated diaphragm valves (requiring pressure to close) have given good service for many years on numerous land treatment systems under all climatic conditions with few operational problems. Electric controls located in the field are generally not suitable because they can be affected by lightening and are a safety hazard. Piloted valves have been unsatisfactory in all cases where the wastewater was used as the operating medium because of clogging, even when strainers are used. The use of a clean, pressurized, external fluid as an operating medium generally provides satisfactory service. Ball, butterfly, and plug valves with external cylinder operators, closed by air pressure and opened by spring tension, provide satisfactory on/off service.

6.6.2 Manual Valves

Ball, plug, and gate valves all provide satisfactory manual on/off service. Butterfly valves are also capable of providing satisfactory service for primary effluent, but are not recommended for use with raw wastewater since solids can interfer with the valve operation. Globe and angle valves generally permit close regulation of flow, and are therefore satisfactory for throttling purposes. Other types of valves should be used for throttling only if specifically recommended for that purpose by the manufacturer.

6.7 References

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CHAPTER 7

TERRACE DESIGN AND CONSTRUCTION

7.1 Importance of Proper Construction

The primary purpose of land grading, also called land forming and land leveling, is to re-shape the soil surface to assure the uniform movement of water across the OF terraces. Attainment and maintenance of smooth sheet flow down each terrace is necessary to achieve optimum OF process performance. Observation of current practices indicates that grading and finishing require more attention in design and during construction. The purpose of this chapter is to provide more detail than that found in the Manual [1].

7.2 Design Methods

The design of OF terraces and drainage channels utilizes many conventional agricultural engineering principles. There are four basic methods which are used for land grading design [2, 3]: (1) the plane method, (2) the profile method, (3) the plan-inspection method, and (4) the contour adjustment method. In all methods, the designer must provide a 10 to 40% excess of cut in relation to fill to compensate for the shrink-swell potential of the soil. With some soils, the percentage can be even higher. Experience with a specific type of soil is the only way to determine the actual percentage of excess cut required. The local USDA Soil Conservation Service personnel can often provide advice in this area.

It is beyond the scope of this text to provide a detailed description of the various land grading design methods. However, since these techniques are not commonly used in municipal wastewater treatment engineering, the consultant is encouraged to seek assistance, if necessary, from an agricultural engineer or other qualified professional for this portion of the design.

7.3 Terrace Configurations

There are four basic types of terrace configurations used in OF design. These are: (1) conventional, (2) step-up, (3) back-to-back, and (4) step-down. The four configurations are shown on Figure 7-1 and described in more detail in the following sections. The choice of which configuration to use should be based on the existing site conditions and the economics of construction. More than one type of terrace can be used on the same site.

