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DESIGN AND OPERATION OF GRAVITY OR SURFACE SYSTEMS

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13.1 BASIC CONSIDERATIONS

13.1.1 General Description

Surface irrigation uses open channel flow to spread water over a field. The driving force in such systems is gravity and hence the alternate term, gravity flooding. Once distributed over the surface of the field and after it has entered the soil, water is often redistributed by forces other than gravity.

Surface irrigation systems generally require a smaller initial investment than do other types of irrigation systems. However, this is not always the case, especially if extensive land forming is needed for an efficient system. In fact, the need for extensive land forming is one of the main reasons why other types of irrigation systems have been developed.

13.1.2 Types

Prior to the early 1900's, most irrigation systems were of the surface type. Initially, water was allowed merely to spill over the banks of rivers and flood adjacent lands. The resulting distribution of water was usually quite nonuniform. This technique is often called uncontrolled flooding. With slight modifications, it has evolved into the technique known as water spreading. A further refinement, in which man-made ditches carry water along the top edges of fields or strips, is known as contour ditch irrigation or wild flooding.

All other methods of surface irrigation can be classified as controlled flooding. The water is guided down an irrigation slope by channels, which may be as wide as 30 meters or as narrow as several millimeters, or is allowed to flood an essentially level area surrounded by dikes. The major types of such systems are discussed in subsequent sections of this chapter. In general, as the level of sophistication increases, so do initial costs and the potential uniformity of water distribution.

13.1.3 Required Design Variables

Depth of water to be applied. The most important design variable is the depth of water to be applied at each irrigation. This is generally given as an average depth for each field even though the soil-water reservoir may not have been uniformly depleted throughout the field, and the water will not be distributed uniformly over the soil as a result of the irrigation. Most surface

irrigation systems in arid and semi-arid areas are designed to raise the soil water content of the root zone to its field capacity even though water may be wasted. This is done in order to utilize water supplies when available, and to reduce the total number of irrigations and hence to also reduce labor. With furrow systems especially, it is sometimes desirable and possible to only partially refill the root zone. In many irrigated areas it is sometimes necessary to apply "excess" water in order to leach out undesirable salts.

Hydraulic variables. As described in the previous chapter, surface irrigation design is a problem in unsteady, nonuniform flow. The main design variables include the field slope and roughness, both of which may vary within a field. Another consideration is the erosiveness of the soil, which will limit maximum inflow rates to a field.

Topographic and related information. The topography of a field limits the types of systems which can be used. Those which have rolling terrain, irregular shapes and shallow soils may be impractical to irrigate with surface systems. If surface systems are used, they will usually be of the non-sophisticated types with relatively low efficiency and non-uniform water distribution when measured on a field basis. On the other hand, flat terrain, fields of regular shapes and deep soils may be adaptable to a wide range of systems, all of which have the potential for high efficiency and uniform water distribution. Thus, a site under consideration for surface irrigation must be mapped to show field boundaries, land elevations and soil depths and textures (also see Chapter 8). Such information will give land slopes and field shapes directly. It will also assist in determining how much water to apply. In some cases minimum intensity mapping is satisfactory, while in other cases detailed mapping is necessary. The latter is usually necessary on relatively flat terrain that will be land-formed prior to irrigating.

Infiltration. The infiltration characteristic of the soil at each irrigation is a primary input variable. It varies with time and space. It is not at all unusual to have 10-fold variations in infiltration rates throughout a field. Such variations can make the design of an efficient irrigation system extremely difficult, if not impossible.

The design procedures of this chapter follow closely those of the Soil Conservation Service (USDA, 1979) and its method of describing water intake by soils is followed. The basis of the SCS design is to classify soils into intake families. The equation of these families is as follows.

$$F = aT^b + c \qquad [13.1]$$

where F is the cumulative intake (mm), T is the time water is in contact with the soil (min), and a, b and c are constants unique to each intake family. Values of the constants are given in Table 13.1, and the families are plotted in Fig. 13.1.

The selection of an appropriate intake family is dependent not only upon the soil, but also upon the irrigation method. Selection can only be made by running field tests. For irrigation systems which have infiltration primarily through a relatively flat, horizontal surface (borders, basins, etc.) ring infiltrometer or basin ponding tests can be run. The measured cumulative infiltration is plotted on Fig. 13.1 and the intake family is that of the curve closest to which the points fall. In these systems, F as defined by equation [13.1], is the depth of infiltrated water.

TABLE 13.1. FURROW INTAKE FAMILY AND ADVANCE COEFFICIENTS

Intake family	a	b	c	f	g
0.05	0.5334	0.618	7.0	7.16	1.088×10^{-4}
0.10	0.6198	0.661	7.0	7.25	1.251×10^{-4}
0.15	0.7110	0.683	7.0	7.34	1.414×10^{-4}
0.20	0.7772	0.699	7.0	7.43	1.578×10^{-4}
0.25	0.8534	0.711	7.0	7.52	1.741×10^{-4}
0.30	0.9246	0.720	7.0	7.61	1.904×10^{-4}
0.35	0.9957	0.729	7.0	7.70	2.067×10^{-4}
0.40	1.064	0.736	7.0	7.79	2.230×10^{-4}
0.45	1.130	0.742	7.0	7.88	2.393×10^{-4}
0.50	1.196	0.748	7.0	7.97	2.556×10^{-4}
0.60	1.321	0.757	7.0	8.15	2.883×10^{-4}
0.70	1.443	0.766	7.0	8.33	3.209×10^{-4}
0.80	1.560	0.773	7.0	8.50	3.535×10^{-4}
0.90	1.674	0.779	7.0	8.68	3.862×10^{-4}
1.00	1.786	0.785	7.0	8.86	4.188×10^{-4}
1.50	2.284	0.799	7.0	9.76	5.819×10^{-4}
2.00	2.753	0.808	7.0	10.65	7.451×10^{-4}

Intake (see equations [13.1] and [13.40]) Advance (see equation [13.35])

$F = (aT^b + c)\,P/W$, mm

T = minutes

$\frac{P}{W} = \frac{\text{Wetted perimeter}}{\text{Furrow spacing}}$

$T_T = \frac{x}{f}\, e^{(gx/QS^{1/2})}$, min

Q = furrow inflow

S = furrow slope

x = distance

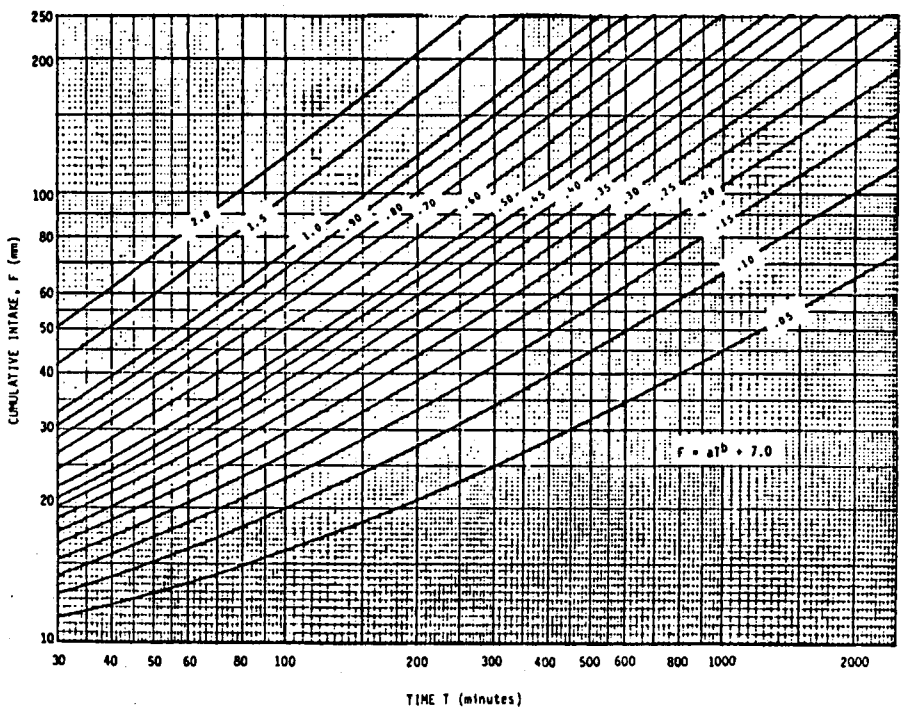


FIG. 13.1 Intake families (USDA, 1979).

Furrow systems require a different approach. An inflow-outflow test is run, and the family is selected using a calculation procedure described in Section 13.6. For furrows, F as defined in equation [13.1] is *not* the depth of infiltration. Rather, the right hand side of that equation must be multiplied by a factor which accounts for the wetted perimeter of the furrow and the furrow spacing. This multiplicative factor is explained in Section 13.6. It is emphasized that a given soil will, in general, be classified into different families for basin type systems and furrow type systems.

13.2 CONTOUR DITCH IRRIGATION

13.2.1 Description

This system, also known as wild flooding, consists of a series of ditches spaced 25 to 100 m apart. These head ditches have a slight slope. The water is removed from the ditches either by making the downslope bank low enough so that the water is not restrained by it when checked, or by making cuts through the downslope bank. In theory the water moves down the slope as a sheet, but in practice it may not. Rather, the water channels and is nonuniformly distributed. In some cases siphons are used for removing water from the ditch, but this is not usual, especially in areas where the land is marginal and the crops grown have a low cash value.

13.2.2 Application

The system is applicable to slopes of 0.5 to 15 percent. It is particularly adaptable to residual soils in foothill areas that have an underlying permeable layer at a rather shallow depth, 0.3 to 0.6 m. This condition allows redistribution of the applied water within the soil profile. It is seldom used on deep sandy soils with high infiltration rates or on clay soils that crack upon drying. The USDA Soil Conservation Service recommends that the system be restricted to soils in the 0.1 to 3.0 intake families.

13.2.3 Advantages

The system has one particular advantage—it is low in first cost. Generally, the systems require no land forming and unlined ditches are common. If a field is on the side of a hill and the soils are underlain by an impermeable layer, then water application efficiencies can be fairly high in properly laid out systems. Successive ditches down the slope pick up the surface runoff and redistribute it to lower portions of the field.

13.2.4 Limitations

Although the range of slopes given was fairly high, there are restrictions due to erosion. If runoff-producing rainfall can be expected then only slopes of 4 percent or less should be considered. Extremely erosive soils will also be carried away by the irrigation practice unless sod-forming crops are grown. High water application efficiencies require frequent short sets which means high labor or an increased cost for automated equipment.

13.2.5 Design

These systems are usually designed from experience. Two general types are recognized—those utilizing a continuous flow on the land, and those utilizing intermittent flows. For continuous flow, Booher (1974) recommends a flow of about 0.7 to 1 L/s per ha (4.5 to 6.5 U.S. gpm/ac). This is equivalent to about 7.4 mm/d (0.29 in./d). For intermittent flow systems, the Soil Conservation Service (USDA, 1967) recommends the use of about 0.028 m^3/s (1 cfs), per 30 m (100 ft) of strip width irrigated.

The distance between head ditches, or between a head ditch and the first pickup ditch, would be about 24 to 90 m (80 to 300 ft), varying with the topographic uniformity.

The field to be irrigated usually receives two cultivation treatments other than that of seedbed preparation. Where soils are erosive, seedings are made without cultivation. If knolls and depressions are a problem, and if soil depths are adequate, slight land forming might be advisable. Alternatively, corrugations (Section 13.6) can run directly down the slope.

13.2.6 Headland Facilities

Earthen head ditches are usually used, although concrete could be used if soils are highly erosive or seepage is a problem. Pickup ditches for redistributing water down the slope are usually earthen.

Outlets from earthen head ditches are often mere cuts in the bank, stabilized by sod or rocks, or spiles can be used. Siphons may be used in either earthen or concrete lined ditches. Cast outlets, with grooves for flashboards, can be installed in concrete ditches. Finally, the ditch can be formed so that there will be discharge over the downslope bank when checks (canvas, plastic, or structural) are activated.

Ditch sizing should be in accordance with the recommendations of Sections 11.2 and 11.3. However, when overlfow from the lower bank is planned, this bank must be on a grade to allow approximately equal overflow at each point along it. This grade must usually be determined by trial-and-error, which often requires that ditches be moved. Common grades on checked ditches are 0.0005 to 0.001 m/m.

When multiple cuts or spiles are used, they should be spaced at intervals of approximately 2 to 3 m (6.5 to 10 ft), according to Booher (1974). If desired, the system can be automated and designed according to the techniques outlined in Section 13.9.

13.3 BASIN IRRIGATION

13.3.1 Description

The field to be irrigated by the basin method is divided into level rectangular areas bounded by dikes or ridges. Water is turned in at one or more points until the desired gross volume has been applied to the area. The flow rate must be large enough to cover the entire basin in approximately 60 to 75 percent of the time required for the soil to absorb the desired amount of water. Water is ponded until infiltrated.

13.3.2 Applicability

Most crops can be irrigated with basin irrigation. It is widely used for close-growing crops such as alfalfa and other legumes, grasses, small grains, mint, and rice. It is used for row crops that can withstand some inundation, such as sugar beets, corn, grain sorghum, and cotton, and for other row crops if they are planted on beds so they will be above the water level. It also is well suited to the irrigation of tree crops, grapes, and berries.

This irrigated method is best suited to soils of moderate to low intake rate (50 mm/h or less). It is an excellent way of applying water to soils that

have a moderately high to high intake rate, but basin areas may need to be very small.

Basin irrigation is best suited to smooth, gentle, uniform land slopes. Undulating or steep slopes can be prepared for basin irrigation, provided the soils are deep enough to permit needed land leveling.

13.3.3 Advantages

High application efficiency can be obtained easily with little labor. Basin irrigation can be used efficiently by inexperienced workers, and can easily be automated. When basins are leveled with laser-controlled scrapers, basins can be as large as 16 ha (Erie and Dedrick, 1979). Many different kinds of crops can be grown in sequence without major changes in design, layout, or operating procedures. There is no irrigation runoff, there is little deep percolation if no excess is applied, and maximum use can be made of rainfall. Leaching is easy and can be done without changing either the layout or operation method.

13.3.4 Limitations

Accurate initial land leveling is essential and level surfaces must be maintained. Adequate basin ridge height may be difficult to maintain on sandy soils or fine-textured soils that crust or crack when dry. Prolonged ponding and crop scalding can occur if the system is poorly managed. In some areas special provisions must be made for surface drainage.

Drop structures, lined ditches, or pipelines may be required to control water on steep slopes that require benching. Relatively large inflow rates are needed for basins and special structures may be needed to prevent erosion.

13.3.5 Design

Water should be applied at a rate that will advance over the basin in a fraction of the infiltration time to achieve high efficiency. The volume of water applied must equal the average gross irrigation application. The intake opportunity time at all points in the basin must be greater than or equal to the time required for the net irrigation to enter the soil. The longest intake opportunity time at any point on the basin area must be sufficiently short to avoid scalding and excessive deep percolation. The depth of water flow must be contained by the basin ridges.

Design limitations. In theory, maximum depth of flow and maximum deep percolation both occur where water is introduced into a basin, usually considered as a "strip" of unit width for computational purposes. For any given set of site conditions, the depth of flow varies directly and the amount of deep percolation varies inversely with the inflow rate per unit width of basin strip. Thus, if a limit is set on flow depth, deep percolation may be reduced only by shortening the length of the basin strip. If limits are established for both depth of flow and deep percolation, then the design limit for length is determined.

Flow at the head end of basin strips must not exceed some practical depth related to the construction and maintenance of basin ridges.

The average deep percolation (the difference between the net and gross irrigation applications) should be minimized. On some sites excess deep percolation causes acute drainage problems. To avoid this condition, the design efficiency usually should not be less than about 80 percent. This efficiency

can be obtained if the time required to cover the basin is not more than 60 percent of the time required for the net application to enter the soil. A design efficiency of less than 70 percent should be considered only for soils having excellent internal drainage. On sites where irrigation water supplies are limited or costly, where subsurface drainage problems are acute, or where crops can be damaged by prolonged surface flooding, design efficiencies in excess of 90 percent are often practical. These efficiencies are easily obtained when laser-controlled scrapers are used (Erie and Dedrick, 1979).

Basin strips usually are designed to be level; however, they may be constructed with a slight grade in the direction of water flow. A slight grade will minimize adverse effects of variations in the finished land surface, such as low areas or reverse grades, which result in a slower rate of advance, reduced efficiency, excessive deep percolation or prolonged flooding that may damage crops. The total fall in the length of the basin strips should not be greater than one-half the net depth of application used as a basis for design. No adjustment is made in the design to compensate for such slight grades.

Drainage facilities may be needed to remove excess water from basins resulting from an accidental overirrigation or heavy rainfall. Large furrows formed when constructing basin ridges facilitate removal of excess rainfall or irrigaton water. They also speed the water coverage rate over the basin and reduce flow depths and deep percolation adjacent to the point or points of water inflow. Surface drainage facilities should be provided for basins on low intake soils, and, in high rainfall areas, on moderate intake soils.

Basin ridges, or levees, should be constructed so that the top width is at least as great as the ridge height. The settled height should be at least equal to the greater of (a) the design gross depth of application, or (b) the design maximum depth of flow plus a freeboard of 25 percent of the maximum depth of flow.

Method of the Soil Conservation Service (USDA, 1974). Design equations are based on equating (a) the volume of water applied to a unit width basin strip during the time period of water advance from the head to the end of the strip, and (b) the volume of intake plus the water in temporary surface storage during the same period.

The designer must know the cumulative intake characteristic of the soil, must select a Manning roughness coefficient (n) appropriate for the crops to be irrigated, and must select the net application depth to be used as a basis for design.

Opportunity Time—The opportunity time required for intake of the selected net application depth can be estimated by solution of the cumulative intake equation in the form

$$T_n = [(F_n - c)/a]^{1/b} \qquad [13.2]$$

where T_n is the time required, also called the net opportunity time (min), and F_n is the desired net application depth (mm).

Advance Time—The time required for the unit inflow rate to advance to the far end of the strip is called the advance time, T_r (min). The required advance time for any desired water application efficiency is determined by multiplying the net opportunity time, T_n, by the efficiency advance ratio, R (Table 13.2).

Water application efficiency is defined as the ratio of average net ap-

length, time of inflow, inflow rate, and depth of flow for any assumed efficiency. An alternative chart would be to exchange efficiency and net application depth. Fig. 13.1 is a sample design chart.

Computation Example—

Given:

Intake family	0.5
Desired efficiency, E	80 percent
Unit/inflow rate, Q_u	0.005 m^2/s
Maximum depth of flow, d	150 mm
Desired depth of application, F_n	100 mm
Manning roughness coefficient, n	0.15

Find:

Opportunity time required, T_n
Basin length, L
Inflow time required, T_a
Maximum depth of flow, d

Solution:

Opportunity time:

$$T_n = \left(\frac{100 - 7.0}{1.186}\right)^{1/0.756} = 320 \text{ min} \quad \text{(equation [13.1])}$$

Ratio, T_r/T_n at 80 percent efficiency = 0.58, from Table 13.2.

Advance time:

$T_r = (0.58)(320) = 186$ min

Basin length:

$$L = \frac{(6 \times 10^4)(0.005)(186)}{\dfrac{(1.186)(186)^{0.756}}{1 + 0.756} + 7.0 + (1798)(0.15)^{3/8}\,(0.005)^{9/16}\,(186)^{3/16}}$$

$$= 345 \text{ m} \quad \text{(equation [13.3])}$$

Inflow time:

$$T_a = \frac{(100)(345)}{(600)(0.005)(80)} = 144 \text{ min} \quad \text{(equation [13.4])}$$

Maximum depth of flow:

$$d = 2250(0.15)^{3/8}\,(0.005)^{9/16}\,(144)^{3/16}$$

$$= 142 \text{ mm} \quad \text{(equation [13.5])}$$

TABLE 13.3 SUGGESTED BASIN AREAS FOR DIFFERENT SOIL TYPES AND RATES OF WATER FLOW (ADAPTED FROM BOOHER, 1974)

Soil type			
Sand	Sandy loam	Clay loam	Clay
Unit areas, ha/100 L/s			
0.067	0.20	0.40	0.67

Empirical method of Booher (1974). Basin sizes are suggested for various soil types and inflow rates, as shown in Table 13.3. The areas are approximations and the table is intended to be used as a guide only.

Soils with high infiltration rates, such as sands, require limited basin size even when large flows of water are available. Basins on clay soils can be large or small, depending on the water inflow rate. The objective in selecting the basin size is to be able to flood the entire area in a reasonable length of time so that the desired depth of water can be applied with a high degree of uniformity over the entire basin.

13.3.6 Selection of Headland Facilities

Water may be conveyed to irrigated basins in lined or unlined ditches, or pipelines installed above or below the ground surface. Adequate structures should be provided in the delivery system to permit control and regulation of the water flow. Such structures include checks, checkdrops, valves, or gates. Measuring structures to determine the delivery flow rate are essential for good irrigation management.

Supply ditches. Supply ditches must convey the design inflow rate of each basin, or multiples of the design flow rate where more than one basin is irrigated simultaneously. The water surface in the ditch should be 0.15 to 0.30 m above the ground surface level in the basin, depending on outlet characteristics. Where possible, the ditches should be constructed with a 0.1 percent grade or less to minimize the number of checks and the labor required.

Ditches may be designed and constructed with the water surface below the ground surface where portable pumps, usually of the low head-high volume propeller type, are used to convey water into basins.

Supply ditch outlets. Outlets to release water from ditches into basins may be of several types. Gated rectangular or trapezoidal outlets installed in the side of the ditch are commonly used where the entire ditch flow is discharged into one basin. Gated orifice-type outlets are desirable when more than one basin is being supplied simultaneously from the ditch, to minimize the effect of pressure head differentials on the discharge rate. Outlet gates are not required when the invert of the outlet is located at or above the normal water surface elevation in the ditch. Water flows through the outlet into the basin when the water surface in the ditch is raised by regulating control structures in the ditch.

Larger diameter siphon tubes may be used to convey water from the ditch into the basin. Capacity varies with the tube size and the differential head between the water surface in the ditch and the basin. Adjustable gates on the discharge end and a small vacuum pump (removable) and valve on top

TABLE 13.2 EFFICIENCY AS A FUNCTION OF THE EFFICIENCY ADVANCE RATIO $R(R = T_t/T_n)$

Efficiency, E	Efficiency advance ratio $R(R = T_t/T_n)$
Percent	—
95	0.16
90	0.28
85	0.40
80	0.58
75	0.80
70	1.08
65	1.45
60	1.90
55	2.45
50	3.20

plication (F_n) to gross application (F_g) expressed as percent.

Basin Length and Inflow Rate—The following mass balance equation can be used to estimate length of the basin strip as a function of unit inflow rate (Q_u) and advance time (T_t).

$$L = \frac{6 \times 10^4 \, Q_u \, T_t}{\frac{a \, T_t^{\,b}}{1+b} + 7.0 + 1798 \, n^{3/8} \, Q_u^{\,9/16} \, T_t^{\,3/16}} \quad \ldots\ldots\ldots\ldots [13.3]$$

where L is the length (m); Q_u is the unit inflow rate (m^2/s); T_t is the required advance time for the desired efficiency (min); a, b, and c are constants in the cumulative intake equation; and n is Manning's coefficient.

The design length of the basin strip can be found for any selected inflow rate, efficiency and associated required advance time, by direct solution of equation [13.3]. A similar solution for the unit inflow rate needed for a selected length and efficiency is not possible. A trial and error procedure must be used.

Inflow Time—The inflow time, the time required to apply the gross application onto the basin strip, can be computed from equation [13.4].

$$T_a = \frac{F_n \, L}{600 \, Q_u \, E} \quad \ldots\ldots\ldots\ldots [13.4]$$

where T_a is the inflow time (min) for the unit inflow rate Q_u (m^2/s), to apply the net application depth F_n (mm) on a basin strip of length L (m), at an efficiency E (percent).

Maximum Depth of Flow—The maximum depth of flow can be estimated from equation [13.5].

$$d = 2250 \, n^{3/8} \, Q_u^{\,9/16} \, T_a^{\,3/16} \quad \ldots\ldots\ldots\ldots [13.5]$$

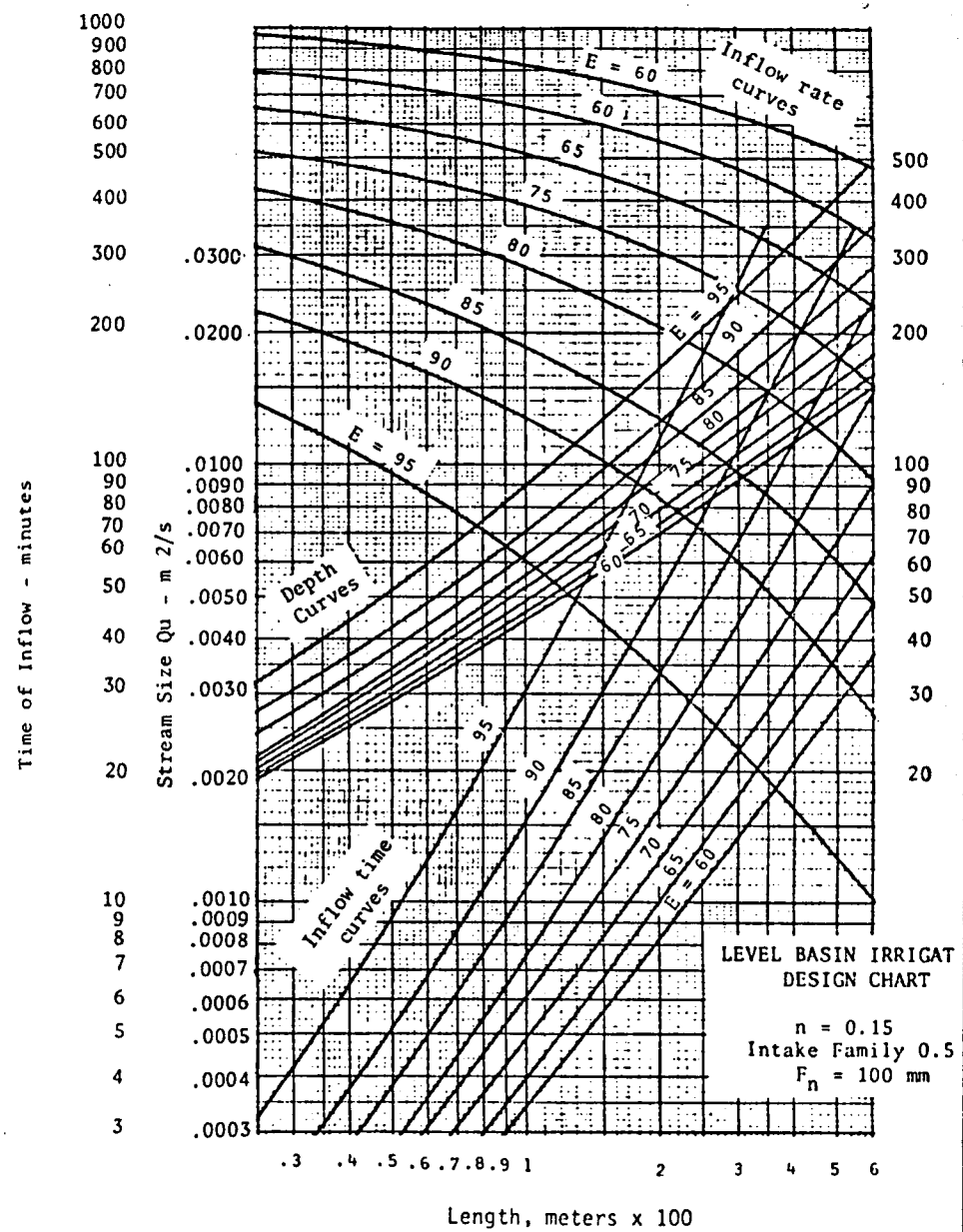


FIG. 13.2 Level basin irrigation sample design (USDA, 1974).

where d is the flow depth at the inlet end of the basin strip (mm). If advance time, T_t is greater than T_a, use T_t in equation [13.5] in place of T_a. The inflow rate for a given maximum depth of flow connot be determined directly. A trial-and-error procedure must be used.

