CHAPTER 9

DRAINAGE SYSTEMS

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Abstract. *This chapter describes the basics of design, operation, and maintenance of subsurface drainage systems as needed for sustained irrigated agriculture. Emphasis is on the control and management of the water table as needed for crop production in harmony with environmental concerns. Two approaches to design of subsurface drainage systems are based on steady state and transient flows. Steady state design of drains is based on Hooghoudt's equations as modified by Moody. Two examples are presented in the form of spreadsheets. Transient flow design is based on procedures developed by Dumm and Glover and others in the U.S. Bureau of Reclamation (USBR). Spreadsheets are presented using the USBR transient approach for parallel relief drains. Spreadsheets are also presented for calculating drain spacing using the Hooghoudt equation and the USBR transient flow approach. Procedures are described for determining design variables. Drainage materials, construction methods, and equipment are described. The chapter concludes with brief summaries of system operation, maintenance, and performance evaluation.*

Keywords. *Drainage systems, Drainage system design, Drainage system materials, Environmental considerations, Hooghoudt equation, Leaching and salinity control, Moody equations, Operation and maintenance, Steady state flow, Subsurface drainage systems, Transient flow, Water table control.*

9.1 INTRODUCTION

This chapter addresses agricultural drainage with emphasis on drainage under irrigated agriculture, in contrast to drainage of building foundations, dams, airport runways, highway embankments, etc. Of course the principles can be applied to these latter cases of drainage, but the focus will be on drainage of agricultural lands for crop production under irrigation. Thus, more attention will be given to control and management of the water table in the soil profile than to removal of excess water from the land surface.

Proper application of principles to drainage of irrigated land should not be in direct conflict with the maintenance and/or restoration of wetlands, since naturally occurring wetlands are not removed. In some cases, as within the Columbia Basin Project in central Washington, large wetlands areas have been created as a result of an irrigation project. Drainage of irrigated fields has provided water to supply wetlands within the project. However, engineers must be very careful with uses of drainage effluent due to the quality of this water, as has been so dramatically demonstrated by the problems in California's Central Valley and Kesterson Wildlife Reserve.

It may appear paradoxical that after large sums are spent on infrastructure to deliver irrigation water to farms, even larger sums must be spent to remove water through drainage systems. It is true that improved irrigation systems and water management may in many situations reduce the costs of drainage systems. However, even after making practical improvements in irrigation water management at various scales, drainage system construction will be necessary.

The goal of this chapter is to develop and discuss the basics of design, operation, and maintenance of subsurface drainage systems to serve irrigated farms. The material presented in this chapter provides scientifically based engineering methods for design of these systems. Because the principals are the same for both humid and arid regions, and because design is more critical in arid regions, the chapter will treat drainage design in arid regions in detail. Any important differences in design related to humid areas will be pointed out as appropriate.

9.1.1 Definition of Drainage Systems

Agricultural drainage systems related to crop production are classified as *surface drainage systems* or *subsurface drainage systems* depending on where the water causing the drainage problem exists. That is, the classification is based on the function of the system not on the location of system components. Surface drainage systems remove excess water resulting from rainfall, snowmelt, and surplus irrigation from the land surface. Subsurface drainage systems remove soil water to keep the water table level below the bottom of the active root zone, to avoid waterlogging. Buried pipes may be part of a surface or subsurface drainage system depending upon the source of the water they are conveying. Likewise, an open ditch may function to control the water table by removing soil water and be part of a subsurface drainage system, or it may carry excess water from the soil surface as part of a surface drainage system. In arid regions, subsurface drains are often deeper than in humid regions because of the need for salinity control; deep subsurface drains make salinity control easier.

Surface drains may be shallow or deep and are normally unlined open ditches. Subsurface drains are deeper, and may be open ditches, but are usually buried perforated pipes. Before the advent of corrugated plastic drain tubing, buried subsurface drains were constructed of rigid clay tiles or concrete pipes with open joints between short lengths of these pipes. The corrugated plastic drain tubing has perforations to admit water and usually is surrounded by a graded gravel envelope. Many buried pipe drainage systems discharge by gravity directly into waterways or constructed open channels. Others discharge to a sump from which the water is pumped into a suitable open channel or waterway.

Typically, drainage systems consist of laterals, collectors, and outlets. The water to be removed by the system first enters the lateral. The water from several laterals enters a collector. If the system is large we may also have sublaterals and subcollectors where the water first enters the sublateral and sequentially goes to the lateral, subcollector, and collector. Depending on system size, the collector or collectors convey the water to the system outlet. The outlet is the discharge point or points of the system where the water enters existing channels or waterways.

9.2 ENVIRONMENTAL CONSIDERATIONS

The time has long since past (if it ever existed) when an engineer can ignore the environmental consequences of designs. This is as true in agricultural drainage as in any other engineering design effort. This section briefly discusses some issues to be considered, including quality of irrigation water, soil water, and drainage system effluent water; the quantity of drainage system effluent water; drainage in relation to wetlands; and the reclamation of salinized soils. It is hoped that through discussion of these environmental factors, the designers of agricultural drainage systems will consider a holistic approach to the designs so that the systems work in harmony with the environment rather than in conflict with other uses of the environment.

9.2.1 Irrigation Water Quality

Irrigation water quality has little to do with the actual physical design of either surface or subsurface drainage systems. If irrigation water is of poor quality from a salinity standpoint, the amount of water that must pass through the soil to control salinity may be larger than when good quality water is used. The subsurface drainage water is naturally of lower quality than the applied irrigation water; and, if irrigation water quality is poor, the subsurface drainage water may need to be kept separated from the water supply to other users. However, it may be possible to reuse most of the drainage water for irrigation. Surface drainage waters can be reused without restriction unless they happen to contain high concentrations of herbicides, pesticides, or pathogens that may damage crops where the waters are used.

9.2.2 Soil Water Quality

For plants to grow unaffected by the quality of the water in the soil pores, some proportion of the applied irrigation water must pass downward through the soil profile. Table 7.4 in Chapter 7 lists the salt tolerance of agricultural crops. If the salinity of the irrigation water is high, a leaching fraction must be added to the water applied in order to keep the soil water salinity below the threshold values shown in the table and avoid crop yield losses. See Chapter 7 for additional details.

9.2.3 Drainage System Effluent Quality

The quality of drainage system effluent is a function of (1) the quality of the water entering the soil surface; (2) the timing and amounts of fertilizers and pesticides applied to the crop; (3) the natural materials in the soil; and (4) the proportion of the water extracted by the plants for evapotranspiration. If the plants extract a high proportion of the water from the soil profile, the salt concentration in the remaining water will be increased significantly because most agronomic plants extract nearly pure water and leave most of the salts in the soil. Highly water-soluble fertilizers and pesticides that are not used or degraded by the soil environment will appear in the drainage water. Some pesticides or their daughter products are molecularly bound to the soil particles and do not remain in the percolating water. Nitrate nitrogen from fertilizer applications may appear in drainwater if it is leached through the profile by excess irrigation or rainfall before the fertilizer is consumed by the plants, and is often used as a general indicator of pollution by other agents as well. If there is doubt about the quality of drainage water, samples should be taken over time and analyzed to determine the level of dissolved constituents in the water. Some soils naturally contain toxic elements (e.g., selenium) that may be mobilized if water is leached through the soil.

9.2.4 Drainage System Effluent Quantity

Water discharged from subsurface agricultural drains must come from water that infiltrated through the soil surface and water seeping into the drains from the water table. In humid areas, the largest volume of drainage water comes from excess rainfall that seeps into the soil and must be removed to prevent a detrimental rise in the water table. At the time of peak discharge, drains in humid areas may discharge as much as 2.3 L/s per ha, or a depth of 20 mm/day, but are normally designed to carry about 0.8 L/s per ha (7 mm/day). In arid areas, drain discharge rates usually do not exceed 0.3 L/s per ha (3 mm/day). Drain discharge rates may be higher if the drains are discharging groundwater that moves into the area from higher lands or canal seepage.

9.2.5 Drainage and Wetlands

The basic purpose of agricultural drainage is to remove the excess water from the soil surface and the soil profile to levels such that crops can be grown successfully. Drainage of a land area will therefore change it from having a wet soil to having a soil dry enough to be cultivated. This can create some interesting economic, institutional, and social issues.

If an area has been wet for long periods and cannot be cultivated, it will develop an ecology that is typical of a “wetland” and may have a microenvironment that serves as habitat for various species of plants and animals. If biological studies show that the wet area serves a public good, it may be mandated that it cannot be drained. A nearby wetland of similar size can sometimes be substituted, allowing artificial drainage of a troublesome wet area in a field to improve the efficiency of the farming operation.

In the U.S., any drainage of an area that could potentially be declared a wetland must be approved according to the procedures and regulations in Section 404 of the U.S. Clean Water Act. To avoid federal penalties, USDA Natural Resource Conservation Service specialists or members of the U.S. Army Corps of Engineers should be contacted for clearance before any construction takes place.

9.2.6 Reclamation of Salinized Soils

Soils become salinized in arid areas because the water table is too close to the surface. Water from a shallow water table can move easily to the soil surface where it evaporates. The salt carried by the water is therefore deposited on or near the soil surface. When the salt content of the soil becomes too high, crop seeds will not germinate and the soil itself may be physically damaged by the salt.

The first step in reclaiming a salinized soil is to take samples for analysis to determine whether drainage alone will remedy the problem or whether it may be necessary to add some source of soluble calcium, such as gypsum, to aid in the reclamation process. If the soil does not need extra calcium, drains can be installed to lower the water table to an appropriate depth. For arid regions, drains should be installed at a depth of at least 2 m to prevent resalination of the soil following reclamation. When the drains are installed and functioning, the soil can be irrigated a few times, in the absence of a crop, to leach the salts out of the surface soils. Once the salt content of the surface soil has been lowered sufficiently that crop seeds can be germinated, salt-tolerant crops can be grown economically. Until the salt content of the surface soil is lowered to a satisfactory amount, overirrigation should be practiced. In some cases, it may be desirable to install the drainage system and then pond the soil surface to quickly leach the soil profile. Refer to Section 7.6.4 to estimate the amount of water that will be needed for reclamation. Once crops are growing, uniform irrigations will eventually remove the excess salt from the profile.

9.3 DRAINAGE SYSTEM REQUIREMENTS

In Pakistan, the people appropriately refer to the “double menace” of waterlogging and salinity when describing drainage problems. Thus, drainage systems in irrigated areas should provide control of the water table and allow for leaching water to control soil salinity in the crop root zone.

In addition to control of the water table and soil salinity, successful agriculture requires appropriate timing of certain critical field operations. Where natural drainage is inadequate, artificial drains are needed to remove excess soil water at times during the year when the soil is generally too wet for these operations. This may apply to entire farms or to only a small area of a large field. In the latter case, localized drains may be installed in a portion of the field to overcome the lack of natural drainage or to intercept seepage from a canal or other sources.

Before proceeding to the details of drainage system design approaches in Section 9.4, it is helpful to consider two important aspects of the need for constructed drainage systems in irrigated agriculture.

9.3.1 Water Table Control

Plants grow best when the water table is kept at or below the bottom of the crop root zone. For control of excess water in the root zone, subsurface drains are generally designed to keep the water table midway between adjacent drains from getting closer to the surface than 1 m. (In organic soils, the water table should be maintained at approximately 0.5 m deep to prevent excess oxidation of the organic soil materials.) The depth and spacing of the subsurface drains determines the midpoint water table depth at the design discharge rate. In humid areas the drains are commonly installed at depths of 1.0 to 1.5 m, while in arid areas drains are typically installed at depths of 2 m or more. Drain spacing in arid areas may be 4 to 5 times greater than in humid areas because there is less excess water to be removed.

9.3.2 Leaching and Salinity Control

If drains are installed at a 2-m depth or deeper in an arid area and if adequate amounts of irrigation water are applied in the course of normal cultivation of a crop, salinity will be controlled and adequate leaching will occur to prevent soil salination. Drains do not need to be made larger for reclamation of saline soils because all movement of water is downward in the soil profile. If the water temporarily exceeds the carrying capacity of the drains, the flow of water through the soil will be slower and the reclamation may be more effective since the salt in the soil has more time to dissolve in the leaching water.