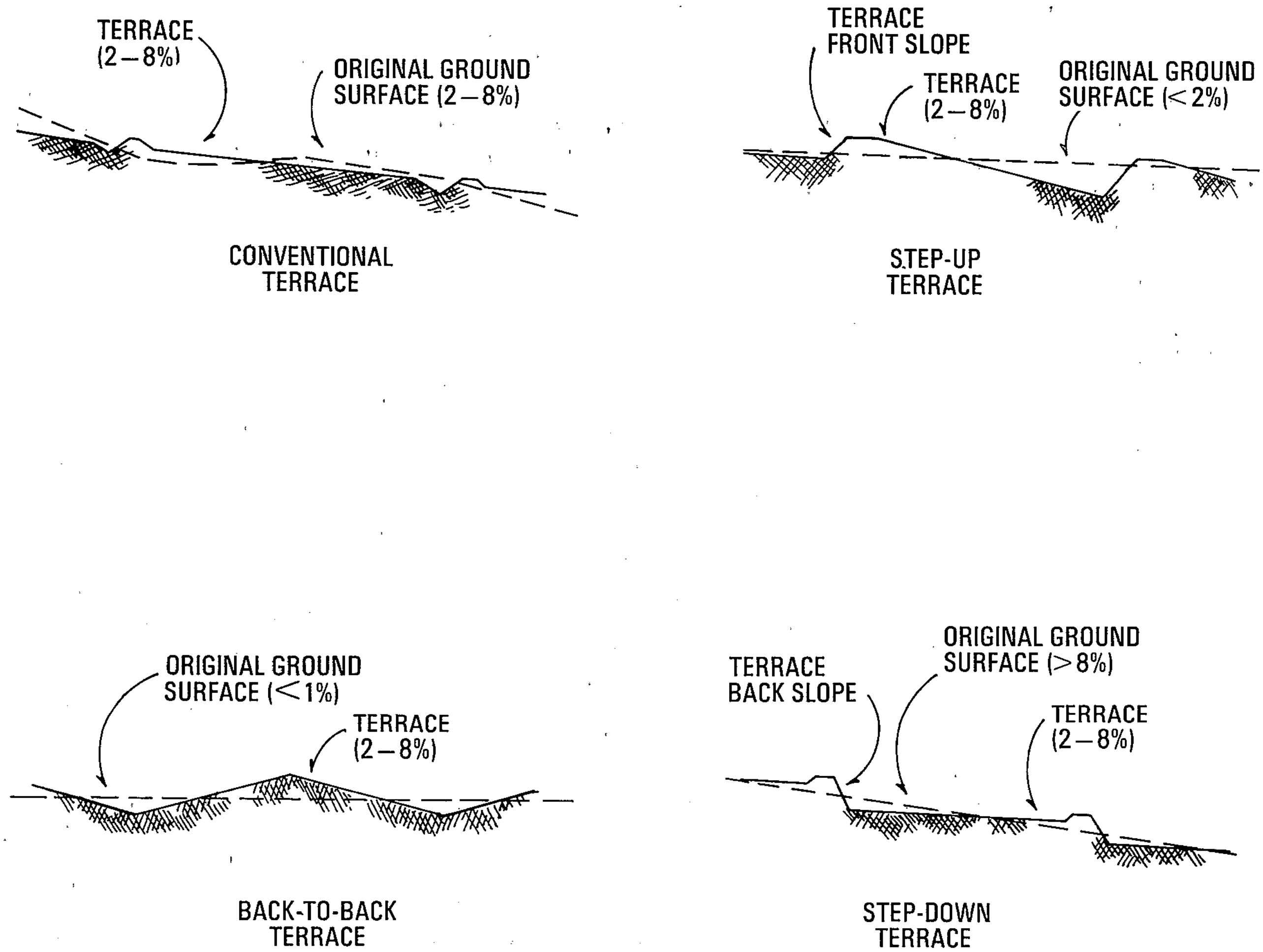


FIGURE 7-1
TYPES OF OVERLAND FLOW TERRACES

7.3.1 Conventional Terraces

Conventional terraces are used where the existing field grade generally meets the criteria (2 - 8%) for overland flow presented in the Manual [1]. Localized cutting and filling is accomplished as necessary to fully meet grade criteria. Individual terraces are then formed by drainage channel construction.

7.3.2 Step-Up Terraces

Step-up terraces are used where the existing field grade is less than that desired for OF (i.e., <2%) and it is necessary to increase existing field grades. The front slope of the adjacent lower terrace provides one side of a v-channel, while the terrace itself provides the other side. If additional channel depth is required, it may be attained by additional excavation, by construction of a ridge on the upper edge of the lower terrace, or a combination of these two methods.

7.3.3 Back-to-Back Terraces

Back-to-back or "humpback" terraces are also used where the existing field grades are less than that desired for OF (i.e., <2%). They are most economical when the existing field grades are very flat (<0.3%). When the existing field grades are between 0.3% and 2.0%, step-up terraces are usually more economical to construct than back-to-back terraces. However, on sites where land availability is extremely limited, back-to-back terraces may be justified when the existing field grades are up to 1.0% because, although they will not be as economical to construct, they will provide higher land utilization (i.e., greater terrace area) than step-up terraces.

Back-to-back terraces do not require construction of individual terrace drainage channels because the entire area of the terraces where the lower portions intersect acts as a broad triangular channel. Where the lower portions intersect at appreciably different heights, the lower portion of the higher terrace should be cut back on a 4:1 or flatter slope to prevent erosion and facilitate mowing.

7.3.4 Step-Down Terraces

Step-down terraces are used to reduce field grades on sites where the existing grades are greater than that desired for OF (i.e., >8%). The drainage channel is constructed at the lower edge of the adjacent higher terrace before the step-down to the lower terrace is started. The upper terrace backslope, or step-down, is typically constructed on a 4:1 slope.

7.3.5 Transitions

The overall grading plan must include transitions from individual terraces to adjacent, unused areas, as well as to other terraces and runoff channels. Safety in construction and future maintenance operations (e.g., mowing) must be considered in designing these transitions. Using ordinary caution, equipment is generally considered safe to operate on 4:1 or flatter slopes [4].

