THE SURFACE IRRIGATION MANUAL

A COMPREHENSIVE GUIDE
TO DESIGN AND OPERATION
OF SURFACE IRRIGATION SYSTEMS

BY DR. CHARLES M. BURT

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A Comprehensive Guide
to Design and Operation
of Surface Irrigation Systems

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By

DR. CHARLES M.SURT

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A WORD FROM THE PUBLISHER

Surface irrigation is the most widely used means of irrigating crops in the world, yet a modern, comprehensive reference that would explain current design practices for surface irrigation systems and proper system management has been lacking. This handbook changes all that. It is complete, starting with basic soil/water/plant relationships, continuing through specific design criteria forvarying types ofsurface systems, evaluation of operating efficiencies, equipment choices, and more. Waterman Industries, Inc. is pleased to have been involved in supporting Dr. Charles Burt in his efforts to bring organization and wisdom to this volume. The agricultural community has long awaited a manual such as this and we believe the collected knowledge contained within will benefit all of us.

ACKNOWLEDGMENTS

The publishers wish to acknowledge valuable assistance given by individuals in the effort to create this manual. Foremost among these is the late Robert J. Tucker, whose prodding and hard work caused this work to be undertaken. We would also like to thank Dr. John Merriam for his work with the author and his valuable lifetime contribution to the science of surface irrigation. Thanks also to Dr. I. Joe Glass who spent numerous hours reading and critiquing draft copies of this work. Finally, we would like to acknowledge the support offered by Don F. Appling, T. Stanley Trapp, and J.B. "Joe" Fiala, without Which, there would be no manual.

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PREFACE

The Surface Irrigation Manual was created to -

- Give the reader the tools for understanding the surface irrigation method.
- Assist readers in implementing their own systems.
- Provide enough technical information to maximize the results of irrigation.
- Allow for a well-founded basis in the system (method) selection.
- Encourage optimum management of the system.
- Generate an awareness that surface irrigation, the most ancient of all methods, is even more viable today.

The intent permeating the following pages is to encourage the proper use of surface irrigation through the recognition of its financial advantages, its relative simplicity, its state-of-the-art technologies, its high soil, crop and water efficiencies obtainable, its design and management guidelines, its automation possibilities, its inherent superior characteristics and its universality.

To effectively present a manual that is informative, educational and practical, the fundamentals of agricultural irrigation (surface irrigation in particular), and the design of these systems are treated in depth. It is hoped that this manual will be used as a single-source aid in understanding, evaluating and designing surface irrigation projects both large and small. It is further hoped that the truths contained herein will underscore the rightful dominance of surface irrigation in agriculture.

Robert J. Tucker

INTRODUCTION

Since time immemorial surface irrigation has been a part of civilization. Earliest records indicate that the Egyptians, Medes, Incas, Romans, American Indians and other early and neo-civilizations channeled water a short distance to selected food sources.

Today, thanks to engineering and construction progress, water has been made available for irrigation even in arid and semi-arid lands. Dams and reservoirs, aqueducts, canals, ditches and pipelines provide the means to bring water to ever-expanding lands, either for underground aquifer replenishment or on the surface for ready use.

Since an overwhelming amount of the irrigation in the United States and the world is by the Surface Irrigation method, it is imperative to understand its functions, its state of the art and its advantages, and to assess its position in relation to the needs of irrigation. This Surface Irrigation Manual is created to foster an understanding of the system, to detail the parameters for proper design and management of a surface system, and to present the opportunity for maximum irrigation efficiency. Surface Irrigation is the simplest of all the methods, requires the least amount of equipment or devices, and uses the least amount of energy. In today's era of ever-rising energy and investment costs, the advantages of surface irrigation become even more pronounced and important than its historical impact.

Before proceeding into the fundamentals and design of surface irrigation systems, it is necessary that an overview of this method be presented as an introduction. In-depth treatment of the subjects appear later in the Manual.

Surface Irrigation

"Surface Irrigation" is the designation given to the irrigation method whereby water is flowed generally by gravity over the surface of the soil to be wetted. Other methods include "sprinkler", which refers to water under higher pressure sprayed by a mechanical device onto the soil, and "drip", "trickle" or "micro" which refer to water emitted in droplet, mist, or reduced amounts from tubing through small apertures onto the soil. "Pivot" and "linear" systems are generally merely mechanical conveyances for sprinklers.

The three principal types of Surface Irrigation are Level Basin, Border Strip and Furrow.

(CONTINUED)

INTRODUCTION

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(CONTINUED)

Level Basin

Level Basin irrigation has received a lot of attention and credibility particularly for growing alfalfa. A large plot of land, usually 10 acres or more, is planed (leveled) finitely to a dead level surface and contained on all sides by a levee, bank or berm. The irrigation, then, is characterized by applying a large volume of water quickly so that the total surface is wetted, ostensibly at once. The water itself as it fans out provides the slope necessary to assure gravity flow to totally inundate the field. It is apparent that an adequate supply of water must be available, and this is generally assured by a well-managed ditch system supply.

Border Strip

Border Strip irrigation is oftentimes employed on grains, alfalfa, trees and pasture grasses. The sloping area to be wetted is contained by berms spaced 20-50 feet apart, and the layout can be graphically compared to the lines on an American football field. The water is flowed in large volumes onto the sloping contained strip through large "alfalfa" valves or other types of equipment. The irrigation frequency is critically dependent upon an adequate soil moisture deficiency so that overwetting does not occur.

Furrow

Furrow irrigation is the dominant type of irrigation in the world. Closely-spaced small ditches or corrugations identify furrow irrigation. The individual plants or trees are situated on the higher ridge soil, while the irrigation water is carried along the adjacent trench (furrow). Ideal for row crops such as cotton, corn, sugar beets, soy beans and the like, furrow irrigation is also widely employed for vineyards and trees.

Water is conveyed by gravity along the furrow, gradually infiltrating the root zone laterally and vertically. Some furrows are level which forces the advancing water to provide its own slope as it moves along. Most furrows, however, slope downward from the water intake location. The degree of the slope and the volume of water introduced are the major determinants of the speed of the water down the furrow, and oftentimes the effectiveness of the irrigation. Refinements of furrow irrigation are treated at length in later chapters.

Robert J. Tucker

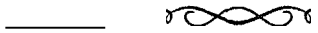
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CHAPTER 1



SOIL - PLANT - WATER RELATIONSHIPS

SOIL - PLANT - WATER RELATIONSHIPS

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SOIL-PLANT-WATER RELATIONSHIPS

Introduction

The role of soil from an agricultural perspective is to:

a. Act as a moisture reservoir between rains and/or irrigations.
b. Act as a nutrient reservoir.
c. Mechanically support and stabilize plants.

Evapotranspiration (ET) is the combination of transpiration and evaporation. Transpiration is the liquid movement of water from the soil, into the roots, up the plant stems, and finally out of the plant leaves into the air as vapor. The type of evaporation considered in ET is evaporation from a wet soil surface, in which case the water does not pass through the plant.

"Irrigation Scheduling" involves managing the soil reservoir so that water is available when it is needed by the plants. Important concepts in irrigation scheduling are:

a. When one should irrigate.
b. How much water should be applied during an irrigation.

These questions are not easy to answer because:

a. Each soil has different moisture holding characteristics. Such characteristics include soil depth, available water holding capacity per foot of soil, and the relationship between soil moisture depletion and matrix potential.
b. Plants differ in their abilities to withdraw water from soils. Strawberries are notoriously sensitive to soil moisture stress, while stressing turf grass by 20% can result in good looking lawns with the benefit of less mowing requirements. Some crops such as cotton may need to be stressed at certain growth stages to maximize yields or crop quality.
c. Plants differ in their water use rates. The maximum evapotranspiration rate of citrus trees is about half of alfalfa.
d. Water quality (salinity) may influence the availability of soil water to plants.
e. Irrigation systems may have inherent characteristics which restrict the frequency, rate, or duration of water application. For example, a gross application of a half inch of water with furrows would typically result in very non-uniform irrigation, since the water would probably not even reach the end of the furrow. A very small application by a center pivot moving at maximum speed in an arid environment may also result in an inefficient irrigation, but for a different reason. With the fast-moving pivot, a very large percentage of the applied water would end up as non-beneficial evaporation from plant and soil water surfaces.
f. Certain irrigation practices may be needed for reasons other than meeting evapotranspiration requirements, such as for frost control, softening a soil crust, or incorporation of a herbicide into the soil.
g. External factors such as labor schedules, availability of water from a supplier, or the need to avoid irrigation of city parks during weekends or daytimes, can dictate irrigation scheduling details.

Soil Texture

Soil texture is defined by the ratios of various sizes of the soil particles. Soils are composed of a mix of clay (smallest particle size), silt, and sand (largest). The classic soil texture triangle is shown in Figure 1. The texture of a soil can only be changed by massive plowing to mix the textures of various soil depths, or by importing huge amounts of soil. In agriculture, deep plowing is common in some areas, but importation of soil is rare. In the landscape industry, many surface soils are imported.

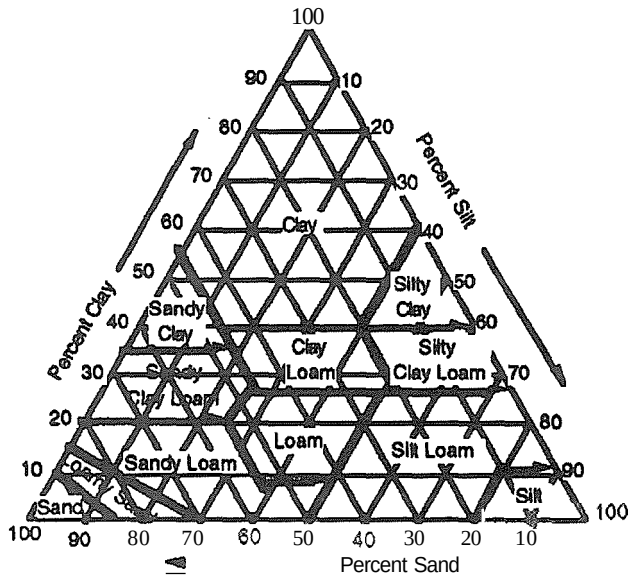


Figure 1. Soil *texture triangle, showing the proportions of sand, silt, and clay particles for different soil textures.*

In a typical soil, the texture varies with depth. In alluvial soils and old marine deposits, there may be tremendous variability of texture with depth; in some cases sandy textures overlay clay textures, and in other cases the reverse is true. In other, non-transported soils, the upper layers are often sandier than deeper layers.

Nutrients are found in the soil water solution. Positively charged nutrients (cations such as Calcium, Ca^{+2}; Ammonium, NH_4^+; and Potassium, $K+$) are also adsorbed (attached rather than absorbed) to the negatively charged surfaces of the clay particles. The "cation exchange capacity" of a soil is a measure of the amount of negative charges a certain weight of soil has (milliequivalents per 100 grams of oven dried soil). Sand and silt particles have little or no cation exchange capacity, and therefore do not store nutrients. Because the soil cannot hold onto negatively charged nutrients (anions such as Nitrate, NO_{3^-}) any irrigation deep percolation will leach (flush) some of these anions from the soil. The low cation exchange capacities of sandy soils, and their inability to store large amounts of nutrients, explain the success of applying frequent, small amounts of nitrogen fertilizer along with the water through well designed irrigation systems (a practice called "fertigation") rather than only applying large amounts of fertilizer to the soil occasionally.

Soil Structure

The structure of a soil is determined by the way various soil particles (sands, silts, and clays) are held together and oriented with respect to each other. A good structure will have many large pore spaces throughout the root zone, allowing water and air to move. The structure can either be improved or destroyed by irrigation and general agricultural management.

The most common causes of poor structure due to general agricultural management include i) equipment compaction due to heavy tractors or an excessive amount of equipment traffic through the field, and ii) depletion of organic matter in the soil. Irrigation management typically affects the top of the soil surface, which in turn affects infiltration rates. Irrigation factors which degrade soil surface structure include i) irrigation with water having a high percent sodium, ii) irrigation with water having a high ratio of magnesium to calcium, iii) irrigation with a very pure water, and iv) sprinkler irrigation at low pressures with large droplets. Soil surface structure problems may be difficult to see; a very thin layer of impermeable soil can reduce infiltration rates as effectively as a thin layer of plastic.

With furrow irrigation, the soil surface structure changes each time one irrigates. At the beginning of a season, a freshly plowed field may have large clods in the furrows bottoms. As the irrigation season progresses, the irrigation water tends to dissolve these large clods, and equipment traffic in the bottom of the furrows further compacts the soil surface. The infiltration rates typically decrease as the season progresses. In addition, surface irrigation will compact the top foot or so of some soils due to the high soil moisture contents during irrigation.

A typical soil structure problem in the western U.S. is the existence of various types of "hardpans" at depths of 2-3 feet, with thicknesses ranging from 0.5 to 3 feet. These hardpans must be ripped or in some other manner be destroyed or broken up in order to develop a reasonably deep soil for adequate root development and water storage.

The USDA Soil Conservation Service (SCS) provides soils maps and information for a large percentage of the U.S. agriCUltural soils, and some urban areas. This information includes a description of textures and structures at different depths, boundaries of various soil series, slope, and available water holding capacities. The maps and books can be obtained at many university libraries and at local SCS offices.

Soil Moisture Terms

This section covers the basic terminology which is used to describe soil-plant-water relationships.

1. Soil moisture reservoir. The soil acts as a reservoir for moisture. It fills up during rains or irrigations and empties due to plant water use (transpiration), or evaporation from the soil surface. Figure 2 shows the typical air/water/solid proportions of a specific soil (loam) at a specific moisture content (field capacity). Not all of the water is available for plant use. In Figure 2 the water content at field capacity (defined below) is 25% by

volume (not by weight), which means that a foot depth of absolutely dry typical loam soil would need 4" of water (25% of 12" depth) to reach field capacity.

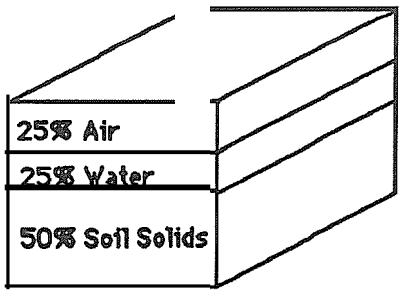


Figure 2. Typical water-air-solid proportions of a loam soil at field capacity.

2. Soil moisture tension (also called matric potential or matrix potential). This is a measure of the affinity of the soil matrix for water. Figure 3 shows that the suction or "matrix potential" in a capillary tube with a small diameter is greater than in one with a large diameter. As a soil dries out, the remaining water is held in smaller and smaller gaps. The water surface tension, which acts at the air-water-solid interface, exerts a tighter holding power on the water as the interface gets smaller. As this happens, it is increasingly difficult for the water to move into plant roots. Most agricultural plants begin to suffer some moisture stress (Le., reduced vegetative growth and reduced transpiration) when the soil moisture tension in the top quarter of the root zone is in the range of 0.5 - 1.5 bars.

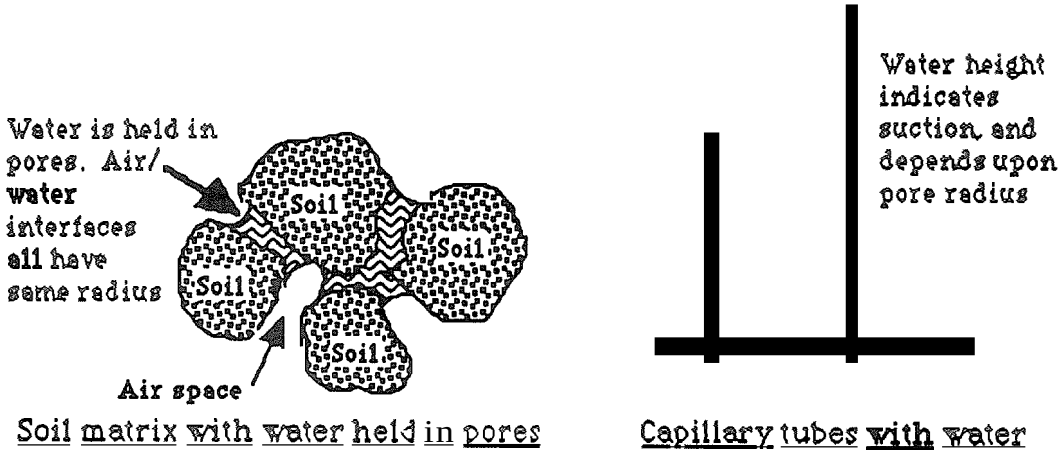


Figure 3. The matrix of a soil holds water by surface tension, just like capillary tubes.

The units of measurement could be "psi" but are more commonly "bars" or "atmospheres" or "kilopascals" (1 bar =1.01 atmosphere =100 kilopascals). The drier the soil, the greater will be the resistance to water extraction. In most soil physics books,

the "tension" is expressed as a negative number; the drier the soil, the more negative the value.

3. Saturation. The soil is filled with water, with virtually no air pockets. The saturated soil moisture tension is 0.0, except for a small saturated area in the soil called the "capillary fringe" which is directly above the water table. During a typical irrigation, the soil profile does not reach saturation except perhaps at the very surface. With surface irrigation, the top several inches of soil may be saturated, but at lower depths the pore spaces will contain significant quantities of air. An exception can occur with very heavy clay soils, which can become saturated during a long irrigation.

4. Field Capacity (FC). This is the upper limit of storable water in the soil once free drainage has occurred after an irrigation. This drainage may be complete in a few hours in a sandy soil, but can take several days or weeks in a heavier, clay soil. For laboratory determinations with disturbed soil samples, the soil moisture content at -0.3 bar is called the soil moisture content at Field Capacity. In the field, the soil moisture tension at Field Capacity is closerto -0.1 bar. Fora soil with actively growing plants, the soil will generally reach Field Capacity within a few hours on a sand or within one to two days on a clay loam soil. A soil with an actively growing plant will reach Field Capacity sooner than one without plants. With plants in the soil, water leaves the soil after an irrigation both from free drainage and from evapotranspiration.

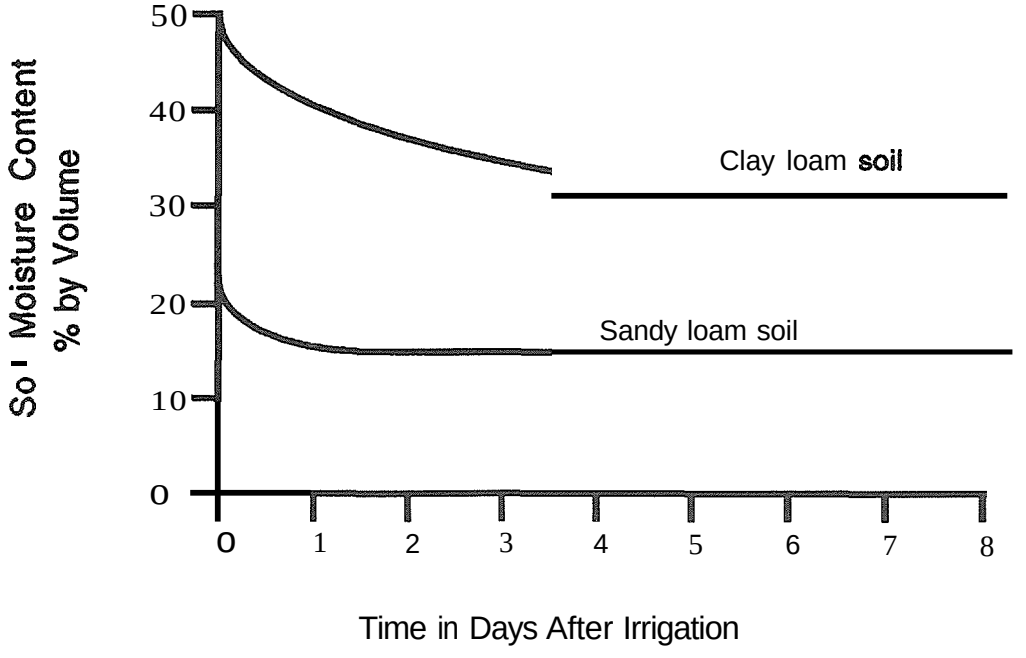


Figure 4. Water content in a soil versus time after a complete irrigation. All changes are due to free drainage, with no soil surface evaporation or plant transpiration.

5. Permanent Wilting Point (PWP). The moisture content in the soil at which plants begin to irreversibly wilt, or the practical lower limit of plant-usable water in the soil. The soil moisture tension is from -10 to -20 bars, depending upon the water use rates, plant type, root density, etc. It is not important to precisely know if the correct value is -20 bars or -10 bars because there is generally very little change in soil moisture between those two values. (see Figure 5). For medium to heavy textured soils, considerable water will remain in the soil at Permanent Wilting Point, but that water will be unavailable for plant use because it is held so tightly by the soil.

6. Available Water Holding Capacity (AWHC). The amount of water held between Field Capacity and Permanent Wilting Point. It is usually expressed as "inches/foot of soil" or as "total inches in the root zone". The "rules ofthumb" in Table 1 give a very rough idea of AWHC for various soil textures.The error for any soil could easily be 25%.

   General information of AWHC can be obtained from private soil science laboratories and from USDA/Soil Conservation Service soil surveys. Because soil textures and structures vary with depth, samples must be taken at least every foot of depth to determine the AWHC of the root zone. The example of Table 2 shows how the AWHC of a root zone might be estimated on the basis of soil texture. The root zone AWHC in a typical landscape or turf irrigation area can vary tremendously from point to point, because the soil is often a mix of cuts and fills.

Table 1. *Approximate AWHC of* soils *by texture.*

Soil Texture	Inches Avail. Water/Ft. of Soil
Coarse sand	.50
Fine sand	.75
Loamy sand	1.00
Sandy loam	1.25
Loam	1.50 - 1.8
Clay loam, silt loam	1.75 - 2.50
Clay	2.20

Table 2. *Sample calculation of root zone AWHC. Root zone depth =2.0 feet.*

Soil Texture	AWHC (in/ft)	Depth Zone	Zone Thickness (ft.)	Available Water (in.)
(Field Data)	(From Table 1)	(Field Data)	(Calculation)	(Calculation)
Loamy sand	1.00	A' - .5'	.5' - 0' = 0.5	0.5' x 1*"1ft* = .5"
Sandy loam	1.25	0.5' - 1.5'	1.5' - 0.5' = 1.0	1.0' x *1.25"/ft* = 1.25
Loam	1.60	1.5' - 2.0'	2.0' - 1.5' = 0.5	0.5' x *1.6"/ft* = .80 Root Zone AWHC =2.5"

If the root zone depth in the soil of Table 2 had been 1.0 foot, the AWHC would have been 1.125" of water (0.5" in the top 6" and 0.625" in the next 6" of soil). In the field, soil moisture values are not known to an accuracy of better than 0.1 inches.

7. Soil moisture characteristic curves. A characteristic curve is determined by laboratory analysis and shows the relationship between water content (expressed either as % by volume or % by mass) and matrix potential. These curves may be considered as graphs of soil water content versus the difficulty which plants have in obtaining that water. At a matrix potential of -15 bars, the difficulty is so extreme that plants "permanently wilt".

   A soil water characteristic curve is valid for only one point in a field and at one depth at that point, unless the soil is completely homogeneous. Because of the detailed and expensive laboratory work necessary to determine such information for a complete field, these curves are rarely available in production agriculture or turf or landscape work. Instead, the moisture contents at Field Capacity and Permanent Wilting Point may be known. The considerable detail provided by the complete curves is seldom needed for daily irrigation work, but can be helpful for detailed research.

   The soil water characteristic curves are a helpful teaching tool, even though they are not used on a daily basis in production agricultural or landscape irrigation management. Many basic facts regarding soil water management are related to the relationship between soil water content and tension.

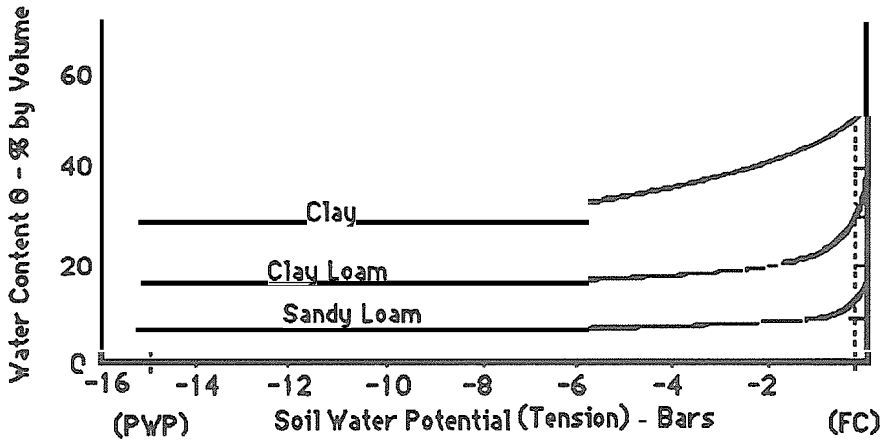


Figure 5. Soil *moisture characteristic curve.*

Figure 5 illustrates the following points:

a. The following numerical data can be obtained from two of the soils depicted:

	Clay	Sandy loam
Water content at Field Capacity (-0.1 bar)	47 % by vol.	13%
Water content at Perm. Wilting Point (-15 bars)	27%	6%

Available Water Holding Capacity [$\frac{\text{inches of water}}{\text{foot of soil}}$]

$\{\frac{\%\ \text{Water at FC}}{100} - \%\ \text{Water at PWP}\} \times \frac{12"\ \text{soil}}{\text{ft of soil}} =$	*2.4"/ft*	*.8"/ft*
Water content at 50% depletion (50% MAD)	37%	9.5%
Soil water potential at 50% MAD	-3.5 bars	-0.8 bars

b. A soil water potential (tension) of -2 bars occurs at a different moisture content for every soil.
c. The percentage of the AWHC which has been depleted at a given soil water potential (e.g., -2 bars) will be different for each soil.
d. A clay has a larger AWHC (Le., the water content between -15 bars and -0.1 bar) than a sandy loam.
e. The moisture held in the AWHC range of a sandy loam is almost all readily available to plants (Le., it is held at a tension close to zero). A clay holds more available inches of water than a sandy loam, but a larger percentage of it is held at higher (more negative) potentials.

8. Management Allowable Depletion (MAD). This is the percentage (%) of the AWHC which an irrigator will allow the plants to deplete before irrigating. The selection of a small MAD will mean more frequent irrigations of smaller amounts than if a large MAD is selected. Frequent irrigations (small MAD) generally result in more applied irrigation water for several reasons:
   a. Some irrigation methods, such as furrows, may be incapable of applying a small amount of water to a field with a high uniformity. Therefore, extra water must be applied just to cover the field, even though the soil may not be very dry.
   b. Wet soil and plant surfaces have more evaporation than dry soil surfaces; a small MAD keeps the surfaces moist.
   c. A wet soil root zone will provide less matrix and salinity stress to a plant, and therefore it will transpire more water than under large MAD scenarios.