9.4 APPROACHES TO DESIGN OF SUBSURFACE DRAINAGE SYSTEMS

There are two general approaches for calculating drain spacing for a subsurface drainage system, the *steady state* and the *transient* approaches. In a steady state design, the water table is assumed to be maintained at a constant level by a continuous slow uniform recharge of water through the soil surface. A steady state design theory was developed by Hooghoudt (1940) in the Netherlands, where rainfall is slow and occurs over a long period of time. When recharge stops, the water level between drains goes down until it reaches the level of the drains. The transient design theory was developed by the U.S. Bureau of Reclamation (USBR, 1993) to more closely represent the periodic recharge to the groundwater that occurs under irrigated conditions. In a transient design, the water table is assumed to rise on the date of irrigation and to fall continuously until the next irrigation or recharge event. In the real world, water tables fluctuate between drains regardless of whether a steady state or a transient method was used to determine spacing.

Constructed drainage systems can be classified according to the nature of the water source causing the drainage problem and the protection offered by the drainage system. If the source of water causing the drainage problem originates within the area protected by the drainage system, the system is called a *relief drainage system*. The drains in this case are referred to as *relief drains*. The spacing of parallel relief drains may be determined by either the steady state or transient design approach (See Section 9.4.1). If the water originates outside the area being protected by the drains, then the drains intercept this water and are called *interceptor drains*. Interceptor drains are a special steady state design case and are treated in Section 9.4.2.

The plan view geometry (or layout) of drainage systems can be modified to meet special physical conditions. NRCS (2001) shows various layouts for field drains that describe their appearance on a map: random, herringbone, or parallel (see also USBR, 1993).

9.4.1 Relief Drains

Drains installed in a field to cause a general lowering of the water table are called relief drains. They effectively "relieve" the high water table. As an alternative to relief drains, networks of small wells can be installed in an area with a high water table for the purpose of lowering the water table. They are called *relief wells* and have the same effect as relief drains installed horizontally below the soil surface. If the groundwater is of acceptable quality, it can be used directly for irrigation or even municipal use. If the groundwater is salty, horizontal relief drains at a relatively close spacing are preferred over wells or deep drains at wide spacing to minimize the amount of deep saline groundwater removed.

9.4.1.1 Steady flow. Drainage systems in both humid and arid areas can be designed using steady state theory. When the appropriate design parameters are selected, the proper design will result. The amount of water to be removed by the drainage system must be determined. This amount is described as a volume per unit area per unit of time and is called the *drainage coefficient*. The units of the drainage coefficient are usually expressed in mm/day or m/day, and must be the same as the units of hydraulic conductivity of the soil that are used in the design calculations. In humid areas, the drainage coefficient is based on local experience and is approximately 7 mm/day. The assumption made is that there will be 7 mm/day of excess water from rainfall that will enter the soil profile and that must be removed by the drainage system to prevent the water table from rising closer than 1 m to the soil surface. In arid areas, a steady state drainage coefficient is computed from information about the average peak rate of evapotranspiration (ET) and the fraction of the applied irrigation water that is not consumed by the crop. The fraction of the irrigation water that infiltrates into the soil, but is not consumed by the crop, is related to the irrigation efficiency or water application efficiency. Any surface runoff water from the irrigation should not be included in the drainage coefficient or the drain spacing will be smaller than necessary. If salinity control is required, a leaching fraction must be added to the drainage coefficient.

Equation 9.1 can be used to determine the drainage coefficient for an arid area:

$$q = ET\,[1 - SF(1 - LF)]/SF \tag{9.1}$$

where q is the drainage coefficient expressed in the same units as ET, ET is expressed as a rate of use per day or as a depth of water consumed over a specific period of time, LF is the leaching fraction (see Chapter 7), and SF is the fraction of the infiltrated irrigation water that is stored in the root zone. The stored fraction (SF) of the irrigation water can be calculated from data for a typical irrigation as

$$SF = ET/[IW(1 - RF) - ET(LF)] \tag{9.2}$$

where IW is the gross amount of water applied, ET is the amount of water consumed by the plants since the last recharge event, and RF is the fraction of the irrigation water that runs off. The stored fraction SF has also been called irrigation efficiency (of the infiltrated water) or water application efficiency (not including runoff).

A typical value for the drainage coefficient for an arid area is from 1 to 3 mm/day. If the equation gives a higher value, the wrong efficiency is being used. The leaching fraction used in the drainage coefficient equation will vary from 0.0 to 0.2 depending on the salt concentration of the irrigation water (see Chapter 7) but will seldom exceed 0.05. Improper irrigation practices that result in poor uniformity (low efficiency) should not be assumed to satisfy the annual leaching requirement. Calculation or selection of a steady flow drainage coefficient provides part of the necessary input to steady state drainage system design.

The steady state design equation used here was developed for an idealized flow system for two parallel buried pipe drains taken from the middle of a larger set of parallel buried pipe drains. The drains are at some distance D above a horizontal impermeable barrier. This idealized flow system is shown in Figure 9.1. It is assumed in Figure 9.1 that the ground surface and the impermeable barrier are horizontal and the two drains are at the same depth. It is customary to assume that the drains are half full (i.e., the design depth for the drains is to the drain pipe centerline.)

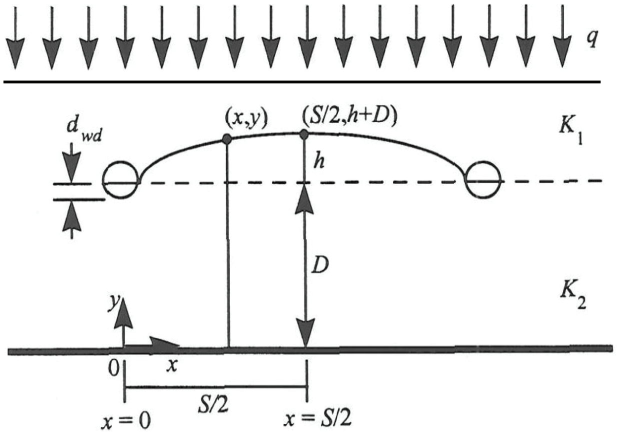


Figure 9.1. Idealized flow system for steady state buried pipe drainage to two drains taken from a set of parallel drains at a distance *D* above an impermeable barrier.

Using the Dupuit-Forchheimer (D-F) assumptions, Hooghoudt (1940) developed an equation of the form:

$$S = \sqrt{(4h/q)(K_1 h + 2K_2 D_e)} \tag{9.3}$$

where S = drain spacing (in the horizontal direction)

q = drainage coefficient

h = height of the water table above the drain pipe centerline midway between the two drains

K_1, = hydraulic conductivity values for the soil above the drains

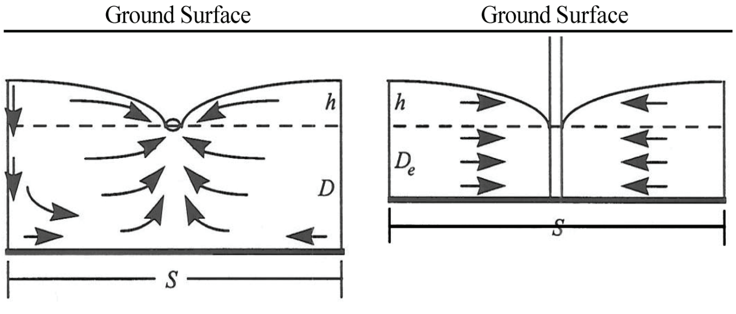
K_2 = hydraulic conductivity values for the soil below the drains

D_e is the Hooghoudt equivalent depth coefficient, which is explained below.

Note from Figure 9.1 and Equation 9.3 that the Hooghoudt equation applies directly to the special layered soil case where the drains lie at the interface between the two soil layers having different values of hydraulic conductivity. Also note that for $K_1 = K_2$, the soil is homogeneous and the equation also applies with $K_1 = K_2 = K$. For the case when $K_2 << K_1$ and the soil below the centerline of the drains may be considered the impermeable barrier, Equation 9.3 still applies with $K_2 = 0$ and $K_1 = K$ (also $D = 0$). Thus, for this case of drains on the barrier, Equation 9.3 reduces to $S = (4Kh^2/q)^{1/2}$ from which S may be calculated directly from value of K, h, and q. Steady state is assumed wherein a constant rate recharge, q (volume per unit area per unit time), is removed by the drains. The recharge is uniformly distributed (uniform rainfall) as it enters the saturated groundwater zone. Also, in the derivation of Equation 9.3, it has been assumed that the depth of water in the drains is small relative to h.

Hooghoudt recognized that the assumption of horizontal flow could be a serious limitation in the use of Equation 9.3, especially for larger values of the ratio D/S. Thus, he developed the concept of *equivalent depth,* D_e. For all calculations, D_e should be used instead of the actual depth from the drains to the impermeable barrier, D.

The Hooghoudt equivalent depth, D_e, is defined as a fictitious depth to a fictitious impermeable barrier such that if the spacing S is computed with Equation 9.3, i.e., using D_e instead of D, the result would be the same as using the actual D in a more accurate theory. Perhaps this concept is more easily understood by using the graphics of Figure 9.2. The real situation with buried pipe drains at a distance D above the barrier is replaced with the theoretical model of fully penetrating ditch drains but with depth of water in the ditch drain of D_e, the Hooghoudt equivalent depth.



Real Situation Theoretical Model

Figure 9.2. A graphical explanation of the concept of Hooghoudt equivalent depth, D_e, as depicted by the difference between the real situation and the theoretical model.

Note that D_e always is either less than or equal to D. The extra resistance of radial flow near the drains is modeled by a shallower total system of the same hydraulic conductivity. In Figure 9.2, note that the hydraulic conductivity is the same in both the real situation and the theoretical model. Of course there may be different values for K_1, above the drains and K_2 below the drains or for homogeneous soils, $K_1 = K_2 = K$. Also, in Figure 9.2 the values of S and h in the real situation are exactly the same as S and h in the theoretical model.

The concept of Hooghoudt equivalent depth is widely accepted as a reasonable way of using the D-F theory to calculate spacing of parallel drains regardless of whether steady state or transient drain spacing theories are applied. Note that the concept is based upon equivalent overall system performance and not upon whether the water flow is steady or transient.

Moody (1966) presented equations to relate the Hooghoudt equivalent depth to the actual depth of the system, the drain radius, and the drain spacing. In the nomenclature used above, his equations are:

$$D_e = D / \{1 + (D/S)\ [(8/\pi)\ln(D/r) - \alpha\ ]\} \quad \text{for } 0 \le D/S \le 0.31 \tag{9.4}$$

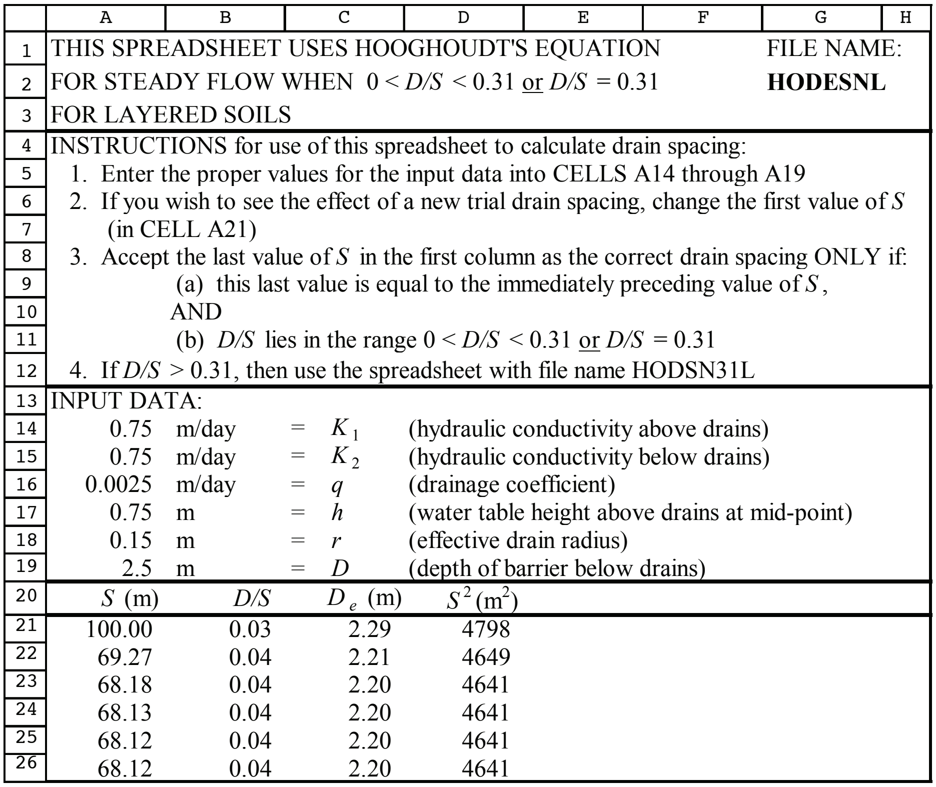
where $\alpha = 3.55 - 1.6(D/S) + 2(D/S)^2$ (9.5)

and r is the drain radius. Moody noted that for practical purposes, $\alpha = 3.4$ can be used with little error. When computers are used to perform the calculations the use of Equation 9.5 to determine α is recommended. For the case where D/S is larger than 0.31, the Moody equation is

$$D_e = S / \left\{ (8/\pi)[\ln(S/r) - 1.15]\ \right\} \quad \text{for } D/S > 0.31 \tag{9.6}$$

Charts to determine Hooghoudt equivalent depth have been prepared and presented in the literature by various authors. We prefer the Moody equations especially for ease of use with computers.