7.4 Drainage Channels

7.4.1 Design

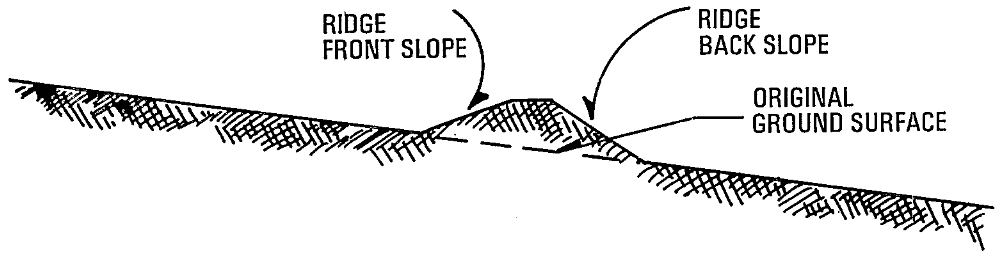
Drainage channels and discharge structures should be designed to handle the discharge from the entire area which they will drain, not just the area of the OF terraces. Drainage channels should have enough discharge capacity to handle the peak rate of runoff from a 25-year/24-hour frequency storm, plus 0.1 to 0.2 m (4-8 in.) freeboard. Channels are ordinarily designed using the Manning formula. For unlined channels, an n value of 0.06 is normally used to design for capacity and an n value of 0.03 to compute for velocity. Channel velocities do not ordinarily exceed 1.5 m/s (5 ft/s), although this limit is influenced by factors such as soil type and vegetation in the channel. Design of drainage channels and discharge structures involves several steps, including runoff computation, selection of channel type, erosion control, and consideration of specific grading techniques.

7.4.2 Runoff Computation

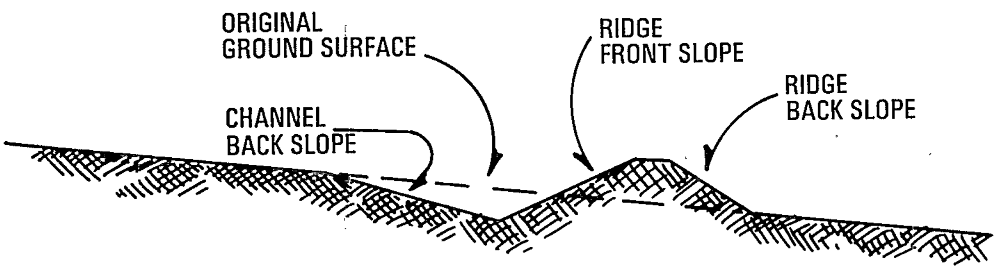
Various satisfactory methods are available for estimation of design runoff. The Rational method [3] is one of the most commonly used and various charts, nomographs, tables, and computer programs are available to expedite design runoff computations. A C value of 0.50 to 0.55 is satisfactory for most overland flow systems. Cook's method [3] is somewhat simpler and gives similar results to the Rational method when the above C value is used. Channels must be designed specifically for each site in order to effect economy of construction.

7.4.3 Channel Types

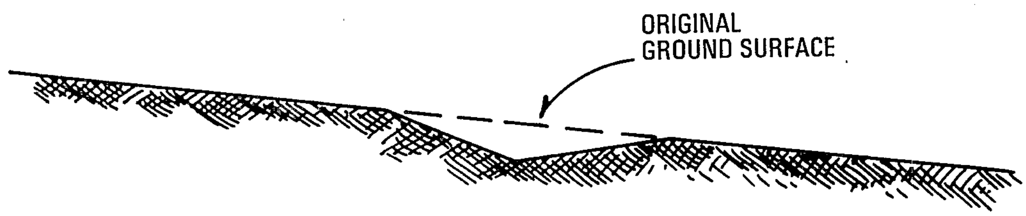
Drainage channels are of three basic types: 1) the ridge type, 2) the more common ridge and trough type, and 3) the trough or ridgeless type. These are shown on Figure 7-2. The ridge front slope provides one side of the V-channel and the terrace provides the other side for ridge type channels. The ridge and trough type channel is constructed by excavating a trough and using the excavated soil to form the ridge. The trough or



RIDGE CHANNEL



RIDGE AND TROUGH CHANNEL



TROUGH CHANNEL

FIGURE 7-2
TYPES OF TERRACE DRAINAGE CHANNELS

ridgeless type channel is usually constructed by excavating the channel and disposing of the excavated soil in another location. For erosion protection and ease of mowing (if grass-lined channels are used), it is generally best to keep the ridge front and back slopes at a 4:1 slope and never steeper than a 3:1 slope.

Any of the channel types may be used with any terrace configuration except the back-to-back terrace. The characteristics of the site determine the relative desirability of the channel types. When working with flat cross slopes, it may be of value to combine the three types of channels in the same terrace, phasing from the ridge type to the ridge and trough type to the trough type. This increases the channel's gradient, thereby improving drainage.

7.4.4 Avoiding Erosion Problems

Observation of operating systems indicates that many runoff channels suffer from excessive erosion before an adequate vegetative cover is established. One cause of this erosion is excessively steep channel sides where water, especially from the terraces, drains into the channel. Such grades should never be steeper than 4:1.