Some situations which would cause a small MAD to be selected are:
   a. Micro-irrigation. Micro-irrigation can usually be irrigated on a daily basis as inexpensively as on a weekly basis. In addition, micro-irrigation may only wet a small percentage of the soil volume, so frequent irrigations are necessary to keep the small moisture reservoir full.
   b. Saline water. Damage from salts can be reduced by keeping the salty soil solution diluted with frequent irrigations. A small MAD should not be confused with salt leaching. MAD involves the concept of how dry a soil will get before it is refilled. Leaching is a different concept, of percolating water through and out of the root zone, and thereby reducing the total amount of salt in the root zone.
   c. Shallow root zone. A shallow root zone has a small available water holding capacity, and it would not be wise to deplete most of it; only a small reservoir would be left. That small reservoir could be easily depleted if an irrigation was a few days late. The idea of a shallow root zone is the same as that of the smaller root zone with some types of drip (micro) irrigation.

d. Uncertain water supply. If the water supply is unreliable, one cannot take the risk of drying out the soil and then not having water to replenish it at the last minute. Therefore, irrigators tend to keep the soil moisture reservoir "topped off' and full.

The following are applicable specifically for agriculture (as opposed to landscape):

e. Certain root plants. Onions and carrots need a moist upper soil for proper shaping of the harvestable portion of the plant.

f. Some plants are very sensitive to moisture stress at all stages of growth, such as strawberries.

g. Plants during certain moisture-sensitive stages of growth. An example would be peaches during the fruit sizing period. If the soil is allowed to dry out and then is irrigated, the subsequent rapid fruit growth maysplitthe peaches. Anothermoisture-sensitive growth stage is pollination for corn.

Some factors contribute to a large (high) MAD selection. These almost all fall within the realm of agriculture or for ornamental bushes. These factors include:

a. Non-solid set sprinkler irrigation systems. Examples include hand move sprinklers and hose-pull sprinklers. These systems are designed so that one piece of equipment (Le., one sprinkler) can cover as many acres as possible, thus reducing the capital cost. Therefore, the sprinklers may have a minimum irrigation frequency of 2 weeks or so, depending upon the number of sprinklers in the field and the number of positions which they must cover.

b. Furrows and border strips with either long runs or sandy soils. Under these conditions, it takes some minimum amount of water just to have the water reach the downstream end of the furrows or border strips. High MAD's are required in order to obtain high Distribution Uniformities (DU). There are some notable exceptions with furrows; ifwater is only applied on the wheel rows, orto every other furrow, or ifsurge flow is used, it may be possible to obtain a high Distribution Uniformity (DU) with a low MAD. Iffurrows with long runs or sandy soils are irrigated very frequently (which may be necessary to avoid plant stress), the irrigation efficiency will generally be low because the DU will be poor.

c. Some plants need stress at specific growth stages. Examples include almond trees prior to harvest (the shakers will strip the bark from the trunks ifthe bark is too moist) and citrus immediately prior to bud formation (dry soil conditions will favor a heavy bloom). Some varieties of cotton need to be stressed on heavy soil in order to develop a sufficient number of cotton bolls, rather than only developing vegetative growth. Some trees and vines need dry trunks (little sap) to avoid frost damage.

9. Soil Moisture

Depletion (SMD) - The inches of moisture depletion at any particular time (Le., how many inches of water are required to fill the root zone up to field capacity)

$$SMD = \frac{MAD \times AWHC}{100}$$

For example, assume that for the soil in Table 2 a MAD of 40% was selected. The SMD at the time of irrigation would be:

$$SMD = \frac{40\% \times 2.5"}{100} = 1.0" \text{ of water}$$

Basic Truths of Soil/Plant Water

Theory and definitions can overwhelm a person trying to learn about soil/plant/water relationships. Therefore, a list of basic ideas which are meaningful to actual irrigation management are provided here, including summaries of some points made previously.

1. The amount of water held at Field Capacity, the amount held at Permanent Wilting Point, and the difference between the two (AWHC) are different for each (i) soil and (ii) horizon (depth) within a soil at a particular point in a field. In other words, a soil moisture characteristic curve is only valid for a single depth at a single point in one field. Nevertheless, there is sufficient horizontal uniformity in many field soils that only one or two sampling holes may be needed to adequately represent the entire field.

2. When water is applied during an irrigation, each upper increment of depth of soil must reach Field Capacity before water will move down to the next depth. This is illustrated in Figure 6.

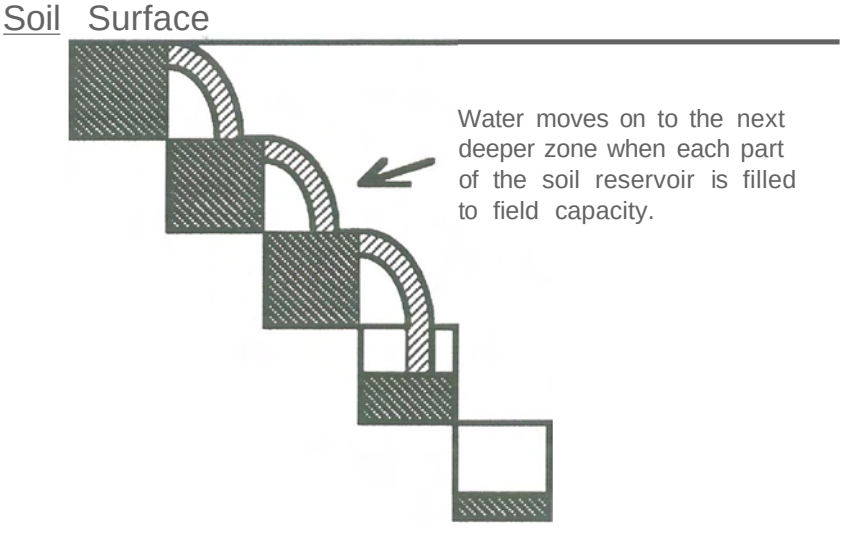


Figure 6. The water holding capacity of soil can be thought of as a series of buckets. In this figure, each bucket represents one foot of a five-foot root zone during an irrigation. If irrigation stopped now, the last foot would not receive any water.

3. There is a distinct wetting front during an irrigation as the water moves into the soil. Saturation will only occur at the very surface of the soil during an irrigation, and then only generally for very high application rates and heavy soils.

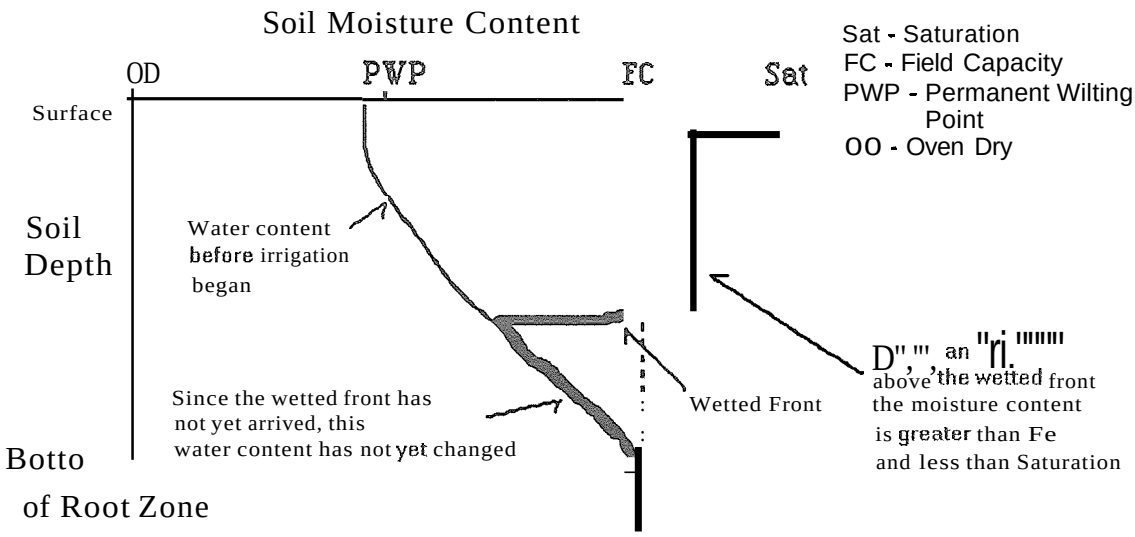


Figure 7. Moisture content in the soil at the time when the wetted front has reached half way down the root zone.

4. Evaporation from a soil surface during a regular irrigation season does not usually affect the soil moisture content beyond 3-4" under the soil surface during the growing season. This is partly because of the rapid plant water uptake at the shallow depths, partly because as a soil dries out the resistance to soil water movement increases, and partly because most surface and sprinkler irrigations are infrequent. However, if a bare ground is kept continually wet by frequent irrigation, the evaporation rate from the wet soil will be almost the same as the evaporation rate from a pool of water.

5. Plants can use moisture if the soil moisture content is above Field Capacity. For example, plants use water even while they are being irrigated. Field Capacity is an upper limit of soil moisture storage, not of plant water use.

6. Plants do not have "pumps". Water moves from the soil into roots, up the plant, and into the air in response to tension gradients (differences of water energy between two points) and the movement is inversely proportional to the resistance of movement. All water movement is caused by a difference in energy between two connected points. Water moves from relatively high energy to one of less energy.

7. The "root zone" depth of a plant is not fixed except for permanent, mature plants such as trees, turf, and shrubs. Even then, rooting densities can vary with time. For example, lower root development increases if water stress occurs at the top of the root zone. Table 3 provides some general information regarding effective rooting depths. Although roots may go deeper than indicated in Table 3, optimum crop growth can usually be achieved by managing the soil depths shown.

Table 3. Typical effective rooting depths of plants. Some plants (e.g., almonds and safflower) will extract water from much greater depths when irrigation is ceased. Rooting depths will be greater on sandy soils than on clay soils.

Plant	Effective rooting depth (ft)
Alfalfa	4-6
Avocado	2-3
Banana	2-3
Barley	3-5
Beans	1-3
Citrus	2-5
Corn	3-5
Cotton	4-6
Decidu. Orchard	4-6
Grains, small	3-4
Grapes	3-6
Grass	3-4
Lettuce	1-2
Melons	2-3
Potatoes	2-3
Safflower	5-6
Sorghum	3-5
Strawberries	1-2
Sugarbeet	3-5
Sugarcane	4-6
Tomatoes	3-5
Turf grass	.5-1.5

8. Roots do not go "hunting" for water. Roots grow well in an environment which supports growth - Le., warm, moist, with nutrients, and good aeration. Roots will not travel several feet through absolutely dry soil to search for a wet area.

9. Soil water extraction patterns by plants vary with time, the plant, and the soil. In many cases the extraction pattern can be approximated by the 40-30-20-10 rule, as illustrated in Figure 8.

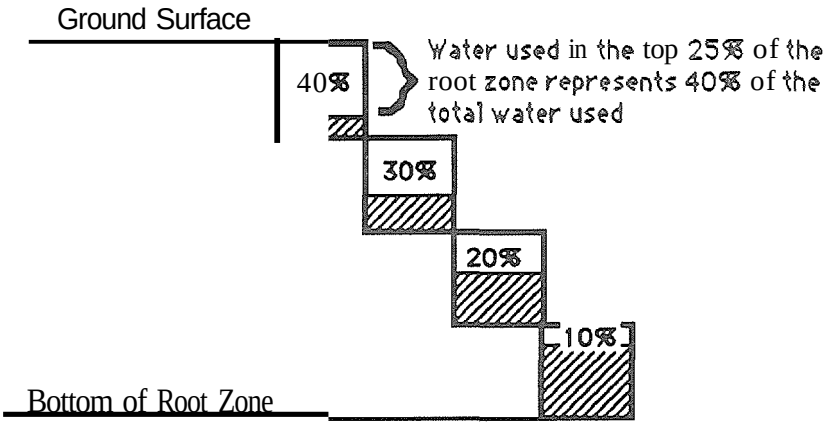


Figure 8. Soil water extraction pattern from the soil for many irrigated plants. (Adapted from Ames Co., 1967)

10. Water can be stored during the winter by pre-irrigation (irrigating bare ground before a plant is planted) or winter rains. That water will be available for use in the summer by plants. In heavier soils such as silty clay loams, it is common to store as much as 8 - 12" for cotton plants by using winter pre-irrigation in arid areas. Note the earlier discussion under "Field Capacity" which describes some loss due to slow drainage. Slow, delayed drainage can be compensated for by not allowing the wetted front reach the bottom of the root zone during a pre-irrigation. The slow drainage will gradually fill the bottom of the root zone in the weeks after the irrigation.

11. The ease of plant water uptake from the soil is affected by the tension at which that water is held in the soil, although farmers frequently use rules of thumb such as "irrigate cotton at a 50% depletion (Le., 50% MAD)". A MAD of 50% in a sandy loam soil may occur at -0.8 bars, but a MAD of 50% in a clay loam soil may occur at -2.0 bars.

12. Given two soils (a clay loam and a sandy loam) with the same root zone depth and irrigated at the same percentage depletion (MAD), the inches of water needed to refill the root zone on the clay loam will be higher than for the sandy loam.

13. Plant roots need oxygen to function in normal conditions. A soil with very readily available moisture (e.g., above Field Capacity) may not be conducive to good growth if the oxygen diffusion rate into the soil is limited. On heavy soils, one must pay particular attention to the balance between water and oxygen availability. With sandy soils, it is not as much of a problem. The availability of oxygen at greater depths on coarse soils is one reason root zones tend to be deeper on sandy soils than on clay soils.

14. Plant ET (evapotranspiration) in a well-watered condition is determined by the climatic demand (Le., the amount of heat available plus the dryness of the air and the existence of wind to transport the water vapor away from the plants). In general, it does not depend

upon the soil type or the method of irrigation. A popular myth used to be that trees irrigated by a micro-irrigation system (which may only wet 50% of the soil volume, for example) will transpire 50% as much water as trees under furrow or sprinkler irrigation. Actually, the ET rate under micro-irrigation is typically about 15% higherthan for furrow or border strip because the soil is kept more moist. The moisture condition will increase soil evaporation and planttranspiration rates. The ET differences are due to differences in availability of water in i) the soil root zone, and ii) at the soil surface.

15. Nutrients move into plant roots via the soil water solution. The majority of two major plant nutrients, potassium and phosphorus, may be found at the soil surface. Therefore, a plant with a medium MAD may not sufferwaterstress but may experience nutrient deficiencies, while a small (low) MAD would prevent both water and nutrient stresses on most soils.

16. Osmotic forces due to salt in the soil water create the same effect as a dry soil in making it more difficult for water to move into the plant roots. A soil water concentration of one mmho/cm (dS/m) is equivalent to approximately -0.36 bars of soil moisture tension. Salinity and dryness effects are additive; a soil water salt concentration of 3.0 mmho/cm (dS/m), plus a soil moisture tension of -2.0 bars results in:

    [3 mmho/cm] x [-0.36 bar/(mmho/cm)] + [-2.0 bar]

    = -3.1 bars tension seen by the plant

Soil Moisture Assessment

There are many techniques used to evaluate soil moisture contents. There can be significant differences between what each device measures. The following is a brief summary:

1. Soil Moisture "by feel". Soil samples are obtained with an auger or soil probe. The soil is then rolled, squashed, squeezed, and otherwise manipulated with hands and fingers, and observations are made as to whether or not itforms a ball, leaves a stain, "ribbons", etc. With the help ofa chart(e.g., Table4), an estimate ofthe soil moisture deficit (inches ofAWHC depletion per foot ofsoil) can be made. This works best on medium and heavy textured soils where there is a substantial difference in the way soil feels and looks at various moisture contents. It is the most extensively used field method of determining soil moisture availability, and it gives reasonable results with practice.

Table 4. Soil moisture and appearance relationship chart. (From Merriam and Keller, 1978).

SMD, inches of water per foot of soil	SOIL TEXTURE CLASSIFICATION				SMD, inches of water per foot of soil
	COARSE (loamy sand)	LIGHT (sandy loam)	MEDIUM (loam)	FINE (clay loam)	
	Field Capacity	Field Capacity	Field Capacity	Field Capacity	
0.0	Leaves wet outline on hand when squeezed.	Appears very dark; leaves wet outline on hand; makes a short ribbon.	Appears very dark; leaves wet ourline on hand; will ribbon out about one inch.	Appears very dark; leaves slight moisture on hand when squeezed; will ribbon out about 2 inches.	0.0
0.2	Appears moist; makes a weak ball.	Quite dark color;			0.2
0.4			Dark color; forms a plastic ball; slicks when rubbed.	Dark color; will slick and ribbon easily.	0.4
	Appears slightly moist; sticks together slightly.				
0.6		Fairly dark color, makes a good ball.	Quite dark, forms a hard ball.	Quite dark, will make a thick ribbon. May slick when rubbed.	0.6
	Very dry, loose; flows through fingers (PWP).				
0.8		Slightly dark color; makes weak ball.	Fairly dark, forms a good ball.	Fairly dark, forms a good ball.	0.8
1.0		Lightly colored by moisture, will not ball.			1.0
1.2		Very slight color due to moisture (PWP).	Slightly dark, forms a weak ball.	Will ball, small clods will flatted out rather than crumble.	1.2
1.4			Lightly colored; small clods crumble fairly easily.	Slightly dark, clods crumble.	1.4
1.6					
1.8			Slight color due to moisture, small clods are hard (PWP).		1.8
2.0					2.0
2.2				Some darkness due to unavailable moisture; clods are hard, cracked.	2.2

2. Oven-drying. A soil sample is taken in the field and then transported into the lab where it is dried with an oven. The before and after weights are compared to determine the moisture content by mass. This is very time consuming, and is rarely done except to develop calibration curves for the other methods. This measurement will tell how much water was in the soil, but not how much of that water was available for plant use.

Water contents by mass (weight) must be converted to water contents by volume in order to determine the inches of water in a soil. Many soil reports do not make this very important conversion. If moisture content values are expressed as "% water", one does not know if the values are by volume or by mass (weight).

$$\% \text{ Water by Mass} = \frac{\text{Wet weight of soil} - \text{Oven Dry weight}}{\text{Oven Dry weight}} \times 100$$

$$\% \text{ Water by Volume} = \frac{[\% \text{ Water by Mass}] \times [\text{Soil Bulk Density}]}{\text{Specific Weight of water}}$$

where the Soil Bulk Density units are $\frac{\text{gm}}{\text{cubic centimeters}}$

and the Specific Weight of water $= 1.0\ gm/cm^3$

For example, consider the following data:

Weight of wet soil plus can =	750 gm.
Weight of oven dried soil plus can =	630 gm.
Weight of can, only =	150 gm.
Soil bulk density =	$1.65\ gm/cm^3$

Determine the % moisture by mass and by volume

Oven dry weight = Weight of oven dried soil plus can - Weight of can
= 630 gm - 150 gm
= 480 gm

Wet soil weight = Weight of wet soil plus can - Weight of can
= 750 gm - 150 gm
= 600 gm

$$\% \text{ Water by Mass} = \frac{\text{Wet weight of soil} - \text{Oven Dry weight}}{\text{Oven Dry weight}} \times 100$$

$$= \frac{600 \text{ gm} - 480 \text{ gm}}{480 \text{ gm}} \times 100$$

$$= 25\%$$

$$\% \text{ Water by Volume} = \frac{[\% \text{ Water by Mass}] \times [\text{Soil Bulk Density}]}{\text{Specific Weight of water}}$$

$$= \frac{25\% \times 1.65\ gm/cm^3}{1.00\ gm/cm^3}$$

$$= 41\%$$

3. Tensiometer - A vacuum gauge is attached to an air-tight tube full of water with a porous ceramic tip on the bottom end. The ceramic tip and tube are buried in the soil at a point of interest. Water can move into and out of the tensiometer through the ceramic tip. As the soil dries out, water moves out, registering a stronger vacuum on the gauge. Therefore, this measures soil moisture tension directly. The scale of 0-100 usually represents 0-1 bar of tension, and the tensiometer will usually "break suction" and have to be reprimed if the tension gets greater than about -.8 bars. The breaking of suction is caused by air entering through the small ceramic pores at the high vacuum.

4. Gypsum blocks, Watermark™, and others. - A block of some type of material with very fine pores. The block is buried with two wires extending to the soil surface. Each device has a slightly different method of monitoring the moisture content of the block. In the case of a gypsum block, the block moisture content can be measured indirectly by sending a current through the wires and using an instrument which reads the electrical resistance in the salt block, and makes a direct conversion to some type of moisture. For a gypsum block, the moister the block (because of a moist soil), the less the resistance to electrical flow between two probes in the small block. Most of these devices are quite reliable and simple to install and use, but the meter readouts generally only indicate a range of soil moisture from some undefined number such as zero, to another undefined number such as 100.

   The moisture content in the blocks is not the same as the moisture content in the soil, because the matrix of the blocks is not the same as the soil matrix. However, water movement between the block and the soil will cease when the matrix potentials of the two are identical. Therefore, these devices can be calibrated in the laboratory, or by the manufacturer, as "meter reading" vs. "soil matrix potential", which should be valid for all soil types. The relationship between "meter reading" vs. "soil moisture content" will vary for each soil, just as with a tensiometer (see earlier section on soil moisture characteristic cUNes). These devices typically work well from about .5 to 10 bars, but the optimum range will depend upon the block matrix.

5. Neutron probe - This device uses a radioactive source of neutrons, which is lowered vertically into a tube which was previously placed in the soil. The neutrons are emitted into the soil, hit hydrogen atoms (found in water), and then bounce back to the probe with a lower velocity. A count of how many thermalized (slow) neutrons return indicates the moisture content (not tension) in the soil. This method is accurate with proper calibration and has the advantages of being able to make measurements at multiple depths very rapidly, and is "non-destructive" in the sense that moisture content can be measured at the same point again the next day or week. With the new data collection modules, the stored readings can be directly fed into a computer.

6. Infrared thermometer - A device which indirectly measures plant temperatures. By pointing an infrared thermometer at a plant canopy, the temperature can be measured without touching the plant. By holding the device at the proper angle and distance, the sample size (Le., number of plants being measured) can be varied. When this

information is combined with other data such as the relative humidity and temperature of the air, an estimate can be made as to the degree of plant stress. Most newer models automatically read all the factors and compute a stress index. For some plants such as corn, grass, and cotton, there are fairly good relationships between readings and plant stress. For other crops, especially trees and vines, it is more difficult to get proper measurements and stress/yield/reading relationships are not as clear.

7. Leaf bomb. The leaf bomb exerts pressure on a sample of one leaf, until sap begins to exit the leaf stem. The pressure at which sap begins to exit the leaf is equivalent to the "leafwater potentiaL" Although this accuracy can be quite accurate, it is limited by the small sample size, the time necessary to conduct the test, and restrictions on the hours of day during which useful measurements can be made.

8. Frequency domain reflectometry (FOR) probe. This device has a probe which is lowered into a PVC tube which is permanently placed in the soil, just as is done with the neutron probe. However, the FOR probe is non-nuclear, and therefore has licensing and safety advantages. It measures changes in the dielectric constant of materials. Most soil particles have dielectric constants in the range of 2.0 - 4.0, whereas water has a dielectric constant of 78. Therefore, changes in dielectric constant of a soil sample reflect the soil moisture change. It has a sample volume of about 10 inches of diameter around the probe.

There are other types of devices and methods, but most measure one of the following:

1. Soil moisture tension. This defines the matrix stress at a single point and depth in the field. It indicates when to irrigate, but not how much to apply. Examples are the tensiometer and gypsum blocks.
2. Soil moisture content. Examples include the neutron probe and the frequency domain reflectometry (FOR) probe. A single measurement from these devices does not indicate the soil moisture depletion, but only the total amount of water in the soil (above oven dry).
3. Soil moisture depletion. The only method which provides this information directly is the "feel" method. By using devices which measure the soil moisture content, the depletion can be calculated if the moisture content at field capacity is known at that point. An instrument measurement must be made of the soil at the time it is at field capacity (1-2 days after an irrigation).
4. Plant water tension. Examples include the leaf bomb and the infrared thermometer (indirectly). This information is similar to soil moisture tension in that it indicates whether or not the plant is under stress at the moment.

It is important to realize that in general, with the amount of time available in commercial agriculture measurements, these devices can neither quantify nor identify the existence of deep percolation. For most soils, one day after an irrigation a root zone which has received exactly the amount needed to rise to Field Capacity will be at the same moisture content (Le., Field Capacity) as another root zone which has received twice as much as needed.

After one or more devices have been selected to provide irrigation management information, the user must address the following points:

1. The number of sites needed throughout a field. In commercial irrigation scheduling, there are typically 1 or 2 sites per field. Although more sites are desirable, they cost too much to install and measure. If there is more than one soil type within a field, measurements are almost always made in the sandier section, since stress will occur there first.
2. The location of sites relative to field boundaries, and direction of irrigation. In general, sites are at least 100 feet away from field boundaries in order to provide a buffer between unusual edge effects. It is usually best to locate the measurements in areas which will receive the least amount of water (e.g., near the tail ends of furrows).
3. The following are specific to soil measurements:
   a. The number of samples needed per site. In general, there will only be one set of samples per site.
   b. The depth of various samples in a set of site measurements. As a minimum, for soil tension measurements at least two are needed - one at about a 1 foot depth for most crops and the other at the bottom of the root zone. The top one indicates the status of the moisture in the very active root zone area, and the bottom will detect any under-irrigation. For measurements of soil moisture content, it is typical to take readings every foot, and sometimes at each half foot of depth.
   c. The sample location relative to the plant. For trees and vines, the sample is generally in the plant row, about 2/3 of the distance between the trunk and the outer edge of the plant canopy. For row crops, the sample is generally in the middle of the plants on the top of the furrow bed.
4. The following are specific to plant measurements:
   a. Tart of the plant to be measured. For each device, manufacturers' instructions should be followed.
   b. The time of day to take measurements. This presents a disadvantage regarding plant measurements. Many of these require measurements in the middle of day, providing only a few hours for effective data gathering.

A neutron probe or other soil moisture monitoring measurement in field and row crops is generally representative for the surrounding soil at that depth. Therefore, one sample site (taking samples at many depths) can be used to estimate the SMD of the surrounding soil.

This is not true in an orchard or vineyard which is irrigated with micro-irrigation. Micro-irrigation only wets a portion of the soil, so measurements taken at different distances from the emitters will give very different soil moisture values. The soil moisture measurements in micro-irrigation cannot be translated into "soil moisture deficits for the field". Instead, the measurements taken over time should be used to indicate trends. If the soil begins to dry out near the emitters, it is an indication that the system should be operated for more hours per week.

In general, almost any device or method can be a valuable tool if it is recognized as a management aid, and not a final answer.

Transpiration

A primary objective of irrigation is to provide water which can be used beneficially by enhancing plant growth. Transpiration is the water which passes from the soil into agricultural plant roots, through the plants, and out the leaves into the air. The majority of water movement through the soil and plant is as a liquid. Once it reaches the leaves, it is converted to water vapor in the stomata (large pores on the underside of leaves). In most irrigated situations, transpiration is the main component of "beneficially used" water, but it may not be the main destination of applied water.