Design Charts—The procedure for design of basins may be simplified by preparation of design charts. A separate chart can be prepared for any combination of Manning coefficient, cumulative intake relationship, and net application depth. Each such chart would describe the relationship between

conditions. The coefficient varies with crops, stages of crop growth, and degree of roughness of the soil surface.

Table 13.4 contains values of Manning coefficient commonly used in design.

A conservatively high value of n should be used in determining maximum flow depth, and a conservatively low value of n used when determining minimum flow rate.

Design equations.

Inflow Rate—Inflow rate per unit width of border strip (Q_u, m^2/s) can be determined for a given net depth of application from equation [13.6].

$$Q_u = \frac{0.00167\, F_n\, L}{(T_n - T_L)\, E} \quad \text{[13.6]}$$

where F_n is the desired net application depth (mm); L is the length of the border strip (m); T_n is the opportunity time (min) required for the desired application depth; T_L is the lag time (min) that water remains on the head end of the strip after inflow stops; and E is the water application efficiency (percent), the ratio of the desired net application depth to the gross application depth. As time lag is a function of flow rate, a direct solution is not possible, unless the slope exceeds 0.4 percent and the time lag becomes insignificent. A trial and error solution is required when the slope is less than 0.4 percent.

Lag Time—High Gradient Borders—The depth of flow approaches normal depth at the upper end of the border strip in a relatively short advance period on borders with steep slopes. Lag time may be ignored in determination of inflow rate on borders with slopes steeper than 0.4 percent. Lag time for high gradient borders may be computed from equation [13.7].

$$T_L = \frac{Q_u^{0.2}\, n^{1.2}}{120\, S_o^{1.6}} \quad \text{[13.7]}$$

where T_L is the lag time (min) at the head end of the strip, n is the Manning coefficient, S_o is the border strip slope (m/m), and Q_u is inflow rate per unit width (m^2/s).

Lag Time—Low Gradient Borders—Lag time is significant in border strips with slopes of 0.4 percent or less on which slopes the depth of flow may not reach normal depth. Lag time for such low gradient borders may be computed from equation [13.8].

$$T_L = \frac{n^{1.2}\, Q_u^{0.5}}{120 \left[S_o + \left(\dfrac{0.0094\, n\, Q_u^{0.175}}{T_n^{0.88}\, S_o^{0.5}} \right) \right]^{1.6}} \quad \text{[13.8]}$$

where T_L is the lag time (min), n is the Manning coefficient, Q_u is the inflow rate per unit width (m^2/s), S_o is the border slope (m/m), and T_n is the opportunity time (min) required for the desired application depth. Equation [13.8] has been developed from water surface profile computations utilizing incremental values of flow rate, border slope, Manning coefficient, and depth. Lag time may also be estimated from Table 13.5.

Inflow Time—Inflow time (T_a) can be determined by subtracting the lag time (T_L) for a specific inflow rate (Q_u), border slope (S_o) and Manning n from the opportunity time (T_n) required for intake of the desired application depth (F_n), as expressed by equation [13.9].

$$T_a = T_n - T_L \quad \text{[13.9]}$$

Design water application efficiency. Design water application efficiency, defined as the ratio of the desired net application depth to the gross application depth, must be selected by the designer based on a particular site under a given set of management conditions. Overestimating the efficiency should be avoided. For a given management level, site conditions have a significant effect on the efficiency achievable in border irrigations. Greater efficiency can be expected on gentle slopes than on steep slopes and on soils that have a moderate to moderately high intake rate, than on soils that have either a low or extremely high intake rate. Table 13.6 shows the efficiencies commonly assigned for designing border irrigation.

TABLE 13.5 RECESSION-LAG TIMES, T_L (min), IN LOW GRADIENT BORDERS

Oppor. time, T_n (min)	Border slope, S_o (m/m): 0.0005				0.001				0.002				0.004			
	Inflow rate, Q_u (m^2/s)				Inflow rate, Q_u (m^2/s)				Inflow rate, Q_u (m^2/s)				Inflow rate, Q_u (m^2/s)			
	0.0001	0.001	0.01	0.02	0.0001	0.001	0.01	0.02	0.0001	0.001	0.01	0.02	0.0001	0.001	0.01	0.02
Manning's n = 0.04																
10	1.9	2.2	2.3	2.3	1.1	1.5	1.9	2.0			1.1	1.1				
25	3.1	4.0	4.8	5.1	1.4	2.0	2.8	3.1			1.2	1.4				
50	3.9	5.4	7.1	7.7	1.6	2.3	3.4	3.8			1.3	1.5				
100	4.4	6.5	9.2	10.1	1.6	2.5	3.8	4.3			1.4	1.6				
200	4.8	7.3	10.8	12.1	1.7	2.6	4.1	4.6			1.4	1.6				
500	5.1	7.9	12.1	13.7	1.7	2.7	4.2	4.9			1.4	1.6				
1000	5.2	8.1	12.6	14.4	1.7	2.7	4.3	4.9			1.4	1.7				
2000	5.2	8.2	12.9	14.8	1.7	2.8	4.4	5.0			1.4	1.7				
Manning's n = 0.15																
10	2.5	2.4	2.2	2.1	2.5	2.7	2.7	2.7	1.6	2.1	2.5	2.6		1.1	1.5	1.6
25	6.1	6.3	6.3	6.2	4.4	5.4	6.2	6.4	2.2	3.0	4.1	4.4		1.3	1.9	2.1
50	10.1	11.6	12.5	12.7	5.7	7.7	9.8	10.4	2.4	3.6	5.1	5.7		1.4	2.1	2.3
100	14.5	18.4	21.9	22.7	6.8	9.7	13.4	14.6	2.6	3.9	5.9	6.6		1.4	2.2	2.5
200	18.4	25.3	32.9	35.2	7.5	11.2	16.3	18.1	2.7	4.2	6.4	7.3		1.4	2.3	2.6
500	22.1	32.5	46.3	51.2	8.1	12.4	18.9	21.4	2.8	4.3	6.8	7.7		1.5	2.3	2.6
1000	23.7	36.0	53.6	60.2	8.3	12.9	20.0	22.8	2.8	4.4	6.9	7.9		1.5	2.3	2.7
2000	24.7	38.2	58.4	66.2	8.4	13.2	20.7	23.6	2.8	4.4	7.0	8.0		1.5	2.3	2.7
Manning's n = 0.25																
10	2.4	2.2	1.9	1.8	2.8	2.8	2.7	2.6	2.2	2.7	2.9	3.0	1.2	1.7	2.1	2.3
25	6.5	6.4	6.0	5.8	5.8	6.6	7.0	7.1	3.4	4.4	5.6	6.0	1.5	2.1	3.0	3.4
50	12.3	13.0	13.1	12.9	8.5	10.6	12.5	12.9	4.1	5.8	7.9	8.5	1.6	2.4	3.5	4.0
100	19.9	23.3	25.6	26.0	10.9	14.9	19.1	20.4	4.5	6.7	9.7	10.8	1.6	2.5	3.9	4.4
200	28.1	36.0	43.5	45.5	12.8	18.5	25.6	28.1	4.8	7.4	11.1	12.4	1.7	2.6	4.1	4.6
500	36.9	52.1	70.5	76.5	14.3	21.7	32.2	36.0	5.0	7.8	12.1	13.8	1.7	2.7	4.2	4.8
1000	41.3	61.2	88.0	97.5	15.0	23.1	35.3	39.9	5.1	8.0	12.5	14.3	1.7	2.7	4.3	4.9
2000	44.1	67.3	100.7	113.3	15.3	24.0	37.2	42.4	5.2	8.1	12.8	14.6	1.7	2.7	4.3	4.9

Note: Recession-lag times of less than one minute are omitted

facilitate removal of air in the tube and initiating flow.

Pipeline outlets. An outlet structure or hydrant is necessary in pipelines to deliver water to the basins. A valve or gate is installed in the vertical riser attached to the pipeline for underground pipelines, or directly to the pipeline on surface installations, to regulate discharge. "Alfalfa" valves are commonly used, and consist of a plate attached to a threaded rod which moves up or down as the handle is turned to regulate flow.

Erosion. The water flow velocity into the basin should not exceed about one meter per second to avoid formation of scour holes or erosion adjacent to the turnout. Turnout structures should be designed with energy dissipation features to limit the discharge velocity.

Outlet location. The number and location of outlets to discharge water into basins varies with the rate of flow required and the width of the basin. One outlet for basin widths up to 60 meters and flow rates up to 0.4 cubic meters per second is common where the outlet incorporates adequate energy dissipation features. Minimizing the number of outlets reduces labor and facilitates use of automatic controls. Spacing the outlets along the basin width, however, may provide a more uniform distribution of water over the basin at the inlet end.

13.4 BORDER IRRIGATION

13.4.1

The field to be border irrigated is divided into graded strips by constructing parallel dikes or border ridges. The ends of the strips are usually not closed. Water is turned in at the upper end and flows as a sheet down the strip. The flow rate must be such that the desired volume of water is applied to the strip in a time equal to, or slightly less than, that needed for the soil to absorb the net amount required. When the desired volume of water has been delivered to the strip, the inflow is turned off. The water not infiltrated is temporarily stored on the ground surface and moves on down the strip to complete the irrigation. Outflow from the strip may be avoided by closing the lower end and ponding the water on the lower reaches of the strip until infiltrated. The discussion in this section follows closely that of the Soil Conservation Service (USDA, 1974).

13.4.2 Applicability

Crop. Border irrigation is suitable for all close-growing, non-cultivated, sown or drilled crops, except rice and other crops grown in ponded water. Legumes, grasses, small gains, and mint are commonly irrigated by this method. It is also used to irrigate orchards and vineyards.

Soils. Border irrigation can be used on most soils. It is, however, best suited to soils with a moderately low to a moderately high intake rate. It is seldom used on coarse sandy soils of extremely high intake rate as excessive deep percolation occurs unless the strip length is very short. Also, it is not suited for use on soils of extremely low intake rate since, to provide adequate intake time without excessive surface runoff, the inflow rate becomes too small to completely cover the border strip.

Slopes. Border irrigation is best suited to slopes of less than 0.5 percent. It can be used on slopes to 2 percent where non-sod crops are grown, and slopes to 4 percent or steeper where sod crops are grown, providing good crop stands are established by supplementary irrigation methods or dependable rainfall. The erosion hazard created by rainfall runoff must be considered in determining the permissible border slopes.

13.4.3. Advantages

Field application efficiency is good to excellent if the border strips are designed and installed properly and good water management practices are followed. Labor requirements are low, and border strip dimensions can be designed for efficient operation of machinery. Within broad limits, border strips can be designed for efficient operation of machinery. Within broad limits, border strips can be designed for irrigation grades that minimize land leveling costs. In areas where surface drainage is critical, borders provide an excellent means for removing excess surface water rapidly. On some sites, the ends of borders may be closed to reduce or eliminate surface runoff.

13.4.4 Limitations

The topography and soil profile characteristics must not restrict land leveling necessary to eliminate cross slope within feasible border widths, and the achievement of a uniform border strip slope. Small flow rates necessitated by low soil intake characteristics or steeper field slopes require complete elimination of cross slope within the border.

13.4.5 Design

Border design involves balancing the water advance and recession curves to achieve an equal opportunity time for intake at any point along a border strip. On sites suitable for border irrigation, advance and recession curves will be reasonably well balanced if the following two conditions are met: (a) the volume of water delivered to the border strip is adequate to cover it to an average depth equal to the gross application; and (b) the intake opportunity time at the upper end of the border strip is equal to the time necessary for the soil to absorb the net application desired.

Soil intake characteristics. Border design requires knowledge of the cumulative intake characteristics of the soils to be irrigated. The series of cumulative intake equations discussed in Section 13.1.3 are used in this section.

Manning coefficient of roughness. Hydraulic calculations in border design are based on the Manning equation, which includes a coefficient (n) that expresses the flow-retardance effects of different hydraulic boundary

TABLE 13.4. COMMON RETARDANCE COEFFICIENT USED IN BORDER DESIGN

Smooth, bare soil surfaces noncultivated, oil-mulch-treated citrus	0.04
Small grain, drill rows parallel to border strip	0.10
Alfalfa, mint, broadcast small grain, and similar crops	0.15
Dense sod crops, small grain with drill rows across the border strip	0.25

where d_n is normal flow depth (mm); Q_u is the inflow rate per unit width (m^2/s); and S_o is the border slope (m/m). The normal flow depth may also be estimated from Table 13.8.

Flow depth—low gradient borders. The depth of flow (mm) at the upper end of border strips with slopes of 0.4 percent or less may be computed from equation [13.13] (Table 13.9).

$$d = 2454\ T_L^{3/16}\ Q_u^{9/16}\ n^{3/8} \quad \ldots\ldots [13.13]$$

where d is the flow depth (mm), T_L is the lag time (min), Q_u is the inflow rate (m^2/s), and n is the Manning coefficient. The depth of flow in low gradient borders is shown in Table 13.9 for selected values of slope, inflow rate, Manning coefficient, and opportunity time required for the desired net application.

Minimum depth of flow. The flow rate must be large enough to spread over the entire border strip. A smaller flow rate is needed on rough surface strips than is required on adequately graded and smooth strips. The minimum inflow rate per unit width can be computed, using equation [13.14].

$$Q_u = (5.95 \times 10^{-6}\ L\ S_o^{0.5})/n \quad \ldots\ldots [13.14]$$

TABLE 13.9 DEPTH OF FLOW, d(mm), LOW GRADIENT BORDERS

Oppor. time, T_n (min)	Border slope, S_o (m/m) 0.0005 Inflow rate, Q_u (m^2/s) 0.0001	0.001	0.01	0.02	0.001 Inflow rate, Q_u (m^2/s) 0.0001	0.001	0.01	0.02	0.002 Inflow rate, Q_u (m^2/s) 0.0001	0.001	0.01	0.02	0.004 Inflow rate, Q_u (m^2/s) 0.0001	0.001	0.01	0.02
	Manning's n = 0.04															
10	4.7	17.4	64.4	95.3	4.2	16.3	62.0	92.5	3.6	14.2	55.2	83.1	3.0	11.8	46.7	70.6
25	5.1	19.5	74.0	110.1	4.4	17.2	66.9	100.6	3.7	14.5	57.2	86.3	3.0	11.9	47.4	71.7
50	5.3	20.7	79.5	119.0	4.5	17.7	69.3	104.5	3.7	14.6	58.0	87.7	3.0	12.0	47.6	72.1
100	5.5	21.4	83.4	125.4	4.5	17.9	70.7	106.9	3.7	14.7	58.4	88.5	3.0	12.0	47.8	72.4
200	5.5	21.9	85.9	129.5	4.6	18.1	71.6	108.3	3.7	14.8	58.7	88.9	3.0	12.0	47.8	72.5
500	5.6	22.2	87.8	132.8	4.6	18.2	72.2	109.3	3.7	14.8	58.9	89.2	3.0	12.0	47.9	72.6
1000	5.6	22.3	88.5	134.0	4.6	18.2	72.4	109.7	3.7	14.8	58.9	89.3	3.0	12.0	47.9	72.6
2000	5.6	22.4	88.9	134.7	4.6	18.2	72.5	109.9	3.7	14.8	59.0	89.4	3.0	12.0	47.9	72.6
	Manning's n = 0.15															
10	8.0	29.1	104.3	153.0	8.0	29.8	109.1	161.0	7.4	28.3	107.1	159.4	6.4	25.1	97.3	146.1
25	9.5	35.0	127.5	187.8	8.9	33.9	127.2	189.0	7.8	30.4	117.5	176.2	6.6	25.9	101.7	153.3
50	10.4	39.2	145.1	214.7	9.4	36.3	138.5	206.8	8.0	31.4	122.7	184.7	6.6	26.2	103.6	156.5
100	11.2	42.7	161.1	239.6	9.7	37.9	146.9	220.4	8.1	32.0	126.0	190.1	6.6	26.4	104.6	158.3
200	11.7	45.3	173.9	260.2	9.9	38.9	152.5	229.7	8.2	32.3	127.9	193.4	6.7	26.5	105.2	159.3
500	12.1	47.5	185.4	279.0	10.0	39.7	156.7	236.8	8.2	32.6	129.3	195.7	6.7	26.6	105.6	160.0
1000	12.3	48.4	190.6	287.6	10.1	40.0	158.4	239.7	8.2	32.7	129.8	196.6	6.7	26.6	105.7	160.2
2000	12.4	49.0	193.7	292.8	10.1	40.1	159.4	241.3	8.2	32.7	130.1	197.1	6.7	26.6	105.8	160.4
	Manning's n = 0.25															
10	9.7	34.6	123.3	180.7	9.9	36.4	131.7	193.7	9.5	36.0	133.8	198.3	8.5	32.9	126.3	188.8
25	11.7	42.4	153.0	224.8	11.4	42.6	157.6	233.1	10.3	39.7	151.3	225.9	8.8	34.5	134.8	202.8
50	13.1	48.5	177.1	261.2	12.3	46.7	175.6	261.0	10.7	41.6	161.0	241.5	8.9	35.2	138.6	209.2
100	14.4	54.1	200.9	297.5	12.8	49.7	190.3	284.4	10.9	42.8	167.6	252.4	9.0	35.6	140.9	213.1
200	15.3	58.7	222.0	330.6	13.2	51.8	201.0	301.9	11.0	43.6	171.7	259.2	9.0	35.9	142.3	215.3
500	16.1	62.9	243.0	364.3	13.5	53.3	209.7	316.4	11.1	44.1	174.6	264.2	9.1	36.0	143.2	216.9
1000	16.5	64.8	253.3	381.3	13.6	54.0	213.4	322.6	11.1	44.3	175.8	266.2	9.1	36.1	143.5	217.4
2000	16.7	66.0	259.8	392.2	13.7	54.3	215.5	325.1	11.2	44.4	176.4	267.2	9.1	36.1	143.7	217.8

NOTE: Values given in this table are solutions to equation [13.13]. In practice calculations to ± 1 mm are adequate.

TABLE 13.10. MINIMUM VALUE OF Q_u/L FOR VARIOUS SLOPES, S_o AND MANNING n's

Border slope, S_o (m/m)	Manning's n 0.04	0.15	0.25
	10^{-5} m^2/s	10^{-5} m^2/s	10^{-5} m^2/s
0.0005	0.3324	0.0886	0.0532
0.001	0.4701	0.1254	0.0752
0.002	0.6649	0.1773	0.1064
0.003	0.8143	0.2171	0.1303
0.004	0.9403	0.2507	0.1504
0.005	1.0512	0.2803	0.1682
0.0075	1.2875	0.3433	0.2060
0.01	1.4867	0.3964	0.2379
0.015	1.8208	0.4855	0.2913
0.02	2.1025	0.5607	0.3364
0.025	2.3506	0.6268	0.3761
0.03	2.5750	0.6867	0.4120
0.04	2.9734	0.7929	0.4757
0.05	3.3243	0.8865	0.5319
0.06	3.6420	0.9711	0.5827

Table 13.10 shows the minimum value of Q_u/L for various slopes and Manning n's.

Maximum slope. The maximum allowable slope for a selected net application depth, efficiency, and given intake family can be estimated from equation [13.15] or Table 13.11.

$$S_{o\,max} = \left(\frac{n}{0.0117\ E} \frac{F_n}{T_n} \right)^2 \quad \ldots\ldots [13.15]$$

Equation [13.15] is based on criteria for minimum depth of flow and does not include slope limitations imposed by erosion hazards due to runoff from rainfall. Although Table 13.11 indicates the theoretical possibility of using border irrigation on very steep slopes, it is better suited to gentle slopes. On slopes over about 4 percent, erosion is an extreme hazard; it is doubtful whether the border method should be considered for slopes in excess of 6 percent.

Maximum length. The theoretical maximum length for open-end borders is limited by the maximum allowable flow rate, as limited by erosion hazard on steep slopes or by the border ridge height on flat slopes. The permissible border length on soils of low intake rate and low slopes, as determined using equation [13.16] may exceed practical limits. The time required to patrol long lengths and the difficulties in determining and making needed inflow rate adjustments usually make these lengths impractical. Border lengths should seldom exceed 400 meters.

$$L_{max} = \frac{Q_u\ E\ (T_n - T_L)}{0.00167\ F_n} \quad \ldots\ldots [13.16]$$

TABLE 13.6 SUGGESTED DESIGN WATER APPLICATION EFFICIENCIES (PERCENT) FOR GRADED BORDER IRRIGATION BY SLOPE AND INTAKE FAMILY

Irrigation slope S_o (m/m)	Intake family 0.3				0.5					1.0					1.5					2.0					3.0				4.0			
	F_n*(mm)				F_n(mm)					F_n(mm)					F_n(mm)					F_n(mm)					F_n(mm)				F_n(mm)			
	25	50	75	100	25	50	75	100	125	25	50	75	100	125	25	50	75	100	125	25	50	75	100	125	25	50	75	100	25	50	75	100
0.0005	65	65	70	70	65	65	70	70	70	75	75	80	80	80	75	75	80	80	80	75	75	80	80	80	65	70	70	70	65	70	70	70
0.0010	60	60	65	65	65	65	70	70	70	70	70	75	75	75	75	75	80	80	80	75	75	80	80	80	65	70	70	70	65	70	70	70
0.0020	60	60	55	50	65	65	70	70	70	65	65	70	70	70	70	70	75	75	75	70	70	75	75	75	65	70	70	70	65	70	70	70
0.0030	55	55	50		60	60	65	65	65	65	65	70	70	70	65	65	70	70	70	65	65	70	70	70	65	70	70	70	65	70	70	70
0.0040	55	50			60	60	65	60	55	60	60	65	65	65	65	65	70	70	70	65	65	70	70	70	65	70	70	70	60	65	65	65
0.0050	50				60	60	60	55	50	60	60	65	65	65	65	65	70	70	70	65	65	70	70	70	65	70	70	70	60	65	65	65
0.0075					55	55	50			60	60	65	65	65	60	60	65	65	65	65	65	70	70	70	65	70	70	70	60	65	65	65
0.0100					55	55				60	60	65	65	65	60	60	65	65	65	60	60	65	65	65	60	65	65	65	60	65	65	65
0.0150					55					55	55	60	60	60	60	60	65	65	65	60	60	65	65	65	60	65	65	65	60	65	65	65
0.0200					50					55	55	60	55	50	60	60	65	65	65	60	60	65	65	65	60	65	65	65	60	65	65	65
0.0250										55	55	55	50		60	60	65	65	65	60	60	65	65	65	60	65	65	65	60	60	60	
0.0300										55	55	50			55	55	60	60	60	55	55	60	60	60	55	60	60	60	60	60	60	
0.0400										50	50				55	55	60	60	55		55	60	60	60		60	60	60				
0.0500															55	55	60	55	50		55	60	60	60								
0.0600															50	50	55	50				55	55	55								

*F_n is the desired net depth of application.

13.4.6 Design Limitations

The design inflow rate, depth of flow, border slope and length should not exceed established limitations.

Maximum flow rates. Flow rates in border irrigation must be nonerosive. The maximum flow rate per unit width should not exceed the flow as given by the following empirical criteria:

For non-sodforming crops, such as alfalfa and small grains:

$$Q_{u\,max} = (1.765 \times 10^{-4})\, S_o^{-0.75} \quad \text{[13.10]}$$

For well-established, dense sod crops

$$Q_{u\,max} = (3.53 \times 10^{-4})\, S_o^{-0.75} \quad \text{[13.11]}$$

where Q_u is the inflow rate (m^2/s), and S_o is the border slope (m/m).

The maximum inflow rate for various border slopes and crop conditions are given in Table 13.7.

Maximum depth of flow. The depth of flow at the head end of the border strip must not exceed the border ridge height, less an allowance for freeboard of approximately 25 percent of the ridge height. Flow depths should generally not exceed 150 mm. Greater depth is practical on some soils, but flow depths exceeding 200 or 250 mm should seldom be considered.

Flow depth—high gradient borders. The normal depth of flow at the upper end of border strips with slopes greater than 0.4 percent may be computed from equation [13.12].

$$d_n = 1000\, Q_u^{0.6}\, n^{0.6}\, S_o^{-0.3} \quad \text{[13.12]}$$

TABLE 13.7 MAXIMUM INFLOW RATES, Qu, FOR NON-SOD AND SOD CROP CONDITIONS

Border slope, S_o (m/m)	Crops: Non-sod	Crops: Sod
	10^{-3} m^2/s	10^{-3} m^2/s
0.0005	52.8	106.0
0.001	31.4	62.8
0.002	18.7	37.3
0.003	13.8	27.5
0.004	11.1	22.2
0.005	9.39	18.8
0.0075	6.93	13.9
0.01	5.58	11.2
0.015	4.12	8.24
0.02	3.32	6.64
0.025	2.81	5.62
0.03	2.45	4.90
0.04	1.97	3.95
0.05	1.67	3.34
0.06	1.46	2.91

TABLE 13.8 NORMAL DEPTH OF FLOW AT UPPER END OF HIGH GRADIENT BORDERS

Border slope, S_o (m/m)	Normal flow depth, d_n: Inflow Rate, Q_u (m^2/s)	Manning's n: 0.04	0.15	0.25
0.005	0.0005		16.4	22.3
	0.001		24.9	33.8
	0.01	44.8	99.1	134.6
	0.02	67.9	150.2	204.0
0.0075	0.0005		14.5	19.8
	0.001		22.0	29.9
	0.01	39.7	87.7	119.2
	0.02	60.2	133.0	180.7
0.01	0.0005		13.3	18.1
	0.001		20.2	27.5
	0.01	36.4	80.5	109.3
	0.02	55.2	122.0	165.7
0.015	0.0005		11.8	16.0
	0.001		17.9	24.3
	0.01	32.2	71.3	96.8
	0.02	48.9	108.0	146.7
0.02	0.0005			14.7
	0.001		16.4	22.3
	0.01	29.6	65.4	88.8
	0.02	44.8	99.1	134.6
0.025	0.0005			13.8
	0.001		15.4	20.9
	0.01	27.7	61.1	83.1
	0.02	41.9	92.7	125.9
0.03	0.0005			13.0
	0.001		14.5	19.7
	0.01	26.2	57.9	78.6
	0.02	39.7	87.7	119.2
0.04	0.0005			12.0
	0.001		13.3	18.1
	0.01	24.0	53.1	72.1
	0.02	36.4	80.5	109.3
0.05	0.0005			
	0.001		12.5	16.9
	0.01	22.5	49.7	67.5
	0.02	34.1	75.3	102.3
0.06	0.0005			
	0.001		11.8	16.0
	0.01	21.3	47.0	63.0
	0.02	32.2	71.3	96.8

NOTE: Values given in this table are solutions to equation [13.12]. In practice, calculations to ± 1 mm are adequate.