By inspection of the Hooghoudt and Moody equations, it is easily seen that the calculation of the proper drain spacing for relief drains is an iterative (or trial and error) procedure because S depends on D_e and D_e depends upon S. In fact, the whole design procedure is iterative. Also note that the depth of the drains below ground surface is not expressly included in the Hooghoudt equation (Figure 9.1, Equation 9.3). The iterative design procedure starts with a depth of root zone appropriate for the crop to be grown. If the designer is inexperienced, calculations for all crops that may be grown should be made and then the most critical one should be chosen as the design crop. Once the depth of the root zone, D_{rz}, has been determined, a trial drain depth is chosen. See Section 9.5.2 for discussion of proper drain depths. Since the drain depth is the sum of D_{rz} and h, the choice of the trial drain depth determines the design value for h. Once the depth from ground surface to the impermeable barrier (see Section 9.5.3) is known, D has also been determined. Next, the designer must choose a trial diameter of drain pipe from which the drain radius r is determined. The effective drain radius is half the drain diameter plus the thickness of gravel envelope surrounding the drain pipe. After the drain spacing has been calculated, the drain discharge must be calculated and compared to the pipe capacity to see if this trial drain diameter is satisfactory.



	A	B	C	D	E	F	G	H
1	THIS SPREADSHEET USES HOOGHOUDT'S EQUATION						FILE NAME:	
2	FOR STEADY FLOW WHEN $0 < D/S < 0.31$ or $D/S = 0.31$						**HODESNL**	
3	FOR LAYERED SOILS							
4	INSTRUCTIONS for use of this spreadsheet to calculate drain spacing:							
5	1. Enter the proper values for the input data into CELLS A14 through A19							
6	2. If you wish to see the effect of a new trial drain spacing, change the first value of S							
7	(in CELL A21)							
8	3. Accept the last value of S in the first column as the correct drain spacing ONLY if:							
9	(a) this last value is equal to the immediately preceding value of S,							
10	AND							
11	(b) D/S lies in the range $0 < D/S < 0.31$ or $D/S = 0.31$							
12	4. If $D/S > 0.31$, then use the spreadsheet with file name HODSN31L							
13	INPUT DATA:							
14	0.75	m/day	= K_1	(hydraulic conductivity above drains)				
15	0.75	m/day	= K_2	(hydraulic conductivity below drains)				
16	0.0025	m/day	= q	(drainage coefficient)				
17	0.75	m	= h	(water table height above drains at mid-point)				
18	0.15	m	= r	(effective drain radius)				
19	2.5	m	= D	(depth of barrier below drains)				
20	S (m)	D/S	D_e (m)	S^2 (m^2)				
21	100.00	0.03	2.29	4798				
22	69.27	0.04	2.21	4649				
23	68.18	0.04	2.20	4641				
24	68.13	0.04	2.20	4641				
25	68.12	0.04	2.20	4641				
26	68.12	0.04	2.20	4641				

Figure 9.3. Spreadsheet using the Hooghoudt and Moody equations to calculate steady state relief drain spacing for $0 < D/S \leq 0.31$.

With the foregoing variables determined and knowing the soil hydraulic conductivity K (or K_1 and K_2) and the drainage requirement q, the designer is ready to use the Hooghoudt and Moody equations to calculate the drain spacing S. While the iterative solution of these equations can be done with a hand calculator, a computer spreadsheet is more useful, especially for the inexperienced designer. Two such spreadsheets are given as Figures 9.3 and 9.4. Figure 9.3 is the one used most often and is for $0 \leq D/S \leq 0.31$. Figure 9.4 applies when $D/S > 0.31$.

Instructions for use of the spreadsheets are included within the spreadsheets. Use of the spreadsheets will be discussed first and then details of how they are constructed will be given. The known or already determined values of the input variables are placed into cells A14 through A19. Next a trial value for drain spacing S is entered into cell A21. The spreadsheet then calculates the values row by row as shown. The correct drain spacing is given as the last value in column A if the last value of S is identical to the next-to-last value of S immediately above and the values of D/S lie in the range $0 \leq D/S \leq 0.31$. If $D/S > 0.31$, then the spreadsheet of Figure 9.4 must be used.

The spreadsheets are constructed as follows. In Figure 9.3, the first trial value of drain spacing is entered into cell A21. Then the value of D from cell A19 is divided by

	A	B	C	D	E	F	G	H
1	THIS SPREADSHEET USES HOOGHOUDT'S EQUATION						FILE NAME:	
2	FOR STEADY FLOW WHEN *D/S* > 0.31						**HODSN31L**	
3	FOR LAYERED SOILS							
4	INSTRUCTIONS for use of this spreadsheet to calculate drain spacing:							
5	1. Enter the proper values for the input data into CELLS A14 through A19							
6	2. If you wish to see the effect of a new trial drain spacing, change the first value of *S*							
7	(in CELL A22)							
8	3. Accept the last value of *S* in the first column as the correct drain spacing ONLY if::							
9	(a) this last value is equal to the immediately preceding value of *S*,							
10	AND							
11	(b) *D/S* lies in the range *D/S* > 0.31							
12	4. If 0 < *D/S* < 0.31 <u>or</u> *D/S* = 0.31, then use the spreadsheet with file name HODESNL							
13	INPUT DATA:							
14	0.75	m/day	= K_1	(hydraulic conductivity above drains)				
15	0.75	m/day	= K_2	(hydraulic conductivity below drains)				
16	0.0025	m/day	= q	(drainage coefficient)				
17	0.75	m	= h	(water table height above drains at mid-point)				
18	0.15	m	= r	(effective drain radius)				
19	50	m	= D	(depth of barrier below drains)				
20	S (m)	D/S	D_e (m)	S^2 (m^2)				
21	200.00	0.25	12.99	24060				
22	155.11	0.32	10.52	19607				
23	140.03	0.36	9.67	18073				
24	134.44	0.37	9.35	17499				
25	132.29	0.38	9.22	17278				
26	131.44	0.38	9.18	17191				
27	131.11	0.38	9.16	17157				
28	130.98	0.38	9.15	17143				
29	130.93	0.38	9.15	17138				
30	130.91	0.38	9.14	17136				
31	130.90	0.38	9.14	17135				
32	130.90	0.38	9.14	17134				

Figure 9.4. Spreadsheet using the Hooghoudt and Moody equations to calculate steady state relief drain spacing for *D/S* > 0.31.

this trial S to give D/S in cell B21. In cell C21, α is calculated from Equation 9.5, and then D_e is calculated from Equation 9.4. Next S^2 is calculated from Equation 9.3 in cell D21. The square root of this last value gives the new trial value of S in cell A22 and the process is repeated for row 22, etc. The construction and use of the spreadsheet of Figure 9.4 is entirely parallel to that of Figure 9.3. Equation 9.6 is used to calculate D_e in Figure 9.4, whereas Equations 9.4 and 9.5 are used for this calculation in Figure 9.3.

9.4.1.2 Steady state relief drain design examples. The spreadsheets of Figure 9.3 and 9.4 also show examples of the drain design process. In Figure 9.3, suppose that the root zone depth to be protected by the drainage system is 1.25 m and the drains are to be placed 2 m deep. Then h is 0.75 m. Suppose the drain pipe is to be 10 cm diameter and the gravel envelope surrounding the drain pipe is 10 cm thick, thus the effective drain radius r is 0.15 m. Also, the drainage coefficient is 2.5 mm/day, the soil is homogeneous with hydraulic conductivity of 0.75 m/day, and the impermeable barrier is

4.5 m below ground surface. For these conditions, the input variables are as given in cells A14 through A19 of Figure 9.3. If a trial drain spacing of 100 m is chosen, Figure 9.3 shows the calculations of this example resulting in a drain spacing of 68 m (rounded to the nearest meter). After constructing the spreadsheet, a designer can easily show that the procedure converges rapidly to the same drain spacing regardless of the value chosen for the first trial drain spacing. A first trial value of 10 m will produce the same result for this example.

Assuming that the drain depth and the root zone depth of this example are satisfactory, all that remains is to determine if the drain pipe diameter of 10 cm is adequate. Since the flow to the drains is steady, the drain discharge per meter length is equal to the product of the drainage coefficient q and the drain spacing S. Thus for this example, the drain discharge is calculated as $Q_m = qS = 0.0025(68) = 0.170$ m^3/day per m of drain. Now if the drain laterals are 300 m long, the discharge from the lateral Q_L is 51.0 m^3/day or 5.9×10^{-4} m^3/s. Using a Manning n value of 0.017 for corrugated plastic drain tubing, the capacity of the drain on a grade of 0.001 is calculated to be 1.25×10^{-3} m^3/s from the Manning equation:

$$Q = VA = (1/n)\, AR^{2/3}\, S_f^{\ 1/2} \tag{9.7}$$

where V = velocity
A = area
R = hydraulic radius
S_f = friction slope
n = the Manning roughness coefficient.

Since the capacity of the lateral (1.25×10^{-3} m^3/s) exceeds the calculated discharge (5.9×10^{-4} m^3/s), the drain pipe diameter of 10 cm is adequate. If the discharge computed with the Manning equation is less than the discharge required from the lateral, choose a larger drain diameter and repeat the spreadsheet calculations with the larger trial drain diameter.

The example shown in Figure 9.4 is for a very deep impermeable barrier. All other values are as given in the previous example. The drains must handle a discharge of 1.14×10^{-3} m^3/s and the 10 cm drain diameter is still adequate for the 300-m drains on a grade of 0.001 m/m.

9.4.1.3 Transient flow. Transient drain spacing designs require the translation of an amount of recharge water entering the soil into a rise of the water table. When an irrigation occurs, no water will reach the water table and cause it to rise until the complete root zone of the plants has been restored to field capacity. Therefore, the amount of water required to refill the root zone is equal to the amount of water that has been consumed by the crop since the last recharge event. If the water added to the soil does not refill the soil profile to replace the water consumed by the plants, the water table will not rise in response to an irrigation. The amount of the irrigation water entering the soil, as well as the storage space in the soil is expressed as a volume of water per unit area of soil, which is equivalent to a depth of water. When the ET since the last recharge event, expressed as a depth of water, is subtracted from the average depth of irrigation water infiltrating the soil (volume per unit area excluding any runoff), the difference will be a depth of water that will cause the water table to rise. The actual water table rise that will occur is the depth of recharge water divided by the *specific* *yield* of the soil. Specific yield is explained in Section 9.5.6, where it is called *drainable porosity*.

The assumption made for transient flow designs is that any recharge causes an instantaneous rise of the water table on the day of recharge. Periods of one day are used in the computations. The day of recharge is the last day of the discharge from the previous recharge event and the zero day for computation for the rate of fall of the water table until the next recharge event.

The transient design procedure presented herein follows closely that developed by Dumm (1968) and others of the U.S. Bureau of Reclamation (USBR, 1993). This design procedure is based upon a concept of dynamic equilibrium wherein the annual cycle of water table fluctuations is relatively consistent from year to year when adequate drainage is achieved for proper water table levels for the crops grown. In some cases this dynamic equilibrium is reached without construction of buried pipe drains, but in many situations natural drainage is inadequate to provide dynamic equilibrium and subsurface drains are required.