Water applied through rainfall or irrigation can also be "consumed" by:

a. Deep percolation
b. Wind drift
c. Soil or plant surface evaporation
d. Droplet evaporation in the air
e. Runoff

The drier and hotter the air, the higher the transpiration rate. The drier the soil, the slower the transpiration rate, because the water is held tighter in the soil (Le., more negative matric and osmotic potentials) as it dries out.

A specific plant variety in a given location has a genetic potential to transpire a certain quantity of water during the growing season. The transpiration rate during any single day depends upon the plant growth stage, weather conditions, the availability of soil water, and general plant health. Figures 9 and 10 illustrate how transpiration will vary with time for an annual plant such as wheat.

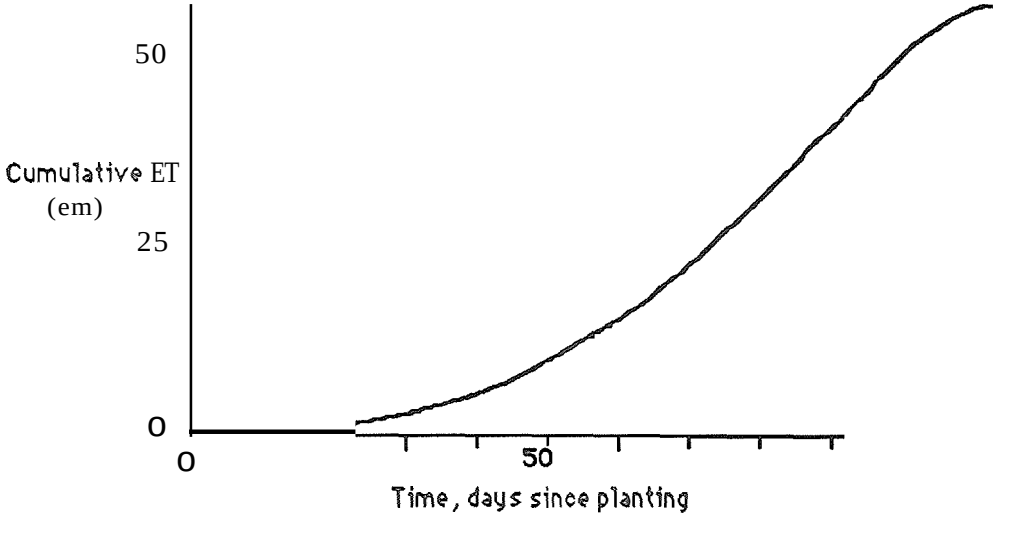


Figure 9. Cumulative Seasonal Transpiration vs. Time.
Generalized curve for annual plant (e.g., Wheat). Non-stressed condition.

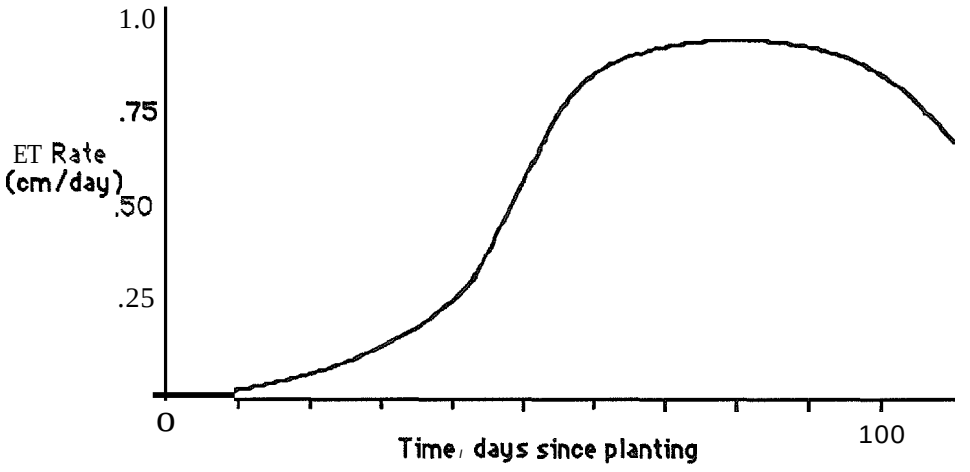


Figure 10. Daily Transpiration rate vs. *Time. Generalized curve for an annual plant (e.g., Wheat). Non-stressed condition.*

Evaporation

Evaporation is water which is converted from liquid to vapor, and which does not pass through the plant. Evaporation may occur from a wet soil or wet plant surface. Figure 11 illustrates the rate of evaporation from an initially wet bare soil surface with time. When plants are young, there can be considerable evaporation losses in this form. After plants achieve 70-80% canopy cover, the soil evaporation losses will only increase the daily Evapotranspiration (soil surface evaporation plus plant transpiration) by 10-25%. This 10-25% increase only occurs immediately after an irrigation, as shown in Figure 11.

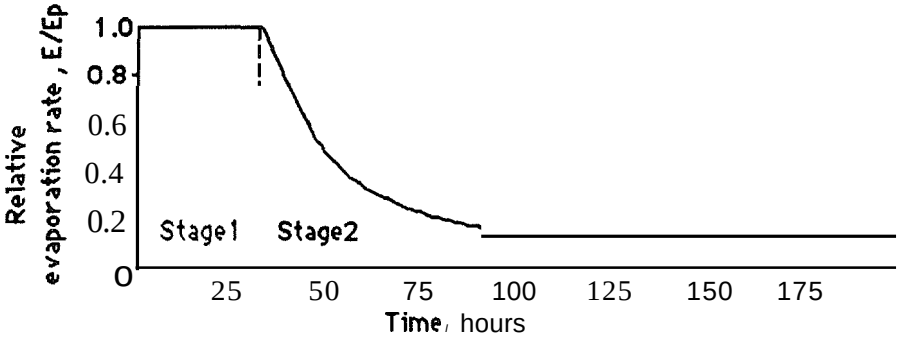


Figure 11. *Evaporation from* a *bare soil surface which* was *initially wet. (from Hanks and Ashcroft, 1980)*

With drip irrigation on trees such as almonds, walnuts, and peaches, the soil evaporation losses from a wet soil surface are approximately the same as for surface or sprinkler irrigation. With drip a small percentage of the soil surface is wet all of the time, resulting in high evaporation rates from that area. With surface and sprinkler, a large percentage of the soil surface is wet, but for only a small amount of time. Irrigation systems with micro-sprayers, haVing a larger wetted soil area than drip systems, may have considerably more soil evaporation losses than surface irrigation systems. By the same token, however, negative effects of the higher evaporation with frequent irrigations can be offset by less stress on plants.

Soil and plant surface evaporation losses can become extremely high (up to 100% of the applied water) in arid regions if sprinklers are operated in frequent, short sets. A common example is center pivots and linear moves which are operated at 100% speed in arid zones. At 100% speed the water applications are so small per pass that only the plant canopy and soil surface are wet, and almost all of that surface water is lost to evaporation. By slowing the pivots and linear moves, the soil and plant surface evaporation loss may be less than 10% of the applied water. The 10% does not include spray and drift losses which occur before the water hits the plant.

More work needs to be done to determine how much of the water which evaporates from wet plant foliage is beneficial and how much is wasted. Some research indicates that this evaporation reduces the transpiration requirement, yet some field experience shows that little or none of this water is beneficial. Figure 12 shows the relationship which may exist between beneficial and non-beneficial evaporation in situations of small and frequent sprinkler applications.

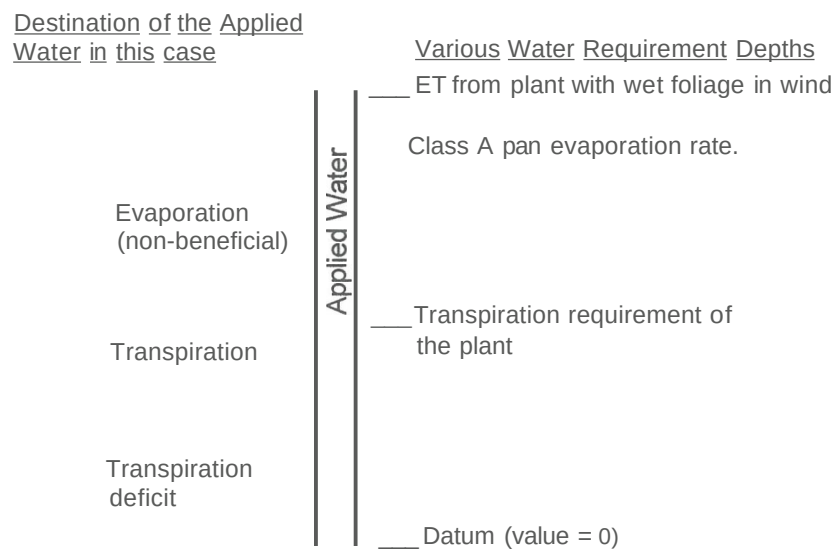


Figure 12. Domination of non-beneficial evaporation with light sprinkler applications.

Evapotranspiration (ET)

Evapotranspiration (ET) is the sum of plant Transpiration (T) and Evaporation (E) from wet plant and soil surfaces. As commonly defined, it does not include other components of evaporation such as evaporation of spray between a sprinkler and the ground or losses of small water droplets due to wind drift (as can occur with some micro-irrigation and sprinkler systems). In good computerized irrigation scheduling programs, the evaporation is usually calculated separately from the transpiration.

Much of the "plant water use" data which is available to designers and farmers is really "ET" rather than "transpiration" only. Table 5 is an example of such a table which is published for one specific area. The ET values are generally estimated and then field verified with soil moisture depletion measurement for "average" irrigation conditions.

More frequent irrigations would result in higher ET rates. Crop varieties also have different season lengths (one of the easiest ways to conserve water in some areas is to switch to short season crops, if applicable). The climate can also vary within a region (e.g., from one side of a valley to the other), so these are approximate numbers. Furthermore, there will be variations between years. In some areas such as the San Joaquin Valley of California, monthly values summer ET rarely vary by more than plus or minus 10%.

Table 5. Example table of ET values for a specific area and year and cultural practices. Similar to the southern San Joaquin Valley of California.

Month	Evapotranspiration Rate, inches/month								
	(ETp) ETr, alfalfa	(ETa) ETr, grass	Alfalfa Hay	Cotton	Citrus	Decid. orch. w/o cover crop	Decid. arch. w/ cover crop	Grape Vines	Small Grains
January	0.88	0.69	0.73		0.58		0.68		0.41
February	2.41	1.97	1.99		1.52		1.98		1.99
March	3.75	3.13	3.11		2.32	1.49	3.33	0.05	3.92
April	6.19	5.24	5.11	0.48	3.75	3.63	5.89	1.16	6.37
May	7.98	6.78	6.71	2.06	4.85	5.58	8.10	4.13	6.24
June	9.03	7.65	7.32	6.68	5.06	6.83	9.08	6.00	0.63
July	9.32	7.92	7.80	10.03	5.27	7.59	9.58	6.72	
August	8.44	7.14	6.92	8.76	4.73	6.85	8.41	5.96	
Sept.	6.03	5.08	5.16	4.47	3.57	4.87	5.89	3.30	
October	4.55	3.75	3.63	0.77	2.69	3.02	3.90	1.22	
November	1.92	1.52	1.61		1.18	0.07	1.58	0.14	
December	0.71	0.55	0.60		0.38		0.50		0.09
TOTAL	61.2	51.4	50.7	33.3	35.9	40.8	58.9	28.7	19.8

Applied Water

Applied water is the amount of irrigation water actually delivered to a field. Applied water is not the same as "plant water requirement". To have maximum yields, the applied water (plus effective rain and soil moisture storage) must always be greaterthan the optimum ET requirements, since irrigation losses such as non-collected runoff, spray and wind drift, and deep percolation are inevitable for all irrigation methods.

An "efficient" irrigation system may result in higher plant transpiration rates than an "inefficient" irrigation system because there will be less dry spots on the field. Irrigation system inefficiencies are caused by poor distribution uniformities of application, irrecoverable losses such as evaporation or wind drift, excess applications which cause deep percolation below the potential root zone, and non-collection of runoff water. The crop yield per unit of irrigation water transpired is higher under good management than under poor management.

Poor design or management of an irrigation system may result in a farmer applying more water than with a better system or management but will rarely result in higher transpiration. Transpiration under a very good irrigation system is generally higher than under a poor irrigation system. High transpiration rates are not bad if they improve yield; a main purpose of irrigation is to increase transpiration and yields.

It is generally understood that plant roots do not know if water came from an emitter, gated pipe, or sprinkler. For the same soil moisture content and frequency of irrigation, the transpiration rate for a plant under all methods should be the same. However, various irrigation methods, soil types, field configurations, and other factors in fact result in different soil moisture contents and irrigation frequencies.

Plant Water Use and Physiology

Plants use tremendous quantities of water. Only about 2% of the transpired water in most field plants actually stays in the plant. The rest simply passes through the plant.

This large quantity of water does not appear to be necessary for nutrient transport inside the plant itself. However, when a plant is stressed for water, the guard cells at the entrances of the leaf stomata close as a protective measure to reduce water loss. Vegetative growth also declines at that point because the flow of CO_2 gas into the stomata is reduced, and CO_2 gas is necessary for plant photosynthesis.

Every plant has a different stomatal response to water stress. The plant response to stress will also depend upon the climate, extent of the root zone, and general health of the plant. However, researchers have been able to define certain basic relationships between yield and water which can assist irrigation designers and managers in making decisions.

ET Rates and Water Potential

At high soil moisture levels, the water is held by the soil matrix rather loosely, allowing easy movement of water into the plant roots. Coarse textured (sandy) soils may have small Available Water Holding Capacities, but most of the available water in them is held more loosely (Le., at a less negative tension) than in heavy textured (clay) soils. Figure 13 shows that seventy to eighty percent of the available water in a coarse textured soil may be depleted before the plant begins to reduce transpiration; transpiration reduction may begin when only thirty percent of the available water is depleted from a clay. Nevertheless, the thirty percent of water from the clay will generally be more inches of water than the seventy percent from the sand.

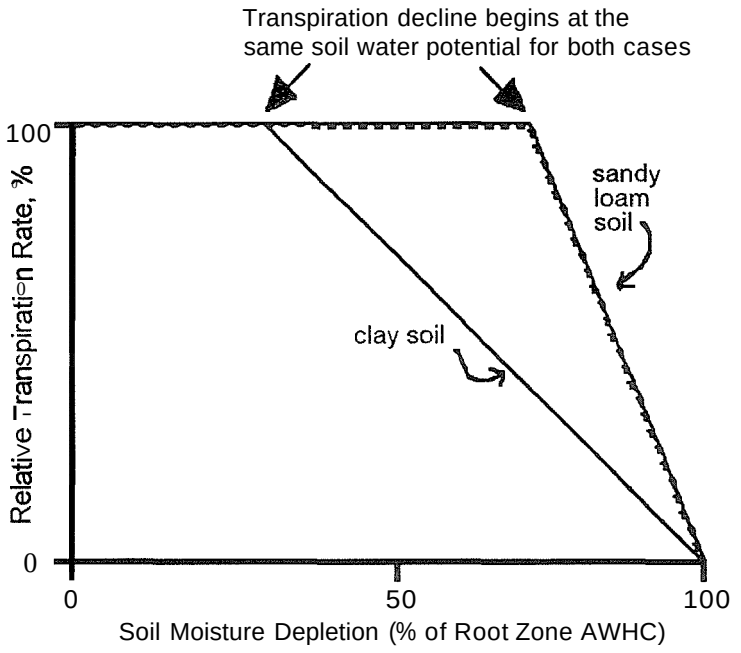


Figure 13. *Relative transpiration rates for two* soils *under varying soil moisture depletions.*

For some plants it may be desirable to stress the plant and reduce transpiration (and stop vegetative growth). Examples are:

1. Stressing citrus to induce blooming (followed by a no-stress period once blooming has begun).
2. Drying the root zone on cotton priorto harvest for leaf defoliation and maturation of bolls.
3. Stressing processing tomatoes prior to harvest to increase the percentage of soluble solids content.
4. Stressing wine grapes prior to harvest to increase the sugar percentage of the berries.
5. Under-irrigating turf grass by approximately 20% does not produce noticeable stress symptoms, but reduces the amount of grass clippings and numbers of cuttings needed.
6. Many ornamental bushes may easily be stressed 20-30% without showing detracting symptoms.

Crop Yield vs. Various Measure of Water

Figures 14 and 15 illustrate relationships between total annual crop Transpiration and Yield. They indicate that yields decline from a potential (Yp) if a crop is stressed at a constant rate throughout the growing season. For each crop, fertility level, and climate there is a different value for Yp.

Figures 14 and 15 indicate what happens if plants are stressed (Le., ifwater is not available for ET). This is different from the concept that yield will increase if more water is available. A plant cannot be forced to transpire more water than its potential.

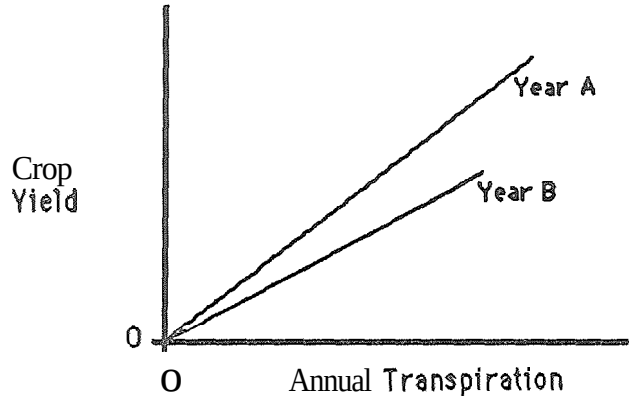


Figure 14. *Yield* vs. *Transpiration for* a *vegetative crop such* as *alfalfa or sugar6eets.*

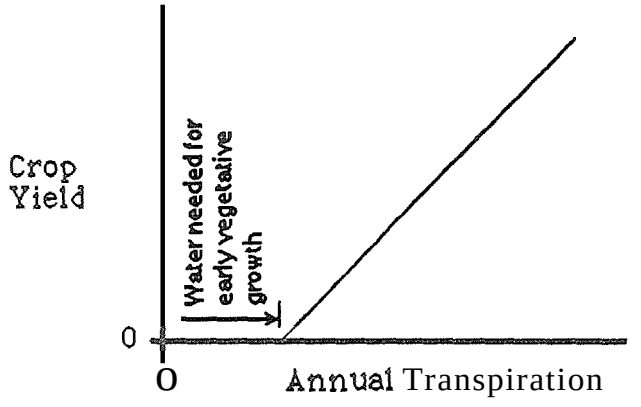


Figure 15. *Yield* vs. *Transpiration for* a *seed or fruit crop.*

The exact values of inches of annual transpiration and tons of crop yield will vary from year to year. Therefore, Figures 14 and 15 are not always transferrable from one location to another. By expressing yield and water as relative values, the functions can be made transferrable between different locations. Figure 16 contains the same type of information as Figure 14, but Figure 16 is transferrable whereas Figure 14 is not.

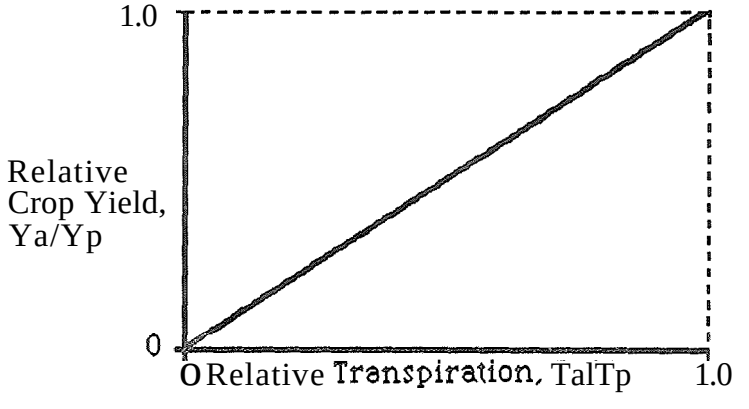


Figure 16. *Relative Yield* vs. *Relative Transpiration for* a *vegetative crop. "Ta"* is *actual Transpiration; "Tp"* is *potential Transpiration.*

Figures 14-16 indicate relationships which occur due to plant stress from a water shortage. In a field, water may be supplied from either rain, a high water table, or irrigation. A plant receives either too little, too much, or just enough water. Figure 17 indicates some possible relationships between Applied Irrigation Water and Yield for a single point in a field. Since a field is composed of many single points, each receiving a different amount of water, the total yield for the field will be the sum of all the points. In order to maximize yield, it is essential that water and fertility i) be applied uniformly, and ii) be applied in the correct amounts.

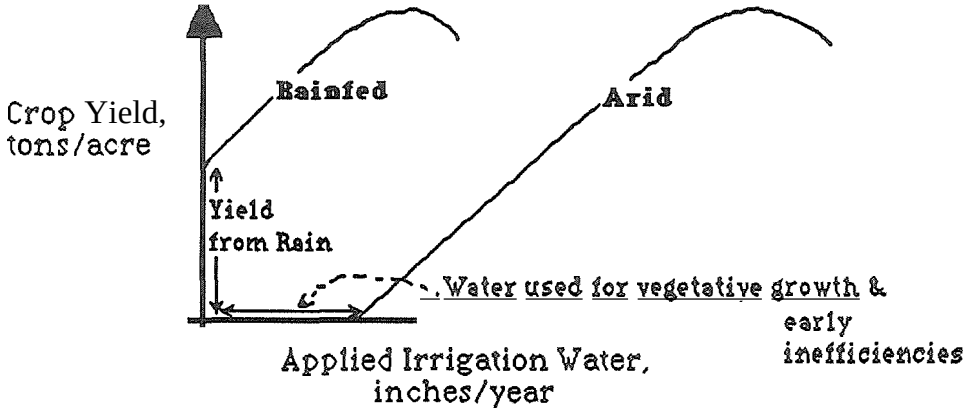


Figure 17. *Yield* vs. *Applied Irrigation Water relationship for a single point in a field.*

Some key points to observe from these curves are:

1. The Yield vs. Applied Irrigation Water curves are not transferrable between farms or regions because of differences in rainfall, drainage conditions, fertility, and irrigation efficiencies. Yield vs. Transpiration are more transferrable, and Relative Yield vs. Relative Transpiration are the most transferrable.
2. Yields decline from excessive applications of water. Declines are due to diseases, poor aeration, cooler soil temperatures, and fertilizer leaching. Unfortunately, when irrigation specialists want to promote water conservation they must discuss this portion of the Yield curves, and very little transferrable data is available regarding excessive water applications. It is difficult to know the exact economic benefits of reducing irrigation applications until carefully conducted, local trials are run.

Yield vs. Fertilizer ad Water

Figure 18 illustrates one relationship between Yield and Applied Water, with three levels of applied fertilizer. It is a non-transferrable figure for the reasons mentioned earlier, but it does indicate that there are definite benefits to coordinating fertilizer and water applications. Most water-fertilizer interaction studies have dealt with nitrogen fertilizers, but increasing work is being done by commercial growers to optimize not just the amount of nitrogen, but also the types and timing of nitrogen fertilizer as well as other fertilizers such as potassium, phosphorus, and micro-nutrients when applied through irrigation water. Most of that work has been done with drip irrigation.

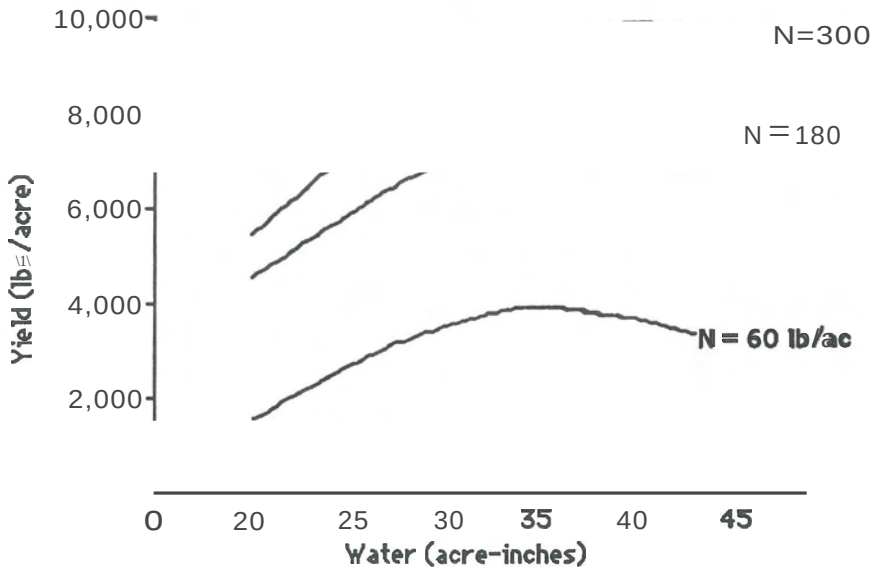


Figure 18. Yield response curves for com (grain) to water assuming specified levels of nitrogen (Hexem and Heady, 1978).

Critical Timing of Irrigations

The water productions shown so far were developed by stressing plants fairly consistently during a season. For some plants such as grains the timing of the water stress is more important than the annual deficit. For example, if corn wilts for only two days during the pollination stage, grain yield can be reduced as much as 20%.

Table 6. Sensitive growth periods for water deficits. (Modified from Doorenbos and Kassam, 1979).

Crop	Growth Period(s) of Maximum Sensitivity
Alfalfa Hay	Immediately after cutting. V. sensitive to both excesses and deficits of water at this time.
Banana	Throughout but particularly during first part of vegetative period, flowering, and yield formation.
Bean	Flowering and pod filling; vegetative period not sensitive when followed by ample water supply.
Cabbage	During head enlargement and ripening.
Citrus	Often desirable to stress before flowering; must have no stress once fruit reaches small marble size.
Grapefruit	Flowering and fruit set> fruit enlargement.
Lemon	Flowering and fruit set> fruit enlargement; heavy flowering may be induced by withholding irrigation just before flowering.
Orange	Flowering and fruit set> fruit enlargement.
Cotton	Flowering and boll formation. Water stress can reduce vegetative growth and induce flowering in heavy clay soils.

Grape	Vegetative period, particularly during shoot enlongation and flowering> fruit filling. Recent work in California shows that even on table grapes, sufficient stress to have only 80% Transpiration reduces vegetative growth and enhances fruit. Stress is needed in later season to develop proper wine grape quality.
Corn	Flowering> grain filling; flowering and pollination states are more sensitive if no prior water deficit.
Olive	Just prior to flowering and yield formation, particularly during the period of stone hardening.
Onion	Bulb enlargement, particularly during rapid bulb growth> vegetative period (and for seed production at flowering).
Pea	Flowering and uield formation> vegetative, ripening for dry peas.
Pepper	Throughout but particularly just prior to and at start of flowering.
Pineapple	During period of vegetative growth.
Potato	Period of stolonization and tuber initiation, yield formation> vegetative period and ripening.
Rice	During period of head development and flowering> vegetative period and ripening.
Safflower	Seed filling and flowering> vegetative.
Sorghum	Flowering yield formation> vegetative; vegetative period less sensitive when followed by ample water supply.
Soybean	Yield formation and flowering; particularly during pod development.
Sugarbeet	Particularly during first month after emergence.
Sugarcane	Vegetative period, particularly during period of tillering and stem elongation> yield formation.
Sunflower	Flowering> Yield formation> late begetative period, particularly during period of bud development.
Tomato	Flowering> yield formation> vegetative period, particularly during and just after transplanting. Some stress may be needed just prior to harvest of processing tomatoies to increase percent solids.
Watermelon	Flowering, fruit filling> vegetative period, particularly during vine development.
Wheat	Flowering> yield formation> vegetative period; winter wheat less sensitive than spring wheat.