Find:

Opportunity time required (T_n)
Lag time (T_L)
Design inflow rate (Q_u)
Required inflow time (T_a)
Reduced inflow rate with end blocks (Q_{ue})
Maximum flow rate (Q_u)
Maximum flow depth (d)
Minimum flow rate (Q_u)
Maximum slope (S)
Maximum border length - open end borders
Allowable length extension with end blocks

Solution:

$$T_n = \left(\frac{100 - 7.0}{1.186}\right)^{1/0.756} = 320 \text{ min} \quad \text{..... (equation [13.2])}$$

From Table 13.5, the recession lag time, T_L, must be such that $7.5 < T_L < 21.4$. Make a first guess as $T_L = 17$ min. Then,

$$Q_u = \frac{(0.00167)(100)(250)}{(320 - 17)(70)} = 0.00197 \text{ m}^2/\text{s} \quad \text{..... (equation [13.6])}$$

Revise $T_L = 13$ min from interpolation in Table 13.5

$$Q_u = \frac{(0.00167)(100)(250)}{(320 - 13)(70)} = 0.00194 \text{ m}^2/\text{s} \quad \text{..... (equation [13.6])}$$

Check T_L:

$$T_L = \frac{(0.15)^{1.2}\,(0.00167)^{0.2}}{120\left[0.001 + \dfrac{(0.0094)(0.15)(0.00167)^{0.175}}{(320)^{0.88}\,(0.001)^{0.5}}\right]^{1.6}} \quad \text{. (equation [13.8])}$$

$$= 13.1 \text{ min} \cong 13 \text{ min}$$

Reduced inflow rate with end blocks, without length extension:

$$Q_{ue} = \frac{0.00194}{1 + (0.80)(0.75)(1 - 0.70)} = 0.00164 \text{ m}^2/\text{s} \quad \text{... (equation [13.19])}$$

Border length extension with end blocks:

$$L_e = 100/[(1000)(0.001)] = 100 \text{ m, based on slope} \quad \text{..... (equation [13.17])}$$

$$L_e = (1 - 0.70)(0.80)(0.75)(250) = 45 \text{ m, based on runoff} \quad \text{..... (equation [13.18])}$$

Length extension limited by runoff, thus extended length =

$250 + 45 = 295$ m.

Maximum flow rate:

$$Q_{u\,max} = (1.765 \times 10^{-4})(0.001)^{-0.75} = 0.031 > 0.00194 \quad \text{..... (equation [13.10])}$$

Maximum flow depth:

$$d = (2454)(13)^{3/16}\,(0.00194)^{9/16}\,(0.15)^{3/8} \quad \text{..... (equation [13.13])}$$

$$= 58 \text{ mm} < 150 \text{ mm}$$

Minimum flow rate:

$$Q_{u\,min} = \left[(5.95 \times 10^{-6})(250)(0.001)^{0.5}\right]/0.15 \quad \text{.. (equation [13.14])}$$

$$= 0.00031 < 0.00194 \text{ m}^2/\text{s}$$

Maximum slope:

$$S_{o\,max} = \left[\frac{(0.15)(100)}{(0.0117)(70)(320)}\right]^2 \quad \text{..... (equation [13.15])}$$

$$= 0.0033 \text{ m/m which is} > 0.001 \text{ m/m}$$

Maximum length—open end border:

$$L_{max} = \frac{(0.031)(70)(320 - 13)}{(0.00167)(100)} = 3989 \text{ m} > 250 \text{ m} \quad \text{.. (equation [13.16])}$$

Preparation of design charts will greatly facilitate use of the design relationships

TABLE 13.11. MAXIMUM SLOPES, $S_{o\,max}$, FOR BORDER IRRIGATION AS LIMITED BY MINIMUM DEPTH OF FLOW REQUIREMENTS OR BY A MINIMUM BORDER LENGTH OF 30 METERS

Intake family	Net appl. depth, F_n (mm)	Manning's n: 0.04 — Water application efficiency, E (percent): 50	55	60	65	70	75	0.15: 50	55	60	65	70	75	0.25: 50	55	60	65	70	75
		percent						percent						percent					
0.3	25	0.22	0.18	0.15	0.13			1.12	0.93	0.78	0.66			3.11	2.57	2.16	1.84		
	50	*	*	*	*			0.40	0.33	0.28	0.24			1.11	0.92	0.77	0.66		
	75	*	*	*	*	*		0.25	0.21	0.18	0.15	0.13		0.70	0.58	0.49	0.42	0.36	
	100	*	*	*	*	*		0.19	0.16	0.13	0.11	0.10		0.52	0.43	0.36	0.31	0.27	
0.5	25	0.22	0.18	0.15	0.13			3.08	2.54	2.14	1.82			8.55	7.06	5.94	5.06		
	50	0.09	0.07	0.06	0.05			1.23	1.02	0.85	0.73			3.42	2.83	2.37	2.02		
	75	0.06	0.05	*	*			0.82	0.68	0.57	0.49	0.42		2.29	1.89	1.59	1.35	1.17	
	100	*	*	*	*			0.64	0.53	0.44	0.38	0.33		1.78	1.47	1.23	1.05	0.91	
	125	*	*	*	*			0.53	0.44	0.37	0.32	0.27		1.48	1.22	1.03	0.88	0.76	
1.0	25	0.80	0.66	0.56	0.48	0.41	0.36	6.9†	7.8†	7.84	6.69	5.76	5.02	6.9†	7.8†	8.7†	9.7†	10.7†	11.8†
	50	0.35	0.29	0.24	0.21	0.79	0.56	4.92	4.07	3.42	2.91	2.51	2.18	11.9†	11.3	9.49	8.09	6.97	6.08
	75	0.24	0.20	0.17	0.14	0.12	0.11	3.45	2.85	2.39	2.04	1.76	1.53	9.57	7.91	6.65	5.66	4.88	4.25
	100	0.20	0.16	0.14	0.12	0.10	0.09	2.76	2.28	1.92	1.63	1.41	1.23	7.66	6.33	5.32	4.54	3.91	3.41
	125	0.17	0.14	0.12	0.10	0.09	0.07	2.35	1.94	1.63	1.39	1.20	1.04	6.53	5.40	4.53	3.86	3.33	2.90
1.5	25	1.66	1.37	1.15	0.98	0.85	-0.74	4.2†	4.8†	5.4†	6.0†	6.6†	7.2†	4.2†	4.8†	5.4†	6.0†	6.6†	7.2†
	50	0.75	0.62	0.52	0.44	0.38	0.33	7.1†	8.1†	7.34†	6.26†	5.4†	4.7†	7.1†	8.1†	9.1†	10.1†	11.2†	12.3†
	75	0.54	0.44	0.37	0.32	0.27	0.24	7.56	6.25	5.25	4.47	3.86	3.36	8.9†	10.1†	11.4†	12.42	10.71	9.33
	100	0.43	0.36	0.30	0.26	0.22	0.19	6.14	5.07	4.26	3.63	3.13	2.73	10.3†	11.6†	11.83	10.09	8.70	7.58
	125	0.38	0.31	0.26	0.22	0.19	0.17	5.28	4.37	3.67	3.13	2.70	2.35	11.3†	12.13	10.19	8.69	7.49	6.52
2.0	25	2.80	2.31	1.94	1.65	1.43	1.24	3.0†	3.4†	3.8†	4.2†	4.7†	5.1†	3.0†	3.4†	3.8†	4.2†	4.7†	5.1†
	50	1.30	1.07	0.90	7.6	6.6	5.8	5.0†	5.6†	6.3†	7.0†	7.8†	8.11	5.0†	5.6†	6.3†	7.0†	7.8†	8.5†
	75	0.94	0.78	0.65	0.56	0.48	0.42	6.2†	7.0†	7.9†	7.81	6.74	5.87	6.2†	7.0†	7.9†	8.7†	9.6†	10.6
	100	0.77	0.64	0.54	0.46	0.39	0.34	7.0†	8.0†	7.51	6.40	5.52	4.81	7.0†	8.0†	8.9†	10.0†	11.0†	12.0†
	125	0.67	0.56	0.46	0.39	0.34	0.30	7.7†	7.75	6.51	5.55	4.78	4.17	7.7†	8.7†	9.8†	10.9†	12.0†	11.57
3.0	25	1.8†	2.7†	2.3†	2.6†			1.8†	2.1†	2.3†	2.6†			1.8†	2.1†	2.3†	2.6		
	50	2.77	2.29	1.92	1.64			3.0†	3.4†	3.8†	4.2†			3.0†	3.4†	3.8†	4.2		
	75	2.02	1.67	1.41	1.20	1.03		3.7†	4.2†	4.7†	5.2†	5.8†		3.7†	4.2†	4.7†	5.2	5.8	
	100	1.67	1.38	1.16	0.99	0.85		4.2†	4.7†	5.3†	5.9†	6.5†		4.2†	4.7†	5.3†	5.9	6.5	
	125	1.46	1.20	1.01	0.86	0.74		4.6†	5.2†	5.8†	6.5†	7.1†		4.6†	5.2†	5.8†	6.5	7.1	
4.0	25	1.3†	1.5†	1.6†	1.8†			1.3†	1.5†	1.6†	1.8†			1.3†	1.5†	1.6†	1.8†		
	50	2.1†	2.4†	2.7†	2.83			2.1†	2.4†	2.7†	3.0†			2.1†	2.4†	2.7†	3.0†		
	75	2.6†	2.87	2.41	2.06	1.77		2.6†	2.9†	3.3†	3.6†	4.0†		2.6†	2.9†	3.3†	3.6†	4.0†	
	100	2.89	2.39	2.00	1.71	1.47		2.9†	3.3†	3.7†	4.1†	4.5†		2.9†	3.3†	3.7†	4.1†	4.5†	
	125	2.53	2.09	1.76	1.50	1.29		3.2†	3.6†	4.0†	4.5†	5.0†		3.2†	3.6†	4.0†	4.5†	5.0†	

*Not adapted for graded borders.
†Slope limited by minimum border length of 30 m.

where Q_u is the maximum flow rate in m²/s, as determined by equations [13.10] or [13.11].

13.4.7 Design of Borders with No Runoff

Higher irrigation application efficiencies and elimination of surface runoff can be achieved by modification of the border design. This may be accomplished by blocking the end and reducing the inflow rate, or extending the border length and impounding the runoff on the length extension.

Border extensions. The length extension is limited by the lesser of:

1 The length, L_e (m), that can be covered by an impoundment whose maximum depth is equal to the desired net application depth:

$$L_e = F_n/1000\, S_o \qquad [13.17]$$

where F_n (mm) is the desired net application depth and S_o is the border slope (m/m). Removal of all or part of the slope at the lower end of the border by land leveling will increase the length extension as limited by slope.

2 The length that can be adequately irrigated with the volume of runoff from the open-end border strip:

$$L_e = (1 - E/100)\, r_i\, r_n\, L \qquad [13.18]$$

where L_e is the allowable length extension with end blocks (m), E is the water application efficiency (percent), r_i and r_n are factors that express the effect of intake and roughness on runoff, and L is the normal design length (m).

Empirical values for factors r_i and r_n are given in Table 13.12.

Borders with end blocks (no extensions). On fields where the length of the border is fixed, use of end blocks and elimination of runoff permits reduction of the inflow rate. The reduced inflow rate required can be estimated from equation [13.19].

$$Q_{ue} = \frac{Q_u}{1 + r_i\, r_n\, (1 - E/100)}, \qquad [13.19]$$

where Q_{ue} is the inflow rate per unit width of border using end blocks (m²/s), Q_u is the inflow rate determined for the border length without end blocks (m²/s), E is the efficiency (percent), and r_i and r_n are empirical factors as given in Table 13.12. Equation [13.19] assumes the reduction in flow rate will not be large enough to result in a significant change in recession-lag time.

TABLE 13.12. INTAKE AND ROUGHNESS FACTORS FOR ESTIMATING POTENTIAL RUNOFF

Intake family	Intake factor, r_i (dimensionless)	Manning coefficient (n)	Roughness factor, r_n (dimensionless)
0.3	0.90	0.10	0.80
0.5	0.80	0.15	0.75
1.0	0.70	0.20	0.70
1.5	0.65	0.25	0.65
2.0	0.60		
3.0	0.50		
4.0	0.40		

13.4.8 Sample Calculation

Given:

Intake family	0.5
Net depth of application (F_n)	100 mm
Border slope (S_o)	0.001 m/m
Manning roughness coefficient (n)	0.15
Estimated water application efficiency (E)	70 percent
Allowable flow depth	150 mm
Border length	250 m
Crop	Alfalfa

13.4.9 Selection of Headland Facilities

Comments applying to basins (Sec. 13.3.6) apply, in general, to border headland facilities.

13.5 CONTOUR LEVEE IRRIGATION

13.5.1 Description*

In contour levee irrigation water is applied to sloping basins. Each basin is bounded by two levees which are on land contours, and two levees (sometimes called checks) which are essentially perpendicular to the contour levees. For non-rice crops, water is introduced into the basin at its highest point until the irrigation requirement has been met, and is then removed through surface drains located along the lower contour levee. For paddy rice, water is usually circulated through the basins throughout most of the season. In contour levee irrigation basins are sloped from contour to contour and emptied primarily by drainage. This is in contrast to basin irrigation (Section 13.3) in which the basins are level and empty by percolation into the soil.

13.5.2 Applicability

Crops. This system is extensively used for paddy rice. It is also used for pasture grasses, hay crops, alfalfa, small grains and row crops which can withstand temporary flooding (e.g., cotton, corn, soybeans, grains and peanuts).

Soils. Soils of medium to fine texture, having waterholding capacities equal to or greater than 100 mm per m, and a total waterholding capacity in the root zone of at least 60 mm are suitable. For all crops other than rice, soil infiltration rates should not exceed Soil Conservation Service family 0.3 (Section 13.1). For rice, only soils whose infiltration rates do not exceed those of SCS intake family 0.1 are suitable.

Slopes. The general land topography upon which contour levee irrigation is applicable must have average slopes of less than 0.5 percent. Slopes within basins are limited on the upper side by soil erosion limitations (usually between 0.05 and 0.3 percent) and on the lower side by the necessity to provide drainage (0.05 to 0.15 percent). Thus, land forming is often needed to alter the slope within basins and to remove minor surface irregularities.

Climate. A large amount of work is necessary in the construction of the levees and checks, and they must be reconstructed after each cultivation. Therefore, except in the case of rice irrigation, unless most early season water is supplied to the crop by rainfall the system is impractical because of the need of frequent reconstruction of levees.

13.5.3 Advantages

The system is applicable to low-intake soils which are difficult to irrigate by other surface methods. It makes maximum use of seasonal rainfall. Water can be uniformly distributed, resulting in high water application efficiencies. The system can be designed to handle high rainfall, with a minimum of soil erosion. Installation costs are low compared to most other methods, especially if little land forming is needed. Large areas can be handled efficiently by a single irrigator.

*This Section and Sections 13.6 to 13.8 are based primarily on USDA (1969).

13.5.4 Disadvantages

Crops must be able to withstand up to 12 hours of flooding. The system is applicable to soils of relatively low intake rates only. Some land forming is often needed. Large irrigation streams are necessary, and small net applications are not feasible with the system. Levees, ditches and structures require frequent maintenance.

13.5.5 Design

Irrigator constraints. The design must consider the time schedule of the irrigator, as well as the physical conditions. Thus, if the irrigator will work 12 hours per day (or at least is available for changing water on such a schedule), and he wishes to work only eight days out of the irrigation interval, this will be a constraint on field layout (and inflow rates). Thus, the possible numbers of basins to be irrigated would conveniently be 8, 16, 24, . . . etc. The average basin size would be the field area divided by the number of basins selected.

Land preparation. Although there is no specific need for land leveling of contour-levee irrigated lands, some land forming is usually desirable. Minor irregularities must be removed if the basins are to be uniformly irrigated. Because the normal vertical interval between contour levees is 100 mm, fields which have not been brought to a uniform slope will have basins of varying widths, and these are difficult to irrigate uniformly. The ideal situation is a series of basins of identical size, shape, and slope. Land may be leveled either before or after the levees are installed. In the former case, basins are usually sloping planes, while in the latter they are level.

The levee system can be layed out once the topographic map is completed and the available water stream is known. Limitations on basin size are based on several factors.

Wind considerations. Winds will develop waves in large basins and these waves can cause erosion of levees. A general rule is to limit the maximum length of the basin in a direction parallel to prevailing winds to 120 m.

Drainage considerations. The drainage system of the basins must remove not only the applied irrigaton water which is in excess of that infiltrated or evaporated form the surface, but also storm water which falls in the basin. The maximum drain length should be 200 m. If a basin has only one outlet location, then its maximum length should be 200 m. If the basin drains from more than one outlet, a longer basin can be used. Special climatic conditions, which could cause large runoff rates from the basin, may impose even more stringent requirements.

Placement of contour levees. As the name implies, the levees are on contour. If they are constructed after land leveling, they are put on the existing contour. If they are layed out before leveling, they must be located where the final contours will be.

Vertical interval. The most common vertical interval, V_i, for contour levees is 60 to 120 mm. In flat lands they may be reduced so that basin widths do not become excessive. In steep lands they may be increased somewhat so that basin widths are not too small. A minimum width of 12 m is about optimal. Greater vertical intervals require higher levees to properly contain the water.

Levee dimensions. Levees must be constructed to the following minimum height.

$$H = V_i + F_g + d_f + S \quad [13.20]$$

where H is the levee height, V_i is the vertical interval between levees, F_g is the gross depth of water to be applied, d_f is the freeboard, and S is the allowance for settlement. The freeboard should be no less then 80 mm and the allowance for settling no less than 90 mm.

Side slopes of levees depend to some extent upon the crop to be grown and tillage practices. If the levees are removed and rebuilt frequently, a common practice when cultivating row crops grown in basins, then side slopes should be no steeper than 1-1/2 horizontal to 1 vertical. When levees are permanent (pasture, forage production, etc.) then side slopes should be no steeper than 3 or 4 to 1. This reduces cattle damage and allows levees to be easily crossed by machinery.

Drainage channels. The drainage channel, constructed parallel to and on the upper side of each levee, should be no less than 150 mm deep and have side slopes no steeper than 1-1/2 horizontal to 1 vertical. Storm drainage requirements may dictate even larger drainage ditches, and crossing by machinery may require different bank slopes. The material excavated in drain construction is used in forming the levees.

Irrigation stream size—non-rice crops. The necessary minimum stream size for all crops except rice is based upon two criteria: (a) The stream must be large enough to meet the entire field's requirement for water—evapotranspiration, intentional leaching required to maintain the salt balance, and unavoidable deep percolation losses; (b) The stream must be large enough (into each basin) to permit coverage of the average size basin in no more than one-fourth the time necessary to infiltrate the required net depth. This latter criterion assures relatively uniform distribution of applied water.

The first criterion is expressed as follows.

$$Q = \frac{A F_g}{360 f h}, \quad [13.21]$$

where Q is the flow rate (m^3/s) into the entire field (sometimes called system capacity), A is the field area in ha, F_g is the gross application depth (mm), f is the actual number of days of irrigating, and h is the number of irrigation hours per irrigation day.

The procedure for designing to meet the second criterion is a quasi-rational one. It requires knowledge of the basin size which will be used. In outline form it is as follows:

1 Determine the time, T_n, to infiltrate the desired net application. This is a function of the net application and the soil infiltration characteristics (Section 13.1).

2 Determine the average depth infiltrated during the time required to cover the entire basin. This time is assumed to be $T_n/4$. In making this calculation it is assumed that by the time the basin has been covered, the average depth infiltrated is equal to the value of the infiltration function at time $T_n/8$. Thus,

$$\bar{z}_1 = z(T_n/8) \quad [13.22]$$

where $\bar{z}_1$ is the average depth infiltrated (mm) and z(t) is the value of the cumulative infiltration function at time t. The infiltration function z(t) may be selected from one of the SCS families.

3 Determine the average depth of water in surface storage at the time the basin is just filled. In this calculation it is assumed that the surface storage volume is wedge-shaped, with zero depth at the upper levee and a depth at the lower levee equal to V_i (mm). (The additional water stored in the drainage ditch is ignored.) Thus,

$$\bar{y}_1 = V_i/2 \quad [13.23]$$

where $\bar{y}_1$ is the average depth of water in surface storage (mm).

4 Determine the required flow rate per unit area irrigated to provide the depth of water equal to the sum of $\bar{z}_1$ (mm) and $\bar{y}_1$ (mm) in the time $T_n/4$ (min).

$$q_1 = \frac{40(\bar{z}_1 + \bar{y}_1)}{60\, T_n} \quad [13.24]$$

where q_1 is the required inflow (m^3/s per ha).

The above four steps represent a quasi-rational method for two reasons. By requiring the water to spread over the basin in the time $T_n/4$ it is reasonable to assume that the average infiltration depth is somewhere between $z(T_n/4)$ and zero. However, there is no reason to believe this average depth would be $z(T_n/8)$ because both the infiltration and advance functions are nonlinear. In addition, the filling of the basin is through an unsteady wave. Thus, at the time the water reaches the upper levee, the water surface is not necessarily horizontal (although it might approximate that very closely).

5 Determine the area which must be irrigated with each set. The total set time, T_s (min) is computed assuming that the basin is covered in one fourth the net infiltration time (T_n), and each point in the basin receives the desired net application or more.

$$T_s = \frac{5}{4} T_n \quad [13.25]$$

This must be less than or equal to 60 h, the irrigation time each day. If this is not the case, the field is not applicable to contour levee irrigation.

6 Determine number of sets per day, N, and total area irrigated per set, A_1.

$$N = \mathrm{Int}\,(60h/T_s) \quad [13.26]$$

where Int(x) means the integer portion of x.

$$A_1 = A/(Nf) \quad [13.27]$$

7 Determine the system capacity based upon the necessary unit inflow (determined from soil and topographic conditions) and the total area irrigated per set.

$$Q_1 = A_1 q_1 \quad \text{[13.28]}$$

8 Complete design by laying out basins according to criteria set forth previously, and the total area irrigated per set. The average basin size must be one, or a simple fraction of, A_1.

Irrigation stream size—rice. Typically, there are three steps in the irrigation of rice. In the *flushing* period, the soil is wet to field capacity after dry planting. The basins may be drained and following that comes *flooding*. In this step all the basins are flooded. They must remain flooded during the growth of the crop, and this is known as *maintaining the flood*. The calculation of appropriate inflow rates for each of these phases follows:

1 **Flushing.** This calculation is carried out according to steps 1, 2 and 3 outlined above, except that the necessary flow rate for one entire basin must be considered. Thus,

$$Q_1 = q_1 \frac{A}{N} \quad \text{[13.29]}$$

where Q_1 is the flow rate required during the flushing stage (m^3/s), N is the number of basins in the field, and q_1 is as defined previously. The basin size, A_1, is A/N. Clearly, Q, the system capacity, must be greater than or equal to Q_1.

2 **Flooding.** During this period the entire irrigated area is covered with water to some predetermined depth, F_f (often 70 to 100 mm). The duration of flooding may be taken as the time necessary for the crop to deplete one half of the available moisture in the root zone, at the maximum evapotranspiration rate.

$$T_e = \frac{720\ R_d}{E_t'}, \quad \text{[13.30]}$$

where T_e is the duration of flooding (min), R_d is the root zone storage capacity (mm) and E_t' is the peak consumptive use rate (mm/d). The depth of water which must be supplied during this period is equal to the sum of the amount necessary to saturate the root zone, plus the average depth of the wedge-shaped surface storage described above ($\bar{y}_1$), plus the depth F_f, plus the amount lost to deep percolation. The flow rate must be determined for the entire irrigated area.

$$Q_2 = \frac{[R_s + (V_i/2) + F_f + L_p]\,A}{6\ T_e} \quad \text{[13.31]}$$

where Q_2 is the flow rate into the field during flooding (m^3/s), R_s is the depth required to saturate the root zone (mm), and L_p is the estimated deep percolation loss (mm) during the time period T_e.

3 **Maintaining the flood.** The water required in this stage is equal to that lost through deep percolation plus that required for consumptive use.

$$Q_3 = \left(\frac{L_p}{6T_e} + \frac{E_t}{8640}\right) A \quad \text{[13.32]}$$

where Q_3 is the required flow rate (m^3/s) to maintain the flood and E_t is the average consumptive use rate (mm/d).