Glover and Dumm (USBR, 1993), using heat flow analogies which neglect convergence of flow toward the drains, presented an equation with the initial water table given by a fourth degree parabola:

$$y(x,0) = (8/S^4)\,[\mathrm{y}(S/2,0)]\,(S^3x - 3S^2x^2 + 4Sx^3 - 2x^4) \quad \text{for } 0 \le x \le S \qquad (9.8)$$

where the coordinates $y(x,z)$ have their origin at the centerline of one of the drains such that $y = 0$ when $z = 0$, y is the water table height above the centerlines of two parallel relief drains taken from the middle of a larger set of parallel relief drains as was done for steady state relief drains (see Section 9.4.1), and S is the drain spacing. By truncating the resulting infinite series solution and using only the first term of the series, an approximate equation results:

$$h/h_0 = (36.37/\pi^3)\exp(-\pi K\bar{D}\,t/[fS^2]) \qquad (9.9)$$

where $\bar{D}$ = the time averaged flow depth for any drainout period assumed to be given as $\bar{D} = D + \mathrm{y}_0/2$ at any location x (at the midpoint between the two drains, $\bar{D} = D + h_0/2$)

h_0 = height of the water table above the drains midway between the two drains at the start of a water table recession or drainout period

h = height of the water table above the drains midway between the two drains at any time t after the start of a drainout period

K = soil hydraulic conductivity

f = specific yield

S = spacing between the two parallel drains.

Solving Equation 9.9 for the drain spacing gives:

$$S = \pi\left\{K\bar{D}t\,/\left[f\,\ln(36.37h_0\,/(\pi^3h))\right]\right\}^{1/2} \qquad (9.10)$$

Van Schilfgaarde (1965) stated that correction for the convergence effect can be made by applying Hooghoudt's equivalent depth concept. There is nothing inherent in the concept of equivalent depth that restricts its application to steady flow. Thus, it can be expected to apply to spacing calculations based upon various transient theories that neglect convergence of flow toward the drains. The Moody (1966) equations for D_e

(Equations 9.4, 9.5, and 9.6) will be used here. Thus, replacing $\bar{D}$ with $\bar{D}_e$ to correct for convergence of flow toward the drains, Equation 9.10 becomes

$$S = \pi\left\{K\bar{D}_e t / \left[f \ln\left(36.37h_0 / \left(\pi^3 / h\right)\right)\right]\right\}^{1/2} \tag{9.11}$$

The equation for drain spacing if the drains sit on an impermeable barrier is (Dumm, 1968):

$$S = \left\{9Kh_0 t / \left[2f(h_0 / h) - 1\right]\right\}^{1/2} \tag{9.12}$$

Dumm (1968) presented a summary of the USBR approach for designing pipe drain systems using a transient equation. He reported that Australian and Canadian research workers and a USBR project provided data for checking the validity of the mathematical developments of the transient flow theory under various field conditions. The reader should refer to Tables 1 and 2 and Figures 2, 3, and 4 of the paper (Dumm, 1968) for these comparisons.

The concept of dynamic equilibrium as it relates to drainage is illustrated by Figure 9.5 of Dumm (1968) and by Figure 5-3 of the USBR Drainage Manual (USBR, 1993). In addition to the earlier discussion of the dynamic equilibrium concept, the main elements of this concept are explained as follows. When recharge and discharge become equal, the highest level and the range in cyclic annual fluctuation of the water table become reasonably constant from year to year. The water table rises during the irrigation season and reaches its highest elevation after the last irrigation. When year-around cropping is practiced, this usually occurs at the end of the peak portion of the irrigation season.

Dumm (1968), gave equations for the transient drain discharge. Replacing $\bar{D}$ with $\bar{D}_e$ in the equation for drains above the barrier gives:

$$Q = 2\pi K\bar{D}_e / S \tag{9.13}$$

and for drains on the barrier:

$$Q = 4Kh^2 / S \tag{9.14}$$

in which Q is the drain discharge in m^3/day per lineal meter of drain, K is in m/day, and $\bar{D}_e$, S, and h are in m. Water balance calculations were shown in Tables 5 and 6 of Dumm (1968) by averaging the beginning and end of each drainout period. Note that Q is a discharge rate and is a function of time because h is a function of time. See comparisons in Figures 9, 10, 11, and 12 of Dumm (1968).

Information needed for the USBR approach include estimates of deep percolation losses for each irrigation, which is given in Table 9.1, and methods for estimating specific yield, f. In estimating the specific yield, hysteresis is ignored in the water content as a function of capillary pressure. The USBR approach is to estimate specific yield from the hydraulic conductivity determined by the auger hole method. Figure 9.5 shows a curve adapted from USBR (1993) for obtaining the specific yield.

9.4.1.4 Transient relief drain design example. Perhaps the best way to explain the design process for relief drains using transient flow theories is by a detailed example. Some of the steps involved are the same as for steady state design of relief drains. First, choose a trial drain depth. Next, from the root zone depth to be protected and the trial drain depth, determine the maximum allowable height of the water table above the drains midway between the drains (maximum h or h_0). Then, from the trial drain

Table 9.1. Approximate deep percolation loss below the root zone for use in drain spacing calculations (adapted from Table 3 of Dumm, 1968).[a]

On Basis of Texture			
Texture	Deep Percolation Percentage[b]	Texture	Deep Percolation Percentage[b]
Loamy sand	30	Sandy clay loam	14
Sandy loam	26	Clay loam	10
Loam	22	Silty clay loam	6
Silt loam	18	Sandy clay, and clay	6
On Basis of Infiltration Rate			
Infiltration Rate (mm/hr)	Deep Percolation Percentage[b]	Infiltration Rate (mm/hr)	Deep Percolation Percentage[b]
1.27	3	25.40	20
2.54	5	31.75	22
5.08	8	38.10	24
7.62	10	50.80	28
10.16	12	63.50	31
12.70	14	76.20	33
15.24	16	101.60	37
20.32	18		

[a] These deep percolation percentages are usually adequate to provide for effective leaching to maintain salt balance in the root zone. However, if the leaching requirement is larger than the percentage given in the table, the larger value should be used in drain spacing calculations.

[b] Percentage is based on the application or net input.

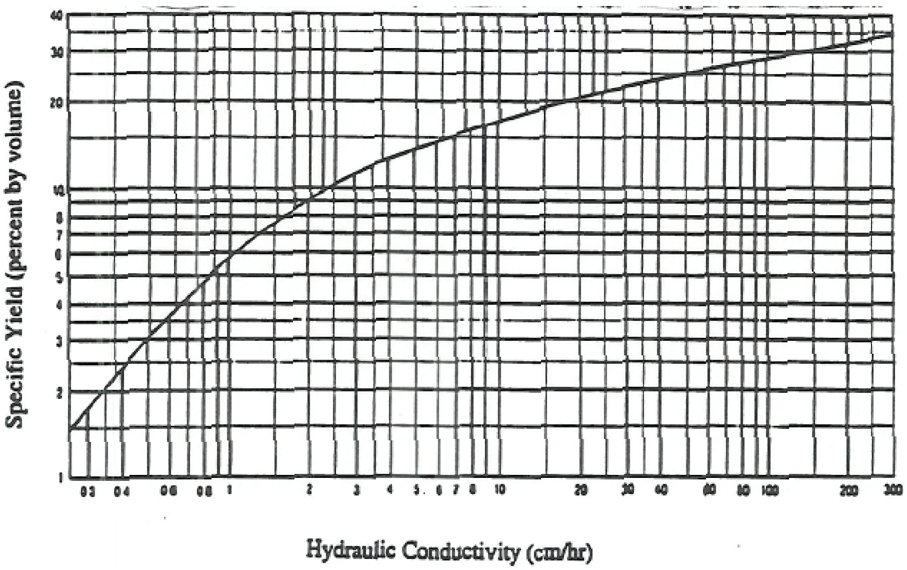


Figure 9.5. Curve used by USBR to estimate specific yield from hydraulic conductivity (adapted from Figure 2-4, USBR, 1993).

Table 9.2. Seasonal ET of sugar beets used for the design example.

Period	ET (mm/day)	No. of Days
151 May – 31 May	3.0	17
1 June – 10 June	5.1	10
11 June – 27 June	5.6	17
28 June – 5 Sept	6.4	70
6 Sept – 25 Sept	5.6	20
26 Sept – 15 Oct	5.1	20

depth and the depth of the impermeable barrier below ground surface, determine *D*. Also, choose a trial drain pipe diameter and envelope thickness to select the effective drain radius *r*. From this point on, additional information must be obtained for transient design beyond that required for steady state design.

For transient design, an irrigation schedule must be developed as well as a schedule for all water additions to the soil surface, such as rainfall and/or snowmelt, that will be significant in raising the water table. An ET schedule for water use from the soil root zone must be developed. A criterion must be developed to estimate how much water is used before irrigation is complete. Deep percolation of water below the root zone for each water addition must also be determined. The specific yield of the soil must be determined so that it can be used to estimate the amount of water table buildup (rise) that will be caused by each water addition to the water table. Once all these calculations have been done, the actual calculations of drain spacing can be undertaken. Again, computer spreadsheets will be used for this purpose.

Now an example will be given. The details and assumptions given herein comprise just one possible scenario. The scenario used for any specific design must match as close as possible the common irrigation and farming practices for the area. If this example does not match practices for the area where the designer is working, modifications will be required to obtain the best schedule for the specific area. For this example, the silt loam soil has an average hydraulic conductivity of 20 mm/hr as measured by the auger hole method and an available water content of 183 mm/m. The depth from ground surface to the barrier is 12.2 m and the root zone depth for the crop of sugar beets is 1.2 m. Choose a trial drain depth of 2.4 m. To develop an irrigation schedule to be used for design, it is decided that the first irrigation will occur on 15 May and ET prior to 15 May is negligible. The design distribution of ET for the growing season is given in Table 9.2.

For the distribution of the sugar beet roots, it is assumed that 40% of the total ET comes from the top quarter of the root zone (top 0.3 m). The irrigation schedule will be based upon use of 75% of the available water in the top quarter of the root zone (top 0.3 m) between irrigations (0.75×183 mm/m $\times 0.3$ m = 41 mm). It is assumed that each irrigation application more than fills the root zone to field capacity. Since 40% of the total ET comes from the top quarter of the root zone, then the total root zone ET per irrigation equals 41/0.40 or 103 mm. Assume that the ET on an irrigation day is extracted as part of the drainout period following that irrigation. Now, 17 days at 3.0 mm/day = 51 mm of water and 103 – 51 = 52 mm can still be used before the next irrigation is needed. The next 10 days at 5.1 mm/day uses 51 mm of water and the total use since the irrigation on 15 May has been 51 + 51 = 102 mm for the first 27 days. One more day (11 June) will use 6 mm, bringing the total to 108 mm since 15 May. This exceeds the allowable use of 103 mm, hence the next (2nd) irrigation will be on 11 June. These calculations are continued (see Table 9.3) to produce the irrigation schedule. Thus, there are nine irrigations scheduled for 15 May, 11 June, 29 June, 15 July, 31 July, 16 August, 1 September, 18 September, and 7 October covering a total span of 145 days (27 + 18 + 16 + 16 + 16 + 16 + 17 + 19). The nonirrigation season then lasts for 365 – 145 = 220 days.

Weather records of the area show that the average year will produce about 25 mm of water from snowmelt, which will recharge to the groundwater below the water table on 20 February. The elapsed time from this snowmelt event until the first irrigation on 15 May is 84 days. The rest of the nonirrigation season amounts to 136 days (220 – 84). Because of possible errors using excessively long drainout time periods, the USBR (1993) recommends splitting this part of the nonirrigation season into two nearly equal drainout time periods. Thus, the 136 days is split into two drainout periods of 68 days each.

The next steps involve calculation of the deep percolation losses from the irrigations and the water table buildup (rise) from each of the water additions. It is assumed that the silt loam soil has an infiltration rate equal to its hydraulic conductivity. From Table 9.1, for the silt loam soil with an infiltration rate of 20 mm/hr, the deep percolation from each irrigation is estimated at 18%. The net water depth added to the groundwater is then obtained from the 103 mm of water added at each irrigation as follows:

Net input = 103/ (1 – 0.18) = 103/0.82 = 126 mm per irrigation.

Deep percolation = 126 –103 = 23 mm per irrigation (or 18% of 126).