Irrigation Scheduling

Accurate estimation of daily plant ET is an accomplished fact on hundreds of thousands (if not millions) of acres. There are many facets to irrigation scheduling. Successful irrigation scheduling programs generally utilize a combination of the following:

1. Daily weather data such as wind run, solar radiation, maximum and minimum temperatures, and relative humidity. The data must be collected at a well sited weather station with good instruments.

2. A calibrated program to calculate reference ET (ETr) with a micro-computer. The most commonly used equations are the modified Penman and the Jensen-Haise. Many programs are available "off-the-shelf', but they generally must be calibrated locally. Failure to do this can result in large errors.
3. Established relationships between the ET of specific plants and the Reference ET, depending upon plant type, stage of growth, and root zone depth.

The requirements (1-3) up to this point are often met by public agencies and serve to provide a data base to farmers, golf course operators, and commercial irrigation scheduling companies. However, actual implementation of a viable irrigation scheduling operation in the field must also include the following components.

4. Weekly or bi-weekly field checks of soil moisture, plant growth, and irrigations.
5. Well trained and experienced field personnel who understand that each plant may need a stress (or no stress) at specific growth stages for maximum growth, and the constraints of the irrigation systems.
6. Excellent communication with the irrigators and farm managers.

Some irrigation schedulers use pan evaporation or simple monitoring of soil moisture variations to predict the timing and quantities of the next irrigation. A major advantage of using a computer is in maintaining records and being able to incorporate more sophisticated computations such as soil drying effects. However, a computer is not necessary for successful scheduling.

Successful schedulers are usually not obsessed with the need to precisely predict ET because they recognize other problems such as insect damage, soil variability, limitations on accuracy of flow measurement, inability to irrigate the whole field in one day with most irrigation systems, non-uniformity of water application, etc.

Currently the greatest limitations to successful scheduling by experienced schedulers do not involve a lack of good ET prediction equations, but rather problems in communication with the irrigators, lack of farmer motivation, poor irrigation system design, and inflexible water deliveries to farms.

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CHAPTER 2

SALINITY

SALINITY

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SALINITY

Introduction

Salts are molecules which break up into positive (cation) and negative (anion) ions when placed in water. For example, sodium chloride (NaCl) is a solid when dry, but when it dissolves in water, it disassociates into two charged particles, the sodium cation (Na^+) and the chloride anion (Cl^-). Total salinity is often referred to as "TDS" or Total Dissolved Solids. Soil particles in the water are not salt; soil in the water is often referred to as "TSS" or Total Suspended Solids.

All irrigation water has salt. The irrigation water quality (chemistry) affects the total salinity and types of salt found in the soil. For this reason, the water and soil chemistry should always be examined before starting a reclamation or irrigation management plan. There are three potential problems associated with this salt:

1. Salt can accumulate in the soil, causing an "osmotic stress" or osmotic potential which has the same effect on plants as a very negative matrix potential when the soil is dry. High salt levels also inhibit plant germination and emergence.

2. If there is a disproportionate amount of Sodium or Bicarbonate and Carbonate in the water the soil surface will seal up, causing infiltration problems. Very pure water causes similar effects.

3. Some specific types of salt, including boron and lithium, are highly toxic to plants in relatively low concentrations.

Some soil salt problems originate from irrigation and fertilization practices. In other cases the original soil had high levels of salinity. In other areas, salinity problems in a field are due to a high water table and salts which are caused by irrigation deep percolation or seepage on fields at higher elevations.

Units of Salinity Measurement

There are three common methods of describing the amount of salt in a sample. These are described below.

1. Electronic Conductivity (EC)
   This value is the easiest to measure, and provides a measure of the total amount of salts in a sample. It is an indication of the number of charged particles in a sample of solution. The measuring procedure uses a current applied between two probes in a water sample; the higher the conductivity of the water, the higher the salinity. The reading is immediate from a dial or digital output on the meter. Because this is a single measurement of total salinity, it does not provide information about the relative concentrations of various constituents of salt in the sample.

   The units of EC are usually decisiemens per meter *(dS/m)* or millimho per centimeter *(mmho/cm).*

   $$1\ dS/m = 1\ mmho/cm$$

   Instruments are now available to provide measurements of the EC of soil profile without the need for taking soil samples. These non-destructive techniques use probes on or near the surface of the soil which send a current through the soil and measure the conductivity. These meters, mounted on a small vehicle, are being used on large district-level surveys for rapid determination of soil salinity levels.

2. Parts per Million (PPM)
   This is a weight (mass) measurement. A common conversion factor is:

   $$\text{ppm} = \frac{\text{milligrams of salt}}{\text{liters of solution}}$$

   For example, consider:

   Total weight of salt in a water sample = 150 mg

   Water sample size = 1.5 liters

   Find: ppm of total salt in the sample

   $$\text{ppm} = \frac{150\ \text{mg}}{1.5\ \text{liters}} = 100\ \text{ppm}$$

The concentrations of individual salt constituents in a sample are measured by various laboratory devices or procedures. The determination of each individual salt constituent ppm is a separate test. The measurement of the total ppm of a sample is the sum of all of the individual salt constituents, and is considerably more difficult than the **EC** determination.

The approximate relationship between EC and total ppm of salt for irrigation waters in the western U.S. is:

$$1\ dS/m = 700\ \text{ppm}$$

For example, consider

Salinity in a river = 1.2 *dS/m*

Find: Estimate of total ppm salt in the river

$$\text{ppm} = 1.2\ dS/m \times \frac{700\ \text{ppm}}{1\ dS/m} = 840\ \text{ppm}$$

This relationship depends upon the specific water sample, as various types of salt of different weights.

3. Millieguivalents per Liter (meqll)

The ppm of each salt constituent is a measurement of weight, but many salinity problems are related more to the number of salt charges than salt weights. Therefore, the ppm values are often converted to meq/l for mathematical computations regarding such things as permeability hazard and the amount of soil or water amendments needed to correct a problem.

The formula for determining the meq/l of a particular ion is:

$$\text{meq/l} = \frac{\text{ppm}}{\text{milliequivalent weight in milligrams}}$$

The formula for determining the total number of millequivalents of a substance is:

$$\text{meq} = \frac{\text{weight of the substance in milligrams}}{\text{milliequivalent weight in milligrams}}$$

For example, using the values in Table 1, the meq/l concentration of a water sample with 120 ppm calcium is:

$$\text{meq/l} = \frac{[120\ \text{ppm}\ Ca^{2+}]\ /\ \text{liter}}{20\ \text{mg/mequiv. wt}} = 6\ \text{meq/l}$$

Table 1. Common constituents of irrigation waters with their weights and charges.

Element Name	Symbol	Charge	Atomic Weight, gm	Milliequivalent Weight, mg
Cations (positive)				
Ammonium	NH_4	+1	17.0	17.0
Calcium	Ca	+2	40.1	20.1
Hydrogen	H	+1	1.0	01.0
Magnesium	Mg	+2	24.3	12.2
Sodium	Na	+1	23.0	23.0
Potassium	K	+1	39.1	39.1
Anions (negative)				
Bicarbonate	HCO_3	-1	61.0	61.0
Carbonate	CO_3	-2	60.0	30.0
Nitrate	NO_3	-1	62.0	62.0
Phosphate	PO_4	-3	79.0	26.3
Sulfate	SO_4	-2	96.1	48.1

Definitions

Acid - A substance with a pH less than 7.0.

Adjusted Sodium Adsorption Ratio (adj. RNa) - An index of permeability problems, based upon water quality.

Alkaline soil - A soil with an exchangeable sodium percentage (ESP) greater than 15%. Symptoms of an alkaline soil include water penetration problems and poor crop growth. Very hard, structureless clods tend to form near the soil surface when the soil dries out. Alkaline soils have excess sodium and insufficient exchangeable calcium on the exchange sites.

EC - Electrical Conductivity. A measurement which indicates the total salinity of a sample of soil or water without considering the individual constituents. A high EC occurs with a high salt level. Units are millimhos/centimeter (mmho/cm) or decisiemens/meter(dS/m). There are three EC measurements of concern with water management:

EC_w - EC of the irrigation water.

EC_{sw} - EC of the soil water solution. This is the salinity that the plant roots "see". The EC_{sw} increases as the soil dries out, because the same amount of salt is contained in less and less water, resulting in a more concentrated solution.

EC_e - EC of the extract of a saturated soil paste. Distilled water is added to a soil sample, mixed up, and then the solution is extracted from the saturated soil sample. The EC of this extract is the EC_e. Estimates of the effects of the EC_{SW} on crop growth are made from this EC_e.

General EC notes:

i) EC_{SW} right after irrigation « EC_{SW} before an irrigation.

ii) Immediately after a heavy irrigation, the ECswabout 1 foot below the soil surface may be approximately equal to the EC_w' since the original soil water would have been replaced by the irrigation water.

iii) For a loam soil at field capacity, the EC_e is about half that of the EC_{SW}' since about half of the pore volume is filled with water at field capacity.

iv) The EC_e of a sample is always less than the EC_{SW}' since the soil water is diluted with distilled water during the process.

v) If a sample is taken right after an irrigation, and then right before the next irrigation, there is the same amount of salt (Ibs/acre) in the soil. However, the EC_{SW} will be very different.

ESP - (Exchangeable Sodium Percentage). The percentage of the cation exchange capacity of a soil which is occupied by sodium.

Leaching - Application of water in excess of crop needs, so that some irrigation water passes completely through the soil root zone.

Leaching Requirement (LR) - The theoretical fraction of applied water required to maintain a desired salinity level in the soil at the spot in the field which requires just enough water to match Evapotranspiration requirements. LR does not include water needed to overcome problems of non-uniformity. LR values are typically .02 -.08 (2-8% of that applied), but may be as high as 0.3-0.4 (30-40%) with a high EC_w' a sensitive crop, and high temperatures. For an area which has no leaching contribution from rainfall,

$$LR = \frac{\text{Inches of irrigation water needed for leaching}}{\text{Total inches of irrigation water applied per year}}$$

Leaching Fraction (LF) - The actual depth of water which infiltrates at the point of the field having the least deep percolation.

pH - A measure of acidity or alkalinity (negative logarithm of the hydrogen ion concentration). Most agricultural plants grow best with a soil pH in the range of 6.5 -7.5.

General Guidelines for Water Quality

Some general water quality guidelines have been established, as seen in Table 2. They must be adjusted for the soil type, frequency of irrigation, and crop type.

Table 2. General guidelines for interpretation of water quality for irrigation (From Ayers, 1977).

Problem and Related Constituent	Water Quality Guidelines		
	No Problem	Increasing Problems	Severe Problems
Salinity (toxicity to plant growth) ECw of irrigation water, in dS/m	< 0.75	0.75 - 3.0	> 3.0
Permeability See Table on Permeability			
Specific Ion Toxicity from ROOT absorption * Sodium (evaluate by adj. SAR) * Chloride Milliequivalents per liter Milligrams per liter (ppm) * Boron, in milligrams per liter (ppm)	 <3 <4 < 142 < 0.5	 3.0 - 9.0 4.0 - 10 142 - 355 0.5 - 2.0	 > 9.0 > 10 > 355 2.0 - 10.0
FOLIAR absorption - sprinklers * Sodium Milliequivalent per liter Milligrams per liter (ppm) * Chloride Milliequivalents per liter Milligrams per liter (ppm)	 < 3.0 < 69 < 3.0 < 106	 > 3.0 > 69 > 3.0 > 106	
Miscellaneous * NH4-N and N03-N, mg/l for sensitive crops (causes excessive vegetative growth)	 <5	 5-30	 > 30
* HC03 (only with overhead sprinklers- causes white spots on tree fruit) Milliequivalents per liter Milligrams per liter (ppm)	 < 1.5 < 90	 1.5 - 8.5 90 - 520	 > 8.5 > 520

Total Salinity - General

Typical irrigation waters import significant amounts of salt to fields. These salts must eventually be leached out of the soil by deep percolation of rain or irrigation water. As an example of the large quantities of salt, consider the example problem below which uses reasonable numbers found in many irrigated arid areas.

Given: EC_w = 0.7 dS/m
Cotton with 40" irrigation water applied per year
100 acre field

Find: Pounds of salt applied/year

Formulas: lbs of salt = lbs of water * ppm/(1 million)

ppm = dS/m x [700 ppm/(dS/m)] (approximately)

1 cu. ft. of water weighs 62.4 lbs.

Solution: Cubic feet of water = (40"/ $\frac{12"}{ft.}$) x 100 acres x (43,560 sq. ft/ac)

= 14,520,000 cu. ft

Lbs.ofwater = (62.4 lb/cu.ft) x 14,520,000 cu. ft

= 9.0605 x 10^8 lb

Lbs of salt = 0.7 dS/m x $\frac{700 \text{ lb salt}}{10^6 \text{lb solution (dS/m)}}$ x (9.0604 x 10^8 lb solution)

= 444,000 lbs of salt per year

Root Zone Salt Profile

Ifthe irrigation water salinity (EC_w) is known, a desired salt concentration in the soil water solution (EC_{sw}) can be maintained by controlling the amount of deep percolation (ie, by maintaining a desired Leaching Requirement, LR). The controlled soil salinity level should be less than that which would harm crops. The effect of different amounts of deep percolation, with the same irrigation water quality (same EC_w) is illustrated in Figure 1.

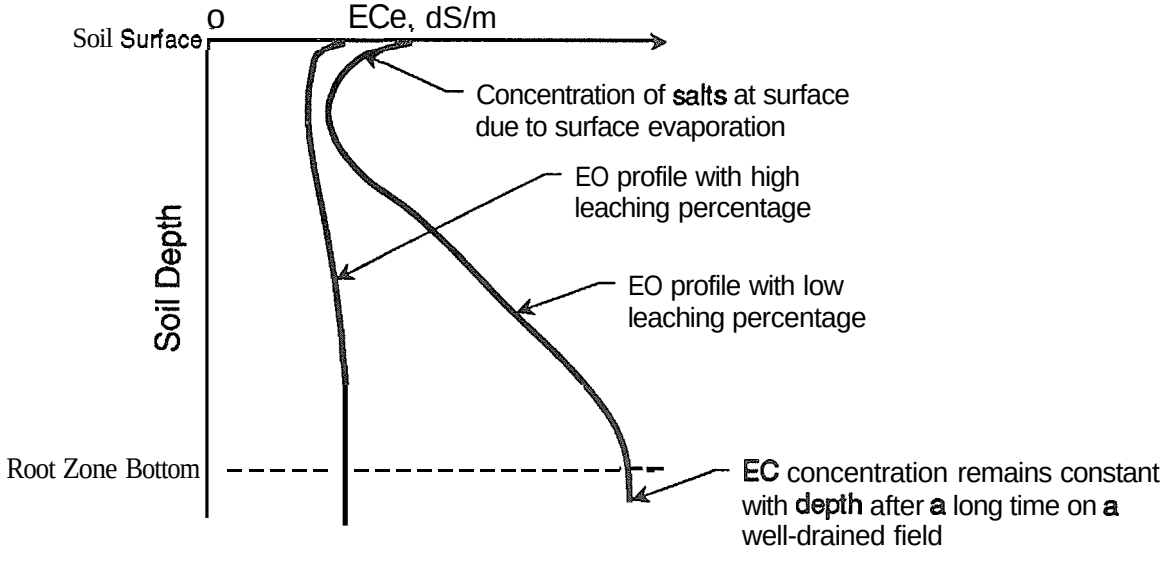


Figure 1. Salinity concentrations at various depths in a root zone which is well drained and which has had the same irrigation practices for some time.

Figure 1 shows that salt concentrations are generally higher in the bottom of the root zone than in the top of the root zone. Without some deep percolation in irrigated fields, the soil salinity gradually builds up until crop yields decline and eventually reach zero. Good natural or artificial subsurface drainage must exist so that the water which ends up as deep percolation can be removed from the root zone.

Crop Sensitivity after Germination

Although plants respond to the EC_{sw}, until recently the EC_{sw} has been difficult to measure directly. Therefore, past researchers have correlated yields with measurements of the EC_e of a soil, which is relatively simple to measure. Critical (threshold) EC_e values are found in Table 3. Current thinking is that once established, plants respond to the average root zone salinity.

Table 3. Salt tolerance of various crops to soil salinity, after germination. (Adapted from Maas *and Hoffman, 1977).*

Crop	Threshold ECe (ECe at initial yield decline) *dS/m*	Yield decrease per unit increase in salinity beyond Threshold % Yield *dS/m*	Crop	Threshold ECe (ECe at initial yield decline) *dS/m*	Yield decrease per unit increase in salinity beyond Threshold % Yield *dS/m*
Alfalfa	2.0	7.3	Lovegrass	2.0	8.4
Almond	1.5	19.0	Meadow foxtail	1.5	9.5
Apricot	1.6	24.0	Onion	1.2	16.0
Avocado	1.3	21.0	Orange	1.7	16.0
Barley (grain)	8.0	5.0	Orchardgrass	1.5	6.2
Bean	1.0	19.0	Peach	1.7	21.0
Beet, garden	4.0	9.0	Peanut	3.2	29.0
Bermudagrass	6.9	6.4	Pepper	1.5	14.0
Blackberry	1.5	22.0	Plum	1.5	18.0
Boysenberry	1.5	22.0	Potato	1.7	12.0
Broadbean	1.6	9.6	Radish	1.2	13.0
Broccoli	2.8	9.2	Rice, paddy	3.0	12.0
Cabbage	1.8	9.7	Ryegrass, perennial	5.6	7.6
Carrot	1.0	14.0	Sesbania	2.3	7.0
Clover; ladino red, strawberry	1.5	12.0	Soybean	5.0	20.0
Clover, berseem	1.5	5.7	Spinach	2.0	7.6
Corn (forage)	1.8	7.4	Strawberry	1.0	33.0
Corn (grain)	1.7	12.0	Sudangrass	2.8	4.3
Corn, sweet	1.7	12.0	Sugarbeet	7.0	5.9
Cowpea	1.3	14.0	Sugarcane	1.7	5.9
Cucumber	2.5	13.0	Sweet Potato	1.5	11.0
Date	4.0	3.6	Tomato	2.5	9.9
Fescue, tall	3.9	5.3	Trefoil, big	2.3	19.0
Flax	1.7	12.0	Frefoil, Birdsfoot narrow	5.0	10.0
Grape	1.5	9.6	Wheat	6.0	7.1
Grapefruit	1.8	16.0	Wheatgrass, crested	3.5	4.0
Harding grass	4.6	7.6	Wheatgrass, fairway	7.5	6.9
Lettuce	1.3	13.0	Whealgrass, tall	7.5	4.2

Graphically, the relationships of salinity effects and yield after plant establishment are shown in Figure 2.

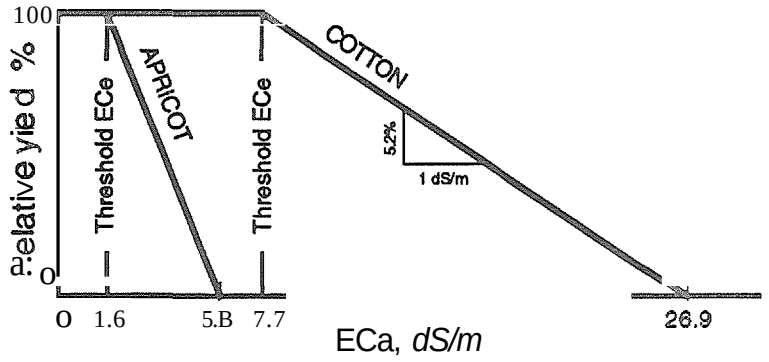


Figure 2. Salinity yield functions for two crops. Valid after emergence.

Table 3 indicates that crops differ greatly in their resistance to soil salt levels. Crops with high tolerances such as sugar beets, cotton, and barley are often the only crops which can be grown in areas with salty water and during initial reclamation of saline soils. In general, salt-resistant crops are of lower value than salt-sensitive crops.

MAD and Salinity

The threshold values of Table 3 are for conventional cyclic irrigation schedules in which plants receive water at a 50-60% MAD. Figure 3 illustrates that immediately before such an irrigation, the EC_{sw} would be rather high because the salts are concentrated in a small soil moisture solution; the EC_{sw} (for the same EC_e) would never get as high if the irrigations were scheduled on a lower MAD. Therefore, the threshold EC_e for plants grown in constantly moist environments (low tension and low MAD) will be higher than the values shown in Table 3. Likewise, plants grown under extreme environmental stress such as very high temperatures have lower threshold values. There has been little research done on the interactions of temperature and salinity.

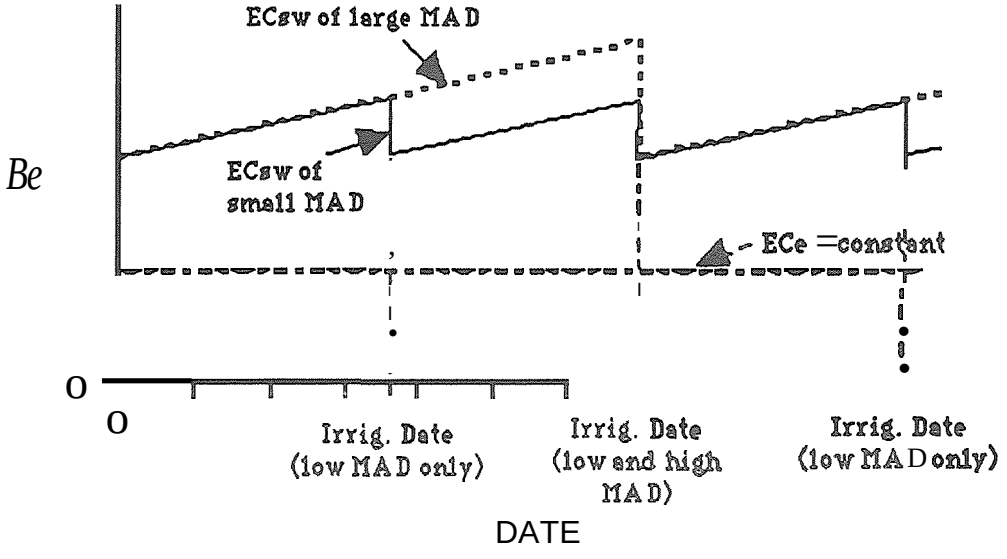


Figure 3. The average EC_{sw} varies with the MAD, but the EC" remains the same.

Figure 3 illustrates that as the soil dryness increases, the EC_{sw} also increases. The greater osmotic potential is accompanied by a greater matrix potential as the soil pores are emptied. The effects are additive. As an approximation,

$$1\ dS/m \text{ of } EC_{sw} = 0.3 \text{ bars of tension}$$

Therefore, the use of a low MAD in salty soils is especially important.

Leaf Burn

Leaf bum only occurs with direct application of salty water onto leaves, which occurs with sprinklers and some micro-sprayers. In a bad case, all the leaves will fall off trees. Leaf burn is especially bad in dry, windy situations because downwind areas receive many hours of spray drift which coat the leaves. In many areas with salty water, surface irrigation (furrows and border strip) is required after plant establishment, to avoid salt burn.

Table 4 provides some guidelines regarding foliar injury. Small, frequent sprinkler irrigations result in higher evaporation losses than large, infrequent sprinkler irrigations. The evaporation of irrigation water concentrates the salt on the plant leaves, which have no protection mechanism against absorption of the salt into the plant cells. Very low moving linear move or center pivot sprinkler systems (with spray nozzles extending below the towers) tend to have less foliar damage than hand move sprinklers because there is less misting and less opportunity time for salts to concentrate on the leaves.

Table 4. Relative tolerance of selected crops to foliar injury from saline water applied by sprinklers. These data are presented only as general guidelines for day-time hand move sprinkler irrigation. (from Pettygrove, 1984)

Na or Cl concentrations (meq/L) causing foliar injury			
<5	5-10	10-20	> 20
Almond Apricot Citrus Plum	Grape Pepper Potato Tomato	Alfalfa Barley Corn Cucumber Safflower Sesame Sorghum	Cauliflower Cotton Sugarbeet Sunflower

Germination Problems

Most discussions of salt damage cover salt tolerances as found in Tables 3 and 4. However, for farmers the most important aspect of salinity management occurs during germination. Simply put, if a crop does not germinate and emerge, there is no need to worry about later crop damage.

The most salt-sensitive growth period for most crops is germination or emergence. Salt problems at this time manifest themselves as bare spots in the field later on. Special care must be given to the salt levels in the seedbed itself. Various practical solutions are as follows:

1. Use the proper furrow shape to avoid concentrating the salt near the seed. Figure 4 illustrates the impact of various furrow shapes.

Furrow Shape	Salt Damage Potential
Single Row Bed Every Furrow Irrigated	Very High
Flat Double Row Bed Every Furrow Irrigated	High
Modified Double Row Bed Every Furrow Irrigated	Mod. Low
Sloping Double Row Bed Every Furrow Irrigated	Low
Sloping Single Row Bed Every Furrow Irrigated	Low

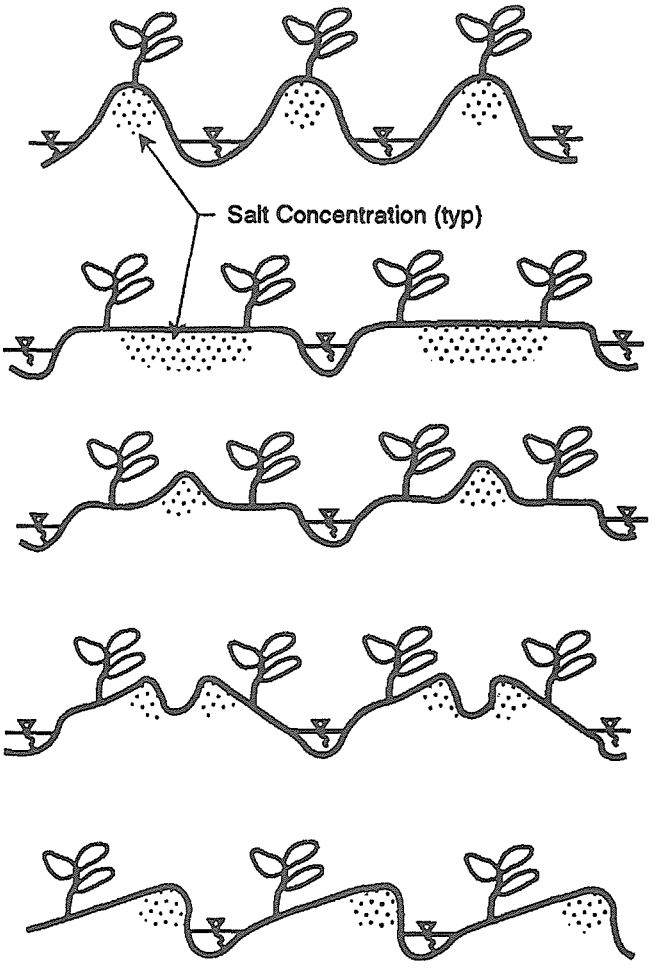


Figure 4. Furrow bed shapes and zones of salt accumulation.