13.5.6 Sample Problem—Non-Rice Crops

Given:

Intake function	$z = 0.78T^{0.691} + 7.0$ mm
Net depth of application (F_n)	65 mm
Vertical interval (V_i)	60 mm
Estimated water application efficiency (E)	70 percent
Irrigation days (f)	8 days
Irrigation time each day (h)	12 h
Total area irrigated (acre)	32 ha
Land slope (average)	0.075 percent

Required:

Total area irrigated at one set (A_1)	ha
System capacity (maximum of Q or Q_1)	m^3/s
Basin dimensions	m X m

Solution:

The intake function is for a soil with a slower intake than a 0.3 SCS family, so it is satisfactory for basins. Land slope is also within the recommended range.

$$F_g = 100\ F_n/E = (100)(65)/70 = 93 \text{ mm}$$

$$Q = \frac{(32)(93)}{(360)(8)(12)} = 0.086\ m^3/s \quad \text{(equation [13.21])}$$

(1) $T_n = [(65 - 7.0)/0.78]^{1/0.691} = 511$ min .. (equation [13.2])

(2) $\bar{z}_1 = 0.78\left(\frac{511}{8}\right)^{0.691} + 7.0 = 21$ mm .. (equation [13.33])

(3) $\bar{y}_1 = 60/2 = 30$ mm ..... (equation [13.23])

(4) $q_1 = \frac{(40)(21 + 30)}{(60)(511)} = 0.0665\ m^3/s$ ..... (equation [13.24])

(5) $T_s = \frac{5}{4}\ 511 = 639$ min ..... (equation [13.25])

(6) $N = \text{Int}\ \frac{(60)(12)}{639} = 1$ ..... (equation [13.26])

$A_1 = \frac{32}{(1)(8)} = 4$ ha ..... (equation [13.27])

(7) $Q_1 = (0.0665)(4) = 0.266\ m^3/s$ ..... (equation [13.28])

Thus, the system capacity is governed by Q_1 (not Q) and is 0.266 m³/s. The correct unit stream size is one fourth this (4 ha per basin, 1 basin irrigated at a time).

$$q_1 = 0.266/4 = 0.066 \text{ m}^3/\text{s}$$

With a vertical interval of 60 mm and an average ground slope of 0.075 percent, the width of a basin is

$$W = 60/0.00075 = 80 \text{ m}$$

The length of a basin is

$$L = A_1 /W = 4 \times 10\,000/80 = 500 \text{ m}$$

This exceeds the allowable length for drainage, and half this length could be used if the basins could drain at both ends. The contour levees should be perpendicular to the prevailing wind, if possible.

13.5.7 Sample problem—Rice Crops

Given:

Intake function	$z = 0.78\ T^{0.691} + 7.0$ mm
Net depth of application (F_n)	45 mm
Vertical interval (V_i)	60 mm
Total area irrigated (acre)	32 ha
Number of basins (N)	16
Available water holding capacity of root zone	91 mm
Saturated moisture capacity of root zone	187 mm
Permeability of restricting layer	0.51 mm/h
Peak-period consumptive use rate (E_t')	7.6 mm/day
Average consumptive use rate (E_t)	6.4 mm/day
Estimated deep percolation loss	10 mm/day

Find:
Minimum stream sizes for flushing, flooding and maintaining flood.

Solution:
This soil is just within the 0.1 SCS intake family band, which can be demonstrated by plotting a few points on Fig. 13.1. It is therefore suitable for rice irrigation.

(1) $$T_n = [(45 - 7.0)/0.78]^{1/0.691} = 277 \text{ min}$$

$$\bar{z}_1 = 0.78\left(\frac{277}{8}\right)^{0.691} + 7.0 = 16 \text{ mm} \quad \text{.... (equation [13.22])}$$

$$\bar{y}_1 = 45/2 = 22.5 \text{ mm} \quad \text{..... (equation [13.23])}$$

$$Q_1 = \frac{(40)(16 + 22.5)(32)}{(60)(277)(16)} = 0.185 \text{ m}^3/\text{s} \quad \text{.. (equation [13.24] and [13.29])}$$

(2) $$T_e = \frac{(720)(91)}{7.6} = 8620 \text{ min} \quad \text{..... (equation [13.30])}$$

$$L_p = \frac{(10)(8620)}{(24)(60)} = 60 \text{ mm}$$

$$Q_2 = \frac{[187 + (60/2) + 85 + 60]\ 32}{(6)(8620)} = 0.224 \text{ m}^3/\text{s}$$

..... (equation [13.31])

(3) $$Q_3 = \left[\frac{60}{(6)(8640)} + \frac{6.4}{8640}\right] 32 = 0.061 \text{ m}^3/\text{s}$$

..... (equation [13.32])

13.6 FURROW AND CORRUGATION IRRIGATION

13.6.1 Description

Small, evenly spaced, shallow channels are installed down or across the slope of the field to be irrigated. Water is turned in at the high end and conveyed in the small channels to the vicinity of plants growing in, or on beds between, the channels. Water is applied until the desired application and lateral penetration is obtained.

The method is separated into types according to the kinds of crops and size of channel. Furrow irrigation is primarily used with clean tilled crops planted in rows, while corrugation irrigation is associated with noncultivated close-growing crops using small closely-spaced channels aligned down the steepest slope of the field. Corrugations are frequently formed after the crop has been seeded, and in the case of perennial crops, reshaped as needed to maintain the desired channel cross section. Water application principles are the same for both furrow and corrugation irrigation. The primary differences are channel size, shape and spacing, and retardance characteristics. The two terms will be used synonymously in this chapter.

Furrows and corrugations vary in shape and size. Most furrows in row crops are either parabolic in cross section or have flat bottoms and about 2 to 1 side slopes. Typical corrugations have 60-mm bottom widths, 1 to 1 side slopes, and depths of 100 to 150 mm.

13.6.2 Applicability

Most crops can be irrigated by the furrow or corrugation method except those grown in ponded water, such as rice. The furrow method is particularly suitable for irrigating crops subject to injury if water covers the crown or stem of the plants, as the crops may be planted on beds between furrows.

This irrigation method is best suited to medium to moderately fine textured soils of relatively high available water holding capacity and conductivities which allow significant water movement in both the horizontal and vertical directions. The method is suited to fine textured very slowly

permeable soils on level sites which permit water impoundment. On sloping sites excessive surface runoff occurs because these soils require very small streams for long periods of time to obtain the desired intake.

The movement of irrigation water applied by furrows or corrugations on coarse textured sands and loamy sands is mainly downward with very little lateral penetration. Efficient furrow or corrugation irrigation on these soils requires very short furrows, small application times, relatively close row spacings and small depths of water application. Furrow grades should be limited so that soil loss from rainfall runoff or irrigation flow is within allowable limits. Furrow grades should generally be 1.0 percent or less, but can be as much as 3.0 percent in arid areas where erosion from rainfall is not a hazard. In humid areas furrow grades should generally not exceed 0.3 percent; however, grades up to 0.5 percent may be permissible if the lengths are sufficiently short. A minimum grade of 0.03 to 0.05 percent in humid and sub-humid areas is necessary to assure adequate surface drainage. Maximum furrow grades for erosive soils can be estimated by the equation:

$$S_{max} \doteq 67/(P_{30})^{1.3} \quad \text{[13.33]}$$

where P_{30} is the 30-min rainfall in mm on a 2-year frequency and S_{max} is the maximum allowable furrow grade in percent. Grades on less erosive soils may be increased by approximately one fourth. Cross slope for furrowed fields with irrigation grades of 0.5 percent or greater should normally be limited to 1.0 percent, and lesser grades to 0.5 percent. Wherever practical, the furrow grade should be uniform.

Corrugations are commonly used on grades of more than 1.0 percent and less than 4.0 percent. They are not recommended in humid areas except for irrigation of perennial crops because of erosion hazards.

13.6.3 Advantages

Moderate to high application efficiency can be obtained if water management practices are followed and the land is properly prepared. Many different kinds of crops can be grown in sequence without major changes in design, layout, or operating procedures. The initial capital investment is relatively low on lands not requiring extensive land forming as the furrows and corrugations are constructed by common farm implements. Soils which form surface crusts when flooded can readily be irrigated because water moves laterally under the surface. Water does not contact plant stems and scalding is thus avoided. Excellent field surface drainage is obtained when furrow grades are sufficient and adequate outlet facilities are provided. Greater utilization of rainfall may be achieved by irrigation of alternate rows because the remaining available soil water storage capacity is greater than when each furrow is irrigated. Also, the initial intake rate is higher in the non-irrigated furrows.

13.6.4 Limitations

Erosion hazards on steep slopes limit use in climatic areas where precipitation intensities and volumes result in surface runoff, which, when concentrated in furrow channels may cause excessive soil erosion or crop damage from flooding. Surface runoff occurs except where the field is level and water is impounded until intake is completed. Labor requirements may

be high as irrigation streams must be carefully regulated to achieve uniform water distribution. Salts from either the soil or water supply may concentrate in the ridges and depress crop yields. Lateral spread of water in coarse textured soils may not be adequate to entirely wet the soil between furrows. Land leveling is normally required to provide uniform furrow or corrugation grades.

13.6.5 Design

A furrow or corrugation system may be designed only after gathering soils, crops, topography, size and shape of irrigable areas, farm equipment available, farmer operational practices, and farmer personal preferences for the proposed area. The designer must know the intake characteristics and water storage capacities of the various soils, which along with the crop to be grown, will determine the design depth of application and whether furrows or corrugations will be used. The topography will determine the direction and grade of furrows and lengths that will fit individual field boundaries. The farm equipment to be used will determine the spacing and maximum capacity of the furrows. The farmer's operational practices will influence the type of furrow or corrugation system to be designed and the irrigation operating schedules to be followed. Furrow flow rate and time of application are both influenced by the operational method to be used. The designs are based on a normal irrigation and adjusted for variations needed in application time, depth, and flow rates for specific irrigations during the season.

For acceptable uniformity and adequacy of application, the minimum time for water at any point is the time for intake of the net design application. The maximum time is limited by excessive deep percolation. The time water is available for intake at any point, the opportunity time, is the time interval between water advance and recession.

Design assumptions. Development of design relationships requires assumptions for intake vs. time, advance and recession rates, flow retardance, and intake as related to the furrow wetted perimeter. Rate of advance is assumed to be a function of water inflow rate, soil intake characteristics, furrow shape, grade, length, and roughness.

Design limitations. Flow rates into furrows must not exceed the channel capacity as limited by cross-sectional shape and size, slope, and hydraulic roughness. The inflow must advance at a rate which will achieve a reasonably uniform opportunity time throughout the length. Maximum flows are also limited to non-erosive velocities. Erosive soils may erode excessively when the flow velocity exceeds approximately 0.15 m/s while less erosive soils may safely withstand velocities of 0.18 m/s. Velocity and depth of flow for a given cross-section and grade depend on the roughness or retardance of the furrows. Manning roughness coefficients of 0.04 for furrows and 0.10 for corrugations are commonly used in estimating flow velocity.

Recession time, the time for water to disappear at any point after inflow ends is primarily affected by flow rate and by furrow length, shape, and slope for a specific soil. Recession time is relatively short and can be ignored when slopes exceed approximately 0.05 percent. Recession time is a very significant portion of the opportunity time on low gradient (< 0.05 percent) or level furrows. Excess opportunity time results in deep percolation, which should not exceed 20 to 25 percent of the design application depth.

Principles of control. There are three principles of water control which

define the type of furrow system. These are (a) gradient with open ends—continuous uniform inflow for the entire irrigation period and recirculation or recovery of surface runoff for reuse; (b) cutback inflow with open ends—reduced inflow rate after water has advanced to the furrow end and continuation of the reduced inflow for the time required to apply the desired application; and (c) level impoundment—impoundment of the the water until intake is achieved, thus eliminating surface runoff. Principle (c) is used for level furrows or where the total fall in the furrow length does not exceed the design depth of application.

Design equations.† Design equations for furrow and corrugation irrigation describe the relationship between length, inflow time, inflow rate, deep percolation, surface runoff, and field application efficiency for selected design values of application depth, soil intake rate, and furrow slope and spacing. Separate design equations and procedures are given for each of the three types. All depths are expressed as equivalent depths over the furrow spacing and unit length to achieve uniformity of expression with other surface irrigation methods where the entire surface is inundated. The water intake per unit length of furrow is directly related to the soil surface in contact with the water, i.e., the wetted perimeter. Intake, however, is in both vertical and horizontal directions in contrast with flooding or sprinkler methods where only vertical intake occurs. The wetted perimeter is increased by an empirical constant to account for horizontal intake caused by soil moisture gradients. This is called the adjusted wetted perimeter. The empirical relationship for adjusted wetted perimeters of typical furrow and corrugation shapes is

$$P = 0.265\,(Qn/S^{0.5})^{0.425} + 0.227 \qquad [13.34]$$

where P is the adjusted wetted perimeter (m), Q is the inflow rate (L/s), S is the slope or hydraulic gradient (m/m), and n is the Manning roughness coefficient. The value of P cannot exceed the furrow spacing W.

The time for water to advance to successive points along the furrow, from regression analysis of trial measurements, is a semi-logarithmic relationship of length, inflow rate, and slope.

$$T_T = \frac{x}{f} e^{\beta} \qquad [13.35]$$

where T_T is the advance time (min), x is the distance (m) from upper end of the furrow to point x (the maximum value of x is L, the field length), Q is the inflow rate (L/s), S is the furrow slope (m/m), f and g are advance coefficients varying with furrow intake family, and $\beta = gx/QS^{1/2}$. Intake family and advance coefficients are listed in Table 13.1. The maximum advance distance is reached when the total intake rate along the furrow equals the inflow rate. Because the intake function monotonically decreases, this condition is never reached. However, it is closely approximated and a maximum advance distance can occur.

Gradient furrows opportunity time. The time water is available for infiltration at any point is equal to the inflow time less the time required to advance plus the time water remains after inflow ends.

†Method of the Soil Conservation Service (USDA) 1979.

$$T_0 = T_1 - T_T + T_r \qquad [13.36]$$

where T_0 is the opportunity time at point x. Inflow time, T_1, is a constant for a specific irrigation. Advance time, T_T, increases at successive points downstream. Recession time, T_r is assumed zero for gradient open end furrows, whether the inflow rate is constant or cut back. With this assumption and equation [13.35], opportunity time for gradient furrows is

$$T_0 = T_1 - \frac{x}{f} e^{\beta} \qquad [13.37]$$

where T_0 and T_1 are in min. (The design inflow, T_1, is the sum of the time to advance to the end, plus the time to fill the root zone.) The average opportunity time, from integration of equation [13.35] between the limits of 0 and x and division by x is

$$T_{(0-x)} = T_1 - \frac{0.0929}{fx \left(\frac{0.305\beta}{x}\right)^2} (\beta - 1)e^{\beta} + 1 \qquad [13.38]$$

where $T_{[0-x]}$ is the average opportunity time (min) over the length x. The average opportunity time for the entire furrow, $T_{[0-L]}$ is determined from equation [13.38] with x = L. The gross water application is

$$F_g = \frac{60\,Q\,T_1}{W\,L} \qquad [13.39]$$

where F_g is gross application in mm, and W is furrow spacing in meters. Cumulative intake is expressed as an equivalent depth over the furrow spacing and unit length by the equation

$$F = (a\,T^b + c)\,P/W \qquad [13.40]$$

where F is the equivalent intake depth in mm, T is time in minutes, and a, b and c are intake family coefficients as listed in Table 13.1. The opportunity time required for intake of the selected net application depth, F_n, can be estimated by solution of equation [13.39] in the form

$$T_n = \left[(F_n \frac{W}{P} - c)/a\right]^{1/b} \qquad [13.41]$$

The average intake, (F_{0-L}), for the entire furrow length is determined by equation [13.40] with time T equal to the average opportunity time $T_{[0-L]}$. Equivalent surface runoff, outflow from the graded furrow, can be estimated as the difference between the gross application, F_g, and the average intake, $F_{[0-L]}$, or:

$$RO = F_g - F_{(0-L)} \qquad [13.42]$$

where RO is the average surface runoff depth in mm. Deep percolation is the average equivalent depth of water which infiltrates the soil in excess of the design application depth.

$$DP = F_{(0\text{-}L)} - F_n \quad \text{[13.43]}$$

where DP is deep percolation in mm. When the design application, at the option of the designer, is to be applied at a distance x which is less than the furrow length L, deep percolation is

$$DP = (F_{(0-x)} - F_n)\frac{x}{L} \quad \text{[13.44]}$$

where $F_{[0-x]}$ is the average intake (mm) over the length x, as computed from equations [13.38] and [13.40].

The application efficiency is

$$AE = 100\ F_n/F_g \quad \text{[13.45]}$$

where AE is the application efficiency (percent). The equation for efficiency when the design application is at a distance x which is less than the furrow length becomes:

$$AE = 100\ (F_{(0\text{-}x)} - DP)/F_g \quad \text{[13.46]}$$

The procedure for design of gradient furrows or corrugations may be simplified by preparation of design charts. Separate design charts such as shown in Figs. 13.3 and 13.4 can be prepared for any combination of roughness coefficient, intake-time relationship, net application depth, and furrow slope. The chart describes the relationship between length and inflow rate with inflow time, runoff, deep percolation, and application efficiency.

Computation example-gradient furrow

Given:

Furrow intake family I_f	0.3
Length, L	275 m
Slope, S	0.004 m/m
Furrow spacing, W	0.75 m
Roughness coefficient, n	0.04
Design application depth, F_n (over full length)	75 mm
Inflow rate, Q	0.6 L/s

Intake and advance coefficient for $I_f = 0.3$, from Table 13.1

a = 0.925	f = 7.61
b = 0.720	g = 1.904 X 10^{-4}
c = 7.0	

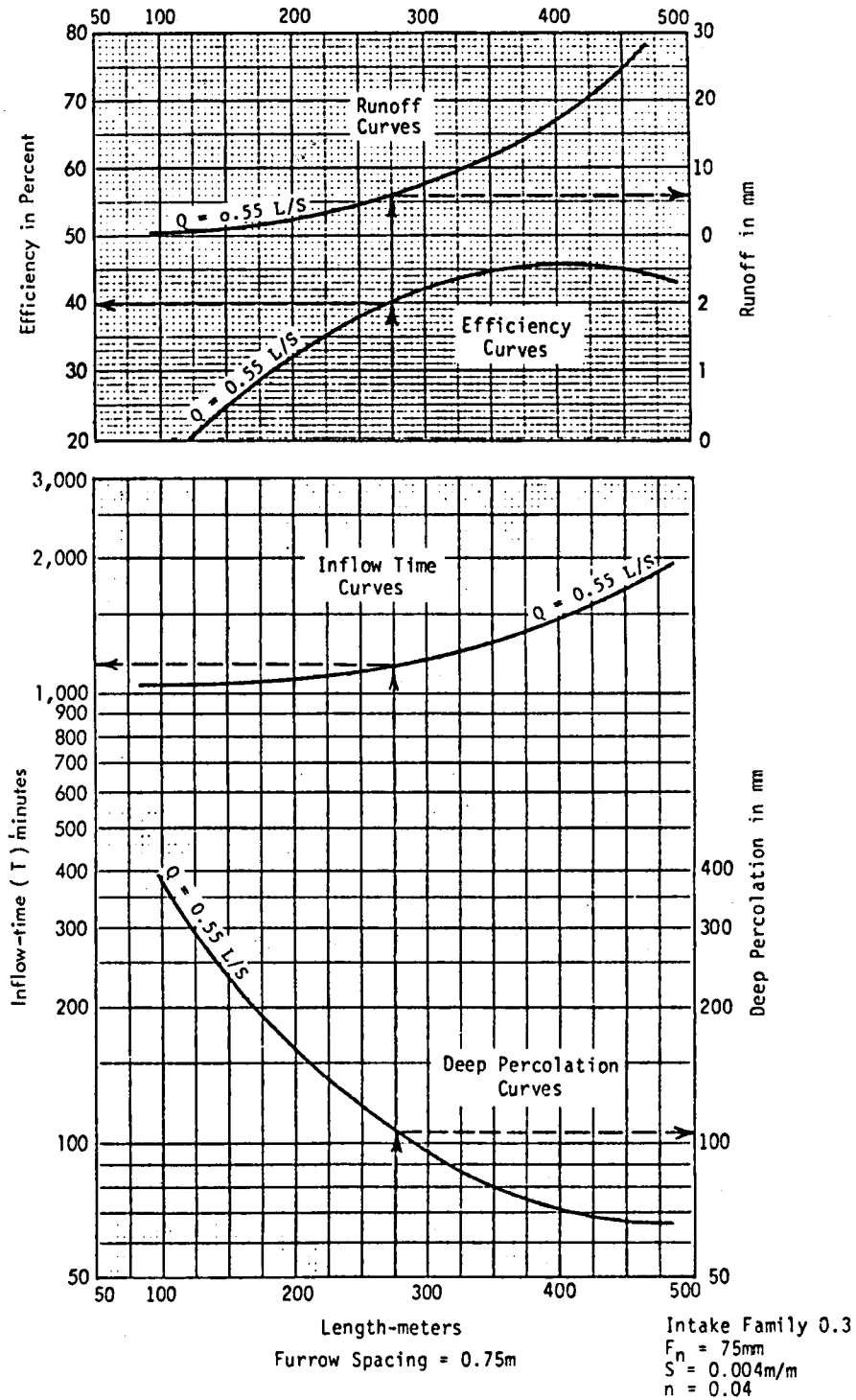


FIG. 13.3 Furrow irrigation design chart (USDA, 1979).

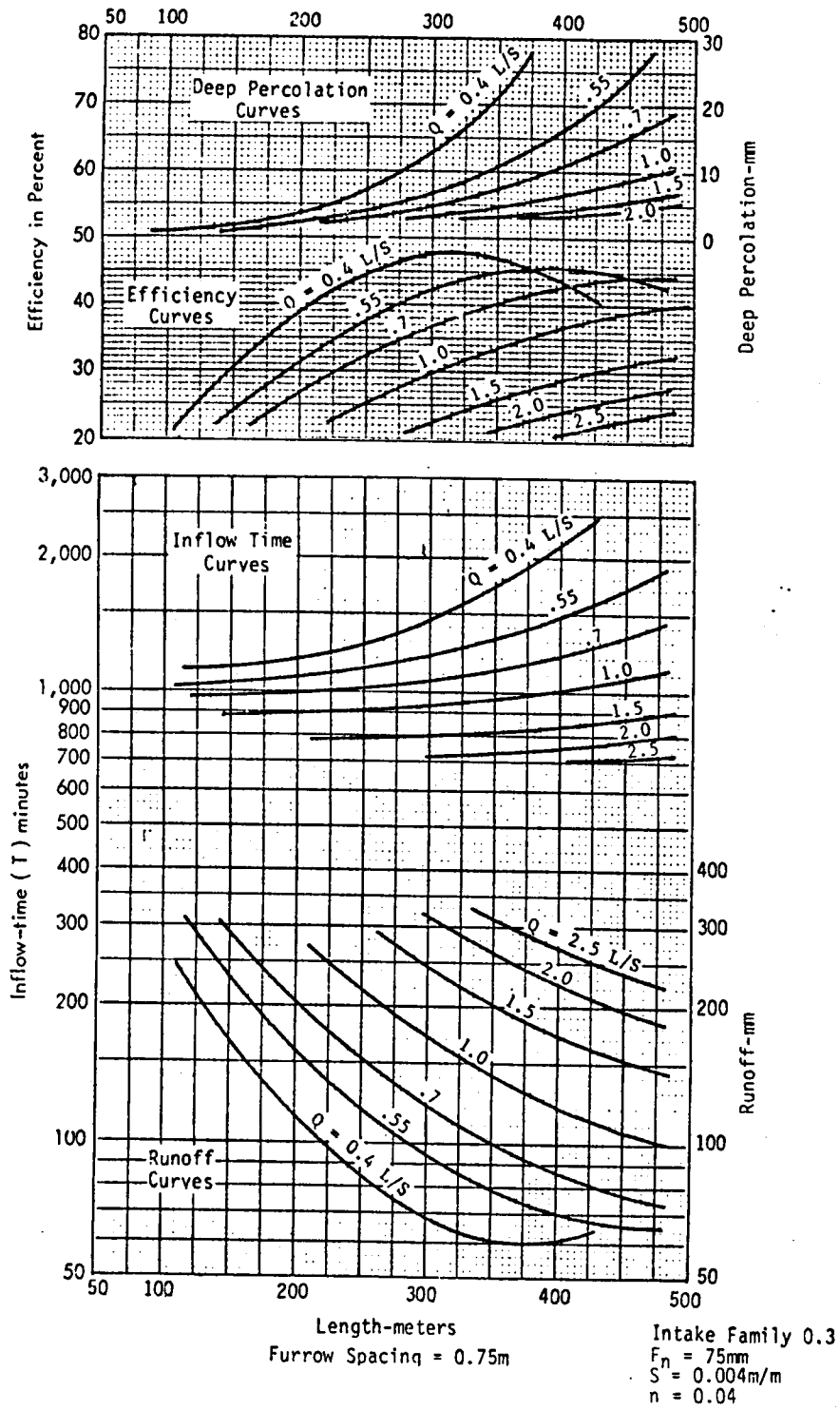


FIG. 13.4 Furrow irrigation design chart (USDA. 1979).

Find:

Design inflow time required, T_1
Surface runoff, RO
Deep percolation, DP
Application efficiency, AE

Solution:
Advance time

$$\beta = \frac{(1.904 \times 10^{-4})275}{0.6\sqrt{0.004}} = 1.38$$

$$T_T = \frac{275}{7.61} e^{1.38} = 143.6 \text{ min} \quad \text{(equation [13.35])}$$

Adjusted wetted perimeter

$$P = 0.265\,(0.6 \times 0.04/\sqrt{0.004})^{0.425} + 0.227 = 0.40 \text{ m}$$

(equation [13.34])

Net opportunity time

$$T_n = \left\{\left[(75 \times 0.75/0.40) - 7.0\right] /0.925\right\}^{1/0.720} = 999 \text{ min}$$

(equation [13.41])

Design inflow time (sum of T_T and T_n)

$$T_1 = 143.6 + 999 = 1143 \text{ min}$$

Gross application

$$F_g = \frac{60(0.6)(1143)}{(0.75)\,(275)} = 200 \text{ mm} \quad \text{(equation [13.39])}$$

Average opportunity time

$$T_{(0\text{-}L)} = 1143 - \frac{0.0929}{7.61(275)\left[\dfrac{(0.305)(1.38)}{275}\right]^2}(1.38 - 1)(e^{1.38} + 1)$$

$$= 1143 - 47.6 = 1095 \text{ min} \quad \text{(equation [13.38])}$$

Average intake

$$F_{(0\text{-}L)} = \left[0.925\,(1095)^{0.720} + 7.0\right] \frac{0.40}{0.75} = 80 \text{ mm}$$

..... (equation [13.40])

Surface runoff

$$RO = 200 - 80 = 120 \text{ mm} \quad \text{..... (equation [13.42])}$$

Deep percolation

$$DP = (80 - 75) = 5 \text{ mm} \quad \text{..... (equation [13.43])}$$

Application efficiency

$$AE = \frac{(100)(75)}{200} = 37.5 \text{ percent} \quad \text{..... (equation [13.45])}$$

Gradient Furrows with Cut-back Inflow

The volume of surface runoff from irrigation with a constant inflow may be reduced, and application efficiency significantly improved, by reducing the inflow rate for a portion of the total application time. This is especially true for soils having intake rates less than that of the 1.0 Intake Family. Where provisions are made for re-use of surface runoff, use of the cut-back inflow method may not be desirable because of complexities in flow regulation and increases in labor requirements.