A second cause of erosion is water from a tributary drainage channel entering a larger drainage channel at a higher elevation than the invert of the larger channel. In such cases, the depth of the tributary channel should be lowered to the invert of the larger channel through a transition section. The transition section should be long enough so that the velocity does not exceed 1.5 m/s (5 ft/s) within it, and the side slopes should be cut back at 4:1. If such a transition is impractical, a concrete flume or some other type of drop structure should be used to convey the water safely into the lower channel.

Another cause of erosion is excessive velocity before the vegetation becomes established. All drainage channels having flow velocities greater than 0.9 m/s (3.0 ft/s) when calculated using a Manning n of 0.25 should be protected with some type of cover material such as staked down jute or nylon matting with wood fiber. Concrete lined channels, rip-rap, and straw or hay mulch using an injected asphaltic or other binder may also be suitable. Finally, pipes may be used to convey the drainage water from the OF terraces to the discharge or some point within the system where it can be safely released. Pipes are particularly suitable when step-down terraces must be used. Piped drainage also provides highly efficient land utilization.

7.5 Land Grading

Although the construction of OF terraces and drainage channels involves the use of some standard construction equipment and practices, it is unique in that it also includes certain equipment (i.e., a land plane) and construction practices which are not commonly used by many of the contractors who bid on OF projects. The following is a discussion of certain aspects of the land grading operations which are specific to OF systems.

7.5.1 Rough Grading

Settlement of the soil after rough grading is a major cause of failure to achieve uniform terrace surfaces. Where the construction schedule will allow, the soil should be left to settle for three months to a year following the rough grading operation. The terraces should then be checked for excessive settlement and corrections made, if necessary. Recognizing that this length of interruption to the construction schedule usually is not practical, the following procedure has been found to provide satisfactory results.

During terrace construction, the earth should be placed in 15 cm (6 inch) layers (or lifts) and compacted, as a minimum, to the density of the adjacent undisturbed soils. Excavation, fill, and compaction should be accomplished with a dozer, pan, scraper, or other appropriate pieces of earth moving equipment. Clods, if any, should be crushed before they are buried. Because it is necessary to avoid excessive compaction, equipment operators should be instructed to avoid a follow-the-leader route, but rather distribute the wheel load over as much of the tract as possible without increasing the earth moving distance. A lift should be built all the way across a fill area before the next lift begins, except when an existing depression occurs within the fill area. Existing depressions should be brought level with adjacent areas using 15 cm (6 in.) lifts before beginning the normal fill procedure.

7.5.2 Topsoil Handling

Observation of construction practices on OF sites shows that topsoil is frequently stripped, stockpiled, and then replaced as the last step in the grading operation. While this may be necessary in some cases, it can be an unnecessary and costly activity in other cases. Utilization of the subsoil may require only the application of the proper fertilizer, which is generally a much less expensive operation than topsoil preservation. Fertilization has proven to be an effective and economical method of utilizing exposed subsoil in several industrial overland flow systems. Soil samples, to the depth of the expected excavation, can be obtained and sent to a soils laboratory for analysis. The laboratory can recommend whether the subsoil can support plant growth and the type and amount of fertilizer and/or other soil amendments required.

If the subsoil cannot support plant growth, even with the addition of fertilizers, the topsoil is stockpiled, the cuts over-excavated, and the topsoil replaced. The fill areas must also be stripped, the fills partially made with the materials available, and the topsoil replaced. This is an expensive procedure, and it increases the difficulty of the final grading operations because the roots and other matter contained in the topsoil often interfere with achieving a smooth surface finish. When topsoil preservation is essential, the topsoil from one terrace should be stockpiled on an area requiring little cut or fill. The cuts and fills in this terrace are then completed and the topsoil from the adjacent terrace stripped and used for dressing the surface of the first terrace. Then, progressing across the field, the topsoil should be moved from the adjacent terrace as the terrace forming is completed until the last terrace is dressed with the stockpile from the first terrace. This procedure minimizes the time and expense of topsoil removal and replacement.

7.5.3 Final Grading

A land plane, also called a land leveler or long frame bottomless scraper, is the ideal equipment for final grading. It is only effective on loose soil. The soil is usually loosened by disking with a heavy disk. If large clods are turned up, the area is re-worked with a lighter disk or sheepsfoot roller to break up the clods. The land plane is then passed over each terrace as a finishing operation. Integrity of the terrace must be maintained during the planing operation. The first two passes of the land plane are usually made at opposite 45° angles to the long axis of the terrace with the last pass being parallel to it. The purpose of this cross planing is to prevent the development of a long sinusoidal wave approximately twice the length of the land plane. If any portion of the terrace is scraped bare of loose soil during the planing operation, that area should be re-scarified and the terrace completely re-planed. Surface deviations not exceeding 1.5 cm (0.05 ft) from the planned slopes can be achieved with three passes of the land plane.