2. Use sprinklers for first germination irrigations, and switch to furrows later. The sprinkler irrigations drive the salt down below the seedbed. Sprinklers are also capable of maintaining low MAD's, whereas it is very difficult to do so efficiently with furrows at planting time.
3. Use pre-irrigations to accomplish the salt leaching, and in the process clean the seedbed.
4. Place fertilizers properly where they won't hurt seedlings. Fertilizer which is banded into a furrow bed between the water and the seed will move with the water and possibly damage the seed. Fertilizer on the distant side of the seed will tend to remain in place, and will not be transported onto the seed.

Specific Ion Toxicities

Some elements such as boron and lithium are extremely toxic to plants in very low concentrations. They may be found in either the water or in the soil.

Infiltration (Permeability) Problems - General

Infiltration rates commonly decrease during the growing season of row crops as the soil surface becomes more compacted. Certain types of salt in the irrigation water can also cause decreases in permeability.

Four water quality conditions contribute to infiltration problems:

1. High Adjusted Sodium Adsorption Ratio (adj. RNa) values.

2. Low EC_w

3. High ratios of Magnesium to Calcium.

4. Fertigation with monovalent cations such as ammonium (NH_4^+) and potassium (K^+)

High Adiusted Sodium Adsorption Ratio (adj. RNa) values

This is the most commonly recognized water quality problem. It contributes to a poor Calcium vs. Sodium ratio on the soil exchange sites. High percentages of Sodium on the exchange sites are bad for water penetration; high percentages of Calcium are good. A high soil ESP (Exchangeable Sodium Percentage of greater than 15%) is often the result of irrigating with poor quality water.

Most water quality analyses from agricultural laboratories include a calculated value of "Adjusted Sodium Adsorption Ratio". At the moment, there is no universal standard method of calculating and interpreting this ratio. Modifications of this permeability index continue to be made. The specific formulas used in the calculations should always be checked before interpretation. One formula used for the adjusted. RNa is:

$$\text{adj. RNa} = \frac{Na}{\sqrt{\frac{Ca_x + Mg}{2}}}$$

where Na and Mg are in milliequivalents/litertaken from the irrigation water quality analysis, and Ca_x (also in meqll) is obtained from Table 5. Interpretations of adjusted. RNa are made using Table 6.

The "Ca_x" value is used instead of "Ca" because the "Ca_x" accounts for precipitation of some of the Calcium with Bicarbonates and Carbonates in the water. If the Calcium precipitates as $CaC0_3$, it essentially ends up as a rock in the soil, and the Calcium is not available for replacement of Sodium on the soil Cation Exchange sites.

Table 5. Ca_x values (meq/l) for use in calculating RNa of irrigation water (from Westcot and Ayers, 1984).

Ratio (HC03 + C03)/Ca	Caxfor salinity of applied water (column headings are ECw - dS/m						
	0.1	0.3	0.5	1.0	1.5	3.0	6.0
.05	13.2	13.92	14.40	15.26	15.91	17.28	19.06
.15	6.34	6.69	6.92	7.34	7.65	8.31	9.17
.25	4.51	4.76	4.92	5.22	5.44	5.91	6.52
.50	2.84	3.00	3.10	3.29	3.43	3.72	4.11
1.00	1.79	1.89	1.96	2.09	2.16	2.35	2.59
2.00	1.13	1.19	1.23	1.31	1.36	1.48	1.63
3.50	0.78	0.82	0.85	0.90	0.94	1.02	1.12
7.00	0.49	0.52	0.53	0.57	0.59	0.64	0.71
20.00	0.24	0.26	0.26	0.28	0.29	0.32	0.35

Table 6. Permeability hazard of irrigation water (Westcot and Ayers 1984). ECw is in dS/m.

adj. RNa	Degree of restriction on use		
	None	Slight - Moderate	Severe
0-03	and ECw = >0.7	0.7 - 0.2	<0.2
03 - 06	>1.2	1.2 - 0.3	<0.3
06 - 12	>1.9	1.9 - 0.5	<0.5
12 - 20	>2.9	2.9 - 1.3	<1.3
20 - 40	>5.0	5.0 - 2.9	<2.9
(ECw is expressed in *dS/m)*			

An example calculation of the Adjusted Sodium Adsorption Ratio follows:

Given: The sample of water quality below

Item	Concentration
Sodium (Na)	2.5 meq/liter
Calcium (Ca)	1.0 meq/liter
Magnesium (Mg)	0.5 meq/liter
Bicarbonate (HC03)	2.0 meq/liter
Carbonate (C03)	0.0 meq/liter
Electrical Conductivity (EC)	1.0 dS/m

Find: Adjusted Sodium Adsorption Ratio
Calculations:

Ca_x : Ratio HCO_3/Ca $= \frac{2.0 + 0.0}{1.0} = 2.0$

= 1.0dS/m

= 1.31 (from Table 5)

adj. RNa

$= 2.6$

From Table 6 (adj. RNa= 2.6 and EC_W= 1.0), the degree of restriction on use is classified as "None".

Low EC_w:

Table 6 shows that permeability problems can be caused by an inter-relationship between EC_w and the relative amounts of various ions. For example, a very high adj. RNa of 30 presents no problem for infiltration if the irrigation water is very salty (greater than 5.0 dS/m). On the other hand, ifthe irrigation water is very pure (EC_W< 0.2 dS/m), there are severe permeability problems even if there is a high proportion of Calcium in the irrigation water.

There is still considerably more to learn about the effects of low ECwon permeability (Tanji, 1990). In many California soils, the irrigation water from the Sierra Nevadas and some wells on the east side of the San Joaquin Valley is very pure (EC_W< 0.25 dS/m). Ifthis irrigation water is applied to soils of low organic matter, with no ground cover and no mechanical disturbance ofthe soil, the intake rates are extremely low. These conditions exist with some furrows on citrus, where clean-cultivation is practiced with herbicides. With micro-irrigation (drip and micro-spray) it is particularly difficult because the ground which receives water is rarely disturbed and is often kept clean with herbicides. On the other hand, irrigation with very pure water in otherareas ofthe U.S. has not presented such problems. Thedifferences may be caused by variations in soil parent materials, organic matter percentages, tillage practices, or the existence of frost in the winter.

High Magnesium/Calcium Ratios.

This is another area with relatively little research and reference in textbooks. However, field experience shows that if the Mg/Ca ratios of irrigation water exceed 1.0, there is a high probability ofpermeability problems. This ratio is frequently exceeded on the Central Coast of California, where the aquifers are in soils with serpentine parent materials.

The problem with Magnesium brings up the question of why Magnesium is considered a beneficial cation in the calculation of the Adj. RNa' Future research by soil scientists may show that Magnesium should not be included in the formula for Adj. RNa'

Fertigation with Monovalent Cations.
Under conditions of low EC_w, any monovalent cation (a positively charged ion with only one charge/ion) resembles Sodium in terms of effects on permeability. Therefore, in some cases it is preferrable to use urea or nitrate fertilizers ratherthan ammonia-based fertilizers when fertigating.

General Types of Salinity Conditions.
A "good" soil from a salinity standpoint is one with a fairly neutral pH (6.5-7.5), low EC_e (less than 1.0 dS/m), and a low ESP (less than 15%). A "good" irrigation water has a moderately low EC_w (0.3 - 0.8 dS/m) and a low adjusted RNa (less than 3.0). These values will vary with soil texture, crop, and the specific mix of salt constituents. However, conditions which are suboptimal are widespread.

Condition #1. Irrigation water quality is poor, but the soil is good before irrigation begins. A very fertile, non-saline and non-alkaline soil can be made saline and/or alkaline within a year if the irrigation water quality is poor and preventative measures are not taken. These preventative measures almost always involve water treatment and/or special irrigation management, and sometimes also require special soil treatments.

Condition #2. Irrigation water quality is good, but the soil is poor before irrigation begins. In some cases the irrigation water may be of very good quality (fairly low salts, and a proper mix of the various salts), but the irrigated soil is saline and/or alkaline due to past management, previous water quality problems, or poor drainage. The solution entails soil treatments and/or heavy leaching.

Condition #3. Irrigation water quality and soil salinity are both poor before irrigation begins. This case has the same solution as Condition #1, with the possibility that the soil treatments may need to be more drastic.

Soil Problems and Specific Solutions
High ECe,. non-alkaline soil.
This condition is also called a "Saline" soil. There is only one solution: reclamation leaching. No chemical amendments to the soil or water are necessary. A rule of thumb for reclamation leaching of high salt levels is that the soil salinity can be reduced by 90% if an equivalent root zone depth of water is applied. For a 5 foot root zone depth, with an average EC_e of 10.0 mmho/cm, leaching with 5 acre-feet ofwater/acre ofgood qualitywater might reduce the EC_e to 1.0 mmho/cm.

This reclamation leaching to reduce an initially high salt concentration from the soil is not to be confused with the use of a "Leaching Requirement". The Leaching Requirement is part of a maintenance program once the soil is no longer saline.

High EC_e, alkaline soil. This condition is also called a "Saline-Alkali" soil. There are two variations of this problem. However, both have one thing in common. The Sodium (alkalinity) problem needs to be taken care of before the EC (salinity) problem. The Sodium problem is one of chemical exchange. The high EC problem is one ofwashing salts out of the soil. If the salts are washed out of the soil first, the soil will seal up (see Table 6) and it will be extremely difficult to take care of the Sodium problem.

The two variations are:

1. The soil has free lime ($CaCO_3$). The existence of free lime can be verified by putting a drop of hydrochloric (muriatic) acid on a soil sample; if there is heavy fizzing, free lime is present. In this case, there is already adequate Calcium in the soil. The Calcium is in a rocklike form and needs to be dissolved into the soil water so it can replace the Sodium which is attached to (adsorbed onto) the soil cation exchange sites. Acid will dissolve the free lime and release the Calcium.

   Generally the solution is to acidify the soil, and then (**but only after acidifying the soil) leach with heavy irrigations. If the alkalinity was originally caused by the irrigation water, the water must also be treated. The following are ways to acidify the soil:

   a. Add Sulfur to the soil. The sulfur particles should be smaller than pea gravel. It must be disced into the topsoil, where it is converted to Sulfuric Acid by soil bacteria. This reaction requires moisture, a warm temperature (greater than about 70 deg. F or 21 deg. C), and many months.

   b. Apply Sulfuric Acid to the soil using a tractor and special injection equipment. This has immediate results. The primary disadvantage is that Sulfuric Acid is extremely dangerous to handle.

   In addition, Calcium can be added to the soil even though free lime is available. This can be done in two ways:

   a. Add Gypsum ($CaSO_4$) to the soil. Gypsum is slightly soluble and will dissolve in the soil, providing "available" Calcium to the soil almost immediately. However, it is usually more expensive than adding the smaller tonnage of Sulfur required to dissolve the Calcium (in the form of $CaCO_3$) which is already in the soil.

   b. Injection of a very pure form of gypsum through the irrigation water. Commercial equipment is available which creates a slurry of very pure gypsum, with particles that can pass a 200 mesh screen.

2. The soil has no free lime. The solution is to add Calcium to the soil or water and then leach (**but only after adding calcium to the soil) with heavy irrigations.

In both situations, the irrigation water quality may have been the original cause of the problem. See the section on water quality problems and solutions for more information on water treatment which must be continued after initial reclamation.

low EGa' alkaline soil .

This is also known as an "Alkaline" soil. The solution is the same as the Saline-Alkali soil, but it is much slower and more difficult to achieve. The Sodium on the soil exchange sites must be replaced by Calcium before a leaching program is begun. Then the leaching will remove the Sodium from the root zone. In some cases, Alkaline soils are sealed upso badly that it takes several years of small irrigations and repeated mixing of gypsum and manure into the topsoil with a disc before water penetration problems are solved. Water treatment is always recommended in the case of this problem.

Water Problems and Specific Solutions

Often two of the irrigation water quality problems described below will occur simultaneously. Both should then be treated.

High Adjusted Sodium Adsorption Ratio (adj. RNa)

The effect is generally on permeability, although the high percentage of sodium may also cause leaf toxicities. There are two variations of this water quality problem:

1. The water has a high percentage of carbonates and bicarbonates (CO_3 and HCO_3). The solution is to remove the bicarbonates and carbonates from the water by acidifying the water. This is most commonly done with S02 Generators or direct injection of sulfuric acid into the water. Sometimes this water treatment can be effective enough to eliminate the need for annual soil amendments, once the soil has been initially reclaimed.
2. The water has a low percentage of CO_3 and HCO_3. The problem is due to high sodium only. In this case, water treatment alone is sometimes successful, but typically annual soil amendment applications are needed, as described under "Soil Problems and Specific Solutions". Water treatment would involve the injection of a gypsum slurry into the irrigation water. In some areas, farmers run their water over gypsum pits before irrigating, but this is less effective than using the gypsum slurry. The pits depend upon dissolving gypsum in the water, and gypsum has a very slow solubility. Also, most mined gypsum (as used in the pits) has a high percentage of clay which inhibits effective dissolving.

Very low EC_w (less than 0.25 *dS/m).*

This problem has already been described in an earlier section on Low EC_w. A common solution is to reduce the CO_3 and HCO_3 levels as much as possible (by acidifying the water) and to also increase the Calcium concentration of the water.

High EC_w

The solutions have already been described in the general section on Total Salinity. Special practices must be used in selection of crop type, avoidance of leaf burn, and eliminating germination problems. However, a major concern is maintenance leaching. The special section on Leaching Requirement Computations is dedicated to that topic.

Leaching Requirement Computations

The extra percentage of water which must infiltrate into the soil beyond the amount needed for meeting the evapotranspiration needs, non-uniformity, and non-infiltrated losses (e.g., spray evaporation, uncollected runoff) is called the Leaching Requirement (LR). It is for a maintenance program, not a reclamation program. The following example illustrates the calculation and use of the Leaching Requirement (LR) value:

Given: Alfalfa
Irrigation Water EC_w = 1.3 *dS/m*

Find: Leaching Requirement (LR) to maintain an average soil root zone EC_e of no greater than the threshold EC_e at the point in the field which receives the least water.

Formula: From Table 3, the threshold EC_e for alfalfa = 2.0 *dS/m*

$$\text{necessary LR} = \frac{\qquad}{[5 \times ECJ - EC_w}$$

where EC_e and EC_w are in *dS/m,* and EC_e is the threshold EC

Solution:
$$\text{necessary LR} = \frac{1.3}{[5 \times 2.0] - 1.3}$$

$$= .15$$

The LR is used in the following equation to determine how much irrigation water should be applied to a field per year:

$$\text{Gross to apply} = \frac{\text{Net Requirement - Effective Rainfall}}{\frac{\text{Irrigation Efficiency with good mgmt} \times (1 - \text{LR})}{100}}$$

The equation above assumes no contribution of water from a high water table, and no net change in soil moisture storage over the crop year. Furthermore, by using this equation to compute the gross to apply, there will be no under-irrigated areas in the field. If a high water table contributes water for ET, plus contributes salt to the root zone, the leaching fraction must be applied for both the irrigation water and high water table components.

The problem below illustrates how to calculate the gross irrigation requirement, and what the various destinations of water are.

Given: . Net ET requirement = *45"/year(assume* there are no other net requirements, such as water for frost protection).
Effective rainfall = *15"/year*
Irrigation Efficiency with good management =80%
Leaching Requirement = 0.15

Find: The gross annual irrigation requirement

Formula:
$$\text{Gross to apply} = \frac{\text{Net Requirement - Effective Rainfall}}{\frac{\text{Irrigation Efficiency with good mgmt} \times (1 - \text{LR})}{100}}$$

Solution:
$$\text{Gross to apply} = \frac{(45" - 15")}{\frac{80\% \times (1 - .15)}{100}}$$

$$= 44.1"$$

The effects of applying 44.1" of irrigation water would be:

1. The extra water which must be applied for salinity control is 15% of the gross irrigation water

   = .15 x 44.1" = 6.6"

2. The use ofthe 80% efficiency in the denominator applies a correction factor to account for losses and non-uniformity of infiltration. The 44.1" would leave no under-irrigated areas in the field. The EC_e at the driest point in the field would be 2.0 *dS/m.*

3. Because of non-uniformity, all points in the field will receive more than the "driest point". Therefore, the EC_e at all other points in the field will be less than 2.0 *dS/m.*

4. The extra 6.6" must only be applied once per year. An extra 15% does not have to be applied each irrigation except in conditions of very high EC_w.

Uniformity and leaching

The "Leaching Requirement" is the factor which will provide the desired salt concentration in the soil at the spot in the field which receives the least amount ofwater. Because of non-uniformity of water application in a field, all other points in the field will receive more deep percolation. Therefore, all other points in the field will have lower ECe's. Some consultants use a map of ECe's to construct a picture of irrigation DU.

Figure 5 illustrates that the water used for LR only represents a small percentage of the total deep percolation.

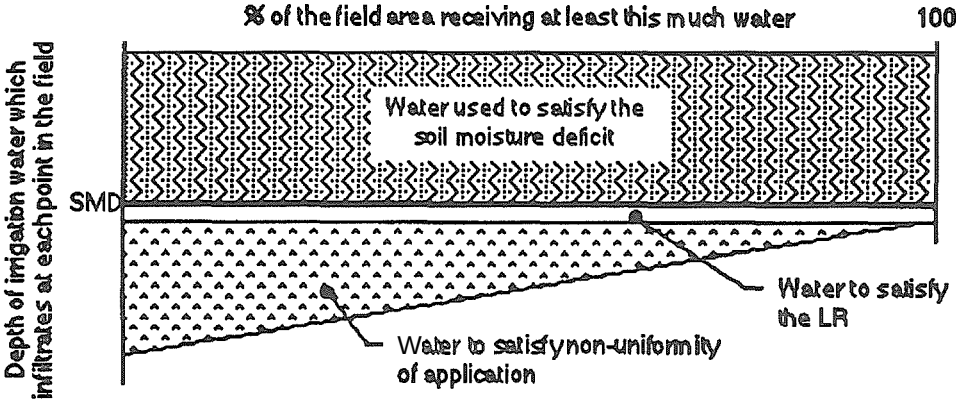


Figure 5. Schematic of non-uniformity of water application throughout a field. The non-uniformity results in different leaching amounts at different points, which in tum results in different EC,:s.

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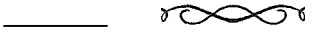
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CHAPTER 4



HYDRAULICS OF WATER MOVEMENT OVER AND INTO SOILS

HYDRAULICS OF WATER MOVEMENT OVER AND INTO SOILS

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HYDRAULICS OF WATER MOVEMENT OVER AND INTO SOILS

INTRODUCTION

Surface irrigation methods have two outstanding hydraulic differences from sprinkler and micro irrigation systems. These are:

1. The soil surface serves as a "pipeline" in conveying water along a border strip or furrow. With sprinkler and micro irrigation systems, the water infiltrates into the soil in the close vicinity of the points of exit from the pipeline.
2. Water is applied to the soil at a rate higher than the infiltration rate of the soil. Unlike pressurized irrigation methods, with surface irrigation the application rate and design does not control the depth of infiltrated water. Instead, the soil controls the amount of water which will infiltrate, almost regardless of the rate at which it is applied. The result is that surface irrigators will not know how much of the applied water will infiltrate unless:

a. Some type of volumetric control is used. The best example of this is level furrows or level basins.
   or
b. The irrigators examine the performance of past irrigations, and with that information are able to predict the amount which will infiltrate in the upcoming irrigation.

If these differences are understood, surface irrigation systems can be properly designed and managed for very high irrigation efficiencies.

TERMS

Advance Curve

A graph with a horizontal axis of distance, and vertical axis of time. The curve is a plot of the elapsed time (Time =0 when inflow began) before water advances to each point along a furrow or border strip. See Figure 8.

Advance Ratio (AR)

A term used with furrow irrigation, with varying definitions by various authors. Often it is a ratio of the "Advance Time" to the "Opportunity Time at the tail end of the furrow." In this book, it is referred to as the ratio of "Advance Time" to the "Opportunity Time at the head of the furrow", because most farmers think in terms of the duration of an irrigation set, which is similar to the "Opportunity Time at the head of the furrow." For sloping furrows, the AR can be approximated by:

$$AR = \frac{\text{Advance time to the end of the furrow}}{\text{Total time water is applied to the furrow}}$$

For level furrows,

$$AR = \frac{\text{Advance time to the end of the furrow}}{\text{Total time water is infiltrating at the top of the furrow}}$$

Advance Time
The time of travel from the head of a furrow, border strip, or basin to a point.

Applied Water
Water which is applied to a furrow, border strip, etc. In calculating "applied" water, runoff water which is collected and not lost to the farmer is subtracted from the water which actually flows onto a field. If all water comes from a reservoir and the collected runoff is returned to that reservoir, the "applied water" of an irrigation is the difference in volume in the reservoir before and after irrigation.

Available Water Holding Capacity (AWHC)
The amount of water (inches) which can be stored in the soil for future use by plants. This "soil moisture reservoir" has an upper limit of field capacity, which is the amount of water which can be held against drainage caused by gravity. The lower limit is permanent wilting point, which is the moisture content at which a plant irreversibly wilts.

Basic Intake Rate
The intake rate at a mathematically specified opportunity time. At that time, the rate of change is equal to 10% of the instantaneous intake rate. Rates are determined using double ring cylinder infiltrometers, and are used as part of a general soils classification. It is a widely used, yet rarely understood term which is of little practical value in actually scheduling irrigation durations.

Distribution Uniformity (DU)
Ratio of the "average of the lowest quarter of the depths infiltrated" to the "average of the depths infiltrated across the whole field."

$$DU = \frac{\text{average of the lowest quarter of depths infiltrated}}{\text{average of the depths infiltrated across the whole field}} \times 100$$

Infiltration Rate (also called "Intake Rate")
The rate at which water is entering the soil at a specified opportunity time. The infiltration rate will vary with depth of ponding, soil surface condition, percentage of wetted area, and other factors. It changes every minute during an irrigation, and decreases with time.

Infiltrated Water
Water which has entered the soil from the soil surface. It may be stored in the root zone (for later ET) or pass through the root zone as deep percolation (which may be wasted or be necessary for salt leaching).

Intake Rate
See Infiltration Rate

Lag Time
The time difference between the inflow shutoff time and the recession time at the top end of a furrow or border strip. Water may continue to be ponded at the top for several minutes or hours after shutoff, depending upon the soil, crop, and slope.

MAD (Management Allowed Deficit)

The percentage (%) of the available water holding capacity in the root zone which is depleted at the time of irrigation. This value is selected by a manager in advance of the irrigation.

$$MAD = \frac{SMD \times 100}{AWHC}$$

where SMD = the desired Soil Moisture Depletion (inches) at irrigation
AWHC = Available Water Holding Capacity (inches)

Opportunity Time

The time available for infiltration at any point along a furrow or border strip. For any point,

Opportunity time = (Time the water disappears) - (Time water arrived)

Recession Curve

A graph with a horizontal axis of distance, and vertical axis of time. The curve is a plot of the elapsed time before water recedes from a point. Timing is started (T=O) when water is first applied to the top of a strip or furrow.

Recession Time

The time at which water has receded from a specific point along a furrow, border strip, or basin. Time is as defined in the Recession Curve definition.

SMD - Soil Moisture Deficit (Depletion)

The depth of water required to fill the potential root zone to field capacity at the time of irrigation.

INFILTRATION RATE

The phenomena of water infiltration into soils has captured the interest of numerous researchers and has been the subject of complete conferences. The following aspects make infiltration an intriguing subject:

- Infiltration rates vary during an irrigation.
- Many design strategies proposed for surface irrigation require knowledge of the precise mathematical constants in advance and infiltration equations.
- Each soil has different infiltration and advance rate equations.
- Infiltration and advance rates vary with subsequent irrigations of the same field.
- Laboratory determinations of mathematical constants for the infiltration rates are not the same as unadjusted field results.
- Infiltration rates have traditionally been very difficult to evaluate in the field.

Several formulas have been developed to describe the advance and infiltration rates as a function of time. For an in-depth study of various formulas, one can refer to virtually any issue of the Transactions of the American Society of Agricultural Engineers (ASAE) or the Irrigation and Drainage Journal of the American Society of Civil Engineers (ASCE).

It is important for irrigation designers and managers to understand the concepts of infiltration and advance in order to understand how various design and management alternatives will effect irrigation efficiency. The basic descriptions in this chapter should be sufficient for that purpose.

Kostiakov Equation for Infiltration

The Kostiakov Equation below is commonly used to describe infiltration rates:

$$I = CT^m$$

where, I = Instantaneous intake rate at a point usually expressed as $\frac{in}{hr}$ or $\frac{cm}{hr}$

e,m = Constants

T = Opportunity time at the point

The constants (C, m) in the equation must be determined for every irrigation. On the same soil with the same moisture content, the (C,m) values for furrows will be different than for border strips. There is no reliable and transferrable table of (C,m) values available for different soils under furrow, border strip, etc.

Some other infiltration equations include a third parameter to account for steady state infiltration rate which begins to occur on some soils after relatively long opportunity times. One such equation form is:

$$I = CT^m + b$$

where, b = the steady state infiltration rate after a long time.

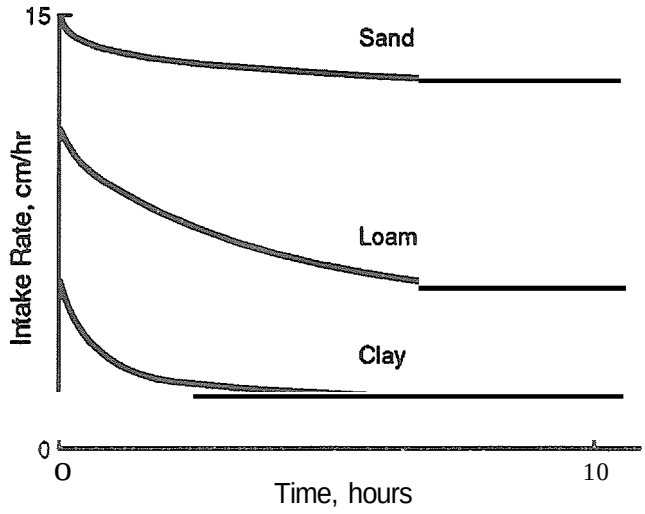


Figure 1. Hypothetical relation of intake rate to time for three soils. Assumes the same percentage of flooded soil surface area.

The actual infiltration rate equation is less interesting to irrigation managers that the equation which defines the depth infiltrated. The most common form of the Depth Infiltrated equation is:

$$D = KTn$$

where, D = the depth infiltrated (usually inches or centimeters)
K = a constant, equivalent to the depth infiltrated at one minute
n = a constant, equivalent to (m+1) where m is from the Kostiakov Equation
T = the opportunity time in minutes

If the constants (K and n) can be determined for a soil and irrigation configuration for a particular event, one can calculate the depth infiltrated at any point if the opportunity time at that point is known.