The degree of reduction of the inflow rate and the time at which the flow is reduced is an option of the designer. The following relationships are based on reducing the initial inflow rate to one-half at the time the initial flow has advanced to the end of the open-end furrow. Appropriate adjustments are required for a different operating procedure. Advance time (or cutback time) is computed from equation [13.35], using the initial inflow rate Q. The adjusted wetted perimeter, P_1, under cut-back flow is determined from equation [13.34] using Q/2 as the flow rate. The opportunity time for intake of the desired net application F_n at length L is calculated from equation [13.41] after substituting P_1 for P. The total inflow time, T_1, is the sum of T_n and T_T. The average opportunity time ($T_{0_{avg}}$) for intake during the advance period is equal to the absolute value of the second term in equation [13.38] with x = L. The average intake under cut-back conditions is the sum of intake during the advance period and intake during the remainder of the inflow time during which the inflow rate is reduced to one-half the initial.

$$F_{(0\text{-}L)} = \left[a\,(T_1 - T_{0\,avg})^b + c\right] \frac{P_1}{W} + \left[(a\,T_{0\,avg}{}^b + c\right] \frac{(P - P_1}{W}$$

.....[13.47]

The gross application for cut-back conditions is

$$F_g = \frac{60}{WL}\left(QT_T + \frac{Q}{2}\,T_n\right) \quad \text{..... [13.48]}$$

Calculations of surface runoff, deep percolation, and application efficiency utilize the same equations as for the noncut-back conditions. Design tables or charts for the cut-back inflow method may be prepared for each combination of intake family, net application depth, slope, roughness and furrow spacing. The curves or charts then give the inflow time, cutback time, runoff, deep percolation and efficiency for any combination furrow length and inflow rate. Fig. 13.5 is an example of a cutback furrow irrigation design chart.

Computation example—cutback gradient furrows
Given:
Same as gradient example.
Find:
Same as gradient furrow example plus time of cutback, T_T
Solution:

Cutback is the time of advance at the full flow, T_T, and is equal to that calculated in the previous example.

$$T_T = 144 \text{ min}$$

Adjusted wetted perimeter during advance is P as calculated in the previous example.

$$P = 0.40 \text{ mm}$$

Adjusted wetted perimeter during reduced flow is calculated with the flow equal to Q/2.

$$P_1 = 0.265 \left[\frac{(0.3)\,(0.04)}{\sqrt{0.004}}\right]^{0.425} + 0.227 = 0.36 \text{ m}$$

..... (equation [13.34])

Net application time is the time water must remain on the surface and is equal to T_n under reduced flow conditions.

$$T_n = \left[\left[\left\{\left[(75)\,(0.75/0.36)\right] - 7.0\right\}/0.925\right]\right]^{1/0.720} = 1165 \text{ min}$$

..... (equation [13.41])

Design inflow time (sum of T_T and T_n)

$$T_1 = 144 + 1165 = 1309 \text{ min}$$

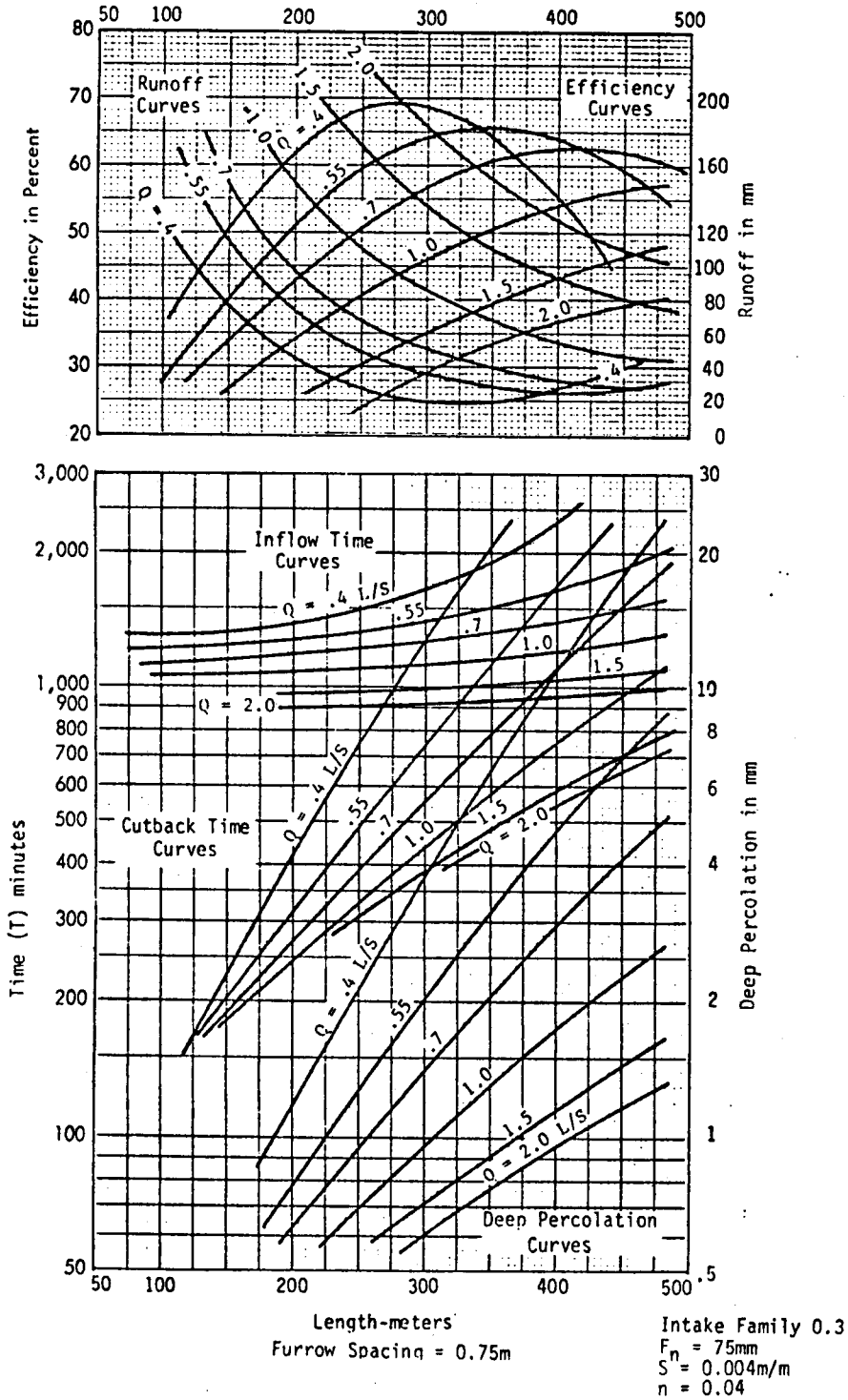


FIG. 13.5 Cutback furrow irrigation design chart (USDA, 1979).

Average opportunity time is the second term of equation [13.38] and was calculated in the previous example as part of $T_{(o-L)}$.

$$T_{o\,avg} = 47.6 \text{ min}$$

Average intake

$$F_{(0\text{-}L)} = (0.925\,(1309\text{-}47.6)^{0.720} + 7.0)\,\frac{0.36}{0.75} + (0.925\,(47.6)^{0.720} + 7)\,\frac{(0.40\text{-}0.36)}{0.75}$$

$$= 81 + 1.2 = 82 \text{ mm} \quad \text{(equation [13.47])}$$

Gross application

$$F_g = \frac{60}{(0.75)(275)}\,(0.6)\,(144) + (0.6/2)\,(1165) \quad \text{(equation [13.48])}$$

$$= 127 \text{ mm}$$

Surface runoff

$$RO = (127 - 82) = 45 \text{ mm} \quad \text{(equation [13.42])}$$

Deep percolation

$$DP = (82 - 75) = 7 \text{ mm} \quad \text{(equation [13.43])}$$

Application efficiency

$$AE = 100\,(75/127) = 59 \text{ percent} \quad \text{(equation [13.45])}$$

Level Impoundment Furrows

Surface runoff is elimated in level furrow systems with diked ends. Water is applied at one end of the furrow at a rate that will provide coverage of the entire length in a relatively short time. The water is then ponded until it infiltrates. The inflow rate should be large enough to advance to the end in not greater than 1.5 times the net opportunity time required for the design application. The rate, however, must not exceed the flow capacity of the furrow nor result in excessive erosion.

The design relationships for level furrows are based on the following conditions or assumptions:

1 The volume of water delivered into the furrow is equal to the average intake over the entire furrow length.

2 The intake opportunity time at the last point covered is equal to the time required for the net application to enter the soil.

3 The longest intake opportunity time at any point along the furrow is such that deep percolation is not excessive.

4 The ends of the furrows are blocked or diked to prevent outflow during the irrigation, and the depth of flow is no greater than can be contained within the furrow.

The inflow depths for level furrows may be approximated by the empirical equation:

$$\text{Inflow depth} = 0.0875\, Q^{0.342} \quad \text{[13.48]}$$

The average hydraulic gradient then becomes:

$$S = \frac{1}{L}(0.0875\, Q^{0.342}) \quad \text{[13.50]}$$

Wetted perimeter, P, is calculated from equation [13.34] and the net opportunity time, T_n, from equation [13.41]. The average opportunity time is the average advance time plus the net opportunity time, or:

$$T_{0\,avg} = T_n + \frac{0.0929}{fL\,\dfrac{0.305\beta}{L}}\,(\beta - 1)e^{\beta} + 1 \quad \text{[13.51]}$$

Inflow time, T_i, to meet the design assumptions, becomes:

$$T_i = \frac{PL}{60Q}\left[aT_{0\,avg}{}^{b} + c\right] \quad \text{[13.52]}$$

The gross application is given by equation [13.37]. Deep percolation, expressed as an average for the furrow length and spacing, is the difference between the gross and net application.

$$DP = F_g - F_n \quad \text{[13.53]}$$

Application efficiency is given by equation [13.5].

Charts may also be prepared to facilitate design of level furrows. Fig. 13.6 illustrates a level furrow irrigation design chart.

Computation Example—Level Impoundment Furrows
Given:

Furrow intake family, I_f	0.3
Length, L	275 m
Furrow spacing, W	0.75 m
Roughness coefficient, n	0.04
Inflow rate, Q	1.25 L/s

Find:

Inflow time required, T_i
Deep percolation, DP
Application efficiency, AE

Solution:
Average hydraulic gradient

$$S = \frac{0.0875}{275}(1.25)^{0.342} = 3.43 \times 10^{-4}\ \text{m/m} \quad \text{(equation [13.50])}$$

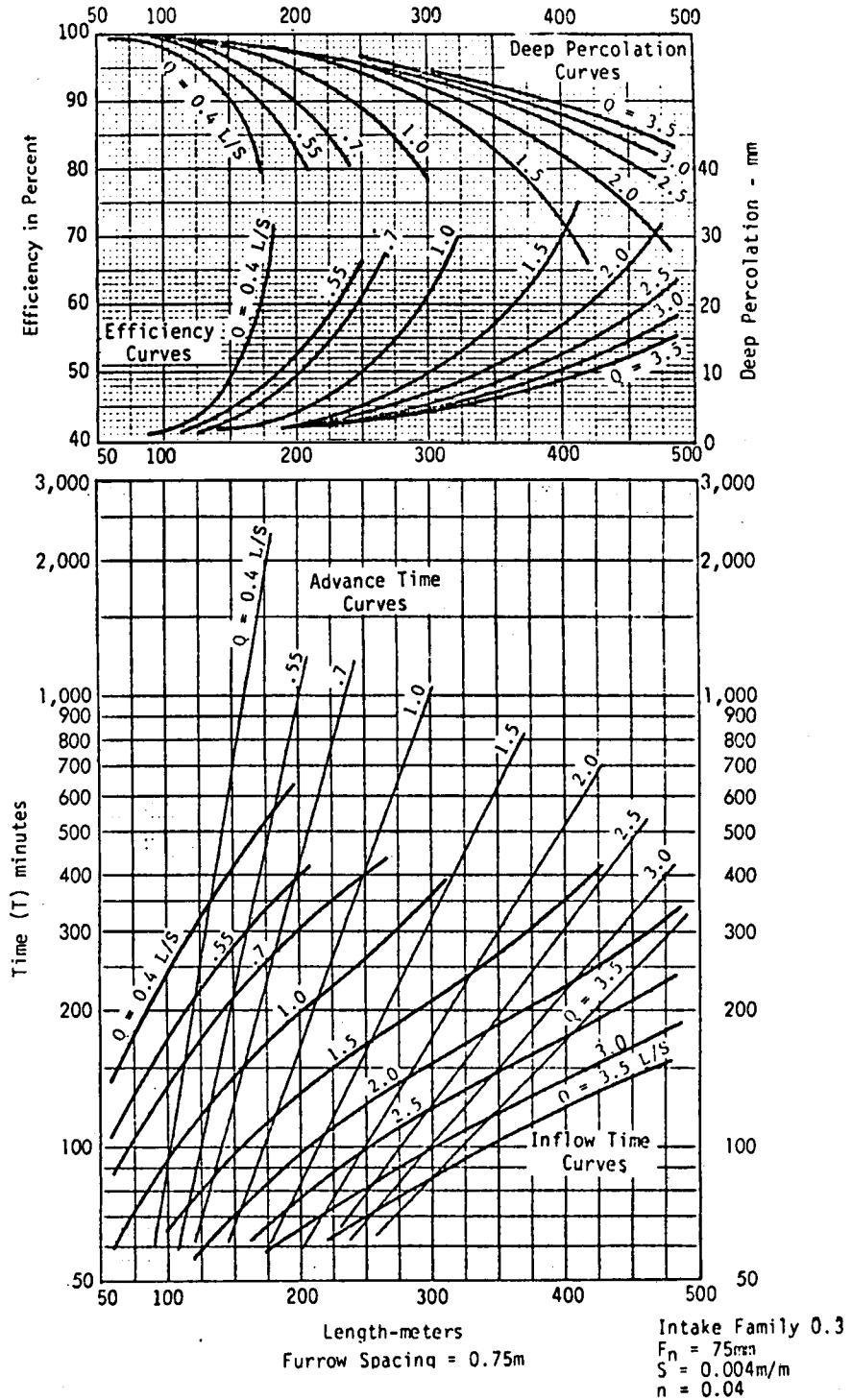


FIG. 13.6 Level furrow irrigation design chart (USDA, 1979).

Adjusted wetted perimeter

$$P = 0.265 \left[\frac{(1.25)(0.04)}{\sqrt{3.43 \times 10^{-4}}} \right]^{0.425} + 0.227 = 0.63 \text{ m}$$

..... (equation [13.34])

Net opportunity time

$$T_n = \left\{ \left[\frac{(75)\,(0.75)}{0.63} - 7.0 \right] / 0.925 \right\}^{1/0.720} = 510 \text{ min}$$

..... (equation [13.41])

Average opportunity time

$$\beta = \frac{(1.904 \times 10^{-4})(275)}{1.25\sqrt{3.43 \times 10^{-4}}} = 2.262$$

$$T_{0\ avg} = 510 + \frac{0.0929}{7.61\,(275) \left[\frac{0.305\,(2.262)}{275} \right]^2} \left[(2.262 - 1)e^{2.262} + 1 \right]$$

..... (equation [13.51])

Inflow time

$$T_1 = \frac{(0.63\,(275)}{(60)\,(1.25)} \left[(0.925)\,(604)^{0.720} + 7.0 \right] = 231 \text{ min}$$

..... (equation [13.52])

Gross application

$$F_g = \frac{60\,(1.25)\,(230)}{0.75\,(275)} = 84 \text{ mm}$$ ..... (equation [13.39])

Average deep percolation

$$DP = (84 - 75) = 9 \text{ mm}$$ ..... (equation [13.53])

Application efficiency

Compute advance time, T_T, and ratio (T_T/T_n) to determine if ratio is equal to or less than the 1.5 limit.

$$T_T = \frac{275}{7.61} e^{2.262} = 347 \text{ min}$$ ..... (equation [13.35])

$T_T/T_n = 347/510 = 0.68 < 1.5$, the ratio is acceptable.

13.6.6 Determination of Furrow Intake-Time Relationships

Intake in a furrow or corrugation, unlike other surface irrigation methods where the entire soil surface is in contact with water, occurs through only a portion of the soil surface. This portion is limited to the wetted perimeter which is independent of the furrow spacing. Field measurements are necessary to determine the intake-time relationship for use in furrow design. Field tests have shown that the relationship may be associated with a series of standard intake-time curves or "families," for most soils. These are shown in Fig. 13.1 and their coefficients are listed in Table 13.1. When field tests show dissimilarity with the standard design families, the on-site measured relationship of intake vs. time should be used. Field evaluations of furrow intake-time relationships require measurement of the hydrographs of inflow and outflow from a furrow(s) with a minimum length of 60 to 90 m for high and 150 to 180 m for low intake rate soils. The furrow cross-sections and grade between inflow and outflow measuring points should be reasonably uniform. Soil water conditions should be measured and tests should be run at a level where a normal irrigation application would be needed. Present and past cropping conditions and soil conditions as influenced by cultural operations should be recorded. Monitoring of an entire irrigation set is desirable; however, an alternative is to monitor the first one-fourth to one-half of the total irrigation. Compute the total volumes of inflow and outflow at a minimum of three intermediate times. The average cumulated intake, $F_{[0\text{-}L]}$ is determined by the equation:

$$F_{(0\text{-}L)} = \frac{1}{LP} (V_{in} - V_{out} - V_s)$$ ..... [13.54]

where L is the furrow length between inflow and outflow measurement points, (m), P is the adjusted wetted perimeter by equation [13.34], V_{in} and V_{out} are water volumes per unit area (L) and V_s is the water volume per unit area in channel storage (L).

The channel volume, V_s, is zero at the end of the irrigation. At intermediate times, V_s may be measured, or estimated by:

$$V_s = \frac{L}{0.305} \left[2.947\,(Q_{in}\,n/S^{1/2})^{0.753} - 0.0217 \right]$$ ..... [13.55]

where Q_{in} is the average inflow rate in L/s. Determine the average opportunity time, $T_{[0\text{-}L]}$ that water is available for infiltration. A simple average of inflow and outflow times may suffice where the advance is reasonably linear. Where the advance relationship is curvilinear, determine $T_{[0\text{-}L]}$ by averaging

advance time curve, dividing by the length and subtracting the resultant average time from the average recession time. Fig. 13.7 illustrates a plotting of 4 points representing average cumulated intake and associated average opportunity times, and comparison to a standard intake family. Fig. 13.8 illustrates cumulative inflow, outflow, and intake. Note that cumulative intake

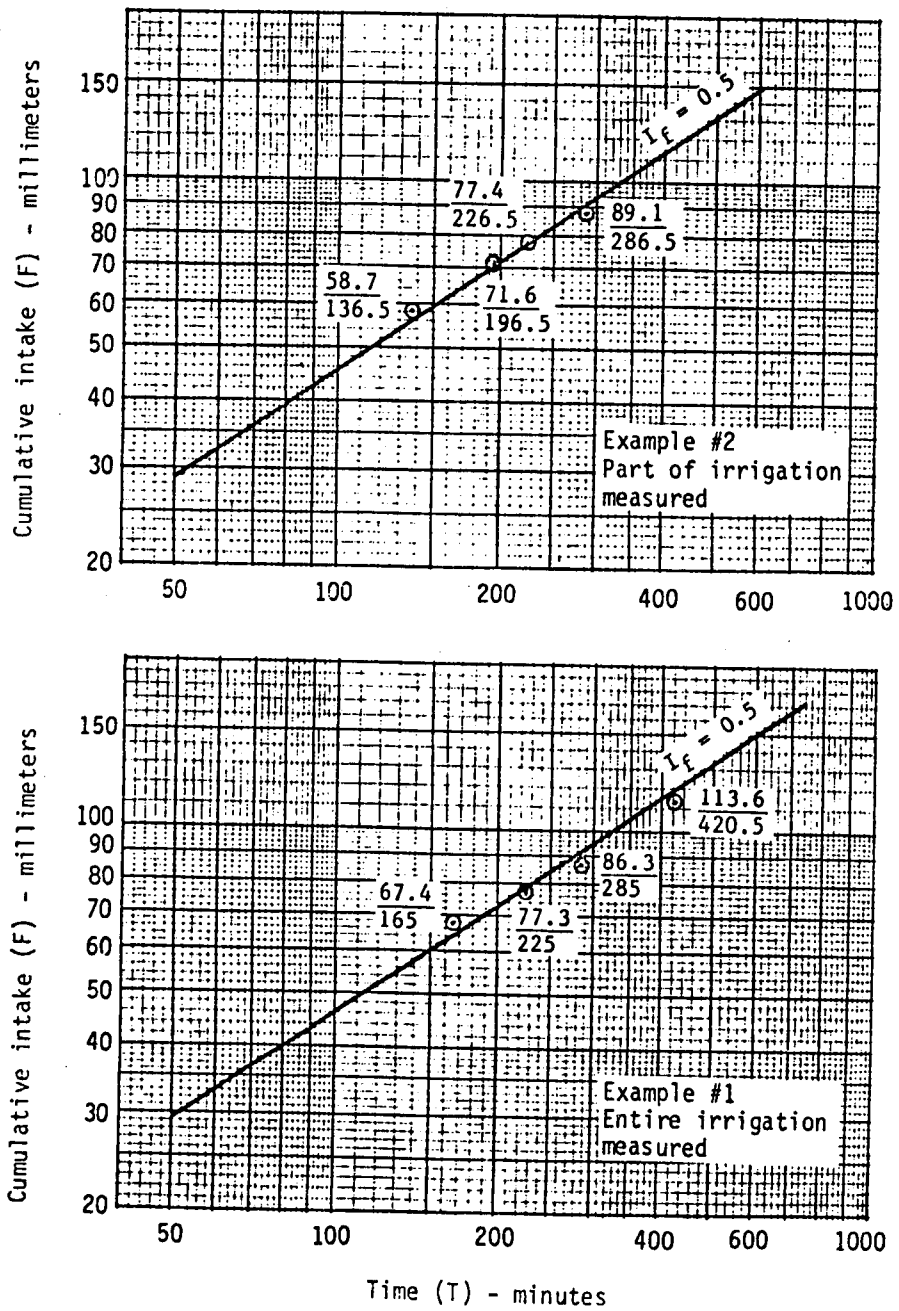


FIG. 13.7 Examples of cumulative intake vs. time evaluations (USDA, 1979).

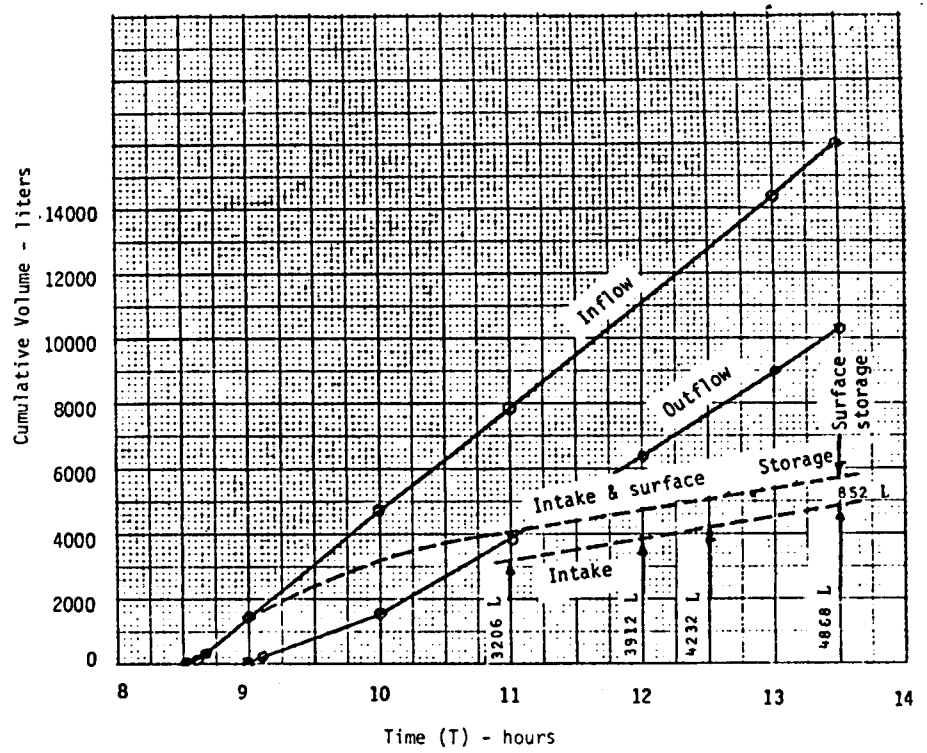


FIG. 13.8 Evaluation example no. 2.

values represent volumes of intake over a width of adjusted wetted perimeters and unit length. Equivalent depth values for the furrow spacing may be obtained by multipying by P and dividing by the furrow spacing W.

13.6.7 Selection of Headland Facilities

Water may be conveyed to fields irrigated by furrows or corrugations in lined or unlined ditches, or pipelines installed above or below the ground surface. Adequate structures must be provided in the delivery system to permit control and regulation of the water flow. Such structures include division checks, checkdrops, inverted siphons, flumes, valves, and gates. Measuring facilities to determine delivery flow rate are essential for proper irrigation management.

Supply ditches. Supply ditches must convey the design inflow rate for the number of furrows irrigated simultaneously. The water surface in the ditch should be 0.15 to 0.30 m above the field surface level. Where possible, the ditches should be constructed with a 0.1 percent grade or less to minimize the number of checks required.

Supply pipelines. Supply pipelines have the same capacity requirements as supply ditches. Pipelines, usually placed underground and either closed or vented to the atmosphere, must be designed with the hydraulic grade line above all points of delivery or have pumping facilities incorporated in the outlets. Non-vented pipelines must have adequate pressure and vacuum release appurtenances, and adequate drainage facilities. The pipe size should be large enough to limit the maximum flow velocity to 1.5 m/s. Pipeline systems may consist of a combination of both underground and surface pipe (Also see Chapter 11).

Supply outlets. Various outlet devices are used to release water into each furrow or corrugation. Outlets of equal size with uniform pressure head are desirable to deliver nearly equal flows to all furrows irrigated at one set. Rates of flow are changed by altering the size of outlets, varying the number of outlets, or changing the operating head. Common types of outlets from ditches are siphon tubes, orifices or spiles, and wier notches. Adjustable gated orifices minimize the effect of pressure head differentials on discharge rate. Ungated orifices and weirs with inverts above the normal water surface in the ditch are used in lined ditches of sufficient depth to permit raising the water surface over the opening by adjustment of regulating control structures.

Outlets from pipelines include hydrants, valves, and vertical stands. Hydrants and stands are used to control discharge into gated pipe or surface ditches which serve a number of furrows. Small valves are used to simultaneously supply flow to several furrows or corrugations.