From Figure 9.5 with $K = 0.02$ m/h, the specific yield is 9% ($f = 0.09$).

Thus for each irrigation, the water table buildup is 23/0.09 = 256 mm or 0.256 m. For the snowmelt the buildup is 25/0.09 = 278 mm or 0.278 m.

Table 9.3. Details of irrigation schedule calculations.

First irrigation on 15 May (17)(3.0) = 51 mm (10)(5.1) = 51 mm 27 days 102 mm	**Second irrigation on 11 June** (17)(5.6) = 95 mm 103 – 95 = 8 mm, 8/6.4 = 1.3 days (1)(6.4) = 6 mm 18 days 101 mm
Third irrigation on 29 June 103/6.4 = 16.1 days (16 days)(6.4) = 102 mm	**Fourth irrigation on 15 July** 103/6.4 = 16.1 days (16 days)(6.4) = 102 mm
Fifth irrigation on 31 July 103/6.4 = 16.1 days (16 days)(6.4) = 102 mm	**Sixth irrigation on 16 August** 103/6.4 = 16.1 days (16 days)(6.4) = 102 mm
Seventh irrigation on 1 September (5)(6.4) = 32 mm 103 – 32 = 71 mm, 71/5.6 = 12.7 days (12)(5.6) = 67 mm 17 days 99 mm	**Eighth irrigation on 18 September** (8)(5.6) = 45 mm 103 – 45 = 58 mm, 58/5.1 = 11.4 days (11)(5.1) = 56 mm 19 days 101 mm
Ninth irrigation on 7 October (9)(5.1) = 46 mm, so a tenth irrigation is unnecessary and ET beyond 15 October is neglected due to beginning of harvest on 16 October.	

As discussed in Section 9.4.1, the USBR approach is based upon a concept of dynamic equilibrium in which the water table fluctuates during the irrigation season, but generally rises and reaches its highest elevation immediately after the last irrigation. Spreadsheets are now used to calculate the proper drain spacing. This example uses the spreadsheet with filename USBR. This spreadsheet is designed for the USBR transient flow approach using the USBR equation for drains above the barrier. The spreadsheet uses the Moody equations for Hooghoudt equivalent depth.

Figure 9.6 shows the first spreadsheet calculation using the input values for this example and a trial drain spacing of 150 m. The instructions for spreadsheet use are included as part of the spreadsheet. An effective drain radius r of 0.15 m (a 10-cm drain diameter with 10-cm thick gravel envelope) is used. Also the trial drain depth is 2.4 m (below ground surface) so that $D = 12.2 - 2.4 = 9.8$ m. The choice of this first trial drain spacing is not critical since we can use linear interpolation or extrapolation after two trials to get values much closer to the correct drain spacing. Note that since the value of h_0 (1.473) at the bottom of Figure 9.6 is greater than the desired value (1.200), the assumed drain spacing of 150 m is too large. Choose a smaller value, say, 120 m, and recalculate using the spreadsheet. Figure 9.7 shows the resulting calculated spreadsheet. Since the last value of h_0 (1.016) is too small, the assumed drain spacing (120 m) is too small. The designer may interpolate between the two calculated drain spacings (150 m and 120 m) to obtain a new trial value of drain spacing (132 m). Instead, once the spreadsheet has been constructed, the designer may just wish to enter new trials of S into cell A18 until the last value of h_0 matches the desired value (1.200). This latter approach was used to produce Figure 9.8 showing the correct drain spacing to be 132.6 m, rounded to 133 m.

9.4.1.5 Discussion of relief drain design examples. Because of all the data requirements and many assumptions upon which the transient method rests, it would be interesting to know how close results of using a simple approach such as Hooghoudt's steady state equation would be to those of the transient method. The spreadsheet HODESNL (Figure 9.3) can be used to calculate drain spacing for comparison. The question arises as to how to choose the value of the drainage coefficient, q. Suppose q is calculated as the average recharge rate for the peak irrigation period, i.e., when the interval between irrigations is the shortest. Then for the transient example given in Section 9.4.1,

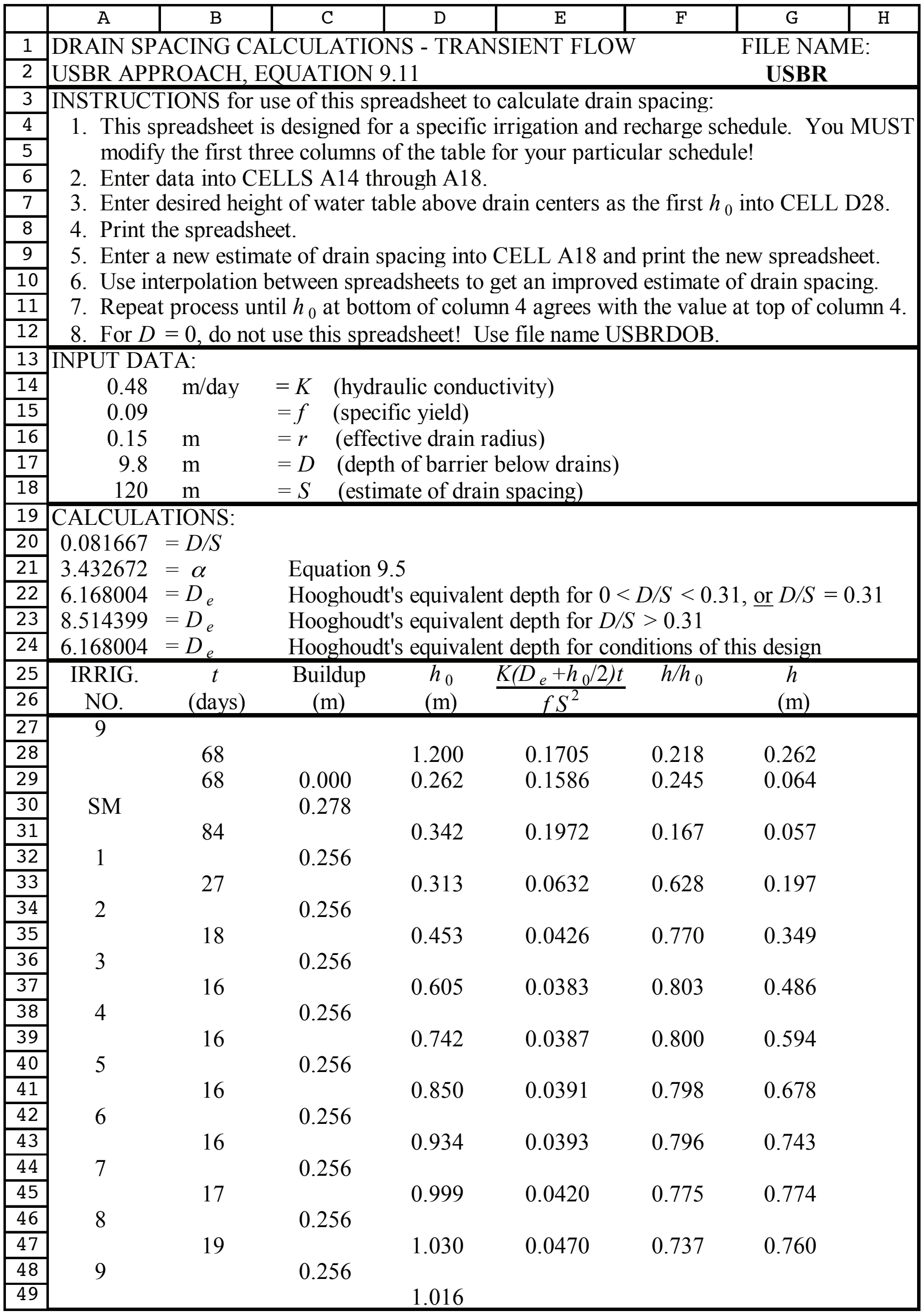
$$q = (0.023 \text{ m})/(16 \text{ days}) = 0.00144 \text{ m/day}$$

The calculation of drain spacing with this value of q in Hooghoudt's equation as shown in Figure 9.9 gives a value of 152.9 m for drain spacing S. This spacing is 15% larger than given by the transient method of calculation (Figure 9.8).

Now assume that the transient method gives a good description of water table position throughout the irrigation season. The spreadsheet USBR (Figure 9.6) can be used to find a starting value of h_0, to satisfy dynamic equilibrium conditions so that the effect on water table position throughout the year from drains placed at the spacing calculated from the Hooghoudt equation can be predicted. This is done by trial and error until the starting and ending values of h_0 in column D match for a drain spacing of 153 m as given by the Hooghoudt equation (Figure 9.9). This result is shown in Figure 9.10 where the dynamic equilibrium value of h_0, is given as 1.527 m for a drain spacing of 153 m.

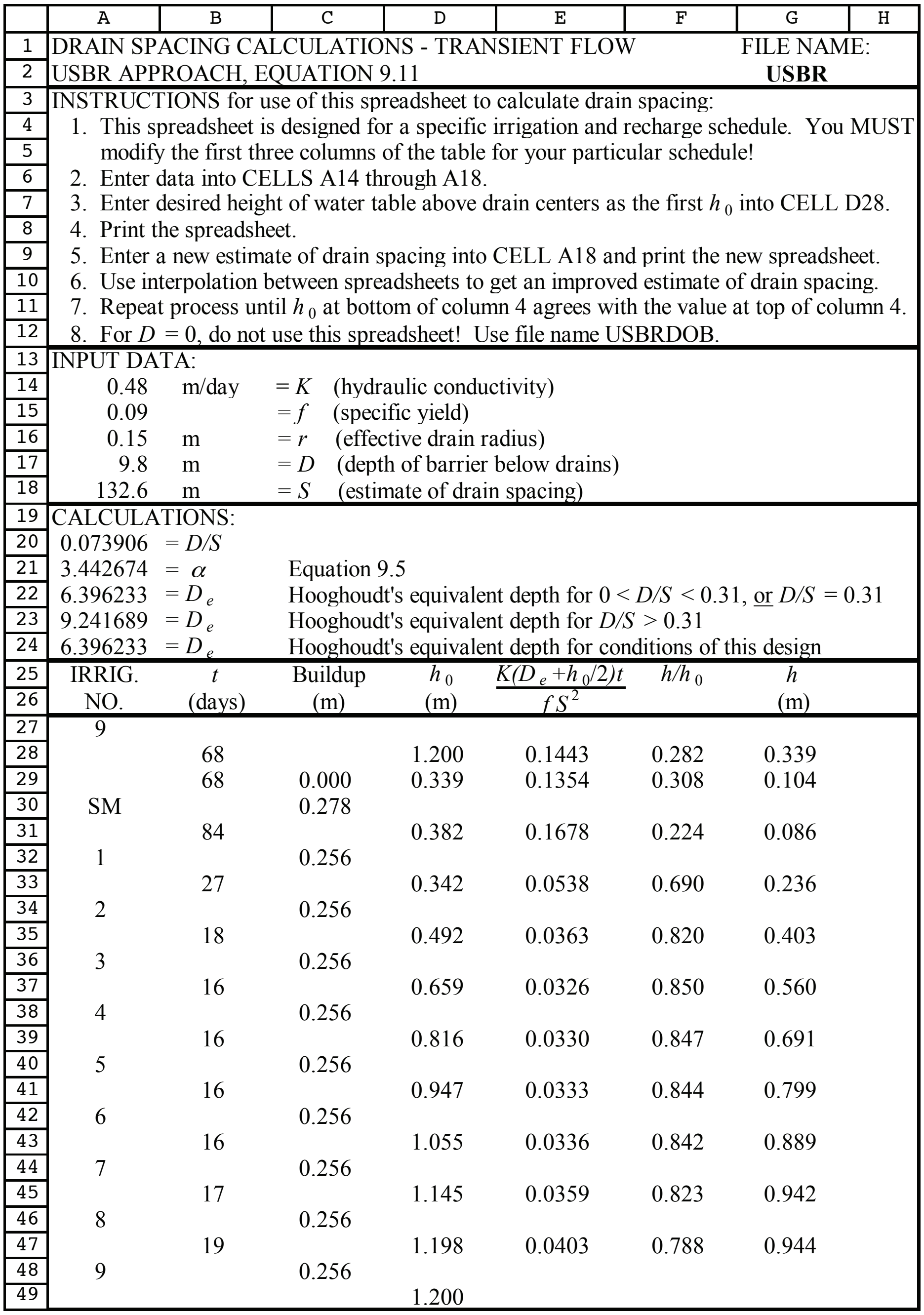
	A	B	C	D	E	F	G	H
1	DRAIN SPACING CALCULATIONS - TRANSIENT FLOW						FILE NAME:	
2	USBR APPROACH, EQUATION 9.11						**USBR**	
3	INSTRUCTIONS for use of this spreadsheet to calculate drain spacing:							
4	1. This spreadsheet is designed for a specific irrigation and recharge schedule. You MUST							
5	modify the first three columns of the table for your particular schedule!							
6	2. Enter data into CELLS A14 through A18.							
7	3. Enter desired height of water table above drain centers as the first h_0 into CELL D28.							
8	4. Print the spreadsheet.							
9	5. Enter a new estimate of drain spacing into CELL A18 and print the new spreadsheet.							
10	6. Use interpolation between spreadsheets to get an improved estimate of drain spacing.							
11	7. Repeat process until h_0 at bottom of column 4 agrees with the value at top of column 4.							
12	8. For D = 0, do not use this spreadsheet! Use file name USBRDOB.							
13	INPUT DATA:							
14	0.48	m/day	= K (hydraulic conductivity)					
15	0.09		= f (specific yield)					
16	0.15	m	= r (effective drain radius)					
17	9.8	m	= D (depth of barrier below drains)					
18	150	m	= S (estimate of drain spacing)					
19	CALCULATIONS:							
20	0.065333	= D/S						
21	3.454004	= α	Equation 9.5					
22	6.668109	= D_e	Hooghoudt's equivalent depth for 0 < D/S < 0.31, or D/S = 0.31					
23	10.23053	= D_e	Hooghoudt's equivalent depth for D/S > 0.31					
24	6.668109	= D_e	Hooghoudt's equivalent depth for conditions of this design					
25	IRRIG.	t	Buildup	h_0	$K(D_e+h_0/2)t$	h/h_0	h	
26	NO.	(days)	(m)	(m)	fS^2		(m)	
27	9							
28		68		1.200	0.1172	0.369	0.443	
29		68	0.000	0.443	0.1110	0.392	0.174	
30	SM		0.278					
31		84		0.452	0.1373	0.303	0.137	
32	1		0.256					
33		27		0.393	0.0439	0.760	0.299	
34	2		0.256					
35		18		0.555	0.0296	0.876	0.486	
36	3		0.256					
37		16		0.742	0.0267	0.901	0.668	
38	4		0.256					
39		16		0.924	0.0270	0.898	0.830	
40	5		0.256					
41		16		1.086	0.0273	0.896	0.973	
42	6		0.256					
43		16		1.229	0.0276	0.893	1.097	
44	7		0.256					
45		17		1.353	0.0296	0.876	1.185	
46	8		0.256					
47		19		1.441	0.0333	0.845	1.217	
48	9		0.256					
49				1.473				