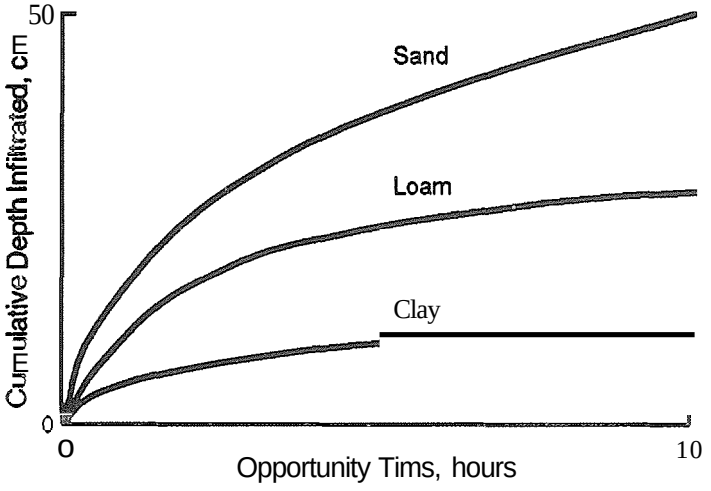


Figure 2. Hypothetical relation between depth infiltrated versus time for three soils. Assumes same percentage of flooded soil surface in all cases.

Factors Affecting the Constants (K and n)

The following factors can cause the intake rate equation for a soil to vary:

1. Method of irrigation. The intake rate equation will be different for furrows than for border strips on the same soil. With border strips all of the soil surface is in contact with the water, so more water will infiltrate in an hour than for furrows.

2. Furrow spacing. Narrow spacings mean more wetted surface area, and therefore a higher intake rate (expressed as in/hr or cm/hr). If the intake rate is expressed as GPMI 100 feet, the intake rate would be approximately the same for different spacings.

3. Depth of water in the furrow or border strip. The depth affects both the pressure of the water (minor effect on infiltration) and for furrows, the wetted surface. A doubling of the wetted perimeter on furrows will almost double the intake rate, according to some studies. However, if the bottom of a furrow is compacted almost all of the infiltration occurs along the edges of the furrow, making water depth vary important as a factor affecting infiltration rate.

4. Furrow shape. Again, the shape affects the wetted area and depth of water.

5. Compaction. Wheel rows (rows down which tractortires always move) have lower intake rates than non-wheel rows.

6. Surface sealing and compaction due to water. Intake rates usually decrease during a growing season on furrows. The furrows begin the season in a loose state and gradually compact due to wetting of subsequent irrigations. On border strip irrigated pasture or alfalfa, there may not be an appreciable difference during the season because the soil surface is not loose at the beginning of the irrigation season due to discing or plowing (as furrows would be). The vegetation also prevents soil surface disturbance and resettlement into compact layers.

7. Tillage. Tillage is frequently used on medium textured soils to break up a surface crust and increase infiltration.

8. Water quality. Low EC_W' high sodium, and high (bi)carbonate concentrations will reduce infiltration rates.

9. Water temperature. Irrigators often notice water "moving back" on a long furrow during a mid-summer afternoon. The infiltration rate increases as the water viscosity increases due to higher temperatures.

10. Adjacent furrows. If alternate furrows are wet, ratherthan every furrow, the intake rate (measured in GPM/foot rather than in.lhr.) will be higher.

11. Cracks. In cracking soils, most ofthe infiltrated water will enter through the cracks and then move laterally underground. The depth infiltrated will depend upon the degree of cracking. In cracking soils the classic equations (e.g., Kostiakov) do not describe the infiltration process. With cracking soils the infiltration rate may be high and constant for a time, and then decrease rapidly to zero. The result is an almost linear advance rate of water across a field on many cracking soils.

12. Initial moisture content. Soils with high initial moisture contents typically have lower infiltration rates than do drier soils.

13. Organic matter and micro-organisms. A high organic matter content tends to increase infiltration. Over-active earthworm populations have been credited for very high intake rates in Idaho.

Importance of Understanding about Infiltration Rates

With sprinklers, systems are usually designed to apply water at less than the minimum intake rate which will occur during the irrigation event. As long as the intake rate is not exceeded, sprinkler irrigation managers can assume that water infiltrates at a constant rate - the application rate. With surface irrigation, the soil controls the intake rate, rather than it being controlled by the hardware as with sprinkler irrigation.

Figure 2 illustrates a fundamental concept of water infiltration into soils. The depth infiltrated versus time of irrigation is shown for three hypothetical soils. There are several observations to be made:

1. A two (2) or six (6) hour irrigation for one soil will not result in the same infiltrated depth as for the other soils.

2. For most soils, the water infiltrates rapidly at first. As time increases, less water infiltrates per unit time (i.e., the intake rate decreases with time).

3. The conclusion must be that no two soils will respond the same with surface irrigation.

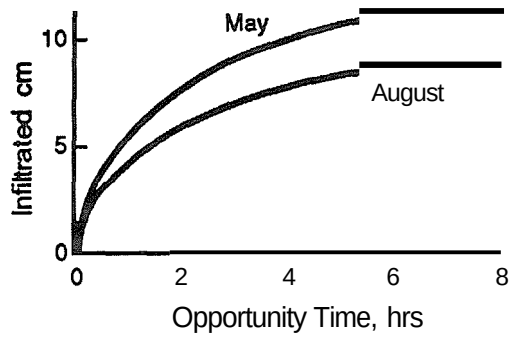


Figure 3. Infiltration for the same soil throughout the growing season.

Figure 3 shows another interesting phenomena which perplexes surface irrigators. Not only does the intake rate change DURING an irrigation, but also BETWEEN irrigations.

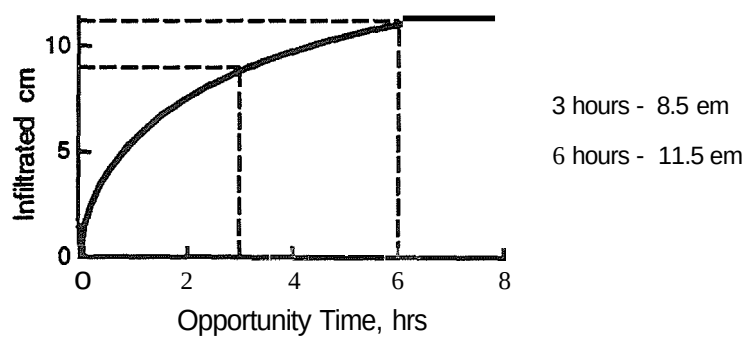


Figure 4. Effect of doubling the opportunity time on the depth infiltrated for a medium, textured soil.

Figure 4 illustrates the concept that by doubling the application time, twice as much water does NOT infiltrate. In other words, over-irrigating by 10% extra time with surface irrigation does not mean a 10% extra deep percolation loss. Instead, the applied water may run off and can be collected and reused. With volumetric control, this does not apply.

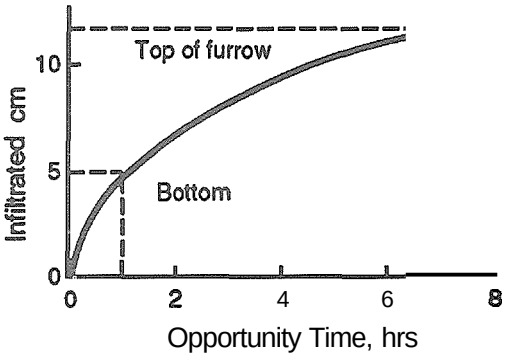


Figure 5. Effect of a poor advance ratio upon distribution uniformity. Water sits at the top (beginning) of a furrow for 7 hours, infiltrating 12 cm. It is shut off soon after advancing to the bottom end of the furrow, providing 1 hour of opportunity time and infiltrating 5 cm at that point.

With surface irrigation, the selected Soil Moisture Deficit (SMD) at the time of irrigation will affect the Distribution Uniformity (DU) for furrows. Considerable water will infiltrate at the top end of a furrow before the water even advances to the lower end, resulting in a poor DU if the purpose is to irrigate a small SMD at the lower end. If the SMD is higher the water must be applied longer and the DU improves.

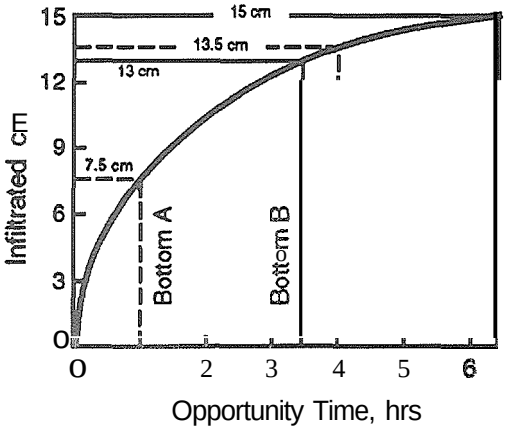


Figure 6. Irrigation of identical furrows at two different SMD's. Both furrows have the same advance time. Furrow "A", with a small SMD, has almost a 100% difference in depth infiltrated at the top and bottom ends of the furrow. Furrow "B", with a larger SMD, has approximately the same depth infiltrated at the top of the furrow (13.5 cm.) as at the bottom end (15 cm.).

Importance of Being able to Estimate Infiltration Rates

Designers and irrigators do not have to know the precise constants of infiltration equations in order to do an effective job. However, there are several questions which frequently arise. They can be answered if the infiltration equation constants are known. Those questions are:

1. What happened? With a level basin or level furrow system, a farmer knows exactly what AVERAGE depth was infiltrated if the onflow rate was measured and timed. However, he doesn't know what depth infiltrated at the head end versus the tail end (the ratio can be estimated, as discussed later in the section on advance ratio).

   With a sloping field, the problem is more fundamental. Most farmers do not measure their runoff (it is usually impractical) and onflow to each set, and do not confine the

applied volume of water to a set of furrows or a border strip. It is often difficult to know how much was applied, much less how much infiltrated on the average, to say nothing about how much infiltrated at different points along the furrow. If the infiltration equation for an irrigation event is known, these questions can be answered by also knowing opportunity times at various points in the field.

2. What is going to happen during this irrigation? If a farmer is planning to irrigate with 12 hour sets (4 hour sets for equivalent border strips) and a certain flow rate per furrow (border strip), will the desired depth of water infiltrate, and will it infiltrate uniformly along the furrow (border strip)? To answer these questions, the infiltration equation constants must be evaluated rapidly before the irrigation is complete. Then appropriate adjustments to flow or set time can be made. As explained earlier, equation constants determined during a prior irrigation would probably not be valid during the subsequent irrigation.

3. How should a new system be designed? This is the hardest question to answer precisely. The infiltration equations can be evaluated before development, but they almost always change (they may increase or decrease) after irrigation.

From a practical design (management) standpoint, it is most important to understand the concepts of infiltration, make some good estimates, and select an appropriate design (management) strategy which is easy to implement and does not require knowledge of complex formulas. A proper design strategy will make the management simple.

Methods of Estimating Infiltration/Intake Rate Equations

Infiltration/Intake measurements must be conducted under conditions which duplicate actual irrigation moisture contents and physical parameters. For valid answers, the measurements must generally be made during an actual irrigation using the proper procedure. Some of the procedures and their applications are described below.

1. Double Ring Cylinder Infiltrometer. This is the most widely used method of determining an intake equation in many areas of the U.S. and abroad. The installation and measurement procedure is well documented in **SCS** and other literature (USDA, 1956). Figure 7 shows the essential installation details. Water is poured into both the buffer and inner rings, and the water level of the inner ring is recorded at various times.

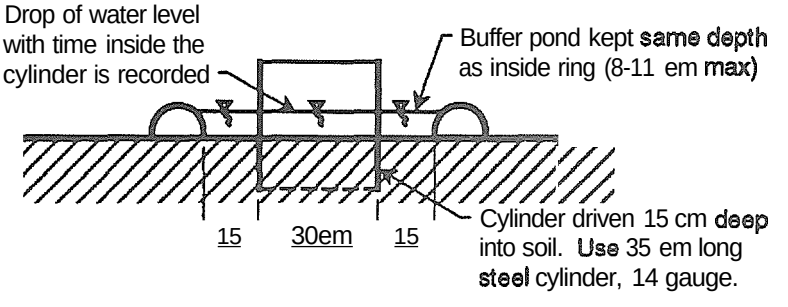


Figure 7. Double ring cylinder infiltrometer installation. Data is useful for a first estimate of intake rates under border strips.

The following characterize cylinder infiltrometers:

a. Because of the buffer ring, they measure one-dimensional infiltration. Therefore, they have intake relationships similar to border strips.
b. The sample area per cylinder is very small, and there are often very large differences between adjacent cylinders. Data from a single ring could not be considered to be representative of a field.
c. When adjacent measurements are "averaged" by various techniques to get a "representative" intake curve, this curve will rarely agree with a volume-balance analysis of a border strip irrigation on the same field. Therefore, the "averaged" curve needs to be further "adjusted" to match actual irrigation conditions.

The time necessary to make double ring cylinder measurements, the variability between cylinder results, and the complexity of adjustments often make this technique impractical for an average evaluator or irrigator to conduct or utilize. However, it is useful in teaching about general concepts such as how intake rates decrease with time, and for getting a sense of magnitude for border strip infiltration. Furthermore, it does give one a good estimate of the exponent of the infiltration rate equation.

2. Furrow Inflow Minus Outflow Measurements. A measure of two dimensional infiltration is needed for furrows. The most common procedure measures flow rates into and out of a 100-200 foot length of furrow. At any time, the difference between inflow and outflow represents infiltration. The procedure is outlined in detail in Merriam and Keller (1978). On some soils this technique gives reasonable results. However, it has the following difficulties:
   a. It is very time consuming, taking approximately 10 man-hours per test or group of furrows.
   b. The sample size is small (one section of one furrow).
   c. Evaluators often leave adjacent furrows dry so they can walk from end to end, resulting in artificially high intake rates.
   d. The measurement devices often dam up the water in the furrow, causing high intake rates. This is especially true if orifice plates are used for flow measurement.
   e. Results must be "adjusted" to match what is observed to actually happen in the field.

3. Volume Balance Techniques. New techniques of estimating infiltration utilize microcomputer programs with input regarding the inflow, advance rates, and sometimes the outflow and recession characteristics of a furrow or border strip. There are several approaches to volume balance, with large differences in the amount of data to collect and mathematical assumptions made. A summary of some of the popular research methods can be found in Elliot and Eisenhauer (1983). The Waterman Program mentioned later is such a technique.

Advance rates reflect soil intake rates. A sandy soil (with a high intake rate) will have a slow advance rate. Therefore, techniques have been developed to indirectly measure intake rates, as a function of furrow advance, onflow rates, and surface storage. The surface storage measurement is the weakest link in this method, as it is difficult to measure. On

low intake rate soils, the surface storage may be a large % of the applied water; therefore a small measurement error of surface storage will destroy the accuracy of the final result. This is not a problem on high intake rate soils.

The basic idea of volume balance is as follows:

a. All measurements are taken during the "advance" phase with furrow irrigation.
b. All the applied water is in one of two destinations during advance- above the ground or in the ground.
c. The applied water volume can be measured. (note - this may be a significant source of error, as this method assumes that the flow rate onto the furrow stays constant; in many farms, the flow rate constantly changes).
d. The surface storage volume can be measured (ie, the volume of water in the furrow which has not yet infiltrated) by knowing the length of wetted furrow and the cross section of the water in the furrow.
e. The unknown quantity, the infiltrated volume along a length of furrow, is:
   Infiltrated volume = Applied - Surface storage.
   The only thing which is unknown is the distribution pattern of this water in the soil. That can be solved for if the infiltration equation is known.
f. There are two unknowns in the infiltrated depth equation, K and n.
g. If volumes infiltrated along two lengths of furrow are known, the two unknowns can be solved for.

The computational procedure of determining K and n is iterative. After the advance equation is obtained (using two points of advance and two times), the depth infiltrated equation is solved for as follows:

$$D = KT_o^n \qquad \text{(Eq. 1)}$$

where

O = cumulative depth infiltrated
K, n = constants
To = opportunity time

and

$$To = T_a - T_x \qquad \text{(Eq.2)}$$

where

T_a = Total time of water application (minutes)
T_X = Time of advance to a distance x

Combining Equations 1 and 2,

$$D_x = K(T_a - T_x)^n \qquad \text{(Eq.3)}$$

Where Ox = Cumulative depth infiltrated at point "x"

There are several forms of advance equations which can be determined by using a "2-point method", in which 2 times and distances of advance are known. The SCS has used the following form:

$$Tx = (xlr)e(xz) \qquad \text{(Eq. 4)}$$

The two unknowns, r and z, are solved for using the 2 measured distances and times of advance.

$$z = (\text{Log}(L2 \times T1) - \text{Log}(L1 \times T2))/(L1-L2) \qquad \text{(Eq.5)}$$

$$r = (L11T1)e(zx\ L1) \qquad \text{(Eq.6)}$$

where

L1 = First distance measured during advance, approximately 1/2 of the furrow length
T1 = Time of advance to distance L1, minutes
L2 = Second distance, near the downstream end of the furrow
T2 = Time of advance to distance L2, minutes

Combining equations 3 and 4,

$$D_x = K[Ta - (xlr)e(xz)]n \qquad \text{(Eq. 7)}$$

This equation can be integrated numerically with respect to furrow distances. It is recommended to divide each advance segment into **20** increments. The use of the trapezoidal rule for numerical integration is sufficient.

The volumes of water infiltrated along the total distance of the two advance distances are:

$$V1 = \int_0^{L1} WX\ K[Ta - (x/r)\ e(xz)]^n\ dx \qquad \text{(Eq.8)}$$

$$V2 = \int_0^{L2} W)(\ K[Ta - (x/r)\ e^{(xz)}]^n\ dx \qquad \text{(Eq.9)}$$

where

W = furrow spacing (L)
V1 = volume infiltrated along distance L1 (LLL)
V2 = volume infiltrated along distance L2 (LLL)

Taking the ratio of the two,

$$\frac{V1}{V2} = \left| \frac{\int_0^{L1} WX\ K[T. - (x/rl.(x,)\ j"\ dx}{\int_0^{L2} W x\ K[T0 - (x/r)\ e^{(xz)}\ l"\ dx} \right| \qquad \text{(Eq.10)}$$

At this point, there are two unknowns, K and n. However, Equation 10 can be further reduced because Wand K cancel out. This leaves only one unknown, m.

$$\frac{V1}{V2} = \left| \frac{\int_0^{L1} [Ta - (x/r)\, e^{(xz)}]^m \, dx}{\int_0^{t-z} [Ta - (x/r)\, e^{(xz)}]^m \, dx} \right| \qquad \text{(Eq.10a)}$$

The values of V1 and V2 can also be found using another set of equations:

$$V1' = Q \times T1 - 51 \qquad \text{(Eq. 11)}$$

where

Q = furrow flow rate (LLUT)
T1 = Advance time to L1 (T)
51 = surface storage along the distance L1 (LLL)

$$V2' = Q \times T2 - 52 \qquad \text{(Eq.12)}$$

The ratio of these two values, is:

$$\frac{V1'}{V2'} = \frac{Q \times T1 - 51}{Q \times T2 - 52} \qquad \text{(Eq.13)}$$

If the values of K and n are correct,

the values of Eq. 13 and Eq. 10a will be equal

$$\frac{V1'}{V2'} = \frac{V1}{V2}$$

The iterative method of solution is as follows:

a. The value of Equation 13 is assumed to be "truth"
b. Equation 10a is solved for using a low assumed value of "n" (.05).
c. The value of Eq. 10 is compared to Eq. 13.
d. If Eq. 10 = Eq. 13, the assumed value of "n" is correct. Iterations stop.
e. If the values are not equal, the iterative solution procedure returns to step (b) with a slightly higher value of "n". (note - "n" is between 0 and 1.0).

Once the correct value of "n" has been determined, equations 9 and 12 are combined to solve for "K"

Most of these techniques suffer from the following field problems:

a. If the inflow to a single furrow varies with time, the results will be incorrect because the computations assume a constant flow rate.
b. The surface storage (water in the furrow, which has not infiltrated) at certain times must be measured. In conditions of very low intake rates, a small error in surface storage measurement will give a very significant error in results. In some cases, programmers assume a certain furrow geometry, but that geometry may not match actual field conditions.

Nevertheless, the sample size (a complete furrow) is much greater than the previous two techniques (rings and inflow-outflow).

An interesting point is that most of the volume-balance computer programs show that as the Kostiakov "K" increases, the n decreases. Ring infiltrometers show the opposite relationship. More research needs to be done to determine which method more accurately characterizes the intake of a soil.

4. Waterman Furrow Evaluation Program. Waterman Industries developed a Furrow Evaluation Program in the 1980's which was one of the first widely used "2 point volume balance" methods. It was a rapid evaluation technique utilizing simple field measurements and a micro-computer to evaluate the present irrigation performance and intake rate equations. The performance under different flow rates, set durations, cutback streams, and field lengths was estimated. As such, it was useful for evaluators in production agriculture. The infiltration and advance functions were included on a separate output page for use in research work. The program estimated the "K" and "n" from the advance and volume measurements of a single furrow.

5. AGWATER approach. Based upon the review of extensive work with the Waterman Furrow Evaluation Program and of research by others, a very simplified approach for estimation of infiltration rate equations was developed to be used in the "AGWATER" expert system (Burt and Parrish, 1991) which was developed for training and evaluation of irrigation methods. The logic looks at what happened during the irrigation of a whole field, not just a set or single furrow:

. Average Depth Infiltrated onto a Field (AD)
= Average Depth applied to head of Field - Runoff from the Field

This AD is adjusted to a larger number because only a percentage of the soil surface was covered by water. The adjusted depth is JD.

$$JD = \frac{AD}{\text{fraction of surface area covered by water}}$$

- The average opportunity time for all spots in the field is known from advance and recession data. This gives an average time, TA
- There is a relationship between uK" and "n" for a flooded surface. Some values for the approximate relationship are:

K	n
6.68	.74
4.0	.68
2.62	.61
1.87	.55
1.47	.48
1.25	.42
1.17	.36

- An initial combination of"K" and "n" are used in an iterative process where "Value" is defined as:

  Value = K x (TA)n

  Where TA is in hours

  Value is in inches

- When the "Value" is equivalent to JD, the appropriate K and n values have been determined.
- If the soil is a cracking clay, it is assumed that "n" equals 0.1

6. Other Methods. Researchers continue to develop other techniques for measurement of border strip and furrow infiltration, with varying degrees of accuracy, complexity, and levels of assumption by the evaluator. Recent issues of the Soil and Water Division of the American Society of Agricultural Engineers (both Transactions and Conference Proceedings) are the best source of current developments. An easy, rapid, accurate, and widely applicable method of estimating the infiltration rate for surface irrigation still eludes the industry.

ADVANCE RATES

Advance Rate Formulas

Several mathematical formulas have been used to describe the advance rate as a function of time (ie, the advance curve). The following are the most commonly used:

$I = aT^r$ Christiansen et. al. (1966)

$T = \left(\frac{L}{C}\right)^{e^{LZ}}$ Collins & Koluvek (1981)

$T = b\,(e^{wI} - 1)$ Davis (1961)

where, T = advance time to a distance I

C,Z,w,b,a, and r = empirical constants

The following conditions contribute to a fast advance with surface irrigation:

1. Large flow rate per foot of border strip width (same as a large flow rate per furrow).
2. Low intake soils.
3. Smooth soil surface.
4. Used furrows (furrows which have conveyed water at least once before, without tillage since then).
5. Steep field slopes.
6. Deep 'Vee" shaped furrows rather than shallow wide furrows."

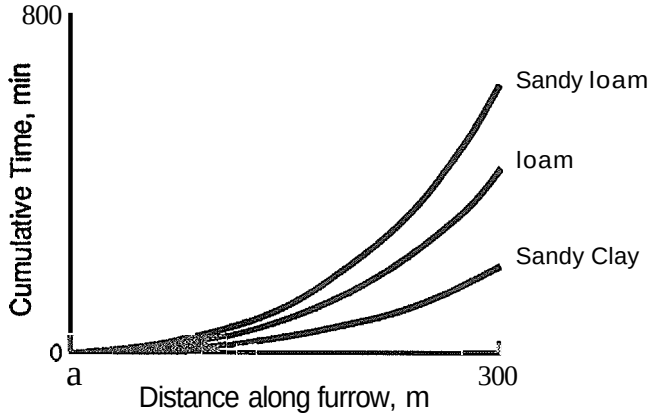


Figure 8. Relative variation of advance rates due to soil texture variation. Same flow rate, slope.

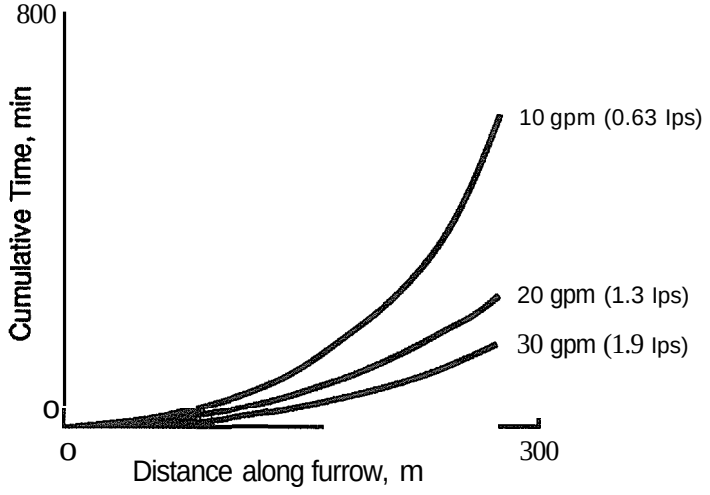


Figure 9. Effect of different flow rates upon advance times. Simulated data on loam soil, slope =.002

RECESSION RATES

The rate of recession is important when water remains on the soil surface long enough to infiltrate a significant depth of water after the inflow is shut off. Recession is quite rapid on most sloping furrows, but very slow on level furrows. It is of major importance in the management of border strips.

With furrows the time of irrigation is long (usually 8-24 hours per set) and the time necessary for water to recede from the soil surface after inflow shutoff is small (20-60 minutes). At

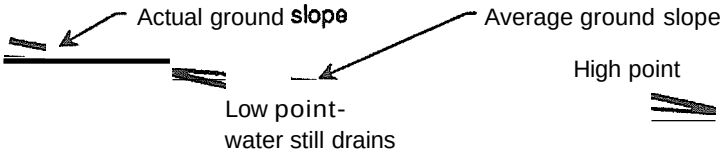
the end of a furrow irrigation, this extra opportunity time does not usually result in a significant extra depth infiltrated.