Gated pipe. Gated pipe, usually portable and rigid or flexible, has uniformly spaced round or rectangular adjustable orifices to discharge flow into individual furrows. Short flexible sleeves may be attached to dissipate energy and minimize erosion at the furrow inlets. Gated pipe, normally placed at the head of a field, may also be located at intermediate locations within a field to reduce furrow length, and to supplement the outflow from upper reaches to achieve the desired inflow rate to the next furrow section. The pipe may, unlike concrete ditches, be temporarily removed to eliminate restrictions on equipment travel.

13.7 WATER SPREADING

13.7.1 General Description

Water spreading, according to the Soil Conservation Service (USDA, undated),

> "is a specialized form of surface irrigation accomplished by diverting flood runoff from natural channels or water-courses and spreading the flow over relatively level areas. The diversion and spreading is controlled by a system of dams, structures, dikes, or ditches, or a combination of these, designed to accomodate a calculated rate and volume of flow"

A major difference between the water spreading system and others discussed in this chapter is that the water spreading system is designed to meet precipitation and runoff conditions of an area and apply runoff to cropped fields while the other systems are designed to deliver water in accordance with plant needs. The systems are commonly designed for 6-hour duration storms of 1.25-year, 2-year or 5-year frequency. They may also be considered as means for controlling runoff to reduce erosion and other damage to the environment (e.g., excessive sediment deposition on range land).

13.7.2 Applicability

Crops. The major crops grown under this system are those found on ranges or pastures. The purpose of the system is to increase the production of forage, hay or seed. Occasionally alfalfa, other legumes, or tame or native grasses are grown under the system. Selection of the crop to be grown will be affected by the dependability of the design precipitation and whether or not ponding must occur.

Soils. Deep, medium to moderately fine textured soils with moderately permeable subsoils and substrata are ideal. Moderately fine to fine textured soils are next best and may be used if land slopes are low enough to allow ponding. Coarse and moderately coarse soils are not suitable because infiltration rates are high and their waterholding capacities are inadequate to maintain crop growth over the infrequent intervals common to this method.

Topography. Land should be smooth and gently sloping. If detention methods are to be used, slopes are limited to 1 or 2 percent so that dikes will not be too widely spaced or too high. If continuous flow systems are used, maximum slopes are limited to 5 percent for uniform topography and 3 percent for undulating topography. Minimum slopes are limited by drainage requirements.

Climate. It is essential that expected runoff events occur at times when the soil can store added water. The water contents of frozen soils or those at field capacity are not increased by a runoff event.

Other considerations. Water must not contain excessive bedload which would deposit in the spreading area. Inaccessible spreading areas must have automatic or semi-automatic systems.

13.7.3 Advantages

Water spreading is an inexpensive means of applying water to an area to supplement rainfall. When properly designed, water spreading systems can result in large returns for relatively small investments.

13.7.4 Disadvantages

Water is applied when runoff occurs. This may or may not coincide with plant water needs, soil water storage availability, or harvest periods. These systems, when developed for inappropriate situations, or when inadequately designed, may damage farm land and property. Concentrated flows may cause serious soil erosion and sediment deposition.

13.7.5 Types of Systems

Designs of these systems can be conveniently divided into flow-type systems and detention-type systems. The flow systems incorporate free drainage from the irrigated area, while detention systems retain the applied water on the irrigated area until it has infiltrated. These two types of systems are further divided into subtypes (Table 13.13).

Spreader-ditch flow systems. These systems distribute a concentrated, short-time flow of water. The ditches are constructed to carry less water when further from the water source, and so the ditch slopes are commonly reduced

TABLE 13.13. TYPES OF WATER SPREADING SYSTEMS

Type	Upper limit of land slope, percent	Maximum flow rate, m^3/s
Flow		
Spreader ditch	5	
Syrup-pan	1.2	0.3
Dike and bleeder	1.2	0.3
Detention		
Manual inlet control	2	
Automatic inlet control	2	

(in equal increments) from about 0.3 or 0.4 percent on the upper end, to 0 percent on the lower end. Water from the upper spreader ditch is collected in the lower spreader ditches and redirected laterally. Thus, although water flows in only one direction in the spreader ditch, it may flow in two directions in the pickup ditches (Fig. 13.9), depending upon land conditions.

Syrup-pan flow systems. In this system there is a single spreader ditch at the upper end of the field, and no pickup ditches. Water spills over the sides of the spreader ditch into the field below. As the water flows down the slope it is intercepted by a contour dike which diverts flow to one end of the field. The dike is broken at that end, and water flows into the next section of the field. It again flows over the field, is picked up by another contour dike and diverted to the opposite end of the field. The system is repeated until the lowest part of the field is reached. Possible ditch and dike layouts and dike construction are given in Fig. 13.10. The maximum distance between dikes is

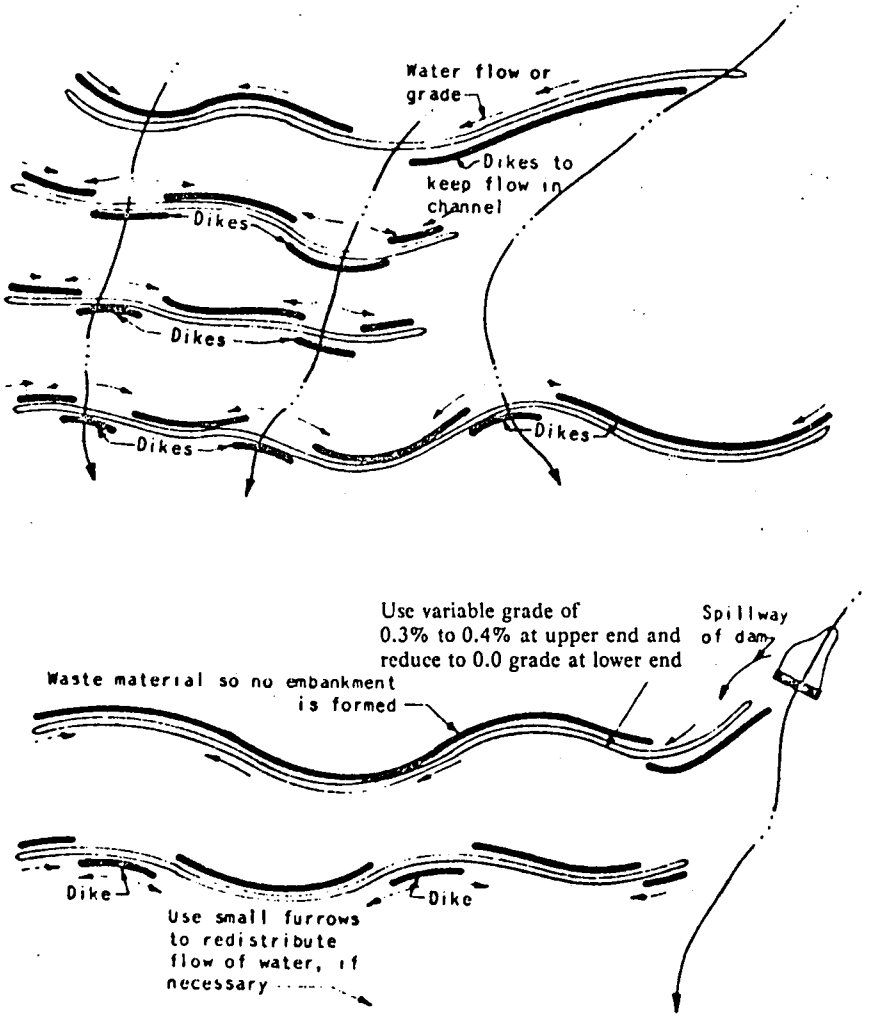


FIG. 13.9 Spreader-ditch flow type systems (after USDA, undated).

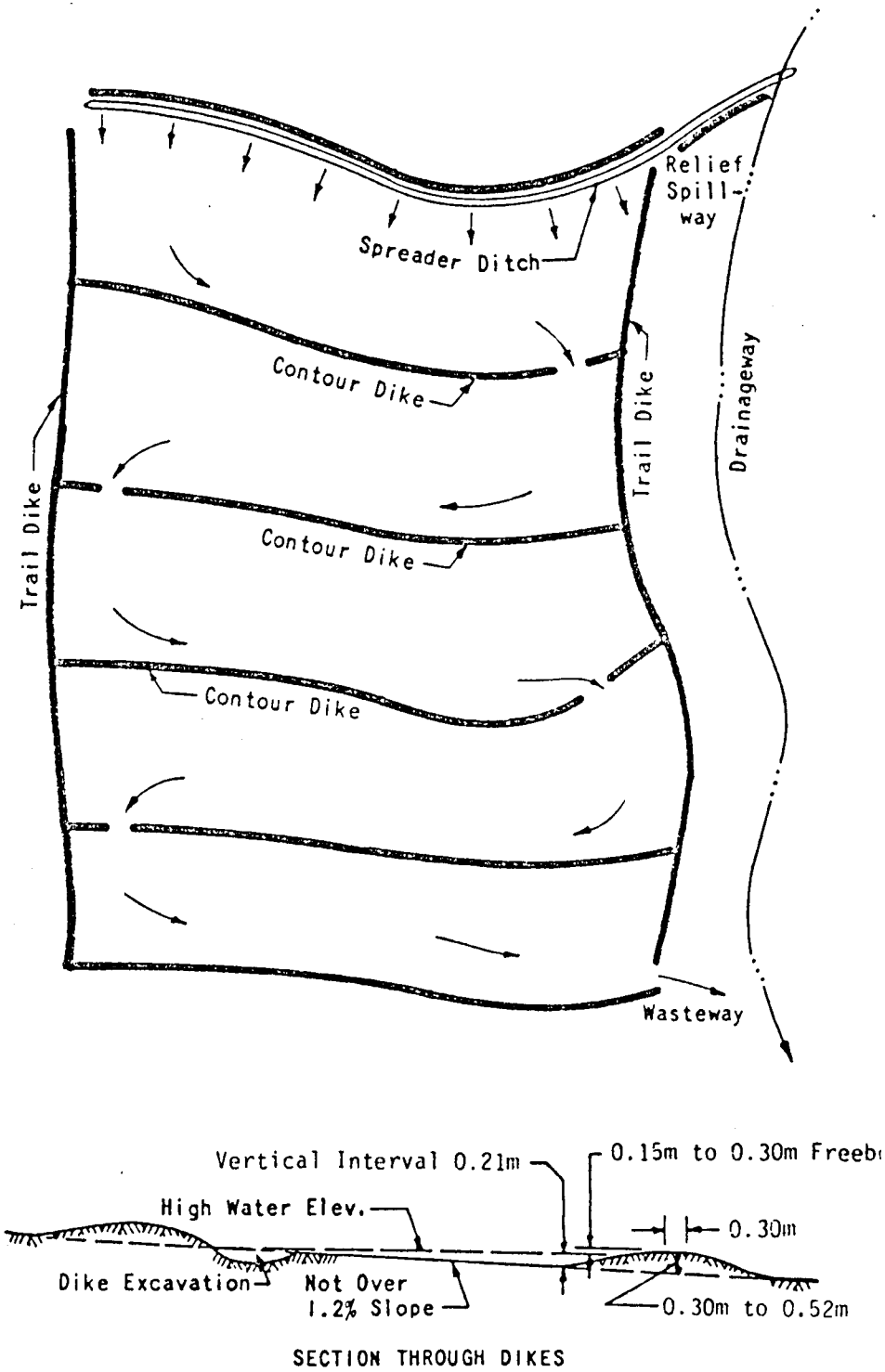
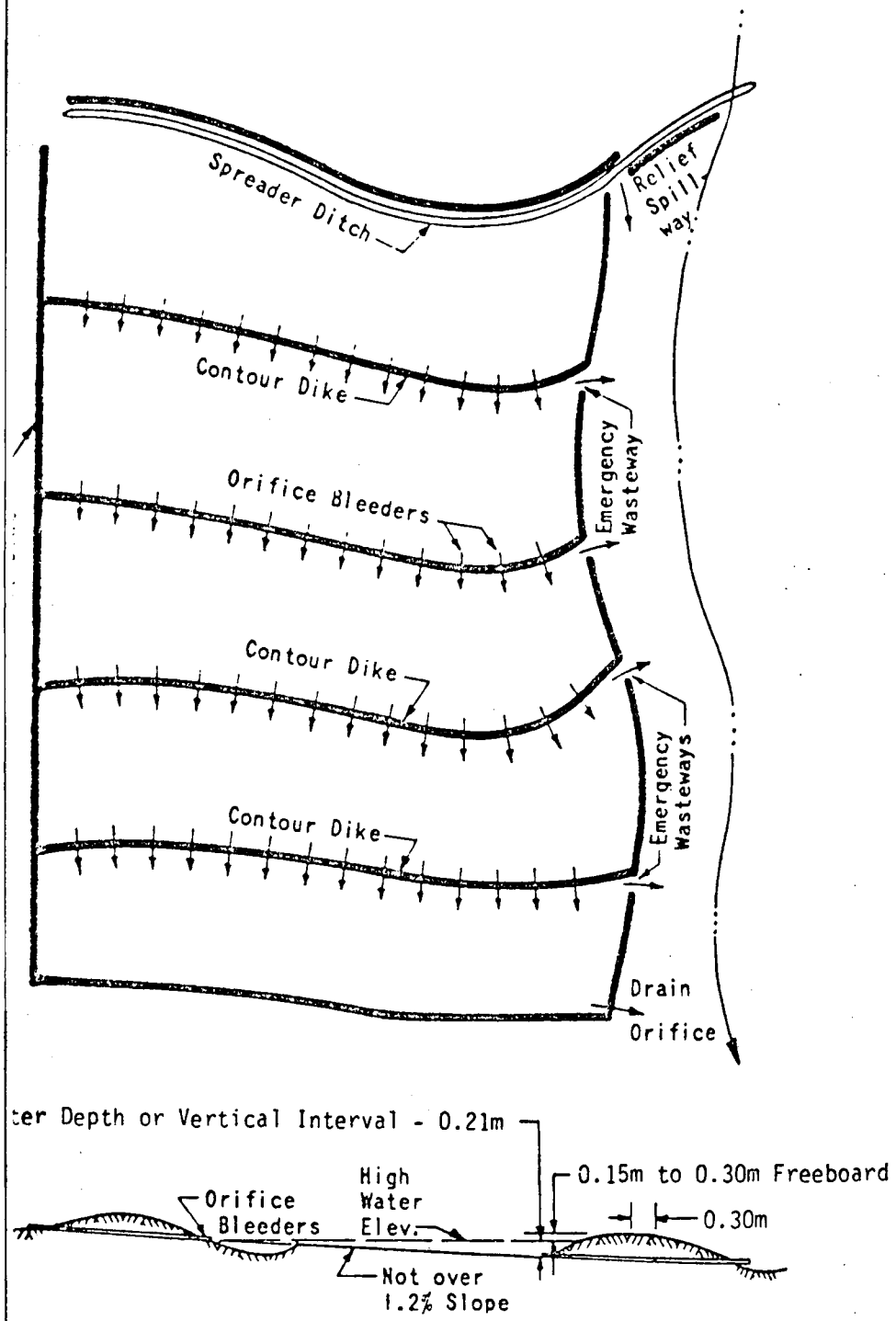


FIG. 13.10 Syrup-pan flow type systems.



SECTION THRU DIKE AND BLEEDER

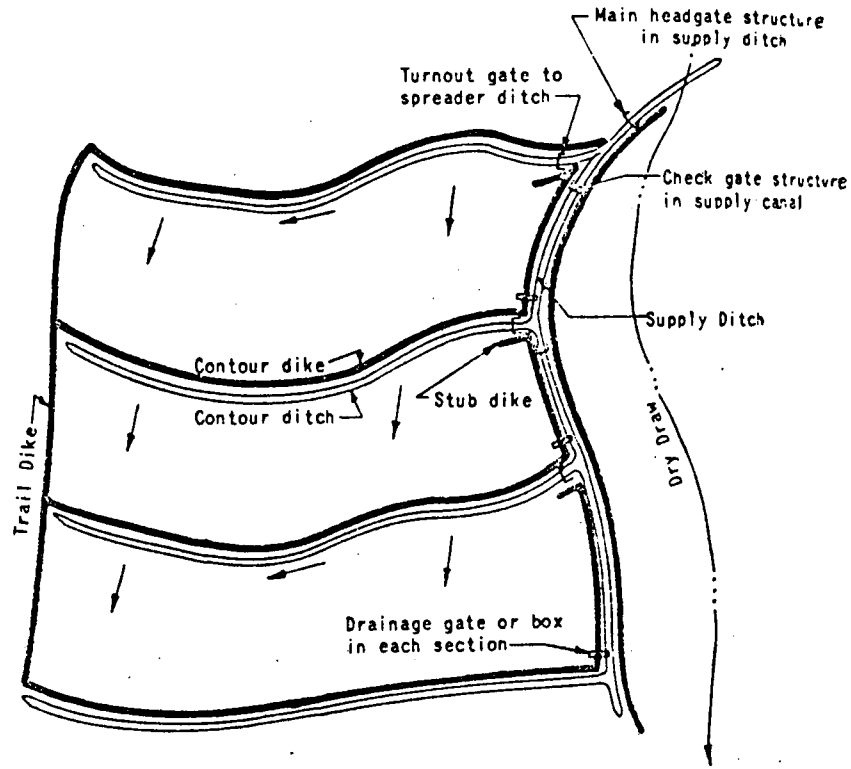
the lesser of 90 m, or that length which limits the water level at the upstream side of a dike to 0.2 m. This latter criterion, and the maximum allowable ground slope of 1.2 percent, would limit dike spacings to about 18 m.

Dikes and bleeder flow systems. This is a modification of the syrup-pan system. Water bleeds through dikes to lower portions of the field via tubes (concrete, clay, metal) or weirs placed at intervals along the dikes (Fig. 13.11). An emergency waterway must be provided to allow for stoppage of tubes.

The above three flow systems are designed for a continuous flow of water. Thus, the area covered must be adequate to infiltrate all this water, or some of it will be lost.

Manual inlet control detention systems. These systems are suitable when long duration flows such as from snow melt, are to be used. These detention systems divert flow into each dike individually (Fig. 13.12) until the desired depth has been applied. Under the maximum allowable ground slope of 2 percent, 0.6-m high dikes, spaced at 30-m intervals allow a freeboard of 0.15 m and an application depth of 0.45 m at the lower dike, and a zero application at the upper dike. Checks and turnouts are needed to divert the water from the supply ditch to each dike. A drain is also needed for each dike.

Automatic inlet control detention systems. These systems are basically similar to the manual ones, except that the control structures are designed to allow only the desired depth of water into the diked areas (Fig. 13.13). A vegetated waterway is used for the supply ditch, and grooves or small chan-



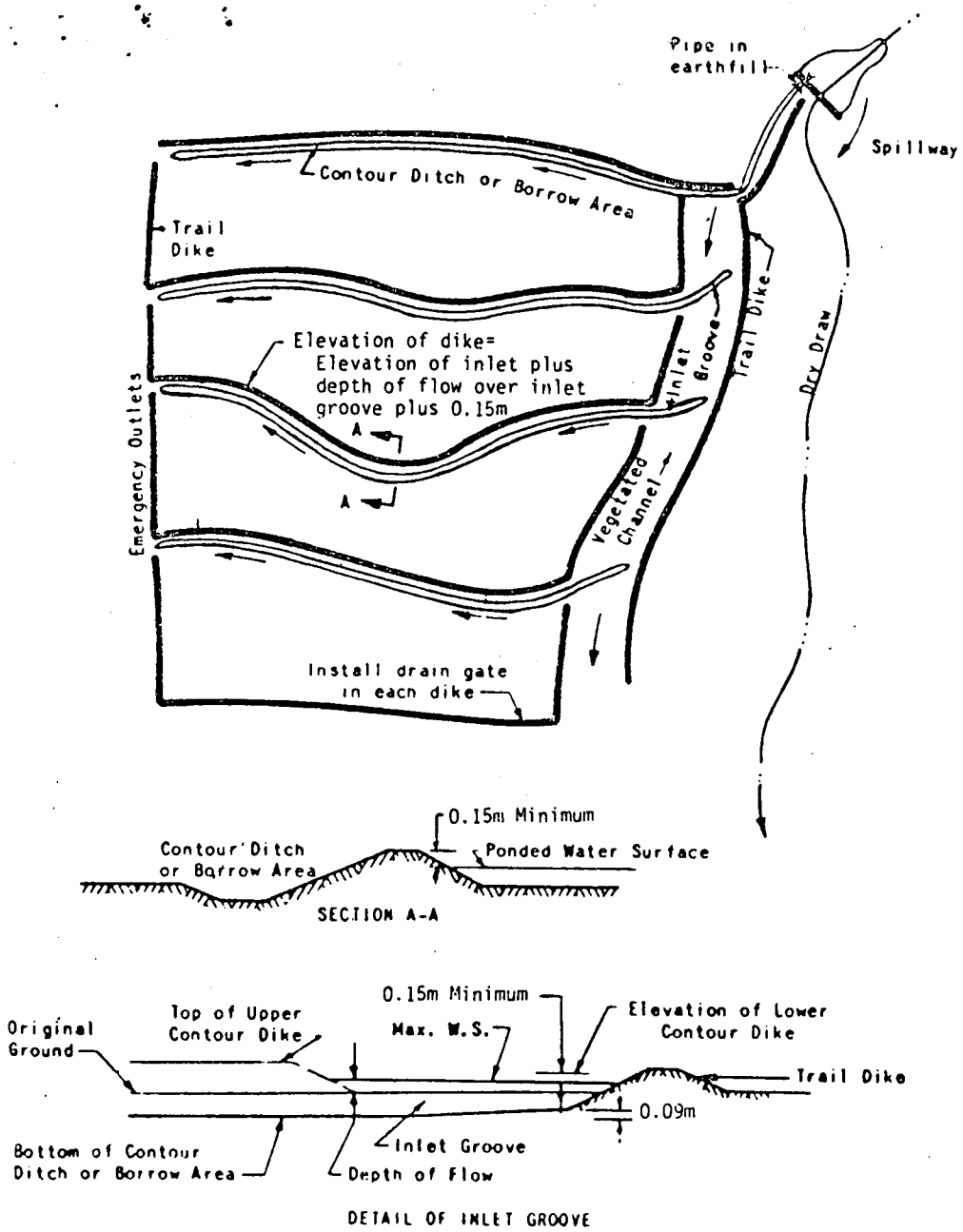


FIG. 13.13 Automatic inlet control detention system.

nels intercept some of the water from the waterway and divert it into the diked area. By appropriate location of the inlet groove, the depth of water in the channel is limited.

13.7.6 Design

Because the purpose of water spreading systems is to divert storm water onto agricultural land, it is necessary to know the runoff characteristics of the area receiving the storm. Standard hydrologic methods can be used to predict volume of storm runoff, V_s, the peak discharge, Q, and the time of concentration of flow, T_c. Consideration must be given to variations in computed runoff rate and volume if there is significant detention storage in canals and reservoirs. All of these calculation are made considering the storm frequencies of Section 13.7.1. The 1.25-year design has an 80 percent chance of occurrence and is called "dependable." The 2-year frequency has a 50 percent chance of occurence and is designated "questionable." The 5-year frequency storm has a 20 percent chance of occurrence and is called "undependable."

Design application depth. In flow systems, the application depth is determined by the duration of flow and the intake characteristics of the soil. The duration of flow is related to the time of concentration, and so a relationship between these three variables can be stated, at least empirically. Table 13.14 can be used to estimate application depths from time of concentration T_c, and infiltration characteristics of the soil, as exemplified by soil type. The table gives the estimated time during which flow would occur over the land, and the depth of water which would be infiltrated under different soil conditions.

In detention systems, the application depth is equal to that which can be held within the plant's root zone (often taken as 1 m). As with flow systems, the total volume of water available determines the area which can be irrigated by a given storm.

Capacity of water supply system. It is not common to use the entire flow from a storm, and the water supply system is designed to convey only that fraction which will be diverted. The required volume to divert, V_d, is

$$V_d = d_a a, \quad \text{[13.55]}$$

TABLE 13.14. DESIGN APPLICATION DEPTH FOR FLOW-TYPE WATER SPREADING SYSTEMS (AFTER USDA, UNDATED)

Drainage characteristics		Soil			
Time of concentration (T_c) hours	Estimated flow duration, hours	M medium texture*	F Mod. fine texture	H fine texture†	V very fine texture†
		Depth of application, mm			
0.5	8.5	215	120‡	75	40
1.0	9.0	225	130‡	75	40
1.5	9.5	235	130‡	80	40
2.0	10.0	245	140‡	80	45
2.5	10.5	250	140‡	80	45
3.0	11.0	260	146‡	85	45
3.5	11.5	270	150‡	85	45
4.0	12.0	280	150‡	90	50
4.5	12.5	285	160*	90	50
5.0	13.0	290	160*	95	50
5.5	13.5	300	165*	95	50
6.0	14.0	310	170*	99	50

*A free flow system is adequate for these soils.
†Detention type systems should be given first choice.
‡These soils are adapted to either flow or detention type systems.

TABLE 13.15. WATER SPREADING VOLUME RATIOS (AFTER USDA, UNDATED)

Ratio $\frac{V_d}{V_s}$	r_q
0.000	0.000
0.100	0.055
0.200	0.110
0.300	0.165
0.400	0.230
0.500	0.295
0.600	0.370
0.650	0.410
0.700	0.455
0.750	0.500
0.800	0.555
0.850	0.610
0.900	0.685
0.950	0.780
1.000	1.000

TABLE 13.16. WATER SPREADING FLOW RATIOS (AFTER USDA, UNDATED)

Ratio $\frac{Q_d}{Q}$	r_v
0.000	0.000
0.100	0.180
0.200	0.350
0.300	0.505
0.400	0.635
0.500	0.750
0.600	0.840
0.650	0.875
0.700	0.910
0.750	0.935
0.800	0.955
0.850	0.975
0.900	0.985
0.950	0.995
1.000	1.000

where d_a is the design application depth and a is the design spreading area. If the volume of the storm, V_s, is known then the diversion flow can be computed from two other equations. First determine the diversion flow, Q_d, from

$$Q_d = r_q Q, \qquad [13.56]$$

where Q is the peak flow, and r_q is an empirical coefficient based upon the ratio V_d/V_s (Table 13.15). On the other hand, if the diversion flow is limited by site considerations, a diversion volume can be found.

$$V_d = r_v V_s, \qquad [13.57]$$

where r_v is an empirical coefficient (Table 13.16) based upon the ratio Q_d/Q.

Water disposal. Excess water is returned to the water supply system, and provisions must be made for this. Erosion, other possible damage, and state and local laws must all be considered.