Figure 9.6. Spreadsheet using the USBR transient approach to calculate parallel relief drain spacing showing first trial drain spacing that is too large.



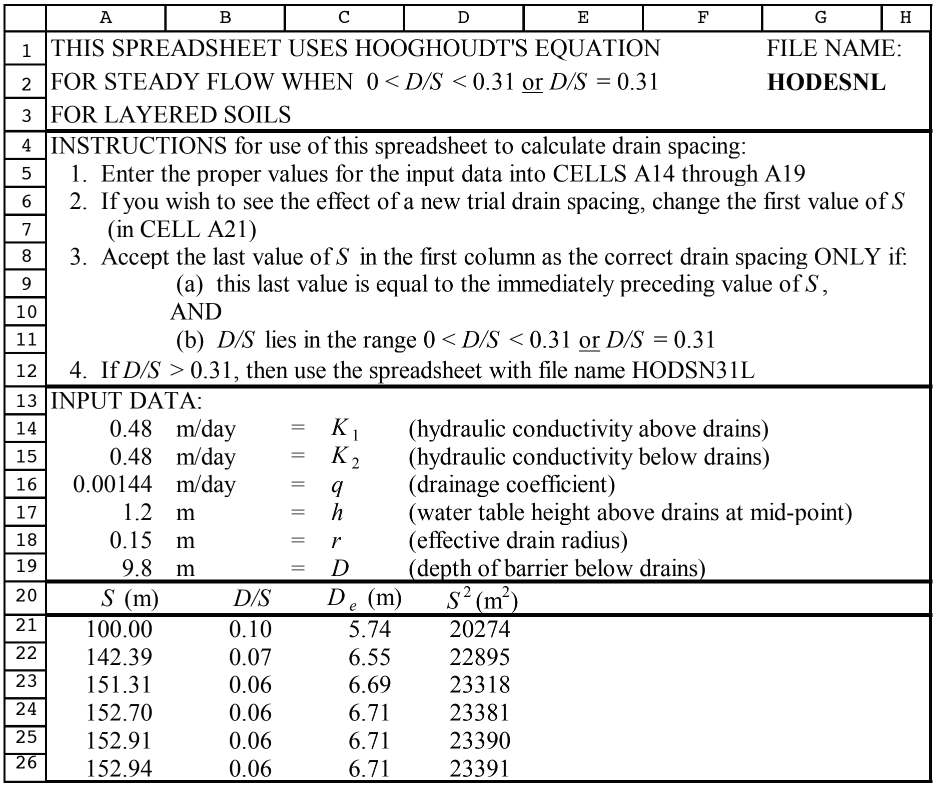
	A	B	C	D	E	F	G	H
1	DRAIN SPACING CALCULATIONS - TRANSIENT FLOW						FILE NAME:	
2	USBR APPROACH, EQUATION 9.11						**USBR**	
3	INSTRUCTIONS for use of this spreadsheet to calculate drain spacing:							
4	1. This spreadsheet is designed for a specific irrigation and recharge schedule. You MUST							
5	modify the first three columns of the table for your particular schedule!							
6	2. Enter data into CELLS A14 through A18.							
7	3. Enter desired height of water table above drain centers as the first h_0 into CELL D28.							
8	4. Print the spreadsheet.							
9	5. Enter a new estimate of drain spacing into CELL A18 and print the new spreadsheet.							
10	6. Use interpolation between spreadsheets to get an improved estimate of drain spacing.							
11	7. Repeat process until h_0 at bottom of column 4 agrees with the value at top of column 4.							
12	8. For $D = 0$, do not use this spreadsheet! Use file name USBRDOB.							
13	INPUT DATA:							
14	0.48	m/day	$= K$ (hydraulic conductivity)					
15	0.09		$= f$ (specific yield)					
16	0.15	m	$= r$ (effective drain radius)					
17	9.8	m	$= D$ (depth of barrier below drains)					
18	120	m	$= S$ (estimate of drain spacing)					
19	CALCULATIONS:							
20	0.081667	$= D/S$						
21	3.432672	$= \alpha$	Equation 9.5					
22	6.168004	$= D_e$	Hooghoudt's equivalent depth for $0 < D/S < 0.31$, <u>or</u> $D/S = 0.31$					
23	8.514399	$= D_e$	Hooghoudt's equivalent depth for $D/S > 0.31$					
24	6.168004	$= D_e$	Hooghoudt's equivalent depth for conditions of this design					
25	IRRIG.	t	Buildup	h_0	$\frac{K(D_e+h_0/2)t}{fS^2}$	h/h_0	h	
26	NO.	(days)	(m)	(m)			(m)	
27	9							
28		68		1.200	0.1705	0.218	0.262	
29		68	0.000	0.262	0.1586	0.245	0.064	
30	SM		0.278					
31		84		0.342	0.1972	0.167	0.057	
32	1		0.256					
33		27		0.313	0.0632	0.628	0.197	
34	2		0.256					
35		18		0.453	0.0426	0.770	0.349	
36	3		0.256					
37		16		0.605	0.0383	0.803	0.486	
38	4		0.256					
39		16		0.742	0.0387	0.800	0.594	
40	5		0.256					
41		16		0.850	0.0391	0.798	0.678	
42	6		0.256					
43		16		0.934	0.0393	0.796	0.743	
44	7		0.256					
45		17		0.999	0.0420	0.775	0.774	
46	8		0.256					
47		19		1.030	0.0470	0.737	0.760	
48	9		0.256					
49				1.016				

Figure 9.7. Spreadsheet using the USBR transient approach to calculate parallel relief drain spacing showing second trial drain spacing that is too small.



	A	B	C	D	E	F	G	H
1	DRAIN SPACING CALCULATIONS - TRANSIENT FLOW						FILE NAME:	
2	USBR APPROACH, EQUATION 9.11						**USBR**	
3	INSTRUCTIONS for use of this spreadsheet to calculate drain spacing:							
4	1. This spreadsheet is designed for a specific irrigation and recharge schedule. You MUST							
5	modify the first three columns of the table for your particular schedule!							
6	2. Enter data into CELLS A14 through A18.							
7	3. Enter desired height of water table above drain centers as the first h_0 into CELL D28.							
8	4. Print the spreadsheet.							
9	5. Enter a new estimate of drain spacing into CELL A18 and print the new spreadsheet.							
10	6. Use interpolation between spreadsheets to get an improved estimate of drain spacing.							
11	7. Repeat process until h_0 at bottom of column 4 agrees with the value at top of column 4.							
12	8. For $D = 0$, do not use this spreadsheet! Use file name USBRDOB.							
13	INPUT DATA:							
14	0.48	m/day	= K (hydraulic conductivity)					
15	0.09		= f (specific yield)					
16	0.15	m	= r (effective drain radius)					
17	9.8	m	= D (depth of barrier below drains)					
18	132.6	m	= S (estimate of drain spacing)					
19	CALCULATIONS:							
20	0.073906	= D/S						
21	3.442674	= α	Equation 9.5					
22	6.396233	= D_e	Hooghoudt's equivalent depth for $0 < D/S < 0.31$, or $D/S = 0.31$					
23	9.241689	= D_e	Hooghoudt's equivalent depth for $D/S > 0.31$					
24	6.396233	= D_e	Hooghoudt's equivalent depth for conditions of this design					
25	IRRIG.	t	Buildup	h_0	$\frac{K(D_e+h_0/2)t}{fS^2}$	h/h_0	h	
26	NO.	(days)	(m)	(m)			(m)	
27	9							
28		68		1.200	0.1443	0.282	0.339	
29		68	0.000	0.339	0.1354	0.308	0.104	
30	SM		0.278					
31		84		0.382	0.1678	0.224	0.086	
32	1		0.256					
33		27		0.342	0.0538	0.690	0.236	
34	2		0.256					
35		18		0.492	0.0363	0.820	0.403	
36	3		0.256					
37		16		0.659	0.0326	0.850	0.560	
38	4		0.256					
39		16		0.816	0.0330	0.847	0.691	
40	5		0.256					
41		16		0.947	0.0333	0.844	0.799	
42	6		0.256					
43		16		1.055	0.0336	0.842	0.889	
44	7		0.256					
45		17		1.145	0.0359	0.823	0.942	
46	8		0.256					
47		19		1.198	0.0403	0.788	0.944	
48	9		0.256					
49				1.200				

Figure 9.8. Spreadsheet using the USBR transient approach to calculate parallel relief drain spacing showing correct drain spacing obtained by successive trials of *S*.



	A	B	C	D	E	F	G	H
1	THIS SPREADSHEET USES HOOGHOUDT'S EQUATION						FILE NAME:	
2	FOR STEADY FLOW WHEN $0 < D/S < 0.31$ or $D/S = 0.31$						**HODESNL**	
3	FOR LAYERED SOILS							
4	INSTRUCTIONS for use of this spreadsheet to calculate drain spacing:							
5	1. Enter the proper values for the input data into CELLS A14 through A19							
6	2. If you wish to see the effect of a new trial drain spacing, change the first value of S							
7	(in CELL A21)							
8	3. Accept the last value of S in the first column as the correct drain spacing ONLY if:							
9	(a) this last value is equal to the immediately preceding value of S,							
10	AND							
11	(b) D/S lies in the range $0 < D/S < 0.31$ or $D/S = 0.31$							
12	4. If $D/S > 0.31$, then use the spreadsheet with file name HODSN31L							
13	INPUT DATA:							
14	0.48	m/day	= K_1	(hydraulic conductivity above drains)				
15	0.48	m/day	= K_2	(hydraulic conductivity below drains)				
16	0.00144	m/day	= q	(drainage coefficient)				
17	1.2	m	= h	(water table height above drains at mid-point)				
18	0.15	m	= r	(effective drain radius)				
19	9.8	m	= D	(depth of barrier below drains)				
20	S (m)	D/S	D_e (m)	S^2 (m^2)				
21	100.00	0.10	5.74	20274				
22	142.39	0.07	6.55	22895				
23	151.31	0.06	6.69	23318				
24	152.70	0.06	6.71	23381				
25	152.91	0.06	6.71	23390				
26	152.94	0.06	6.71	23391				

Figure 9.9. Drain spacing estimated from Hooghoutd's equation with q as the average recharge rate for the peak irrigation period of this example.