Infiltration rates under border strips are much higher than furrows, and therefore the irrigation durations are considerably less. At the time of inflow shutoff an appreciable percentage of the applied water is in surface storage, and must drain off or infiltrate. Because intake rates are high at this time, the extra depths infiltrated during the recession phase can sometimes represent the majority of the irrigation on parts of the field.
The following conditions contribute to a fast recession:

1. High intake rate
2. Smooth soil surface
3. Sparse vegetation
4. Steep slope

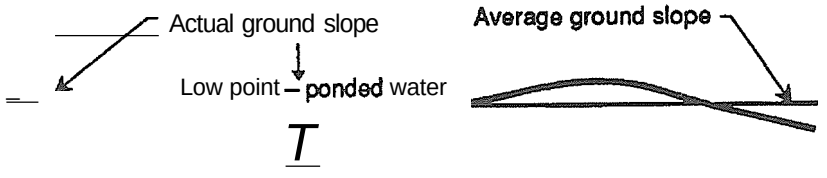
The quality of land grading will affect the recession characteristics to varying degrees, depending upon the surface irrigation method. Waterwill run offsloping furrows and border strips even if there are minor inconsistencies in grade down the run. Inconsistencies in slope across the run are of less importance (except within a single border strip).

For level surface irrigation methods, minor inconsistencies in land grading can create major differences in recession time and depth infiltrated in the field.



Case (a). Sloping furrow. Water drains from complete furrow length despite slight uneveness of land grading. Result: no difference in infiltrated water depths along the furrow caused by slight uneveness.

Figure 10. Effect of slight non-uniformities of land grading on sloping furrows.



Case (b). level furrow Uneveness of land grading causes water to pond in some spots and drain quickly from others. Result: large non-uniformity of water infiltration.

Figure 11. Effect of slight non-uniformities of land grading on surface drainage and distribution uniformity with level furrows.

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CHAPTER 13



IRRIGATION EFFICIENCY

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IRRIGATION EFFICIENCY

General

There are in fact many expressions of "uniformity" and "efficiency", and each expression has a specific mathematical equation. Furthermore, there are specific procedures to use in obtaining the values to be placed into the equations. Any discussion of "efficiency" or "uniformity" must first reach an agreement upon the definitions to be used; failure to this results in different values for the same situation and confusion.

This chapter will use three basic terms to describe irrigation performance:

1. Distribution Uniformity (DU)
2. Irrigation Efficiency (IE)
3. Water Use Efficiency (WUE)

For all three terms, the following must always be specified:

1. The area underconsideration. Forexample, the DU ofwater throughout an irrigation field is lower than along a single furrow, because of additional items causing non-uniformity throughout the field such as differences between individual furrow intake rates. Field irrigation efficiencies cannot be confused with irrigation district irrigation efficiencies or basin irrigation efficiencies.
2. The time under consideration. For example, the irrigation efficiency of a single irrigation event may be higher or lower than a seasonal irrigation efficiency.

There is no problem in comparing the DU and IE of various irrigation methods (drip, sprinkler, surface) if calculations are done properly. Many published comparisons have been inaccurate because they have inadequately defined the performance of all methods in a comparison.

Distribution Uniformity

Definition

The "Distribution Uniformity" describes how evenly water is made available to plants throughout a field. Distribution Uniformity (DU) is defined (Kruse, 1978) as:

$$DU = \frac{\text{Minimum depth infiltrated}}{\text{Average depth infiltrated}} \times 100$$

where the "minimum" is the average depth infiltrated in the region of the field receiving the lowest 25% of the water. Although there have been many mathematical expressions of uniformity developed overtheyears, the definition ofDU above is nowstandard in California (Burt, et al. 1991). Values such as "Christiansen's Coefficient of Uniformity" are calculated differently; for example, Christiansen's Uniformity Coefficient (UC) produces a higher uniformity numberwith the same data. The major difference between reported DU values, however, is that most reports only look at one of the many uniformity components which must be measured to give a field DU.

Ideally, irrigation methods should apply water uniformly to each plant in a field (assuming that plants are evenly spaced, which is generally true except in orchards). Such a method

would have a DU of 100%. In fact, no system is capable of applying water so that every plant in a field receives the same amount of water. For all methods, some points in the field are always over and/or under-irrigated. A low DU results in water and energy wastage because excess water must be pumped onto a field to apply enough water at the dry points.

Graphical Depiction of DU

The following two-dimensional sketches present the ideas of uniformity and efficiency. They represent depths of water which have infiltrated into various areas of a field, and do not include other things such as evaporation or uncollected runoff.

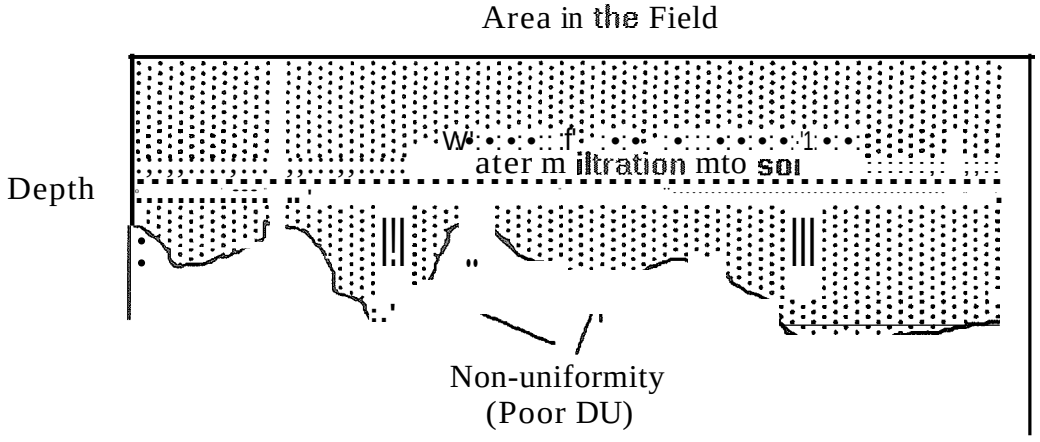


Figure 1. The concept of non-uniformity. Regardless of the irrigation method, there are differences in the amount infiltrated. Some sprinklers and emitters are always partially plugged and have pressure differences. Water sits at the head end of a furrow longer than at the tail end. These are just a few of the many causes of non-uniformity.

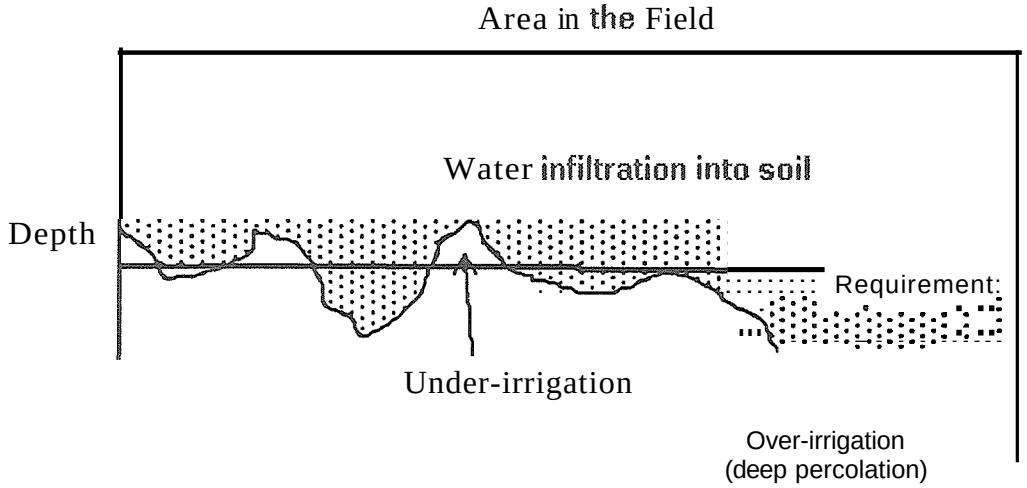


Figure 2. The concept of adequacy of irrigation. In the case shown above, some of the field receives enough water and some is under-irrigated. The deep percolation is "lost" to that field, along with nutrients which have leached down with the water.

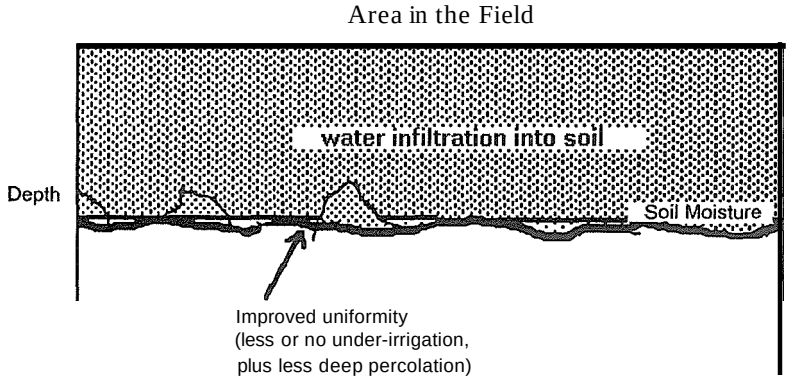


Figure 3. The same amount of water as for Figure 2, but now with a better DU. The result is less or no under-irrigation, plus less deep percolation.

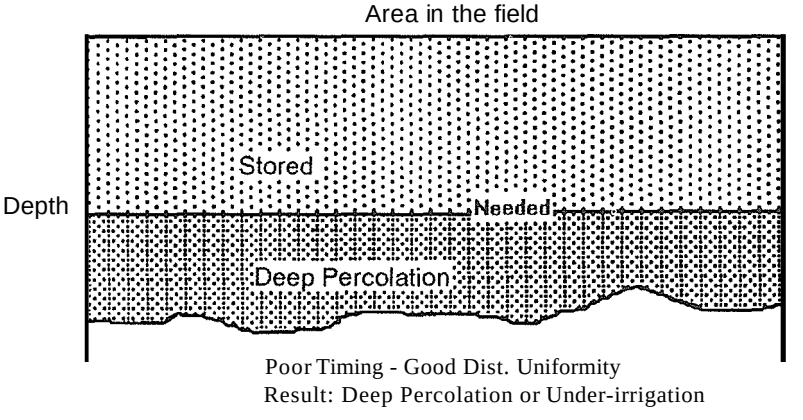


Figure 4. A good DU, but the water was not shut off at the correct time, resulting in large amounts of deep percolation.

Figure 4 points out the importance of the two major items affecting Irrigation Efficiency (discussed later in more detail):

1. DU - in this case, it is good
2. Timing - in this case, it is bad.

BOTH items need to be addressed in order to obtain a good irrigation efficiency and no under-irrigation.

The economics of irrigation system design may dictate that less than 100% of the area be adequately irrigated. Where the average of the low quarter is equal to the desired depth of application, approximately 12% of the area of an irrigated field will be under-irrigated.

Most designers believe that a uniformity of 75% is quite good. Figure 5 illustrates that a DU of 75% really means that one spot on the field will receive twice as much water as another spot.

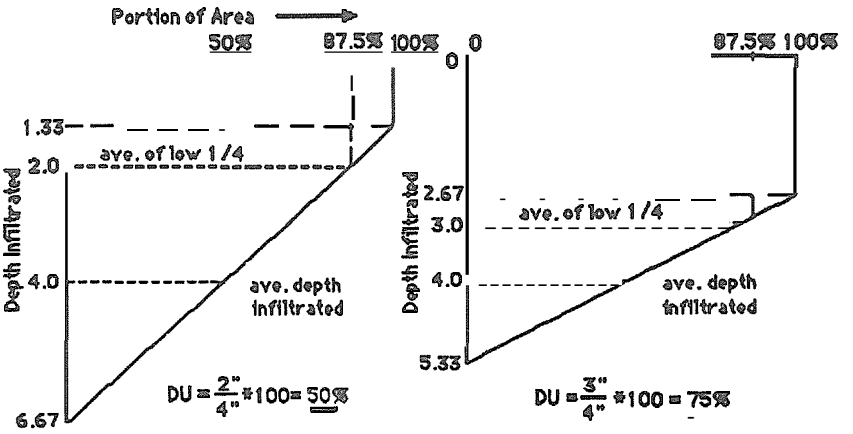


Figure 5. Simplified infiltration pattern with two DU's. Same average depth infiltrated in both cases This could be for any irrigation method.

The DU is Not a Measure of Efficiency, because it does not quantify beneficial use or even deal with non-infiltrated water. However, a high irrigation efficiency with a fully irrigated field is only possible if there is a high DU.

Potential DU

An item of much interest is the value of potential uniformities of various irrigation systems. Table 1 provides such estimates.

Table 1. Potential Field DU's for Moderately Well Designed Irrigation Systems.

Irrigation Method	Potential Field DU
Permanent Undertree Sprinkler	94
Linear Move	92
Orchard Drip	90
Sloping Furrow	89
Level Furrow	87
Border Strip	85
Row Crop Drip	90
Hand Move Sprinkler *(w/alternate* sets)	85
Hand Move Sprinkler *(w/o* alternate sets)	75

Various persons will assign slightly different numbers in tables such as Table 1. The important point is that "Actual" DU's are almost always lower than the "Potential" values. Uniformities under field conditions depend upon:

1. Irrigation system design. Systems must be designed and installed properly.
2. Suitability. Each system has a particular set of conditions under which it will operate satisfactorily. For example, furrows are not suited for sandy, non-homogeneous soils. Drip is not suitable for areas with large rodent populations because the rodents eat the hose.
3. Management. As an example, large furrow flow rates are needed to attain a good advance ratio for furrows.

Field DU vs. Catch Can DU vs. Single Furrow DU vs. New Emitter DU

As mentioned earlier, much confusion has arisen because many published values of DU have been based upon incomplete assessments. In general, as more items of uniformity are considered, the DU value for any particularsystem falls. Table 2 lists the most important components of DU for various irrigation methods in agriculture. All of those factors must be considered in order to calculate the field, or system, DU.

Table 2. Components of Irrigation System DU for Various Irrigation Methods.

Method	DU Components	Factors Within Each Component
Drip	Flow rate variation	Pressure differences down a hose
		Pressure differences between hoses
		Plugging
		Manufacturing variation
		Leaks
		Emitter part wear
	Unequal drainage	Low spots may drain for hours
	Variable tree and emitter spacing	Number of emitters/tree in various blocks of trees
Undertree Sprinkler	Flow rate variation	Pressure variations along lateral
		Pressure variations between laterals
		Nozzle plugging
		Nozzle wear (sand)
		Different nozzle sizes
	Unequal drainage	
	Variable tree and sprinkler spacing	
	•• Catch can uniformity is NOT a component if the sprinklers are reasonably spaced, on a mature orchard	
Hand Move Sprinkler	Catch can uniformity	Nozzle pressure
		Sprinkler spacing
		Wind, angle of nozzle
	Flow rate difference	Pressure variations along lateral
		Pressure variations between laterals
		Nozzle plugging
		Nozzel wear (sand)
		Different nozzle sizes
	Leaks	
	Unequal drainage	

chart continued on following page

Method	DU Components	Factors Within Each Component
Sloping Furrow	Opportunity time differences	Advance time ratio for a single furrow • flow rate • furrow length • soil intake characteristics • slope ·smd
		Set time differences
		Uneven land grading
	Soil variation	
	Flow rate variation	Different flow rates to various sets
	Furrow differences	Wheel row vs. non-wheel row

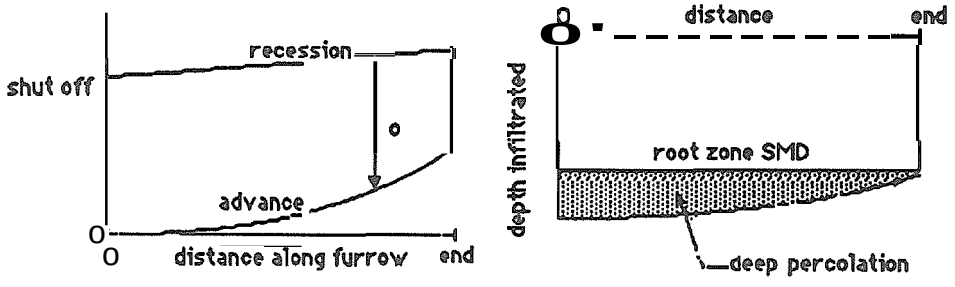
Detailed procedures for estimating the various components of DU for each system are contained in an Irrigation System Evaluation Manual by Burt, et al. (1991). Three day short courses are offered several times annually at Cal Poly to train irrigation specialists in these techniques, and software is provided for computations and output.

In summary, several key concepts must be defined before any discussion of irrigation DU can be meaningful:

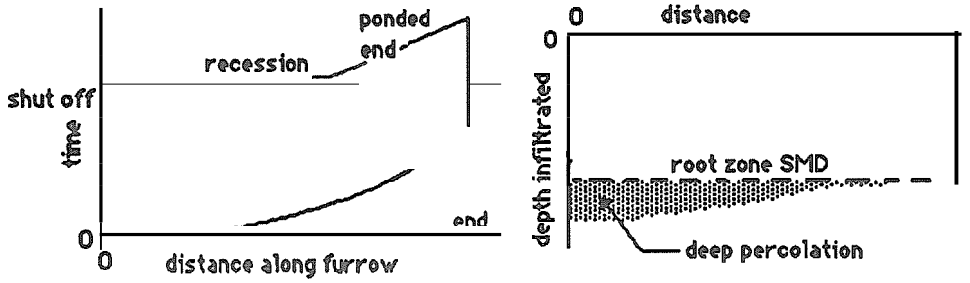
1. The equation of DU must be defined.
2. The DU must be a FIELD DU, not just the DU along a single furrow or for the overlap between 4 sprinklers.
3. Actual DU's after a few years operation are very different from "Potential DU's.
4. The concept of DU is actually a concept of the evenness of application to each plant.

Surface Irrigation DU

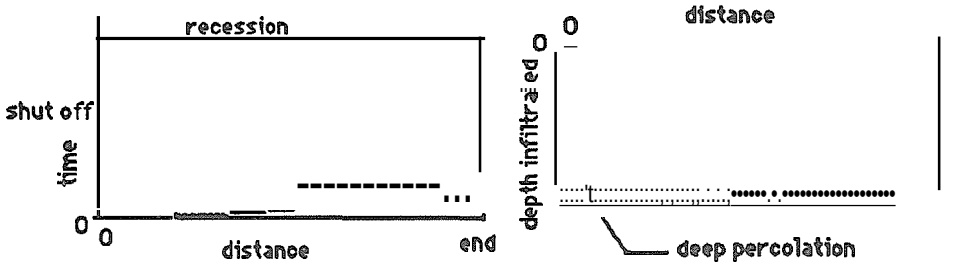
In most conditions of (i) uniform soil with (ii) good land grading, the distribution uniformity of water infiltration in a surface irrigated field is primarily dependent upon the uniformity of Opportunity Times. "Opportunity Time" refers to the amount of time water sits on the soil surface at a point. Figure 6 illustrates possible advance-recession relationships and the corresponding DU's along single furrows or border strips with uniform soils.



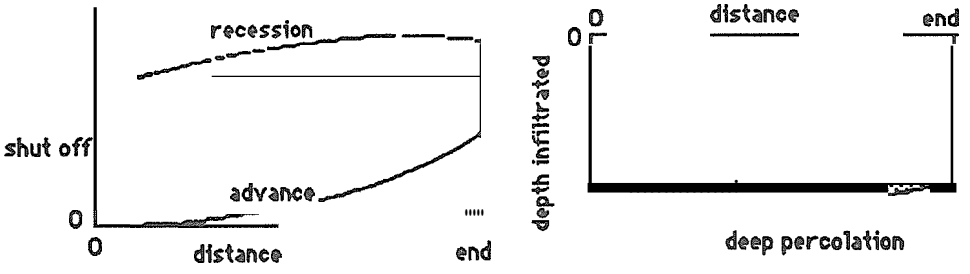
(a) Sloping furrow with runoff.



(b) Sloping furrow with blocked end, preventing runoff. Lower flow rate than case (a).



(c) Level furrow. No runoff. Water is applied, then shut off and ponded until it infiltrates. Same *flow rate* as *case (a), shorter field length.*



(d) Graded border strip. Runoff allowed.

Figure 6 *(a)-(d). Advance-recession and DU graphs of various surface irrigation methods. DU's are for* a *single furrow or border strip, and represent the effect of differences of opportunity times at various points along the furrow/strip.* A *field DU must also include soil and set time differences.*

For both level and sloping furrow systems, high uniformities (greater than 85%) require a reasonably small Advance Ratio. This gives similar Opportunity Times at both ends of the furrow. A rule of thumb is to attempt for an advance ratio of 1/3-1/2. Faster advances give better uniformity, but more runoff on sloping systems.

For border strips a fast advance may not be desirable. The optimum advance rate is determined by the shape of the recession curve. For maximum uniformity, the onfJow rate must produce an advance curve which will approximately parallel the recession curve.

Irrigation Efficiency

Definition. Irrigation Efficiency (IE) is defined as the percentage of the irrigation water which is beneficially used in the application area.

$$IE = \frac{\text{Irrigation Water Beneficially Used}}{\text{Irrigation Water Applied}} \times 100$$

Note that the word "Irrigation" is in both the numerator and denominator. This avoids the possibility that there can be an IE value of greater than 100%, which commonly occurs when the value for "crop evapotranspiration" is used instead of "irrigation water beneficially used". The error in this instance generally arises from one or more of the following situations:

1. The crop evapotranspiration is calculated from a theoretical unstressed crop situation, whereas the actual crop is stressed and has a lower evapotranspiration.
2. The crop evapotranspiration has been estimated correctly, but part of the water for that evapotranspiration came from:
   a. A high water table supplied externally, or
   b. Effective rainfall, or
   c. Change in soil moisture storage throughout the time period.

Beneficial Use

Water applied by irrigation has many possible destinations. The water may go into the air (evaporation, spray loss, or transpiration), be stored in the soil root zone, run off the surface of a field, or exit the root zone as deep percolation. This deep percolation can contribute to high water tables and/or recharge streams and ground water aquifers. The deep percolation can be either beneficial or harmful to neighboring farms. For on-farm irrigation efficiency definitions, the following table defines the major forms of "beneficial" vs. "non-beneficial" water uses.

Table 3. Beneficial and Non-Beneficial Uses of Water for On-Farm Irrigation

Beneficial Uses	Non-Beneficial Uses
Crop transpiration (**major use)	Weed Transpiration
Leaching for salinity control	Deep percolation in excess of leaching requirement
Special Practices: * packing the soil for harvest * weed germination * climate control	Evaporation from wet soil surfaces
	Evaporation from wet foliage
	Canal and pipe seepage
	Spray losses

There is ample justification for improving low on-farm irrigation efficiencies. Low on-farm irrigation efficiencies can result in:

1. Excess pumping
2. Fertilizer leaching

3. Low crop yields
4. Water quality degradation
5. Drainage problems
6. Excess water costs
7. Reduction in the acreage which can be irrigated with a fixed volume of water available on a farm

In agriculture, many farmers are not yet convinced that improved irrigation efficiencies will result in higher paybacks, and therefore they are often notwilling to make further investment in improved water management. In fact, many farmers are able to offset poor irrigation efficiencies with increased applications of fertilizers and chemicals. However, society and environmental organizations have begun to demand higher irrigation efficiencies in the hopes of reducing ground and surface water quality problems and drainage problems, and to maintain higher in-stream flows for fish by reducing irrigation water diversions in natural rivers.

Reasonable vs. Beneficial Use

it is unrealisticto expect all irrigation waterto be beneficially used, due to physical limitations such as imprecise uniformities, unexpected rainfall, uncertainties ofthe exact soil moisture deficit, and unavoidable evaporation losses. Furthermore, there are economic constraints on farmers which limittheircapabilities to invest time and/ormoney. Therefore, water rights allocations by courts generally consider both "reasonable" and "beneficial" uses of water. Courts may decide upon a value for irrigation efficiency which can be "reasonably" expected of farmers in a specified combination of climate, economics, topography, water quality, etc.

Spray and Evaporation Losses

Under normal irrigation practices, spray and evaporation losses constitute a rather small percentage of the total applied water. Typical values are given in Table 4.

Table 4. Typical Annual Spray and Evaporation (wet soil and plant surface) Losses for Various Irrigation Methods.

Irrigation Method	Spray Losses, %	Plant Leaf and Soil Surface Evaporation Losses, %
Hand Move Sprinkler	4-6 (depends upon nozzle size, pressure, wind speed, and ET rate)	6-30 (depends upon irrigation frequency, nozzle size & P, adjacent sprinklers, and ET rate)
Micro-spray	1	4-25 (depends upon irrigation frequency, nozzle size & P, and ET rate)
OrchardNineyard Drip	0	0-4
Undertree Sprinkler	1	3-20 (depends upon irrigation frequency, nozzle size & P, adjacent sprinklers, and ET rate)
Surface (not rice)	0	0-6 (depends upon canopy cover, irrigation frequency, and ET rate)

Water Destination Diagrams

Figure 7 illustrates an irrigation in which the applied water has the following destinations:

1. Evaporation losses.
2. Runoff (which may or may not be lost).
3. Root zone storage.
4. Deep percolation, below the root zone.

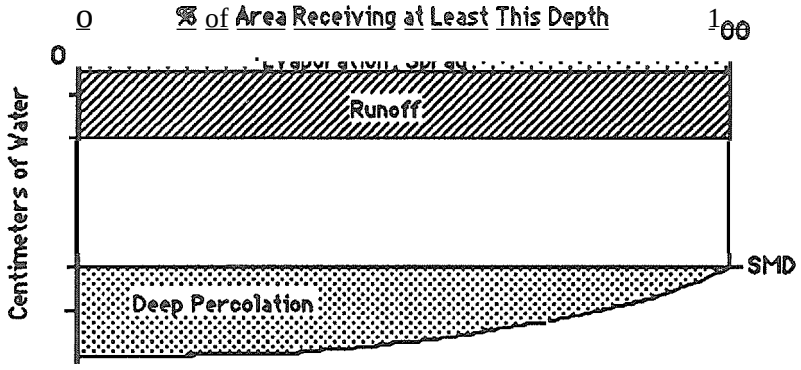


*Figure 7. Destination of applied water for a case of perfect **irrigation timing**. The "stored" and "deep percolation" water represents water which infiltrated into the soil. All of the field is adequately irrigated. All deep **percolation is** due **to non-uniformity.***

Figure 7 illustrates the fact that if no spot in a field is under-irrigated, then all the field is over-irrigated. Figure 8 illustrates a situation of over-irrigation at the driest point due to improper timing. The sprinkler, drip, or furrow set length was too long.