13.7.7 Sample Calculations

Nomenclature:

a	Spreading area, ha
P_n	Design precipitation for n-year storm, mm
R	Runoff from design rainstorm, mm
A	Drainage area, km^2
T_c	Time of concentration, h
d_a	Design application depth, mm
q	Unit peak discharge, m^3/s per 10^3 m^3 of runoff
Q	Peak discharge, m^3/s
Q_d	Diverted flow, m^3/s
R	Design storm runoff, mm
V_s	Storm runoff volume, m^3
V_d	Diverted volume, m^3

Given:

Drainage area	A = 15.3 km^2
Time of concentration	T_c = 1.77 h
Maximum available size of water spreading area	a = 56.7 ha
Design storm runoff	
5-yr (20 percent chance)	R = 9.65 mm
2-yr (50 percent chance)	R = 4.32 mm
1.25-yr (80 percent chance)	R = 1.78 mm
Unit peak discharge	q = 0.0978 m^3/s per 10^3 m^3
Type of system	Flow
Soil texture on spreading area	F (moderately fine)

Find:

Area, a, for undependable, questionable and dependable supplies (a) using entire flood flow, and (b) for the flood flow of the dependable system.

Solution:

Case (a). Compute, for each frequency, the storm volume, V_s (= A R), in 10^3 m^3; the peak discharge Q (= q A R) in m^3/s; the design application depth, d_a (Table 13.14, by interpolation); and the maximum spreading area a (= V_s/d_a). These results are tabulated below for the entire flood flow.

Class of system	R, mm	V_s, $10^3 m^3$	Q, m^3/s	d_a, mm	a, ha
Undependable	9.65	150	14.66	132	114
Questionable	4.32	67	6.56	132	51
Dependable	1.78	28	2.70	132	21

The entire flood flow is a dependable supply for 21 ha, a questionable supply for 51 ha and an undependable supply for 114 ha.

Case (b). The peak flow of the dependable system is 2.70 m^3/s. This can be applied to the three cases by computing the ratio Q_d/Q; finding r_v from Table 13.16; determining the volume diverted, V_d (= $r_v V_s$); and computing the area a (= V_d/d_a). The results are tabulated below.

Class of system	Q, m^3/s	Q_d, m^3/s	Q_d/Q	r_v	V_s, 10^3 m	V_d, 10^3 m	d_a, mm	a, ha
Undependable	14.66	2.70	0.18	0.32	150	48	132	36
Questionable	6.50	2.70	0.41	0.65	67	44	132	33
Dependable	2.70	2.70	1.00	1.00	28	28	132	21

13.8 REUSE SYSTEMS

13.8.1 Description

Reuse systems collect irrigation runoff water from a field and make it available for reuse. They consist of collection ditches or diked areas at the lower end of a field, an open channel or pipe drain which directs the collected water to a storage area, and a means for returning the collected water to the same field or delivering it to a different field. The return/delivery system may include a pump and a pipeline, or open channels.

13.8.2 Applicability

Reuse systems are particularly applicable where legal constraints require that water which is delivered to a farm must be used on that farm. This

is often the case in groundwater control districts. It is also desirable when surface drainage might otherwise cause inundation and consequent damage to neighboring lands, and where water is pumped from aquifers since re-pumping tail water is less expensive.

Reuse systems are most commonly used with furrow irrigation systems. A reuse system allows large stream sizes to be used throughout the irrigation without excessive loss of water. Providing the economics are favorable, any surface irrigation system might benefit from a tailwater reuse system.

13.8.3 Definitions

Application duration is the elapsed time from the beginning of an irrigation set to the time at which the inlet flow is terminated for that set.

Cutback irrigation results if the initial inflow stream is reduced during an irrigation set.

Cycling-sump systems immediately recycle runoff water, as soon as the small (minimum size) sump accumulates enough water to provide proper pump operation. Pump operations may be as frequent as 15 cycles per hour.

Pumpback systems are reuse systems which deliver runoff by pumping, usually to a point at equal or greater elevation than the collection point.

Sequence systems deliver runoff by gravity to a point at lower elevation than the collection point.

Storage facilities accumulate runoff for later or immediate reuse.

Time of cutback is the elapsed time from the beginning of an irrigation set to the time at which cutback is affected.

13.8.4 Basic Principles of Design

Bondurant (1969) analyzed reuse systems and arrived at general design requirements, many of which are included in the following paragraphs.

Runoff water should be applied to a set different from that on which it occurs (unless cutback is practiced). Recirculating runoff to the same irrigation set that is generating runoff, without substantial reduction of the primary inflow, results only in temporarily storing water on the field. This will not significantly increase the infiltration rate, but will increase the rate of runoff and will probably increase erosion in a furrow. The practice also involves appreciable labor to start new streams from a constantly increasing runoff flow (unless placed in temporary storage), and usually results in different durations of application in various parts of the field.

When computed over the time interval required to irrigate the area contributing to the cycling-sump system, runoff water will have to be returned to the system at the same average rate that it is accumulated if all runoff is to be reused. If temporary storage is provided, stored runoff will eventually have to be recirculated at a rate equal to the average storage accumulation rate to prevent loss by overflow.

Maximum improvement in total water use on the farm will result if stored runoff water is used to achieve a reduced stream size for cutback irrigation; i.e., stored runoff water is pumped from a reservoir to increase the stream size (on another set) during the advance period and pumping is stopped after the set has started to produce runoff. This reduces deep percolation and runoff so that a minimum amount of water must be recirculated.

Reservior storage can affect maximum savings of labor and water if the runoff water is placed in a reservoir adequate to retain it all for later regulated use. The use of such a reservoir to also contain the initial supply to save labor is usually economical. With such a reservoir, cutback streams are usually not needed, thereby saving labor. Moderate size initial streams can provide reasonably uniform distribution along furrows and more runoff is produced than if a cutback system is used. However, this runoff can be retained on the farm by returning it to the reservoir. Erosion is increased, however, with some soils, and the extra sediment collected in the reservoir must be removed, which increases cost of operation. With erosive soils, stream size must be kept as small as practical to keep erosion to a minimum.

Runoff rate and total volume are necessary inputs to the design of any reuse system. These variables may be estimated with the methods of Section 13.6, and general guidelines for runoff prediction are given by ASAE (1980).

13.8.5 System Design

All systems should be designed in conformance with local regulations (reservoir construction, safety precautions, etc.). Nevertheless, there are certain features of design which are dependent directly on the operation of the reuse system.

There are five main components of a reuse system.

1 A system of drains to intercept and carry the runoff to a desilting basin.

2 A desilting basin to settle out excess suspended matter carried in runoff waters. This may not be needed in all cases, and when it is not needed, the collecting system carries water directly to the storage area.

3 The storage area, which is a sump, dugout, reservoir, pond, etc.

4 A pump with its inlet facilities, power unit and controls (automatic or manual). The inlet facilities should include provision for removing floating debris and weed seeds. Although pumps are not needed in all reuse systems, the functions of the inlet facilities must be met in any case.

5 A conveyance system of pipes or open channels which delivers the water back to the main irrigation system, either at the same field from which runoff originated, or another field.

The above facilities will be discussed in the following paragraphs. The single most important item is the selection of reservoir size, and that is dependent upon the type of reuse system under consideration. Closely related to reservoir size is pump size, and that item is included with the discussion of reservoir sizes.

Cycling-sump systems. Cycling-sump systems (Davis, 1964) must not cycle more than 15 times per hour to maintain reasonable pumping plant efficiencies and to conform to pump manufacturers' recommendations.

When designing for maximum cycle rate, 15 times per h, and hence minimum reservoir and pump size,

$$S = 60P = 60I_d \qquad [13.58]$$

where S is the storage capacity of the sump in liters, P is the pumping rate in L/s and I_d is the design inflow rate to the sump in L/s. If all runoff water is to be reused (no waste)

$$I_d = I_{max} \qquad [13.59]$$

where I_{max} is the maximum runoff rate.

TABLE 13.17 MINIMUM SUMP SIZE FOR TAILWATER RECOVERY SYSTEMS (AFTER DAVIS, 1964)

Tailwater inflow, L/s	Inside diameter of circular sump: Depth in storage in sump between on and off levels, m			
	cm	cm	cm	cm
3.1	64	46	38	33
6.3	89	64	53	46
9.5	109	79	64	56
12.6	127	89	74	64
15.8	142	102	81	71
18.9	155	109	89	79
22.0	168	119	97	84
25.2	178	127	104	89
28.4	191	135	109	97
31.5	201	142	117	102
34.7	211	147	122	107
37.9	218	155	127	109

Sump sizes should conform to those given in Table 13.17 and the following additional restrictions.

1 The inside diameter of the sump should be at least five times the inside diameter of the pump column.

2 The clearance between the sump floor and the strainer must be at least one-half the inside diameter of the pump column.

3 The velocity of inflow to the sump should not exceed 0.3 m/s.

4 The lowest water level should provide submergence over the pump strainer of at least nine times the pump diameter.

The pump should be set off-center in the sump to reduce vortex formation.

Fluctuations in pressure and flow make water deliveries from a cycling-sump system difficult to handle. Thus, such systems must usually deliver the pump flow to regulating reservoirs or major supply reservoirs.

Reservoir systems for continuous pumping. There are two principle procedures used to design the reservoir systems for continuous pumping.

1 Design considering runoff rule only (Davis, 1964)—The size of a reservoir from which accumulated runoff will be pumped continuously is

$$S = V_d\left(1 - \frac{\bar{I}_d}{P}\right) \quad \text{[13.60]}$$

where S is the reservoir capacity (not including dead storage), V_d is the design volume of runoff and I_d is the average runoff inflow rate corresponding to the design volume of inflow. If all the runoff is to be utilized

$$V_d = V \quad \text{[13.61]}$$

where V is the total runoff. Otherwise V_d is limited by some other criteria (e.g., 90 percent of runoff will be recycled, the remainder will be wasted). The average inflow is based on the design volume of runoff and the time over which runoff occurs.

$$\bar{I}_d = \frac{V_d}{60\,T_t} \quad \text{[13.62]}$$

where T_t is the duration of runoff in minutes. The time of pumping is

$$T_p = \frac{\bar{I}_d T_t}{P} \quad \text{[13.63]}$$

where T_p is the time (min) at which the pump should be started, measured from the time at which runoff starts.

2 Design considering reapplication of runoff to the same field (Stringham and Hamad, 1975a, 1975b)—In this case, the first set is irrigated by the supply stream only, and the last set (n + 1) is irrigated by the water stored from runoff. Intermediate sets are irrigated from both sources. The number of furrows irrigated in sets 2 through n is constant, while the number irrigated in set 1 and set (n + 1) is less than in the others.

If the total application time is T, and the duration of each set is t (constant, regardless of source of water) the total number of sets irrigated by the supply is n.

$$n = \frac{T}{t} \quad \text{[13.64]}$$

The time T is determined from previous knowledge of the gross water to be applied and the inflow rate of the supply, Q_s. The duration of each set is determined by the designer. If the inflow to each furrow is q, the total number of furrows irrigated by the supply stream, f_s, is

$$f_s = \frac{Q_s}{q} \quad \text{[13.65]}$$

and that by the pumped back runoff water, f_p is

$$f_p = \frac{Q_p}{q} \quad \text{[13.66]}$$

where Q_p is the pump back flow rate. Clearly,

$$F = n(f_s + f_p) \quad \text{[13.67]}$$

where F is the total number of furrows to be irrigated. This is determined from the field size and furrow spacing. It is obvious that there must be a balance between the chosen pump back rate, the application time, and the supply rate.

The volume of water in storage in the reservoir at the end of any given set, i, is V_i (m^3).

$$V_i = 3.6\left\{\left[iQ_s + (i-1)Q_p\right]R_f - (i-1)Q_p\right\}t, \quad \text{[13.68]}$$

where Q_s is the supply flow rate (L/s), Q_p is the pump back rate (L/s), R_f is the total runoff volume expressed as a fraction of the applied volume, and t is the application time for each set (h). The runoff fraction can be determined

by one of the methods outlined in Section 13.4. The maximum value of V_i is the necessary reservoir size.

An alternative design is to have a variable number of furrows in each of sets 2 through n to effect a minimum resercoir size (and maximum pump back rate). This design, along with tables and graphs to assist in obtaining a solution, is given by Stringham and Hamad (1975).

Other reservoir systems (Davis, 1964). Design of other reservoir systems involve storage of either the entire runoff flow or part of the runoff flow.

1 Storage of the entire runoff design flow—The most flexible systems are those which store the entire design runoff flow. These allow use of pumps of any convenient size (to deliver cutback or main streams), but they require large reservoirs.

$$S = \bar{I}_d T_t \qquad [13.69]$$

No cycling is required during the emptying of the reservoir, and pumping can commence at the convenience of the irrigator.

2 Storage of a portion of runoff design flow—Storage of a portion of runoff design flow allows use of a smaller reservoir than when the entire design flow is stored. Some pump recycling is allowed (in contrast to the systems for continuous pumping).

13.8.6 Equipment

In the following paragraphs, the information on pumps and controls, conveyance systems, desilting provisions, drop structures, high velocity chutes, and protective dikes are taken *verbatim* (except for slight editorial changes) from Fischbach and Bondurant (1970). The information on trash and weed seed screens was provided by Pugh (1975).

Pumps and controls. Single stage reuse type turbines, low lift centrifugal, submerged centrifugal, self-priming centrifugal, or sump pumps are used in reuse systems. The electric driven single stage, reuse type turbine pump makes a convenient pumping plant (Bondurant, 1969) and an overall efficiency of 60 percent or more is easily attained. If the reuse pumping plant is automated with water level controls, absolute fail-safe priming is necessary. Reuse pumps are also powered by internal combustion engines.

Reuse systems are easily adapted to automatic controls. Automatic controls are generally of two types: (a) water level controls in the storage reservoir; and (b) time controls. Water level controls are either air-cell gage switches, float-operated switches, or electrode sensors. The air-cell gage switch uses an air cell located near the bottom of the reservoir connected to a water-level gage switch. The high or low water level contact are adjustable to make the pumping plant start or stop at preset water levels in the reservoir. The float-operated switch turns the reuse pumping plant on and off by the water level float activating a mechanical switch. The electrode sensors use the water as a conductor in the circuit. Time controls (clock operated) are sometimes used on the dugout or larger type reservoirs to turn the reuse pumping plant on and off, or off after an irrigation set has been completed.

Conveyance systems. Most reuse systems will require a return pipe line, either to another field or to the main supply ditch, gated pipe or buried pipe line. The sizes will vary according to the capacity of the reuse pump but probably will be a 100-, 150-, or 200-mm diameter pipe. Pipe lines made from plastic, concrete, asbestos-cement, steel, plastic coated aluminum or fiberglass can be used.

The accessories needed are those which are normally used for pipe lines and pumping plants such as air relief, pressure relief and vacuum relief valves. If the reuse system is connected directly to a gated pipe or pipe line with the main irrigation supply, check valves will be needed on both the reuse and main supply line pump. (Refer to appropriate ASAE standards for design, installation and performance of underground piping systems.)

Desilting provisions. Although surface irrigation is recommended on slopes that do not exceed 1½ percent, many surface irrigation systems are operating today on steeper slopes. Usually, but not always, some erosion takes place on these steeper slopes causing a silt problem. The irrigator may have too long a run for his particular slope requiring a stream size that causes some erosion. Reuse systems operated under these conditions probably will require a desilting basin located ahead of the storage pond, dugout or sump. These desilting basins may need to be cleaned each year or more often with the soil transported back onto the field.

If special desilting provisions must be made, the design criteria summarized by Brown (1950) may be used. The design requires a knowledge of the silt sizes to be removed by settling.

Drop structure, high velocity chutes and protective dikes. The sump or dugout type of storage reservoir needs some means of controlling erosion as the water from the drainage ditch enters the storage reservoir. For some sump-type installations which are constructed of concrete, concrete block, steel casings, etc., the trash screen is attached to the structure and no added drop structure is needed. However, all earth sumps and reservoirs need some structure to prevent serious erosion of the inlet to the reservoir. Cantilevered pipe inlets or most any type of drop structure or high velocity chute works well.

A dike should be constructed around the reservoir to protect it from flood damage due to rainfall or melting snow.

Trash and weed seed screens (Pugh, 1975). The type of screen used will primarily be determined by the design of the reuse systems. Trash in the runoff water must be removed to prevent damage to the return flow pump.

All the water used in a reuse system should be screened before it returns to the field, whether the water source is a reservoir or a cycling-sump system. The screen should be sufficiently small to remove all weed seeds so that fields will not be reinfested with waterborne seeds. Screens are either stationary or moving.

Stationary screens—Stationary screens are the least expensive to construct and maintain. Construction details will vary for each type of screen (Pugh and Evans, 1964).

Horizontal screens set level in both directions are generally best suited to remove both trash and weed seeds. The screen fabric should be 1.6 to 2.4 mesh/mm with 20 m^2 of area for each m^3/s of flow. There must be at least 200 mm of free fall onto the screen to assure self cleaning. The grain of the screen fabric should be set parallel to the direction of the water flow.

Vertical screens placed perpendicular to the direction of flow can be used where there is no fall in the reuse system. The mesh size must be 1.6 to 2.4 mesh/mm to assure removal of the weed seeds. This screen must be manually cleaned and is easily clogged.

Basket outlet screens need at least 80 mm of fall to work properly. The basket can be removed for cleaning, but must be manually cleaned.

Sock-type screens are simple tubes of screen fabric with draw strings on each end. One end is fastened to the outflow pipe and the other is closed to trap the debris. The vertical, basket and sock-type screens are all designed for open ditch type of collection systems. Large screen areas of 65 m^2 per m^3/s of flow are necessary for each of the above screens. These screens also require frequent manual cleaning to be effective. Commercially made filter traps and sand filters will effectively remove small debris and weed seeds, but will only work on the pressure side of the pump system. These filters may be back-flushed manually or automatically if electric power is available.

Paddle wheel screens utilize moving brushes across the screen fabric surface. The screen is fine mesh (1.6 to 2.4 mesh/mm) with a slope of 1 in 10 upward in the downstream direction. Provide a surface area of 20 m^2 per m^3/s of flow.

Backing material and fabric for any of the metallic screens should be the same to prevent corrosion. The plastic or nylon screen fabrics are generally very satisfactory and eliminate any galvanic action.

Moving screens—The most common moving screens are electric or water powered. The electric powered screens are generally used where high flows are anticipated. They use a rotating tubular screen that is self cleaning, using high pressure spray jets to remove the debris.

Rotary cone-shaped drums can be used where the return flow is pumped through the drum. Propellers inside the drum provide the rotation. Screen area necessary per unit of flow depends upon the kind of debris and the effectiveness of the self-cleaning action. Generally, the screen area is similar to that for the stationary screens.

Screens should be kept tight and free from holes. Great care should be taken during the cleaning operation, as fine mesh screens are easily damaged.

All trash and weed seeds should be carefully stored and disposed of to prevent reinfestation of the fields.

13.9 AUTOMATION AND REMOTE CONTROL OF SURFACE IRRIGATION SYSTEMS

13.9.1

Automated irrigation systems reduce labor, energy and water inputs and maintain or increase farm irrigation efficiency. Automation is the use of mechanical gates, structures, controllers, and other devices and systems to automatically divert the desired amount of water onto an agricultural field to satisfy the water requirements of a growing crop.

Border and basin systems are well suited for automation and have received the most attention. Furrow and corrugation systems are much more difficult to automate because water must be uniformly distributed to each furrow or corrugation. A large number of outlets are needed per unit area and each outlet must be relatively inexpensive for an automated system to be cost effective.

Research and development by the USDA, state experiment stations and industry have produced some successful structure, controls, and other devices to automatically control irrigation water on the farm. However, automated surface irrigation is still in its infancy; few commercially

ed systems and components are currently available. About two-thirds of the irrigated land in the United States is surface irrigated and most of this and the surface irrigated land in developing countries is suited for automation. To date, automation has been applied to only a small fraction of this large potential acreage.

Irrigation systems and their associated components are classified as either automatic or semiautomatic, depending upon their method of operation. Automatic systems normally operate without operator attention except for periodic inspections and routine maintenance. The irrigator may determine when and how long to irrigate and turn water into the system or start programmed controllers to initiate the automated functions. Fully automatic systems may use soil moisture sensors, such as tensiometers or electrical resistance blocks, to activate electrical controls when soil water is depleted to pre-determined levels. Meteorological data, using climate-based sensors, can also be used to predict when to irrigate and the output from a microprocessor controller can automatically begin irrigation. Once irrigation has been started, water is diverted into the farm distribution system and irrigation is completed without operator intervention. Irrigation duration may be controlled by programmed timers, soil moisture sensors or surface water sensors. Fully automatic systems require a water supply available "on demand" such as from wells or farm reservoirs. Most farm systems, however, do not have the flexibility required for complete automation.

Semiautomatic systems and controls require manual attention at each irrigation and are usually simpler and less costly than automatic systems. Most current semiautomated systems use mechanical or electronic timers to activate control structures at predetermined times. The irrigator usually determines when to begin irrigation and its duration and manually resets or returns the devices to their original positions or moves them from one location to another before the next irrigation. Parts of a given system may be automatic while other parts are semiautomatic or manually operated. The terms automatic or automated are commonly used in practice and in this chapter when referring to automated systems in general and include fully automatic, automatic and semiautomatic systems.

Most automated and some semiautomated system components are remotely controlled by centrally-located controllers. Such systems require communication between the controller and system components located in the field. Communication may be by direct interconnecting electrical wires, by hydraulic or pneumatic conduits, or by radio telemetry. Spurious signals and interference can sometimes be problems when telemetry is used.

13.9.2 Automation Principles and Design Considerations

System components. An automated surface system is similar to a regular system except it must include: (1) a field prepared for controlled irrigation water flow; (2) water supply controls including structures or valves that are automatically controlled; (3) turnout or discharge outlets that deliver a specified flow into each segment of the field being irrigated; and (4) activating mechanisims or devices to open and close gates or valves automatically in a selected sequence. A tailwater pickup and water reuse system that automatically recirculates irrigation runoff, or stores it for future use, is also usually needed. The mechanical components and structures that are unique to automated surface systems are discussed later.

Operational sequence. In general operating principles of automated systems are similar for pipeline and open-channel systems even though the structures, valves and other devices are different. Irrigation usually proceeds either downstream or upstream as each field segment is irrigated in sequence. With open distribution channels and an irrigation sequence that proceeds upstream from the lower end of the ditch, the field segment at the lower end of the ditch is irrigated first. Water is automatically checked consecutively at each upstream turnout in the head ditch and diverted onto the field through an automatically controlled gate or a fixed opening. When irrigation proceeds from the upper end of the ditch towards the downstream end, the sequence is reversed. An advantage of irrigation in a downstream sequence is that the ditch can be used to convey water between irrigations before the turnout gates are reset for the next irrigation. Thus, the ditch can be used to convey water to other parts of the field or to carry runoff or excess flood waters. Also, the ditch is naturally drained after each irrigation without dead storage remaining between checks. An advantage of irrigating in an upstream sequence is that in case of a gate failure, only one part of the field is missed because the next gate upstream operates as scheduled. If a malfunction occurs when irrigating in a downstream sequence, water may continue to flow on the same set until the problem is corrected. Another advantage of irrigating in an upstream direction is that flow can be more conveniently diverted into another distribution ditch or channel since the last field segment to be irrigated is at the upper end of the ditch. A disadvantage of irrigating in the upstream direction is that all timers must be operating simultaneously until each segment is irrigated. For long irrigation periods, the total irrigation time may exceed the time capacity of mechanical timers.

Random sequence is possible with automatic systems having gates or valves that can open or close against a head of water.

Irrigation timing. Irrigation duration is usually timed with mechanical (alarm clocks and 24-hour timers), electromechanical or electronic timers. Since electronic timers have become more reliable and can operate for extended time periods using battery power, they are now preferred. Although still commonly used, mechanical timers provide only one timing function, whereas irrigation requires two timing functions—one for the delay until irrigation begins and the other for the irrigation duration. Electronic timers can satisfy these two functions with a single timer and also provide an electrical output to actuate a secondary device for tripping a gate or operating a valve. Most mechanical timers use a direct mechanical linkage to trip or actuate a gate or valve.

Irrigation duration can be controlled with soil moisture sensors such as tensiometers or electrical resistance blocks or with water sensing devices such as floats if certain obstacles are overcome. The sensors can compensate for changes in advance time through a field from one irrigation to the next. One disadvantage of using a sensor to determine irrigation duration is that some means of communication between the sensor and the controller is required. The controller is usually located near the upper end of the field or near the farmstead. Thus, the control system is most feasibly restricted to radio telemetry because direct wire or fluid conduit communication from the lower end of the field is too cumbersome for multiple irrigation sets. Also, satisfactorily terminating furrow irrigation with sensors is difficult because of the interrelationships between stream size, length of run, soil intake rate and sensor location.

Use of sensors to begin irrigation, however, is feasible. With the present state-of-the-art, it is more satisfactory to use sensors to begin an irrigation sequence and then use a timer or volumetric flow measurement to determine irrigation duration. An established sequence must be used once irrigation is started. If irrigation is randomly controlled completely by sensors, management of a stream of water on the farm is very difficult.

Electromechanical and electronic programmed controllers used for sprinkler and drip systems can sometimes be used directly or modified for surface irrigation systems. Some of these controllers do not have the required time duration capacity nor can they be powered from batteries.

Cutback irrigation. One advantage of an automated system is that cutback furrow streams can be used to reduce runoff. Manual cutback is seldom practiced because of the extra labor required and the problem of handling the excess water during the cutback mode. One technique to achieve automatic cutback streams is to pump additional water from a reuse pond only during the initial or wetting phase of the irrigation (Fischbach, 1968). Another method is the split-set technique where the total set or field segment is divided into two parts (Humpherys, 1978a). The first half of the set is irrigated with the entire stream until water runs off the field. The entire stream is then directed onto the other half of the field segment for the same length of time. Water is then reintroduced into the furrows of the first half so that the entire stream of water is distributed across the total set for the remainder of the irrigation. The experimental surge flow concept of Stringham and Keller (1979) is discussed in Section 13.9.5.

Cutback streams from lined ditches can be achieved by constructing the ditch in a series of level bays with spile outlets at equal elevation along the side of the ditch. Water is released sequentially downstream from one bay to the next by timed check gates. As the water advances to the next check, the water level in the upper bay is lowered and flow from the upper bay outlets is reduced (Garton, 1966; Humpherys, 1971; Nicolaescu and Kruse, 1971; Hart and Borrelli, 1972; Evans, 1977).

Flow measurement. Flow measurement or volumetric flow control devices are necessary for an efficient water management system. With volumetric flow control, the volume rather than the time determines the irrigation duration. This method is particularly adaptable to level basin systems having a variable supply flow. With this method, a predetermined volume of water is metered into the basin and the flow measuring or volume control structure is instrumented to terminate irrigation or change irrigation sets when the required volume of water has been applied.

Constraints and limitations. A number of limiting factors need to be considered when designing automated surface irrigation systems. Some of these factors may not be applicable to a specific installation but each can affect the practicality of installing a system.