Careful study of Figure 9.10 shows that after irrigations 6, 7, 8, and 9, h_0 exceeds the desired value of 1.200 m and h exceeds this value at the ends of the drainout periods following irrigations 7 and 8. At all other times of the year, the water table is below the 1.200 m depth as desired for the crop of this example. A designer must evaluate the economic damages of such water table positions before deciding whether one is justified in using the simpler steady state approach instead of the much more data-demanding transient approach to design the drainage system. Of course, many more sets of conditions and situations than considered here would have to be evaluated before one could reach any general conclusions on this question. Further work is left to the individual designer.

Of course, as was mentioned in Section 9.4.1 on steady state design, the designer must complete the work by ensuring that the trial drain pipe diameter is sufficient to carry the expected drain discharge (refer to Equation 9.7). One could also develop a spreadsheet for the special transient design case where the drains rest upon the impermeable barrier using Equation 9.1.

The designer is cautioned to use the design tools presented here with judgment tempered by experience. Especially with the transient approach, an appropriate irrigation schedule must be built into the spreadsheet to handle the specific situation for the design conditions at hand.

	A	B	C	D	E	F	G	H
1	DRAIN SPACING CALCULATIONS - TRANSIENT FLOW						FILE NAME:	
2	USBR APPROACH, EQUATION 9.11						**USBR**	
3	INSTRUCTIONS for use of this spreadsheet to calculate drain spacing:							
4	1. This spreadsheet is designed for a specific irrigation and recharge schedule. You MUST							
5	modify the first three columns of the table for your particular schedule!							
6	2. Enter data into CELLS A14 through A18.							
7	3. Enter desired height of water table above drain centers as the first h_0 into CELL D28.							
8	4. Print the spreadsheet.							
9	5. Enter a new estimate of drain spacing into CELL A18 and print the new spreadsheet.							
10	6. Use interpolation between spreadsheets to get an improved estimate of drain spacing.							
11	7. Repeat process until h_0 at bottom of column 4 agrees with the value at top of column 4.							
12	8. For $D = 0$, do not use this spreadsheet! Use file name USBRDOB.							
13	INPUT DATA:							
14	0.48	m/day	$= K$ (hydraulic conductivity)					
15	0.09		$= f$ (specific yield)					
16	0.15	m	$= r$ (effective drain radius)					
17	9.8	m	$= D$ (depth of barrier below drains)					
18	153	m	$= S$ (estimate of drain spacing)					
19	CALCULATIONS:							
20	0.064052	$= D/S$						
21	3.455722	$= \alpha$	Equation 9.5					
22	6.710662	$= D_e$	Hooghoudt's equivalent depth for $0 < D/S < 0.31$, <u>or</u> $D/S = 0.31$					
23	10.39937	$= D_e$	Hooghoudt's equivalent depth for $D/S > 0.31$					
24	6.710662	$= D_e$	Hooghoudt's equivalent depth for conditions of this design					
25	IRRIG.	t	Buildup	h_0	$K(D_e+h_0/2)t$	h/h_0	h	
26	NO.	(days)	(m)	(m)	fS^2		(m)	
27	9							
28		68		1.527	0.1158	0.374	0.571	
29		68	0.000	0.571	0.1084	0.402	0.230	
30	SM		0.278					
31		84		0.508	0.1333	0.315	0.160	
32	1		0.256					
33		27		0.416	0.0426	0.771	0.320	
34	2		0.256					
35		18		0.576	0.0287	0.884	0.509	
36	3		0.256					
37		16		0.765	0.0259	0.909	0.696	
38	4		0.256					
39		16		0.952	0.0262	0.906	0.862	
40	5		0.256					
41		16		1.118	0.0265	0.903	1.009	
42	6		0.256					
43		16		1.265	0.0268	0.901	1.140	
44	7		0.256					
45		17		1.396	0.0287	0.884	1.233	
46	8		0.256					
47		19		1.489	0.0323	0.853	1.271	
48	9		0.256					
49				1.527				

Figure 9.10. Dynamic equilibrium at a drain spacing of 153 m for this example.

9.4.2 Interceptor Drains

Interceptor drains are single drains installed nearly along a ground surface contour at a depth to intercept subsurface water moving downslope from higher lands. In contrast, relief drainage assumes that all the water removed by the drains originates from the soil surface above the system of drains and that there is no lateral movement of water into the area drained.

Interceptor drains can be open drains or buried pipes. Both kinds are equally effective if functioning as designed. We prefer the buried pipe interceptors. The main advantages of buried pipe interceptors are no loss of farmed area, no restriction to equipment movements, no weeds, and low maintenance. Open ditch interceptors introduce problems with weeds, instability of banks, restrictions of farming operations, loss of significant farm area, and difficutly of maintaining hydraulic grade. The depth of the drains and the proportion of the subsurface flow cross-section intercepted by the drain determine the amount of water that will be collected by the drain.

The principal design factor for interceptor drains is the quantity of water entering the drains, which determines the best pipe size. The depth and location of the drain is controlled by the local site conditions, particularly the hydraulic conductivity of the soil, the slope of the groundwater table in the vicinity of the drain, and the depth from the water table to any low-permeability (restrictive) layer in the soil profile.

Darcy’s law is used for the design of interceptor drains. Consider a vertical section aligned with the direction of flow of water to be intercepted. The original flow rate under normal conditions is calculated for a unit width of soil (length of drain) in this vertical section from Equation 9.15.

$$Q = Kd(dh/dl) \tag{9.15}$$

where Q = flow rate, m^3/day per m width of soil (length of drain)
K = hydraulic conductivity, m/day
d = depth from the water table to the restrictive layer, in m
dh/dl = hydraulic gradient.

The depth of the drain is selected based on soil profile characteristics and the kind of excavating equipment available for construction. If the low-permeability layer can be reached, the drain should be placed on top of the layer. The drain should not be installed in or under a restrictive layer. The distance of the interceptor drain below the original groundwater surface divided by the original total depth of the flowing water above the restrictive layer multiplied by the calculated original flow will be the flow per unit length of interceptor. That is, the interceptor can be considered to skim all water arriving between its depth and the original (pre-construction) groundwater surface. The flow per unit length of drain can then be used to choose a pipe size.

The post-construction groundwater surface downslope of the interceptor will be parallel to the original groundwater surface and at the depth of the interceptor. The groundwater surface in the soil upslope from the drain will be at the original level beyond a distance of 10 times the depth of the drain below the original groundwater surface. The upslope drawdown curve to the drain has an approximate parabolic shape. The designer is referred to USBR (1993) for additional details on interceptor drains, including how to place additional drains downslope in an irrigated area requiring a combination of interceptor and relief drains.

Keller and Robinson (1959) presented results of an extensive sand-tank flume study of interceptor drains that can be used as a guide for design. Their results include ways of estimating the increased seepage loss from a canal when an interceptor drain is installed near the canal. The interceptor drain actually induces more seepage from the canal than existed before the drain was constructed. As long as the proper size of drain pipe is used, the interceptor drain will protect the land downslope as intended.

9.5 DETERMINING DESIGN VARIABLES

Drainage design is not an exact science because of the great variability that exists in natural soils. Frequently, the design variables needed can only be approximated, but every possible attempt should be made to use accurate and precise information. Rainfall is not constant and irrigations are not perfectly uniform, so drainage coefficients are not precise. Likewise, neither the soil hydraulic conductivity nor the depth to the impermeable barrier can be determined with a high degree of accuracy under any circumstances, so drainage design also requires the use of reasonable engineering judgment.

9.5.1 Water Table Depth

As given in Section 9.3.1, the design depth to water table at the midpoint between a system of parallel relief drains is normally taken to be 1.0 m. The USBR (1993) specifies a design depth to water table of 1.2 m. The deeper depth gives a factor of safety in areas where salinity control is important. If a steady state drain spacing is calculated using a water table depth of 1 m, a drain spacing calculated by the transient method will be less, even though the physical variables used in the calculation are identical. The preferred method is to calculate the steady state drain spacing with a water table depth of 1 m and then use the calculated steady state drain spacing to calculate the maximum height of the water table at the time of recharge using a transient design equation. The fluctuations of the water table will be controlled by the drain depth and spacing and the recharge events, and not by the design method, steady state or transient, used to determine the drain spacing.

9.5.2 Drain Depth

Drains should be installed on or above a low-permeability layer and in highly permeable layers that are at or near the proper depth. Beyond those guidelines, the depth of drains is determined primarily by the type of excavating equipment available. Plows and trenchers used to install subsurface drains have a limited depth capability, depending on their design. Open drains can be as deep as necessary, but to maintain stable side slopes, deep open drains take large areas of land out of service and create serious maintenance problems. Special trenchers have been manufactured to install buried subsurface drains to depths of 4 to 5 m, but they are uncommon. The USBR (1993) made an economic analysis of drain depth. Deeper drains can be installed at wider spacing, but excavation costs are higher. The balance between excavation costs, speed of construction, and cost of materials resulted in an optimum drain depth of approximately 2 m. Shallower drains will cost more because more materials will be needed for the closer spacing. Deeper drains will cost more because labor and excavation costs are higher. Local conditions may cause the true economic drain depth to be different.

9.5.3 Depth to Barrier

In the design of subsurface drainage systems, one of the necessary variables is the distance from the drain to some layer in the soil that would be a barrier to vertical water movement. Since permeability of the soil is a relative value, any horizontal soil layer with a hydraulic conductivity less than a tenth to a fifth of the weighted average hydraulic conductivity of all the overlying layers that are below the water table is considered to be a barrier. Work of Toksoz and Kirkham (1971a,b) can be used to evaluate this assumption of barrier depth. The designer may also use the appropriate spreadsheet in this chapter to evaluate the effects of different barrier depths on calculated drain spacing. In most drain spacing equations, the depth to barrier is replaced by an "equivalent depth" (see Section 9.4.1). Since the equivalent depth equation contains the drain spacing and the drain spacing equation contains the equivalent depth, the solution to the equations must be by trial and error. If the depth to the barrier is unknown from soil borings or well logs or some other source, the equivalent depth can be assumed to be 25% of the drain spacing.

9.5.4 Hydraulic Conductivity

In subsurface drain spacing calculations one of the most important factors is the permeability or hydraulic conductivity of the soil. Hydraulic conductivity can be determined in the field by means of an auger hole test (USBR, 1993; Matsuno, 1991) or a shallow well pump-in test (USBR, 1993). In the absence of any other information, the basic intake rate of the soil can be used as an estimate of hydraulic conductivity. It should be remembered, however, that the basic intake rate applies to the highly structured surface soil, but the water flowing to the drains is moving through the deeper soil layers.

9.5.5 Drainage Requirements

The drainage requirements and goals in humid and arid areas are different (see Section 9.1.1). In humid regions, excess water removal is the objective of subsurface drainage. The hydraulic conductivity of soils in humid areas tends to decrease with depth so that drains must be closely spaced at a shallow depth. The normal installation depth for humid area drains is 1.2 m and the selected design steady state water table height above the drains is 0.2 to 0.3 m. The design depth to the midpoint water table is therefore about 1 m. Local experience is used to decide on the drainage coefficient, which is in the range of 7 to 8 mm per day. Humid area soils should not be overdrained, i.e., drained to such depths that the plants become short of water between precipitation events or irrigations. The depths below the water table can be used as a soil water storage reservoir. Organic soils should not be drained below a depth of 0.5 m to protect the soils against accelerated oxidation.