There are various types and sizes of land planes and the selection of one for a specific site is important. A land plane capable of doing excellent work on a field 1.6 by 1.6 km (1 mi by 1 mi) probably cannot be turned around on a terrace 30 m (100 ft) long by 60 m (200 ft) wide. A laser controlled land plane, the ultimate in finish grading equipment, can be used if the terrace was designed as a plane. Planing need be accomplished only in the direction of the longest axis of the terrace. The many turns required in cross planing can be eliminated if a laser controlled land plane is used.

Although other equipment can be used for final grading, it is usually with greater difficulty and/or poorer results than with the use of a land plane. Laser equipped dozers are efficient for finishing some terraces. Motor graders can also be used for final grading. The use of a landscape box is suggested to smooth out the tracks and other irregularities left by a motor grader. These alternatives are recommended only if a land plane is not available.

7.6 Supervision and Acceptance

Although it is beyond the scope of this chapter to provide detailed instructions for supervising the terrace construction operations, the following is a check list of some items which are critical to the satisfactory completion of the terraces:

1. The specified lift depth (max. 15 cm) is not exceeded.

2. The specified compaction (to pre-construction density) is maintained in each lift.

3. The specified grades are attained on the edges of the terraces.

4. The specified channel depth and width are maintained.

5. The channel transitions are constructed as shown on the plans.

6. Terrace areas which are scraped bare of loose soil during the land planing operation, are re-scarified and the terrace completely re-planed.

7. The pipeline backfill is properly placed and compacted.

8. Stumps, concrete, and other materials are not buried within the overland flow site.

7.7 References

1. U. S. Environmental Protection Agency. Technology Transfer Process Design Manual for Land Treatment of Municipal Wastewater. EPA 625/1-81-013. U. S. EPA, Center for Environmental Research Information, Cincinnati, OH. October 1981.

2. Land Leveling Irrigation. Chapter 12. SCS National Engineering Handbook. Section 15. U.S. Department of Agriculture, Soil Conservation Service. March 1959.

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CHAPTER 8

VEGETATION SELECTION AND ESTABLISHMENT

8.1 Function

A well established vegetative cover is essential for the efficient performance of OF systems. The purpose of this chapter is to provide greater detail on vegetation selection and establishment than found in Sections 6.8.3 and 6.10.2.2 of the Manual [1].

8.2 Selection

8.2.1 Objectives

The Manual [1] lists the desirable characteristics for an OF cover crop. However, a high percentage of new OF systems appear to be placing great emphasis on cash crop production to the detriment of slope protection and wastewater treatment. This is especially important in the early stages of cover crop establishment and in the start-up of wastewater treatment. Some grasses, most notably some of the improved Bermuda grass varieties, are established only by sprigging or sodding. Sodding is usually too expensive, while sprigging at standard agricultural rates does not provide adequate initial coverage for either slope protection or the start-up of wastewater treatment. The problem is further compounded when the early cuttings are removed as hay rather than being allowed to remain on the terraces as a protective mulch.

8.2.2 Grass Types

Grasses suitable for use in OF systems are described in Table 8-1. Additional information on these grasses can be found in various references on forage grasses [2, 3]. The primary purpose of the vegetation in an overland flow system is to facilitate the treatment of wastewater. The market value of the crop is only of secondary importance. If a grass will not grow under a particular set of OF conditions, no matter what its other desirable characteristics, it is of no benefit. The most common grasses used on OF systems have been Reed canary grass and various Bermuda grass varieties.

8.2.2.1 Reed Canary Grass

While Reed canary grass is sometimes slow in establishment, it forms a very tough, dense sod, spreading by seed, rooting from the joints of stems in soil contact, and vigorous thick rhizomes which push out from the crown. It has one of the longest growing seasons of all the cool season grasses and

TABLE 8-1

SUITABLE OVERLAND FLOW GRASSES

Common Name	Perennial or Annual	Rooting Characteristics	Method of Establishment[a]	Growing Height (cm)
Cool Season Grases				
Reed canary	Perennial	sod	seed	120-210
Tall fescue	Perennial	bunch	seed	90-120
Rye grass	Annual[b]	sod	seed	60- 90
Redtop	Perennial	sod	seed	60- 90
Kentucky bluegrass	Perennial	sod	seed	30- 75
Orchard grass	Perennial	bunch	seed	15- 60
Warm Season Grasses				
Common Bermuda	Perennial	sod	seed	30- 45
Coastal Bermuda[c]	Perennial	sod	sprig	30- 60
Dallis grass	Perennial	bunch	seed	60-120
Bahia	Perennial	sod	seed	60-120

a. Optimum planting period for a specific grass varies according to geographical location.
b. Used as a nurse crop
c. Includes other improved Bermuda grass varieties

continues growing throughout hot summers if adequate moisture is available. It is one of the most drought-resistant of the cool season grasses and may be expected to become the dominant grass on many OF sites.