1 **Flexible water supply.** The degree of automation depends largely on the farm water supply. Semiautomatic controls are usually used when farms receive water on a rotation basis. Water on demand with flexibility in frequency, rate, and duration is needed for fully automated systems. Most existing open channel delivery systems do not have the capability to respond to variable, unscheduled deliveries as automated farm systems accept and reject water. Level top canals equipped with automatic constant water level control gates can help provide the needed flexibility (Merriam, 1974, 1977).

If farm regulating reservoirs or storage ponds are used to accumulate continuous or intermittent canal deliveries, water can be supplied to the farm distribution system with greatest flexibility. Farm runoff can be largely eliminated when a reuse system is used with these reservoirs. Trash and outlet plugging problems are also reduced because water withdrawn from the reservoir is usually cleaner than that supplied by a canal. Farm reservoirs should be located at the upper end of the field or farm. This allows continuous water delivery to the reservoir from a canal while irrigating from the reservoir at a different flow rate. Also, when irrigation is completed, the irrigation system can be shut off without having to make further provision for water being received from the canal. Water delivery from the reservoir to the irrigation distribution system can be efficiently controlled with float valves (Humpherys, 1978b).

2 **Variable soil intake rates.** The operation and management of an automated furrow system is complicated by variable furrows intake rates. One of the primary causes of different intake rates between furrows is unequal tractor wheel traffic. Irrigating every other furrow reduces this problem where alternate furrows receive the same amount of traffic. Variable field slopes also can cause unequal furrow intake rates. Fields that are automated should be planed to a uniform slope to prevent sediment deposition and furrow overtopping. Plant leaves, stems and residues in the furrows and rodent activity affect the furrow intake rate. One of the primary objectives in automating irrigation systems, that of reducing labor, is partially defeated if gates or other distribution outlets must be individually adjusted throughout the season or during irrigations to compensate for variable intake rates. This problem is greatest with easily erodible soils because the stream size must be controlled to minimize furrow erosion. Where a return flow system is used and soils do not erode easily, runoff does not have to be carefully controlled and relatively large streams can be used. Besides reducing the effect of variable intake rates, large streams improve uniformity and increase the probability of all furrows being completely irrigated.

3 **Equipment factors.** One of the constraints to automating surface irrigation systems is the lack of well designed, self-contained, complete commercial systems or system components. Currently, equipment and components needed for an automated system often must be modified or adapted from other uses.

A frequent limitation is the lack of AC electrical power in the field where system components are located. In contrast to sprinkler systems where AC electrical energy is usually provided at a central location, such as for a pump, automated surface systems require very small amounts of energy at a number of locations. Because of this small demand, installing or extending a power line to provide service for automated surface system controls is seldom feasible. Battery or solar-powered electrical components have only very recently become available. Solid state electronic devices, latching relays and latching solenoids that use very little energy and can be powered by batteries are required in most systems. Solar energy will be increasingly used as improved technology lowers component costs.

If used to apply fertilizer, irrigation water may contain high concentrations of soluble salts. System components must be carefully chosen and mounted to avoid corrosion damage. This problem can be minimized by use of plastic, but certain plastics that deteriorate when exposed to direct sunlight and high temperatures should be avoided. Timers and electrical components should be adequately dust and waterproofed. Water-filled or -actuated components also must be protected from freezing. Pneumatic components can be used to avoid frost damage, but maintenance of airtightness is sometimes difficult.

4 **Distribution outlets.** The cost of automated furrow systems depends upon the number of distribution outlets required. The cost of each outlet must be relatively low for the total system cost to be feasible. For a 400-m length of run, and depending upon the furrow spacing, the number of outlets will vary from about 22 to 32 per ha. By comparison, the number of sprinklers on a mechanical-move center pivot system ranges from about 0.8 to 1.9 per ha.

Tubes, notched outlets, and weir outlets have been used to control flow into each furrow but have not been very satisfactory. Gated pipe currently is the most feasible method of distributing water in an automated furrow system.

5 **Erosion.** Excessive erosion may occur where the streams from gated pipe strike the soil surface. This problem is aggravated on steeply sloping land that increases pipeline pressure. Fabric tubes can be used over each pipe gate to minimize erosion, but they are a nuisance and add to the total system cost.

Orifice plates or other energy dissipating devices installed in the pipe are sometimes used to control pipe water pressure. Small overflow stands similar to concrete stands used in gravity pipeline systems can also be used to limit the amount of head that can develop in the pipe on steep slopes.

6 **Trash and debris.** Trash is a common problem where water is supplied from a canal. Clogging of gates and furrow discharge outlets cannot be tolerated in an automated system where an operator is not present to keep outlets clean. Clean water is essential; it is difficult to find a screen that is satisfactory for many canal turnouts, particularly where electric power is not available.

7 **Rodents.** Automated systems using air supply lines are particularly susceptible to damage by rodents. The most satisfactory solution has been to encase the plastic tubing and other susceptible components in rodent-proof material. Air lines are being installed in concrete lined ditches to reduce this problem. Furrow blockage by gopher mounds may cause nonuniform irrigation with permanent or semipermanent crops such as pasture, alfalfa and orchards.

13.9.3 Pipeline Distribution Systems

Pipeline systems are easier to automate than are open channel systems. Pipelines and associated facilities may be buried or placed on the surface. Many existing systems can be equipped with automated valves and other components to reduce the cost of converting to an automated system. Pipelines can be designed using the criteria presented in Chapter 11. Automated valves, outlets, or both are used to sequence water from one set or field segment to another.

Pneumatic valves. One of the first pneumatically operated valves, developed by Haise et al. (1965), consists of an inflatable O-ring, or doughnut-shaped diaphragm, constructed from a rubber inner tube and supported with a butyl rubber cover. When mounted on an alfalfa valve and in-

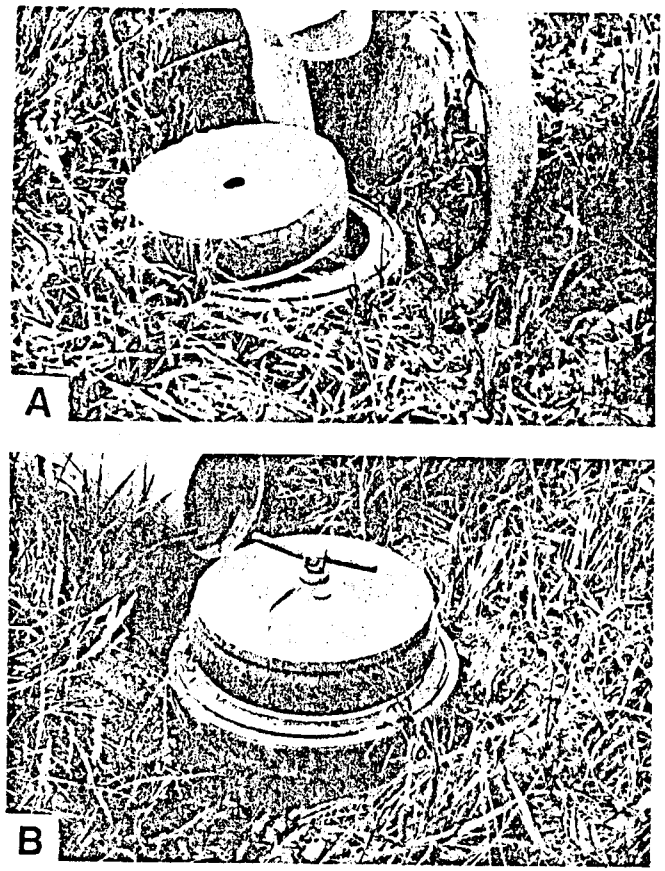


FIG. 13.14 Pneumatic irrigation valve for automating pipelines. Valve being installed on an existing alfalfa valve (A), and inflated valve stopping flow of water (B).

flated with air, the tube forms an annular seal betwen the alfalfa valve seat and lid as shown in Fig. 13.14. In addition to controlling discharge from buried pipelines into borders and basins, it can be used with a hydrant for gated pipe irrigation (Haise et al., 1980), and also can be used with portable controls (Edling et al., 1978). A modification by Fischbach and Goodding (1971), shown in Fig. 13.15, is commercially produced.* The commercial valve has a specially fabricated, pneumatic diaphragm. It is constructed with a male pipe fitting on the bottom inlet and is installed directly on top of a riser from an underground buried pipeline. Pneumatic valves, except the portable models, require an air compressor to provide air for actuation.

Water-operated valves. Self-closing and regulating valves using water for actuation were developed by Haise et al. (1980). These can be used for both on/off and modulating discharge control. Water-filled bladder valves developed by Humpherys and Stacey (1975) are used to control irrigation through gated pipe and buried lateral distribution pipelines. The valves

*Manufactured by the Econogation Valve Co., Humboldt, NE. Trade and company names are shown for the benefit of the reader and do not imply endorsement or preferential treatment of the company or products listed.

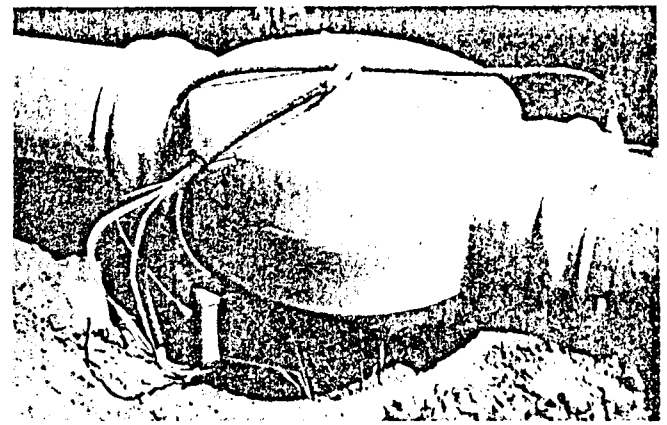


FIG. 13.15 Pneumatic valve for automating gated pipe systems.

operate as independent units without an outside energy source. Water from the pipeline is used to close the valve. Valve opening and closing are controlled by battery powered, timer activated, 3-way pilot valves. The valve model shown in Fig. 13.16 is commercially available.†

Buried distribution laterals. A system developed by Varlev (1973, 1978) using buried distribution laterals is used in Bulgaria. This system consists of a buried pipeline with a telescoping riser for each furrow. The risers are extended to the surface hydraulically at the beginning of the irrigation season and pushed down by hand at the end. Water is sequenced from one buried pipeline to another with automatic valves that utilize water pressure in the pipeline for activation (similar to those of Humpherys and Stacey, 1975). A similar system that currently uses flexible risers and calibrated orifices at each furrow outlet was described by Worstell (1976, 1979).

The advantage of a buried system is that field tillage and cultural operations can be performed over the top of the irrigation pipeline. A multi-set

†Manufactured by Hastings Irrigation Pipe Co., Hastings, NE.

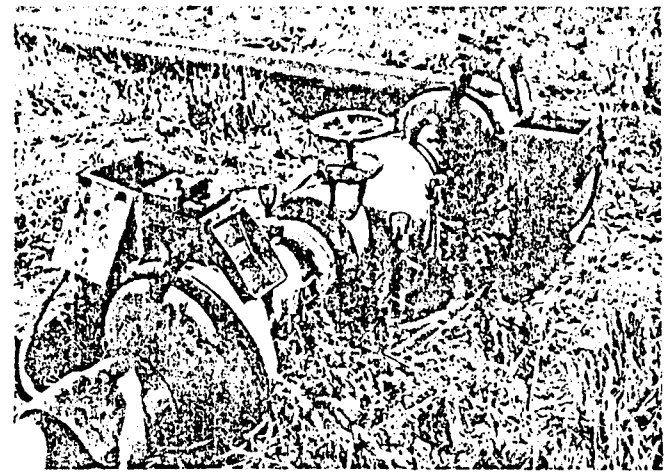


FIG. 13.16 Automated valves that use water from the pipeline for operation.

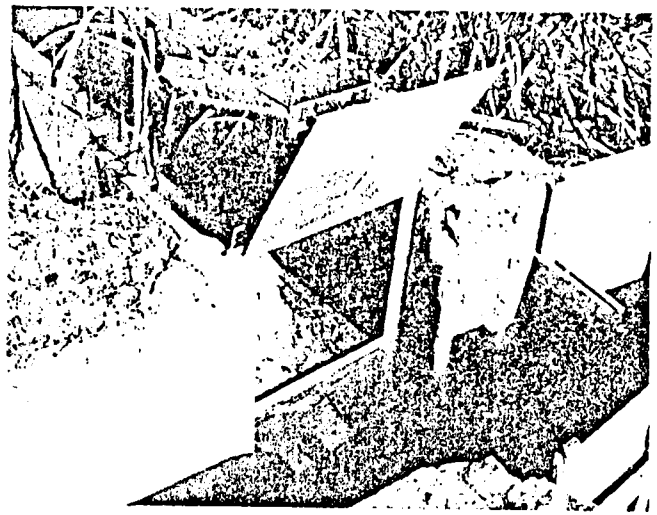


FIG. 13.17 Timer-controlled, semiautomatic drop closed check gate.

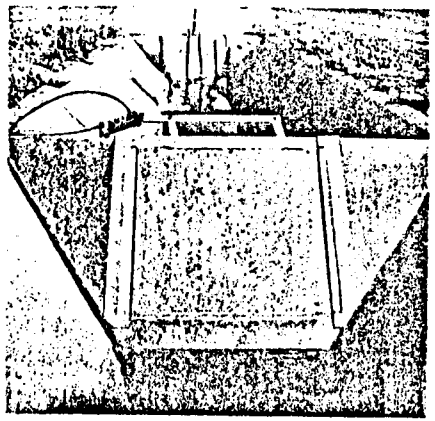
concept (Rasmussen et al., 1973) can be used to introduce water into irrigation furrows at one or more midfield locations. This shortens the effective length of run and makes small stream sizes possible. Using small furrow streams reduces erosion and runoff from a field because field runoff occurs primarily from the last subset at the lower end of the field.

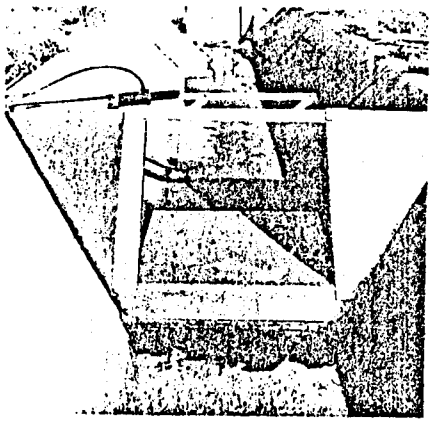
Reuse systems. With the appropriate interfacing controls, reuse systems (Section 13.8) can be an important part of an automated pipeline system.

13.9.4 Open Channel Distribution Systems

Simple, timer-controlled gates of various designs have been used in farm ditches for years. Individual farmers have made many of these in their own shops.

Drop open and drop closed gates. The most common gates for open channel systems are the drop closed and drop open types. The drop closed gate is used to divert water directly onto irrigated fields or from one ditch into another. In the open position, it is suspended over a flow opening and, when tripped, falls by its own weight to stop the flow of water (Fig. 13.17). It is





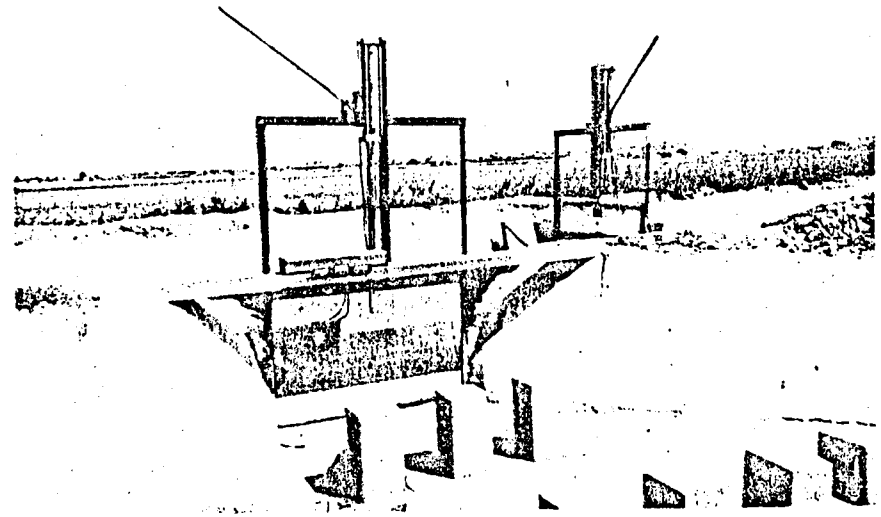
used in lined and unlined ditches and may be permanently or probably mounted. Semiautomatic drop closed gates and dams tripped by mechanical timers are extensively used in New Zealand with sill- or weir-type side outlets and borders to irrigate pastures (Taylor, 1965; Stoker, 1978).

The drop open gate is hinged so that when tripped, it either falls or swings open to allow water to flow downstream (Fig. 13.18). It is normally used as a companion gate to the drop closed gate. When used in pairs, one gate opens while the other closes to divert water from one turnout or flow opening into another. They may be tripped by different actuating devices including mechanical timers, solenoids, floats, or pneumatic and hydraulic cylinders. Both types of gates were widely used in Hawaii for sugar cane irrigation before the advent of drip irrigation (Reynolds, 1968). Gates of various configurations and designs have been used by different investigators (Calder and Weston, 1966; Kimberlin, 1966; Humpherys, 1969; Hart and Borrelli, 1970; Lorimor, 1973; Evans, 1977; Haise et al., 1980).

Drop closed and drop open gates are sometimes mounted together on the same structure to form a "combination" gate (Humpherys, 1974). These are used as turnouts into a border or basin where it is necessary to first divert water onto the field and then terminate irrigation by closing the opening against a head of water. They are needed where head ditch slopes are relatively flat and there is more than one turnout between each ditch check.

Automated lift-gates and tile outlets. Lift-gates are commonly used in Arizona where large irrigation streams are used. Dedrick and Erie (1978) and Haise et al. (1980) described equipment used to automate lift-gate systems to irrigate level basins. The lift-gates shown in Fig. 13.19 are actuated with air cylinders. Compressed air is diverted through a four-way pneumatically- or electrically-operated pilot valve to the bottom of the cylinder to open the gate or to the top of the cylinder to close the gate. The automated gates are used both as check gates and as diversion structures.

Concrete pipe tile outlets 410 mm in diameter are used with the lift-gates to divert water from a supply ditch into level basins (Erie and Dedrick, 1978;



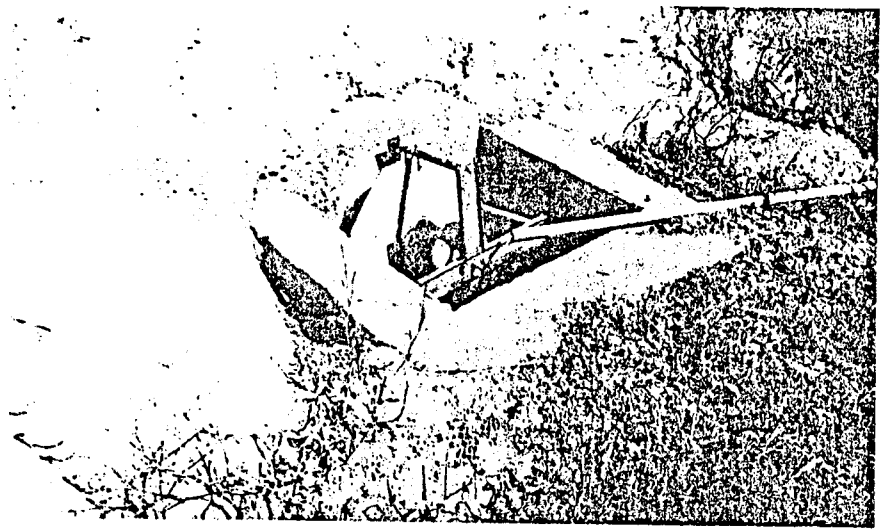


FIG. 13.20 Pneumatic pillow-disc valve installed on a concrete pipe tile outlet. Note the dissipator box.

Haise et al., 1980). A pillow-disc valve, consisting of a metal ring insert onto which are attached a permanent stop, an air pillow, and a moveable plate or disc, is mounted on the discharge end of the outlet as shown in Fig. 13.20. The plate is forced against the seat by the air inflated pillow to stop the flow of water from the outlet. Concrete energy-dissipator boxes are used at the end of the outlet to control erosion. About 80 kPa air pressure is needed to operate the air pillows, and 345 kPa is used to operate the lift-gates. The air compressor, irrigation controller, and other associated components are located in a control center storage shed where AC electrical power is available.

Traveling dams. Machines that divert water continuously from an irrigation ditch are sometimes used with close growing forage and grain crops where large streams of water are available for surface flooding. These slow-moving commercial machines, powered by small gasoline engines, straddle the ditch and pull canvas, plastic, or rubber dams that cause the water to overflow the ditch banks. The ditches are usually constructed so that the upper bank is higher than the lower bank so that the water always flows over the lower downstream ditch bank.

Other gates and outlets. A number of other different types of gates and outlets have been used to a limited extent. Circular skimming and rectangular weir type outlets with automatic check gates have been used in Australia‡ (Robinson, 1972). Weir-type outlets were described by Sweeten and Garton (1970). Several types of pneumatically- and hydraulically-operated butterfly, modulating, push-off, pillow-disc, self-closing and regulating, and fluidic diverter outlets and check gates were developed by Haise and Kruse (1969) and by Haise et al. (1980). Pillow-disc valves are used as discharge outlets from ditch turnouts, buried pipeline risers and individual furrow gates. They consist of a disc that is forced against a seat by an inflated air bladder or pillow positioned directly over the disc. Bowman (1969) tried radio controls with soil moisture sensors and center-of-pressure gates.

‡Metseal automatic irrigation structures developed and licensed by the State Rivers and Water Supply Commission of Victoria, Armadale, Australia.

13.9.5 Experimental Systems

Most automated systems are still in the experimental stage of development. Several systems have not progressed so far that they have been used but have potential for practical application with further development.

Single-pipe system. Most automated furrow irrigation systems require two parallel pipelines at the upper end of an irrigation run. The conveyance or main pipeline is either buried or placed on the surface. The second pipeline, usually gated pipe, serves as the distribution line. The system could be simplified and the cost reduced if a single pipeline could serve both functions. Several attempts have been made to accomplish this and experimental work is still underway. Reynolds (1968) described the "miniwai" system in Hawaii which uses a membrane installed inside the distribution pipe to cover a group of furrow outlets or openings simultaneously when water flows above the membrane. Water is discharged from the openings when flow inside the pipe is below the membrane. Fischbach used an air cylinder connected to cables or rods to operate sliding pipe gates (Haise and Fischbach, 1970). Stringham and Keller (1979) used a bank of automatically controlled individual pneumatic valves in a single pipe to test the surge flow irrigations concept. Haise et al. (1980) automated individual openings in gated distribution pipe with hydraulic pipe gates and pneumatic pillow-disc valves.

Surge flow. The surge flow concept of automatic cutback irrigation was investigated by Stringham and Keller (1979). Cutback is achieved by using intermittent surge flows. Banks of furrow valves are automatically controlled to be either completely open or closed. When the valves are open half the time and closed half the time, then the full flow running half the time in a given furrow produces about the same average stream size as half the flow running full time. The cycle time can be variable.

Traveling irrigator. A mechanical, continuous-move distribution lateral is being developed for furrow irrigation by Lyle and Bordovsky (1979). This system used a modified rectilinear-move sprinkler lateral. Modifications include replacing sprinklers with drop tubes and orifice-controlled emitters for each furrow and adding the components necessary for propulsion, guidance and control. The system operates on 70 to 170 kPa and moves in a continuous rectilinear pattern through the field. It is used in conjunction with microbasins formed in the furrows. Tillage implements form the microbasins which are used to maximize both irrigation efficiency and rainfall utilization.

Controllers. Solid state electronic technology using microprocessors is developing so rapidly that it is impractical to describe specific units, although some units are available commercially. New concepts include a battery-powered controller capable of integrating flow rates through a nonlinear open channel flow measurement device (Duke et al. 1978). The controller can be programmed to control a large number of turnouts in any sequence and to terminate irrigation after the programmed volume of water has been applied. An experimental soil moisture monitor system that provides a visual indication of soil moisture status and the capability of controlling irrigation automatically was described (Anon. 1975).

13.9.6 Benefits and Costs

Because of the experimental nature and limited use of most automatic surface irrigation systems, measured irrigation efficiency and labor input data are limited. Although there are many potential benefits, the dominant reasons that irrigators automate are convenience and labor reduction. Stoker (1978) reported that with semiautomated border systems and a minimum 0.23 m^3/s stream size, 24 ha can be irrigated with one manhour of labor and with a fully automated system using sensors, 81 ha can be irrigated with one manhour. In contrast, only about 0.8 ha/h can be irrigated with nonautomated border systems. With semiautomated level basins in Nevada, labor requirements were reduced 80 percent compared to manual irrigation using graded borders (Kimberlin, 1966), and irrigation efficiency was increased 13 percent by using semiautomation in Hawaii (Reynolds, 1968). Only 1/6 as much labor was required for semiautomated mountain meadow systems as for manual irrigation systems and only half as much water was used (Lorimor, 1973). Fischbach and Somerhalder (1971) reported average irrigation efficiencies of 65 percent for automated gated pipe without a reuse system and 92 percent with a reuse system. The average runoff, 27 percent without reuse, was recovered when the reuse system was used. Humpherys, (1978a) reported a 20 percent decrease in runoff with a corresponding increase in applied water use efficiency for a semiautomated system using gated pipe on plots of sugarbeets and corn as compared with nonautomated plots.

Irrigated sets of either 12 or 24 h are commonly used because of the inconvenience of making irrigation sets more frequent or at time intervals other than 12 or 24 hours. With automation, irrigation sets of different durations can be made as required by soil and plant conditions.

Fertilizer loss can be reduced by automation because of the reduction in runoff and deep percolation and the increased use of reuse systems. Automation can reduce energy costs by making surface irrigation more acceptable than alternative systems that use more energy. Batty et al. (1975) estimated that sprinkler systems use from 4 to 13 times more total energy than do surface systems.

Automation costs are highly variable and depend upon the water supply; degree of automation (automatic or semiautomatic); existing facilities; irrigation method (whether furrow, border, level basin, etc.); and field conditions such as degree of leveling or land surface preparation required, length of run, slope, soil texture, field size and shape, etc. Depending upon local conditions, costs can range from as little as 1/4 to 1/2 the cost of a self-move wheel roll sprinkler system for some of the simple low-cost surface systems to as much as 1-1/2 times the cost of a center pivot sprinkler system for a more elaborate buried lateral multiset system. The cost of a double pipe automated gated pipe system is approximately comparable to that of a self-move side roll sprinkler system.

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