In arid areas, the primary purpose of subsurface drainage is to keep the water table well below the soil surface so that excessive movement of salts from the water table to the root zone or the soil surface is avoided. In fine-textured soils, water can move upward by capillarity a distance of 2 m or more, so to avoid salinity problems drains must be installed at a depth of at least 2 m. In undrained arid areas, natural water tables should be below 3 m. In arid areas that are irrigated throughout the year or throughout the full growing season, drains can be installed at depths shallower than 2 m, but the land should not be allowed to stand fallow for any significant period of time or the surface will become salinized from capillary rise of water. With relatively frequent irrigations, salts will be leached downward and will not accumulate on the soil surface. The drainage requirement can be calculated using Equation 9.1 to determine the proper drainage coefficient for use in drain spacing equations.

9.5.6 Drainable Porosity

A saturated soil, in the absence of restrictive drainage conditions, will drain to field capacity. The difference in water content of soil between saturation and field capacity is the *drainable porosity*. If the drainable porosity of the soil is 10%, adding 10 mm of water to a soil at an equilibrium field capacity water content will cause the water table to rise 100 mm. A better term to use in the design of drainage systems is *specific yield*. The definition of specific yield is the volume of water released by a unit area of soil for a unit drop in the water table. Specific yield changes with depth to the water table until the water table is deeper than about 3 m. Specific yield is correlated with hydraulic conductivity for normal drainage depths and the USBR (1993) has published a curve that can be used to estimate specific yield when the hydraulic conductivity is known. Field measurements of drain discharge as the depth to water table changes can also be used to estimate specific yield. The most common laboratory method for determining the specific yield requires interpretation of a soil water retention curve for the soil in question.

9.6 DRAINAGE SYSTEM MATERIALS

Drainage design is not finished nor can discussion of the design procedure be complete without some consideration of the materials from which the system is to be constructed. This section considers the tubing and pipe, envelope, outlet, and manholes for buried pipe drainage systems.

9.6.1 Tubing and Pipe

The first pipes used for agricultural subsurface drains were short lengths (0.3 m) of unglazed fired clay pipe with an internal diameter of 0.1 m. Since the pipes were made in the same kilns as other types of clay tiles, they were called *drain tile* and drains constructed of these rigid pipes were called *tile drains*. The reader is referred to Weaver (1964) for a more complete history of tile drains in the U.S. The pipes were merely butted together in the bottom of a trench and depended on the irregularity of the ends of the pipes to admit the drainage water. Standards were developed for strength and the pipes were used successfully for many years.

Concrete pipes also have been used for subsurface drains. They were first made in 0.3-m lengths with square ends and were called *concrete tiles*. Later, special concrete drain pipes were made in approximately 1-m lengths with tongue and groove or bell and spigot joints that fit loosely to admit water.

In the 1960s, corrugated and perforated plastic tubing, made primarily of high density polyethylene in the U.S. and polyvinyl chloride in Canada and Europe, was developed and was rapidly accepted as a substitute for clay tile and concrete pipe as subsurface drains. Today, concrete pipes are normally used for buried subsurface collector drains larger than 0.3 m and corrugated plastic tubing is the dominant material for smaller-diameter collector drains and laterals. Clay and concrete pipe are rigid pipes and have strength and loading characteristics that are very different than the flexible corrugated plastic tubing. Corrugated plastic tubing (called simply plastic drain in the following) is classified as flexible even though it feels rigid. Plastic drains must be carefully bedded to prevent being crushed by the weight of the backfill in the drain trench. If a gravel envelope is not used, plastic drains should be laid in a specially formed 90° or semicircular groove (with the same diameter as the outside of the drain tubing) made in the bottom of the trench. Otherwise, the tubing may be deformed or crushed. The full resistive strength of plastic drains can only be attained with the proper bedding described above. Placing a plastic drain in a flat-bottomed trench with loose material along the sides as backfill can result in flattening or failure of the drain. Surrounding plastic drains with a fine gravel or coarse sand envelope will provide the drain with adequate structural support in a flat-bottomed trench.

There are ASTM standards for both rigid pipes and flexible drain tubing. The standards provide specifications for strength and durability and protection of plastic drains from ultraviolet light damage. The USBR (1993) and the NRCS (2001) have drainage handbooks containing additional information about pipe and tubing strength and installation requirements.

Studies in Europe (Dierickx, 1992) have shown that the open area for entry of water into drain pipes or tubing should be 20 cm^2 per meter length as a minimum and that open area up to 50 cm^2 per meter improves drain performance. Plastic drains should have at least one perforation in every corrugation.

The advantages of plastic drains over clay or concrete pipe are light weight, ease of handling, lack of loose joints to become misaligned, low cost, and durability. Disadvantages are its low structural strength without proper bedding, susceptibility to crushing from point loads during backfilling, and greater hydraulic roughness than rigid pipes.

Any pipe that will not deteriorate while buried in the soil can be used as a drain pipe if it has adequate open area for the entry of water. Pipe perforations must be small enough to prevent the entry of sediment if the pipe is not protected by synthetic or natural envelope materials.

9.6.2 Envelope

Porous materials such as coarse sand or fine gravel are placed around drains in unstable soils to prevent movement of soil particles into the drains. These materials are called *drain envelopes*. Early in the history of buried pipe drainage these envelopes were called *filters*. A filter is exactly what we do not want to place around our drains. Since a filter is designed to trap particles, it will clog over time and prevent water movement into the drain. Conversely, a well-designed envelope of properly graded granular material will naturally “bridge” to prevent most soil particles from passing through but will allow very fine soil particles into the drain. These very fine particles will be transported on out through the system with the normal water flow. The NRCS (2001) and USBR (1993) have published criteria for the design of granular material envelopes for drains.

Soils (particularly fine sands and coarse silts) in arid regions are frequently structurally unstable and thus most subsurface drains require the use of envelopes. Drain envelopes will always improve the hydraulic functioning of a drain. Soils in humid areas tend to be structurally stable so that drain envelopes are not normally necessary.

In recent years, various synthetic fabrics called *geotextiles* have been used for drain envelopes. While they may be cheaper and easier to install, they are not as effective as granular materials. Where success with geotextiles has been reported, probably no envelope was needed. We recommend envelopes of properly graded granular materials.

9.6.3 Outlet

As stated in Section 9.1.1, the outlet(s) is the discharge point(s) of the drainage system where the water enters an existing channel or waterway. Small drainage systems may have only one outlet but larger, more complex systems usually have multiple outlets. The importance of the outlet should not be underestimated. The adequacy of the entire drainage system depends on a properly designed, constructed, and maintained outlet.

Gravity flow of all water from the drains to the receiving bodies of water is usually most desirable. When this is not possible or is difficult to do, sumps equipped with water-level controlled pumps may be used to lift the drainage effluent to the proper elevation.

Regardless of whether the drainage system uses sump pumps or discharges by gravity flow, the last section of buried pipe should be solid rigid pipe that extends over the receiving body of water. This pipe is necessary to prevent damage to the system from livestock and/or wildlife. Usually the last section of pipe is specified as corrugated metal culvert pipe of suitable diameter. To protect the outlet pipe from encroachment by muskrats or other animals, flap valves, grates, or other devices may be installed at the exit end of the pipe (NRCS, 2001). We recommend that the invert of the outlet pipe be at least 0.3 m above the high-water stage of the waterway.

9.6.4 Receiving Bodies of Water

Although formally not a part of the drainage system, the existing waterway or other receiving body of water must be carefully considered. The stage-frequency relationship of the waterway must be known, as well as its capacity to carry the drain discharge together with its normal flow. The quality of water in the receiving water body must be such that the drainage water can be safely discharged into it. Subsurface drainage outlets have been classified as point source pollution if they damage the water quality of the receiving stream.

In determining the effective elevation of a proposed drainage outlet, the elevation of the water surface throughout the year should be determined to eliminate the possibility of submerging the drain system outlet at a time when drainage is most needed.

9.6.5 Manholes

Manholes in subsurface drainage systems serve the same purpose as manholes in a sewer system; they provide access for inspection and cleaning. As a general rule, manholes should not be used in a subsurface drainage system unless absolutely necessary. Manholes interfere with cultivation if they are placed in a field. If they are left open they can be dangerous to children and animals, and are entry points for trash that may clog the drainlines. Manholes are useful at points where collector systems come together so that performance of individual systems can be observed. Rather than installing manholes at regular intervals, it is more economical to dig down to a drain with a suspected problem.

9.7 CONSTRUCTION METHODS AND EQUIPMENT

Subsurface drains are installed in trenches that may be hand dug or opened by a variety of digging machines. Most drains are installed by mechanical trenchers designed and equipped especially for drain installation. Fouss (1974) shows photographs of drainage equipment which has not changed much in recent decades except for more widespread use of laser grade control. Fouss shows a modern chain-digger trencher equipped with a shield to prevent collapse of the trench before the pipe can be installed in an unstable soil, and a drainplow that can install drains to depths of 1.5 m without making an excavation. (Drainplows are uneconomical for installations deeper than 1.5 m.) Trenching machines are classified according to the digging method used: a wheel, flexible ladder, or digging chain. The chain-type trencher is used for narrow trenches common for small diameter lateral drains. Wheel and ladder trenchers are used for the wide trenches needed for large diameter drains. Drainplows are used for small-diameter drains installed with or without geotextile envelope materials, but not with gravel/sand envelopes.

Trenches can be opened with a backhoe or other type of excavating machine, but extra effort is then needed to bring the bottom of the trench to the necessary firm uniform grade. Drain pipes should be laid straight or on gently curving uniform grades without vertical deviations greater than 15% of the pipe diameter. Vertical deviations (humps or valleys) in a drainline reduce the flow gradient and make a deadwater area upslope of the hump where sediment can collect and reduce the flow capacity of the pipe. Depressions in a drainline also collect sediment and reduce flow capacity. Over-excavation of a trench and bringing the trench to grade by throwing in loose material may result in vertical grade deviations, pipe misalignment, and structural failure of the pipe. Thus, bringing an over-excavated trench back onto grade should be done only with gravel material—an expensive process. The foregoing discussion points to the need for good grade control during the construction of drains. Controlling grade with a laser is now commonly used with drain plows and drain trenchers to assure adequate control of grade and alignment.

9.8 DRAINAGE SYSTEM OPERATION AND MAINTENANCE

Subsurface drainage systems with gravity outlets operate continuously and automatically with minimum maintenance. If the outlet is a sump with a pump, then continuous maintenance of that part of the system is necessary to assure adequate drainage. The operation of the system is fixed at the time of the design and consists of the design rate of water removal or the rate of lowering of the water table at the midpoint between drains. If the drain flows respond to irrigations or rainfalls in excess of the water storage capacity of the soil, the drains are assumed to be operating properly. The appearance of new wet spots near a drainline in a field is an indication that the drain is clogged at that point. The drain can then be excavated, examined, and repaired. The crop can be watched for indications of inadequate drainage, such as salt accumulations on the soil surface or differences in crop growth (poor growth in wet periods or good crop growth in dry periods). In periods of water shortage, plant roots can invade and clog the drains. Sugar beet, cotton, willows, poplars, and deep-rooted desert plants can clog drains in a relatively short period during times of water shortage. Use of rigid drain pipes is recommended if such conditions are expected.

Maintenance of buried subsurface rigid-pipe drains is primarily accomplished by high-pressure water jetting machines similar to sewer cleaners used in the city (Grass et al., 1975). Sections of drains up to 200 m long can be easily cleared of roots, sediment, and iron ochre deposits by jet cleaning. Pump pressures are approximately 4 MPa. The drains are cleaned in sections from excavated access holes. After an appropriate plug is placed into the downstream section of drain at the access hole, cleaning is done in an upstream direction from this plug and the water and debris are pumped out of the access hole. When cleaning of this section is finished, the drain pipe and envelope are repaired and the access hole is filled. Cleaning costs are usually 20% or less than the cost of installing a new drain.

9.9 DRAINAGE SYSTEM PERFORMANCE EVALUATION

Drainage system performance criteria are fixed at the time of design. Drain depths are set by the capacity of the available digging machines, the requirements of the area, and the stratification of the soil. Drains should not be installed in or below soil layers that are slowly permeable. The minimum allowable depth to the water table at the midpoint between drains is fixed by local experience and is usually approximately 1 m. The drainage requirement or design drainage coefficient determines the rate of water removal necessary and then the soil hydraulic conductivity is used in a suitable equation to determine the appropriate drain spacing. If the soil drains acceptably, the performance criteria are assumed to be met. If the soil does not drain properly, further investigation will be needed to determine whether the drain is faulty or whether additional drains are necessary.

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