The market value of Reed canary grass is often described as low by many references which show it to have relatively low protein values. Under normal growing conditions, it is raised on wet land where it is usually past its prime before it can be harvested as a hay crop. However, some research [4] has shown that Reed canary grass grown on an OF system can exceed the commonly reported values for alfalfa in both protein and nutrient content.

8.2.2.2 Bermuda Grass

Common Bermuda grass becomes readily established from seed during warm weather and may establish an initial dominance during the warm season. Unless used as a summer nurse crop (see Section 8.2.2.3), however, Bermuda should be planted only as far north as it grows naturally. This is true for any of the warm season grasses. If planted in the fall or early spring in a mixture of cool season grasses, only unhulled seed should be planted, since unhulled seed (as opposed to hulled seed) is more likely to survive during conditions which are not conducive to germination. The various Bermuda grasses will generally become dormant after the first hard frost, but will continue to provide the thick turf needed for satisfactory treatment performance. If an improved Bermuda grass variety which requires sprigging (e.g., Coastal Bermuda) is selected, overseeding with a nurse crop is required to establish a satisfactory cover until the sprigged grass is well established.

8.2.2.3 Nurse Crop

When the primary grass selected is a slow starting species or one with a low initial density, a number of other varieties may be used as a nurse crop to provide the initial density needed, even though some of these nurse crops may last no longer than the first growing season. Various warm season varieties of grass could be selected for this purpose, including common Bermuda, which will provide rapid cover when planted during hot weather and will eventually be crowded out by the improved variety Coastal. If the time of year is suitable, annual rye grass or another quick growing cool season grass will serve the same purpose.

8.2.3 Combining Grass Species

<u>Planting a mixture of several grasses generally achieves the most satisfactory results</u> by providing a high initial density

and the capability to apply wastewater as quickly as possible. The selection of grasses to include in a mixture is dependent on the climate, the time of planting, and the availability of irrigation water during start-up. In areas where warm season grasses such as Bermuda will grow, a mixture of warm season and cool season grasses is recommended. In cooler climates, a mixture of all cool season grasses is best.

Recommendations from local grass specialists can be helpful in the selection process. Such specialists might be found through the local USDA Soil Conservation Service, county agents, or other local agricultural specialists. However, when using agricultural advisors, it is very important for them to understand that the primary objective of the project is wastewater treatment and the grasses selected must adapt to the imposed wastewater application conditions. If this is not done, inappropriate recommendations may result.

8.3 Planting

8.3.1 Density

The amount of seed to plant depends on the type of grass selected, expected germination, water availability, and time available for cover crop development. Table 8-2 provides general guidelines for seed densities which can be used when constructing a new OF system. The low density rates are used for temporary stabilization and erosion control or if water for cover crop development is in short supply. These rates are equivalent to commonly used seeding rates for pastures. The moderate density rates represent a typical range of seeding when planting at the optimum time for development by natural rainfall. The high density rates will provide the fastest development of a dense cover crop, but these rates require an adequate supply of irrigation water whenever sufficient moisture for germination and growth is not provided by natural rainfall.

It is necessary to convert the density values in Table 8-2 to a seeding rate (kg/ha or lb/ac) by dividing by the number of seeds per kilogram (or per pound) for a specific grass. These latter values can be provided by suppliers or found in standard references [2,3]. If a mixture of several grasses is used, the seeding rate for each grass must be adjusted proportionally.

8.3.2 Scheduling

Planting time is affected by location, climate, variety of grass, availability of irrigation water, capability of sprinkler irrigation, construction schedule, expected rainfall, and other factors. In general, cool season grasses should be planted from spring through early summer or early fall through late fall. The availability of water and the capability of sprinkler irrigation adds considerable flexibility to these planting times. For example, with proper irrigation, cool season grasses can even be started in the middle of summer. However, this requires considerable attention to detail and is not a recommended practice. Warm season grasses generally should be planted from late spring through early fall, although this can vary according to area and climate.

TABLE 8-2
SEEDING DENSITY FOR OVERLAND FLOW SYSTEMS

Level	Density (seeds/m^2)
Low Density	1,450 - 2,690
Moderate Density	4,300 - 8,600
High Density	15,500 - 20,450

seeds/m^2 x 0.093 = seeds/ft^2

Local agricultural advisors can give advice regarding the optimum time as well as the limits of time for grass planting. The timing of the planting schedule is complicated when it is desired to plant both cool and warm season grasses. However, there are time overlaps in both spring and fall for most grasses. Also, some varieties of grass seed, if planted out of season and protected by a growing cover, simply will lie dormant until climatic conditions are suitable for their germination and growth. A second option is to plant the two seed types separately at the optimum planting time for each. The second planting (commonly called overseeding) will require a seeder which will not damage the initial stand or mar the soil surface.

In order to have the greatest potential for establishing a good vegetative cover, it is important to plan the construction schedule so that completion of the final grading of the terraces coincides with the optimum time for planting the selected grasses. If this cannot be accomplished, it is necessary to plant a nurse crop which will germinate and grow at the time construction is completed and then to overseed with the primary grasses at their optimum planting time. If the construction is completed at a time when even a nurse crop will not germinate, the terraces and drainage channels will require protection (straw mulch or other suitable covering) from erosion. The construction and planting schedule is most critical in cold climates where there is generally less time available for establishing vegetation.

8.3.3 Soil Preparation

At the completion of finish grading, fertilizer and soil amendments (e.g., lime) are applied according to soil test recommendations and immediately incorporated to a depth of 10 cm (4 in.) or more. Any standard equipment can be used for applying this material provided it gives uniform spreading and does not damage the slopes. A smoothing harrow is usually pulled behind the disk harrow to eliminate ridges and provide additional cultivation in forming a seed bed. Additional passes of the same or other equipment may be necessary to prepare an adequate seed bed. Any procedure is satisfactory provided it leaves the soil surface in a smooth condition and does not create ridges or leave vehicle tracks.

8.3.4 Seeding

Brillon seeders have been found to be very effective planters for OF systems. This equipment distributes the seed and covers it with a small amount of soil (commonly called cultipacking) in the same operation. Although, other planters can be used satisfactorily, the best results are generally achieved when the seed is cultipacked into the soil. <u>Only light tractors should be used for seeding or other operations such as cultipacking and there should be no wheel tracks left on the terraces. The seeding and cultipacking operations should be carried out parallel to the steepest terrace slopes, even though this is clearly contrary to the conventional wisdom for erosion control.</u> If contour planting is followed, the depressions are so shallow that the slightest runoff breaks through the minute depressions or weak spots, creating a rill or wash. Up and downslope planting prevents the concentration of water so that only sheet erosion or that from raindrop splash takes place. Actual soil movement may be greater, but the smooth uniform surface is not as likely to be destroyed. Areas other than terraces may be seeded according to the best operating characteristics of the planting equipment.

8.3.5 Sprigging

Broadcast planting is the perferred method of sprigging an OF terrace. The soil is prepared in the same manner as for seeding. Then 3.5 to 4.4 m^3/ha (40-50 bu/ac) of sprigs are broadcast. Sprigs are pressed into the soil using a weighted disk harrow with the disks set straight. The soil is firmed by cultipacking up and down the slope. One cm (0.5 in.) of irrigation water should be applied by a sprinkler system immediately. Broadcasting freshly cut stems is an alternative method of propogating Coastal Bermuda grass. The quantity should be increased to 7.8 to 8.7 m^3/ha (90-100 bu/ac) but the same procedure as for sprigging is used. Immediate irrigation is even more critical when using stems than sprigs and no delays should be allowed between cutting the stems and planting.

Other planting methods are satisfactory, provided they leave the soil surface in a smooth condition and do not leave ridges or vehicle tracks. <u>If less than 2.4 m^3/ha (27 bu/ac) are planted, the terraces require overseeding with a nurse crop to provide cover until a permanent sod forms. A nurse crop should also be overseeded if the planting is late in the season when a sod may not have time to form before frost.</u>

Certain precautions are necessary in planting sprigs. Only live, freshly dug sprigs should be planted. If they cannot be planted immediately following digging, they should be kept moist and cool. They must be wet down and turned often to prevent heating. Sprigs must be well covered during hauling to prevent drying by sun and wind. Allowing sprigs to dry out before planting is probably the most common cause of failures to obtain stands. The tips of sprigs should be above ground as sprigs buried more than two inches deep may not grow. The area from which sprigs are dug should be free of weeds, soil borne diseases, and insect pests.

8.4 Vegetation Development

When seeding and/or sprigging is used, a period of growth and acclimation is necessary between planting the vegetation and the application of wastewater at full design loading rates. Certain management practices can be used to accelerate the development of the vegetation and to minimize the acclimation period.

8.4.1 Start-Up Irrigation

8.4.1.1 Methods

Grass growth on seeded systems can usually be started with natural precipitation by selecting the appropriate planting time. However, irrigation is strongly recommended when sprigging an OF system. In either case, the use of irrigation will almost always provide more rapid grass development than relying on natural precipitation. Since irrigation can add considerable cost to the development of the system, it is usually used only when the construction and start-up schedule does not provide sufficient time for grass development by rainfall.

Either fresh water or wastewater can be used to promote the growth of vegetation, with the irrigation rates selected to provide sufficient moisture for grass growth, but not enough to create runoff from the system. On seeded systems using medium to high pressure sprinkler distribution systems, the sprinkler system can be used for the application of irrigation water. Surface and low pressure distribution systems cannot be used for such purpose because they do not provide uniform coverage of irrigation water. A temporary system of portable irrigation pipe with medium to high pressure sprinklers is required for these systems. Temporary irrigation pipe and sprinklers are required for sprigged terraces, even if the wastewater distribution system consists of high pressure sprinklers. The temporary system is required to wet the lower portion of the terrace beyond the reach of the sprinkler patterns.

8.4.1.2 Procedures

The following irrigation procedures have been used successfully in providing rapid vegetative growth on new OF systems. They have been found to be applicable in many areas of the country and with many types of grasses [5].

The first watering is initiated as soon as possible after planting the grass, but is not started unless there is an adequate water (or wastewater) supply to continue irrigation as needed. The first watering of each terrace lasts for 15 minutes. Subsequent waterings of 15 minutes each are repeated hourly up to the point of runoff within the sprinkler pattern. Irrigation is stopped as soon as a surface film of water appears within the sprinkler pattern of a terrace. Fifteen minute waterings, repeated hourly, are re-started as soon as the soil surface appears dry. The sprinklers should be operated at pressures recommended by the manufacturer. Whenever rainfall occurs to the extent that it serves the same purpose as the irrigation, the irrigation can be discontinued until the soil surface within the sprinkler pattern appears dry.

When the grass within the sprinkler pattern reaches an average height of 2.5 cm (1 in.), the irrigation schedule can be changed to one watering per day for one week. The following week, the irrigation schedule can be reduced to one application of water every other day and then twice a week for the following week, each time applying water long enough to achieve total wetting of the soil, but allowing no runoff. Upon completion of the above schedule, water should continue to be applied to achieve a steady growth pattern. Application frequency will vary with daily temperatures and rainfall, but watering to the point of runoff should not be needed more than twice per week.

When the terraces are irrigated only by the wastewater distribution system (with medium to high pressure sprinklers), the same procedure is used until the grass within the sprinkler pattern reaches an average height of approximately 2.54 cm (1 in.). At this point, the number of watering cycles can be successively increased to force water downslope toward the bottom of the terrace. Extreme caution is necessary at this point, as runoff must never be allowed to the extent of causing erosion. Therefore, it is important to supervise all applications of irrigation water. As the grass development proceeds to the extent that the entire terrace can be wetted, the irrigation schedule can be changed to one watering per day for one week. The following week, the irrigation schedule can be reduced to every other day and then to twice a week for the following week, each time applying water long enough to achieve total wetting of the terrace, but stopping irrigation just before runoff into the terrace channel occurs.

8.4.2 Initial Management

The initial level of treatment at new OF systems may not meet the effluent discharge limitations, particularly if the hydraulic loading rate is at the full design rate. The system begins to become acclimated during the first few months of wastewater application, and treated effluent quality continues to improve during this period.

One practice which has been found to improve the efficiency of an OF system during the start-up and acclimation period is to allow at least the first three cuttings of grass to remain on the soil surface. This mulch layer helps to prevent erosion, and increases the organic and nutrient content of the soil, thereby providing improved conditions for biological activity. If wastewater must be treated at the earliest possible time, the terraces can be sodded or seeded at the highest density shown on Table 8-2 and irrigated according to the guidance provided in Section 8.4.1 in order to accelerate development. The use of wastewater for irrigation to hasten the development of the vegetative cover not only shortens the OF system acclimation period and produces a higher initial quality of effluent, but also reduces alternate treatment and/or storage requirements.

Short application periods with short rest periods, such as 15 to 30 minutes of wastewater application with 45 to 90 minutes rest, will also produce a better initial quality of effluent and may further shorten the acclimation period. This technique also reduces or prevents damage from erosion, since it reduces flow velocities while still allowing the soil microorganisms to become acclimated to the waste. These short application and rest cycles can be gradually increased over several weeks to the regularly planned application periods.

8.5 Erosion Control

The first several months after seeding is the time that the system is most susceptible to erosion damage. Frequent inspections are necessary during this period to note and repair erosion damage. The inspection is especially important after a heavy rainfall. Small channels can be corrected easily with hand labor by filling or by making small coffer dams of soil and/or mulch in the channels, and re-seeding or sodding. More extensive damage may necessitate repair with equipment (i.e., a land plane or other device), but the use of equipment on the terraces after seeding should be avoided except when absolutely necessary.

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