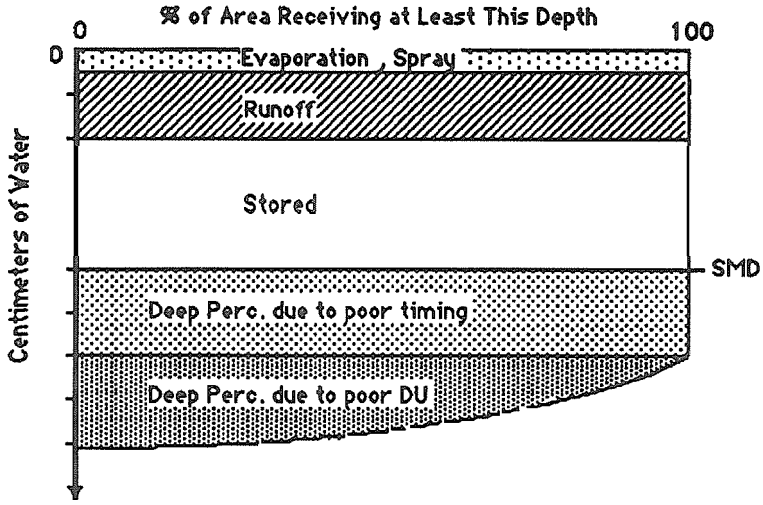


Figure 8. Destination of applied water for a case of over-irrigation. Deep percolation is due to both poor timing and non-uniformity.

Figure 9 shows a different concept. Some farmers believe that they have no deep percolation losses if the average depth infiltrated equals the soil moisture deficit. As can be seen, this would result in both deep percolation and under-irrigation.

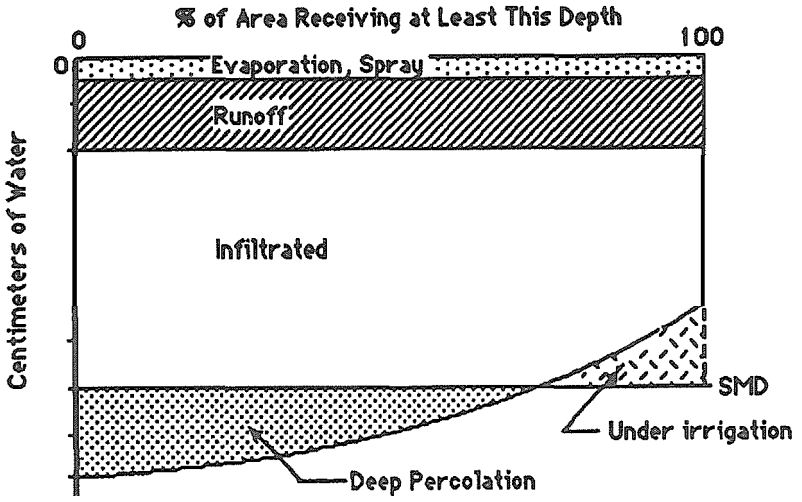


Figure 9. Destination of applied water for the case *in which the average infiltrated water depth exactly equals the* SMD.

Figure 10 illustrates a situation which may be possible only on buried drip systems after the crop has been germinated by another system. There is no runoff or evaporation loss from the dry soil surface at this time. In this case, a 100% Irrigation Efficiency can be achieved by under-irrigating the complete field. All the water will be used by the crop. The problem with this under-irrigation approach is that the yields are unacceptably low, and salinity problems would develop.

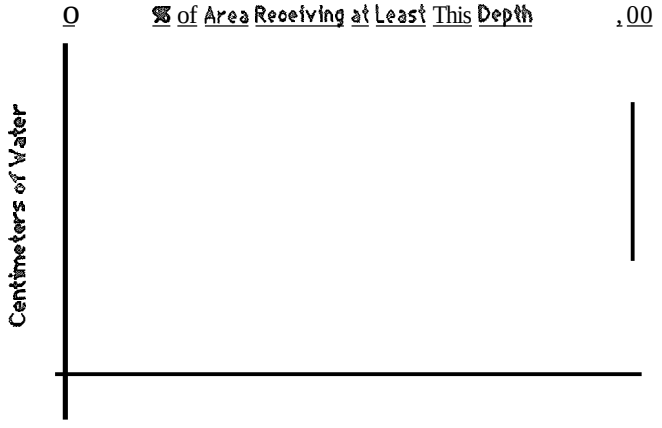


Figure 10. Destination of applied water in a case *of 100% irrigation efficiency. Dry soil surface, buried water supply, under-irrigation of all the field except one point.*

Comparing On-Farm IE with DU

On-farm irrigation efficiency can exceed, equal, or be less than the DU. The following are guidelines for comparison:

1. If IE is greater than DU, there is under-irrigation. A very high IE indicates that almost all of the applied irrigation water is beneficially used, which can only be possible if there is almost no deep percolation (see Fig.1 0).
2. IE can approximate DU if surface runoff is collected and recylced, if evaporation losses are minimal, and if there is excellent irrigation timing.

3. IE is lower than DU if set durations are excessively long, resulting in deep percolation at the spot in the field which receives the least amount of water. It can also occur with surface irrigation if there is uncollected runoff (see Fig. 8).

Fertilizer leaching

Water which percolates below the root zone (leaches) carries salt with it. Some leaching is necessary to remove the salts which are brought in by the irrigation water itself. However, the excessive leaching caused by non-uniformity and poor irrigation scheduling results in heavy fertilizer losses. This fertilizer later shows up in the groundwater, and is beginning to cause serious concerns throughout the U.S.

Nitrogen fertilizers are particularly susceptible to leaching loss. In addition to the ground water pollution aspect, this is an important energy conservation concern. One ton of Nitrogen fertilizer requires approximately 42 Million BTU's for manufacture. It has been estimated that only 54% of the applied Nitrogen in the San Joaquin Valley in California is used by the plants. The remainder is either lost in leaching or volatilization.

Runoff and Irrigation Efficiency

Runoff (tailwater) with surface irrigation systems does NOT decrease efficiencies if it is collected for reuse later somewhere on the farm. Runoff does not have to be recycled on the same field or irrigation set in order to qualify as "beneficial use." Runoff (which is collected) is a sign of good management in arid areas, because the existence oftailwater indicates a good advance ratio and good DU.

Obtaining a Good Irrigation Efficiency

In summary, a good Irrigation Efficiency can be obtained by:

1. Achieving a good Distribution Uniformity
2. Proper scheduling of irrigations (when and how much)
3. Collecting runoff
4. Minimizing evaporation and spray losses (**Except for some moving systems such as linear moves, or fast cycling systems such as solid set sprinklers, this may be a minor concern relative to the other factors).

Water Use Efficiency (WUE)

Irrigation efficiency is only one part of the total picture in improving yields. A farmer must also plant at the right time and at the right depth, manage labor properly, apply pesticides in a timely and proper fashion, and must do numerous other things correctly in orderto have high yields.

The term "Water Use Efficiency" was developed to assess the impact of irrigation water application on crop yield.

$$\text{Water Use Efficiency} = \frac{\text{Crop Yield}}{\text{Irrigation Water Applied}} \times 100$$

There is strong rationale behind using such a term. For example, a cotton crop with a yield of 1.5 bales/acre may have about the same evapotranspiration as one with a yield of 3.0 bales/acre. In fact, the lower yielding field may actually have a higher evapotranspiration, but as a result the plant may have produced excess vegetation at the expense of cotton bolls. The Irrigation Efficiency might be identical on both fields, but the WUE on the 3.0 bale field would be double that of the 1.5 bale field. In terms of product per amount of natural resource actually consumed, the 3.0 bale field is best.

In the future, we can expect to see the demand for both high WUE and IE, as the competition for water increases and the environmental concerns are better understood.

Other Aspects of Efficiency

Some of the other aspects of efficiency include return on investment, adaptability to existing social patterns, dependability of operation, etc. These are generally much more important to the success of a project than a simple measure of "Irrigation Efficiency" or "Water Use Efficiency" as defined earlier.

On some irrigation projects the limiting factor may be something as simple as the ability to obtain spare parts for the system. Table 5 was developed to address such factors affecting irrigation efficiency.

It can be seen that all irrigation systems have merits and disadvantages, and special designs and considerations always provide exceptions to the rule. Certainly there is no such thing as a perfect irrigation system. For each situation, the proper irrigation system should be selected after an investigation of the factors affecting efficiency, cost, and manageability.

Table 5. Factors affecting irrigation, power, and operating efficiency.

Item	Level Furrow	Sloping Furrow *w/* Return Flow	IRRIGATION METHOD			
			Sprinkler			Drip
			Linear Move	Center Pivot	Hand Move	
Dirt in water	No effect	No effect	Sand separator and/or filter needed. Extra system pressure necessary =3 - 10 psi.			Extremely sensitive filtration; chemical injection needed
Max. Slope	Level	up to 3%	4%	15%	15%	Anything farmable
Min. economic Field size, ha.	No Miimum	4	100	50	1	2
Field Shape	Any	Any	Long and Rectangular	Circular or Modified Circle	Any, but major edge effects on odd shapes	Any
Energy for installation & materials	Medium	Mod. Low	Mod. High	Medium	Mod. High	Mod. High
Annual pumping $	None-Very low	Low	Medium	Mod. High	High	Mod. High
Technical expertise needed by farmer	Medium	Medium	Very High	High	Low	High

continued on next page

Item	Level Furrow	Sloping Furrow *wl* Retum Flow	IRRIGATION METHOD			
			Sprinkler			Drip
			Linear Move	Center Pivot	Hand Move	
Relative # of *moving* or complex parts (1=few, 10=many)	1	2	6	5	3	10
High salt water	Special bed shapes required, highest sensitivity during germination & seedling stages		Toxicity and leaf bum after plant has leaves. Good for germination		More serious problems than linear *move* or pivots	Best, but need leaching by other means *every 5-6* years in arid areas
Non-uniform Soils	Sensitive		Non-sensitive			
Inflexible water delivery system	Moderately Tolerant		Very Sensitive	Sensitive	Moderately Tolerant	Sensitive
Min. SMD, em.	4-7	2.5 -7	0.3	0.3	2.5	0.3
Fertigation ease	Good	Good	Excellent		Good	Excellent

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CHAPTER 14



IRRIGATION EVALUATIONS

IRRIGATION EVALUATIONS

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IRRIGATION EVALUATIONS

Introduction

Irrigation system evaluations are quite varied with respect to the types of measurements made, conclusions reached, and objectives. Common variations are outlined in the tables below.

Table 1. Classification of irrigation evaluations by time and area.

Category	Sub-Category	Comments
Time Period	Single Event	- The evaluation of one irrigation event will provide information about design problems, misadjusted valves, DU, and management constraints. It will not provide annual irrigation efficiency values or annual cost information.
	Annual Performance	- Irrigation costs may be accurately estimated only if the total annual water use, ET requirement, and power costs are examined. Annual irrigation efficiency will be different from a single event efficiency.
Area	Single Field	- If management and design conditions are similar throughout a farm the evaluation of a single field can be valuable in extrapolating recommendations to a larger area. Runoff losses from a single field should not be considered as losses if the water is used on other parts of the farm.
	Farm	- It may be difficult to identify how much water goes to each field on a farm, so the water balance on the complete farm may be needed to estimate irrigation efficiency.
	Hydrologic Basin	- In arid regions the irrigation efficiency within a hydrologic basin is often higher than on individual farms or fields. Water quality and drainage problems which are caused by inefficient on-farm irrigation may show up in areas downstream of farms with inefficient on-farm irrigation. Canal leakage and spills, river flow, etc. must be included. Water rights adjudication should always consider hydrologic basin water balances.

Evaluations need to be focused on an objective. There is such a large number offactors which can be measured that an evaluation procedure which arbitrarily measures various factors may be worthless or misleading.

As a different example, an evaluation designed to collect statistics for a government study will concentrate on mathematical accuracy, correct sample sizes, and detailed measurement of certain items. It may not collect the type of data needed to tell the farmer how to correct problems.

Table 2. Classification of on-farm irrigation evaluations by motivation.

Motivation	Sub-Category
Determination of water rights	
Obtaining statistics regarding present performance	
Water conservation	- Water is expensive, or - Water is scarce, or - Society problems from over-irrigation such as: - Poor drainage in an area - Reduced water quality
Improvement of crop yields or quality	
Reducing other irrigation-related costs	- Fertilizer - Labor - Tile drainage
Energy conservation	- Energy is expensive, or - Energy is scarce

Simple undefined terms such as "efficiency" can be misinterpreted. A case in point is a recent California election issue in which urban debaters quoted low on-farm irrigation efficiency values to substantiate an argument regarding the water balance in a basin. On-farm efficiencies (measured as water applied vs. water used by the crop) are very low in some areas for economically justifiable reasons. For example, they may be 50% if runoff is not collected. However, if that runoff is used downstream by another farmer or the same farmer on another field, the basin efficiency may exceed 90%. Therefore, the arguments of the debaters should have been confined to facts regarding basin, not on-farm water balances.

Low on-farm irrigation efficiencies are not justified by high basin efficiencies. In most cases, good on-farm irrigation efficiency results in a healthier environment, higher crop yields, and lower expenses. A properly designed and conducted irrigation evaluation is an excellent tool to improve on-farm irrigation efficiencies.

This chapter will concentrate on on-farm evaluation aspects which interest the farmer and irrigation dealer. Therefore, the focus will be on how to identify factors and solutions which affect farming costs.

Facts Needed to Determine an On-Farm Irrigation Efficiency

A good estimation of the irrigation efficiency on a typical 65 hectare (160 acre) field, plus the measurements required to make recommendations for improvement, requires one to four man-days of work. The time will depend upon the availability of information such as water delivery records, the complexity of the irrigation system and soils, and the design of the evaluation procedure. Table 3 lists the basic data which are required for the evaluation.

Table 3. Basic data required to evaluate the irrigation efficiency (IE) on a field or farm.

IE of a Single Event	Annual IE
Distribution Uniformity**	Distribution Uniformity**
Water applied SMD	Schedules and amounts of irrigation and rainfall Runoff and evaporation
Runoff and evaporation	ET, LR, etc. Water holding capacities of soils.
** The DU must be a complete field DU.	

The most common errors in published irrigation evaluation results are due to:

1. The quoted "irrigation efficiency" is really a measurement of DU.
2. The measured DU is only one component of the DU. The most common example is the use of "catch-can DU" as a measure of hand move sprinkler DU.
3. Infiltration parameters are measured incorrectly in surface irrigation evaluations.
4. The wrong values are measured. Forexample, the catch-can uniformity is unimportant in the DU measurement of a frequently irrigated under-tree sprinkler system.

Interrelationship Between Factors

A farmer may be interested in more than irrigation efficiency. Low pumping plant efficiencies, for example, can be as important to irrigation costs as poor Distribution Uniformities. A research study to quantify on-farm irrigation efficiency would typically not examine irrigation management strategies orcauses ofdrainage problems (Le., why things happen). A recommendation-oriented evaluation will identify and quantify the various causes of energy and water inefficiencies.

Benefits of Evaluations

General. Considerable savings can sometimes be pinpointed by simple irrigation system evaluations. The amount of savings will depend upon the present performance and costs, but many on-farm irrigation efficiencies are in the neighborhood of 50%. Yield improvements are difficult to estimate because crop yields depend upon much more than irrigation alone.

Energy savings. Energy savings are relatively easy to quantify. A 20% reduction in horsepower will result in a 20% energy savings. Similarly, a 20% reduction in the flow rate (assuming this means a 20% reduction in the annual volume of water pumped) will reduce the energy consumption by 20%. Figure 1 shows results from a typical study of energy savings possible by installing a tailwater recovery system. The benefits do not include increased yield, reduced labor, etc.

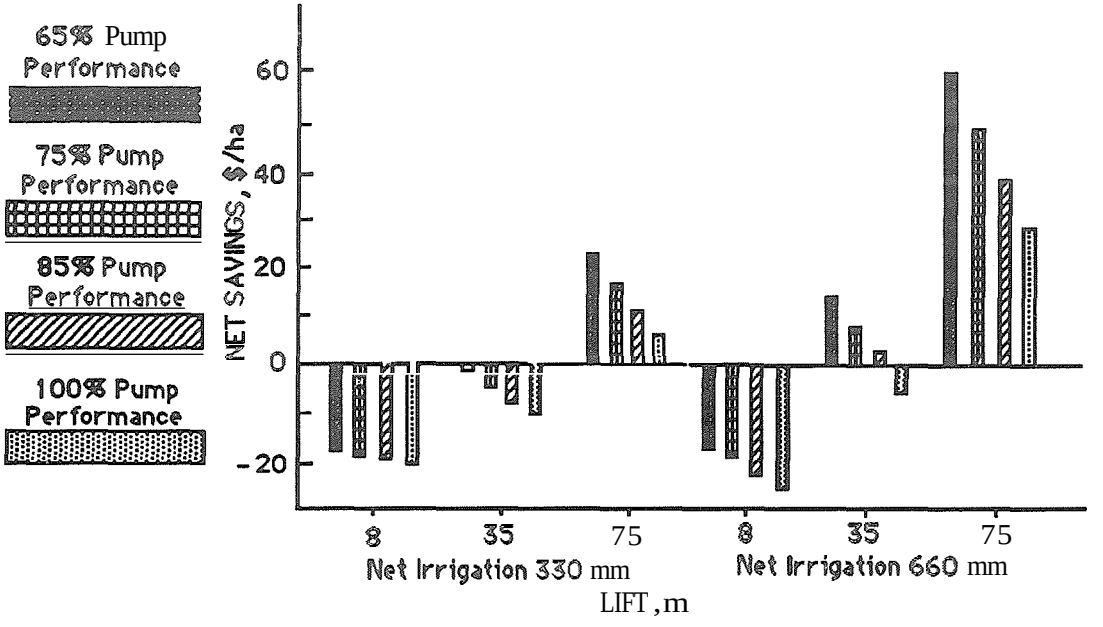


Fig. 1. Net economic benefits from installation of runoff reuse systems for gated pipe surface irrigation systems (Gilley and Supal/a, 1983).

Pacific Gas and Electric CO., a major electric utility in California, began an Irrigation System Survey program in 1982. Irrigation systems receive a simple and rapid evaluation, followed by energy-conserving recommendations made to the grower. After six months of operation, the Irrigation System Survey had identified 8.1 million kilowatt-hours and $1.45 million in potential savings for approximately 18,500 hectares (46,000 acres). (Kah, 1983)

Water and power savings not always possible. Evaluations may determine that a farmer is under-irrigating or operating at too low of a pressure. In such cases, the cost of power and water may actually increase. However, the objective of making modifications would be to increase the net profit through improved yields.

Accuracy of Evaluations

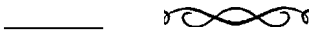
In irrigation it is generally impossible to be accurate to more than a few percentage points in most measurements. For some measurements, a 5% accuracy is more than realistic. This must be kept in mind when planning an evaluation and certainly is important when interpreting results.

Table 4. Accuracy and ease of measurement of various evaluation items.

Item Estimated or Measured	Ease of Measurement	Condition	Typical maximum measurement error, % of	
			Total Water Applied	Value of the Measured Item
Annual ET (last year)	2	Commercial irrigation scheduling companies are using ET models and field checks locally.	5	7
	3	Commercial schedulers are not working in the area.	12	15
Gross annual water applied to a field	1	One volumetric (totalizing) water meter or one well is used for each field.	4	4
	3	There are several metered water sources but runoff is not confined to the field it comes from.	18	18
	5	Canal water is used and there is no volumetric water meter.	35	35
Annual conveyance losses	1	Lined ditch or pipeline conveys water	3	5
	4	Unlined earth ditch is used	10	25
Water applied annually for special purposes (frost, weeds, etc.)	3			
Effective rainfall	4	Depends upon intensity and timing	8	20
Intake rate equation	2	Using Waterman program	-	10
	5	Using standard measurement tech.	-	100
Distribution Uniformity - Due to opportunity time differences along a single furrow.	2		-	6
- Due to soil variation	2	Fairly uniform soil	-	10
	5	Non-uniform soil	-	40
- Due to labor schedules	2		-	10
Constraints (either management or hardware)	1			

* Scale of measurement ease (includes time and $)
1- very easy
2- relatively easy
3- average
4- relatively difficult
5- very difficult

CHAPTER 15



BASIC CALCULATIONS

BASIC CALCULATIONS

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Water Requirements

Depth of water. "One centimeter (inch) of water" refers to a one centimeter (inch) depth of water over an arbitrary area. When a depth of one inch is applied to an area of one acre, this refers to a volume of:

$$\frac{1\text{" deep}}{12\text{"/ft.}} \times \frac{43560 \text{ square feet}}{\text{acre}} \times 1 \text{ acre} = 3630 \text{ cubic feet of water}$$

Water use by plants is described in terms of "inches of water per day" or "inches of water used per season". The actual volume (gallons, cubic feet, etc.) of water needed by a tree in a day depends upon the tree size and area it occupies.

Plants have large water requirements. On-farm reservoirs are generally not capable of storing more than enough water to supply an irrigation system for a few hours. The following example illustrates the need for large water storage facilities in the form of dams or underground aquifers in arid regions:

Example problem:

A 100 acre field of alfalfa.
The annual gross irrigation water requirement is 48 in.
All of the annual water supply must be stored a reservoir with a surface area of 1 acre.
The inside walls of the reservoir are vertical.

Find: The depth of water in a reservoir

Formula: $\text{Depth} = \frac{\text{Volume}}{\text{Area}}$

Solution: $\text{Volume required} = 100 \text{ acres} \times \frac{43560 \text{ sq. ft.}}{\text{acre}} \times \frac{48 \text{ inches}}{12\text{"/ft}}$

$= 17{,}424{,}000 \text{ cubic feet}$

$$\text{Depth} = \frac{17424000 \text{ cubic feet}}{1 \text{ acre} \times 43560 \text{ sq } ftlac} \times 400 \text{ feet}$$

Obviously, most on-farm reservoirs are not large and deep enough to store a season's water requirement.

With English units there is sometimes confusion with terminology. Farmers in the western U.S. frequently refer to their irrigation water entitlement from an irrigation district in terms of "acre-feet of water." One acre-foot of water on one acre is the same as "one foot of water." Farmers generally mean "3 feet of water" when they say "3 acre-feet of water" because the entitlement is per acre.

Flow rate. A flow rate is defined as: $\frac{\text{volume of water}}{\text{time}}$

Common flow rate units in the English system are:

GPM = Gallons per minute
Furrow flow rates will generally vary from 15 - 60 GPM.
A typical row crop sprinkler on a hand move sprinkler system will have a flow rate of 3 - 8 GPM.

CFS = Cubic feet per second.
CFS is used to describe larger flow rates, such as found in canals.

1 CFS = 449 GPM.

Example problem: Convert a flow rate of 1000 GPM to CFS

Solution:

$$\text{CFS} = \frac{1000 \text{ GPM}}{449 \text{ GPM/CFS}} = 2.23 \text{ CFS}$$

Converting Flow Rates to Depths. The following relationship can be used:

$$\text{Depth of water} = (\text{Flow rate}) \times \frac{\text{Time}}{\text{Area}}$$

Diminsions: $L = \frac{(L^3) \times T \times L^2}{T}$

The following formula can be used with English units:

$$\frac{\text{Inch}}{\text{hr}} = \frac{\text{GPM} \times 96.3}{\text{Area}}$$

where

Area is in units of square feet.

Example problem: A flow rate of 550 GPM is applied to a 10 acre field for 12 hours with sloping furrows?

Find: (1) How many inches of water are applied?
(2) How many inches of water infiltrate?

Formulas:

(1) $\text{inches} = \frac{\text{GPM} \times 96.3 \times \text{hours}}{\text{square feet}}$

(2) Infiltrated depth cannot be determined with the information given, unless volumetric control is used.

Solution: (1) $\text{inches} = \frac{550 \text{ GPM} \times 96.3 \times 12 \text{ hours}}{10 \text{ acres} \times 43560 \text{ sq. ft/acre}}$

$= 1.46$ in.

Example problem: A group of 40 level furrows is irrigated at once.
Furrow length =880 feet
Furrow spacing = 40 inches
It is desired to infiltrate an average of 4.5 inches
Furrow flow rate = 60 GPM

Find: Necessary application time in hours

Formula: $\text{Hours} = \frac{\text{Inch} \times (\text{area in square feet})}{\text{GPM} \times 96.3}$

Solution: Solving using the area and flow of one furrow:

$$\text{Hours} = \frac{4.5 \text{ inches} \times 880' \times [40''/(12/\text{foot})]}{60 \text{ GPM} \times 96.3}$$

= 2.28 hr.

Net versus gross. Evapotranspiration requirements are considered "net" values. Applied water quantitities are "gross" values. Irrigation managers must decide how much extra water to apply to compensate for non-uniformities and inefficiencies. The basic definition relating the two is:

$$\text{Gross} = \frac{\text{Net}}{(\text{Efficiency}/100)}$$

Gross irrigation requirement in the summer. During the middle of the summer, no extra water need be applied for leaching; the extra for leaching can be applied once a year during the spring or winter. The following problem illustrates how to determine the gross to apply for an irrigation during the middle of the summer:

Example problem: The soil SMD will be 4" at the time of irrigation.
Estimated DU = 85%
The irrigation will have perfect timing; that is, waterwill be shut off at exactly the right moment.
Of the water which is delivered from the source, the following losses are anticipated:
- All runoff is collected.
- 10% of applied will be lost in ditch seepage en-route to the furrows.

Find: The gross irrigation depth required.

Formulas: First draw a sketch (Figure 1) to help visualize the problem, then put it into equation form.

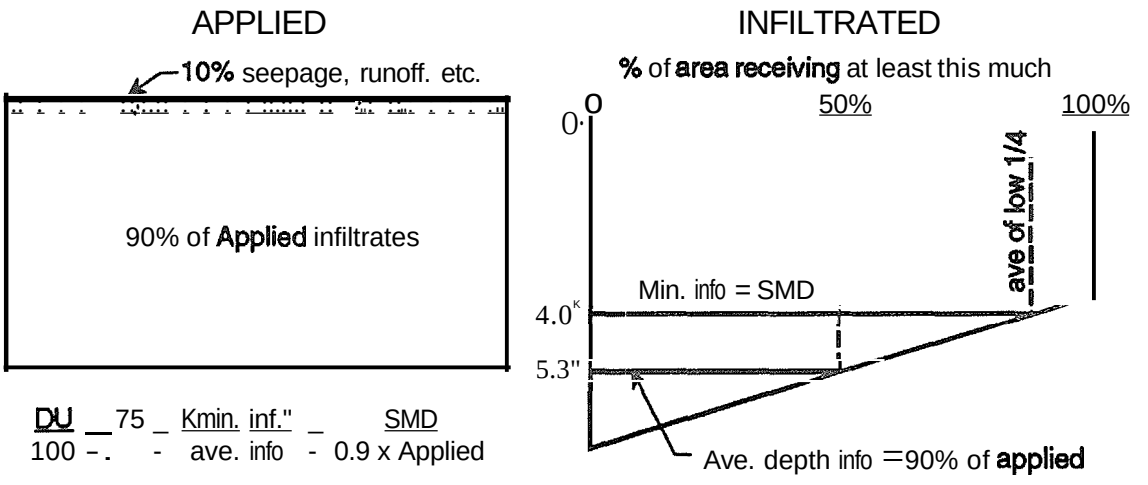


Figure 1. Destination of applied water for example problem; 75 % DU. 10% of applied is lost to canal seepage before reaching furrows. Note that the "minimum" infiltrated in this example is not the absolute minimum, but the amount which infiltrates at the "average of the low 1/4" point.

Relations which are known:

$$\text{Average Infiltrated Depth} = \frac{(100\% - 10\%\text{seepage}) \times \text{Applied}}{100}$$

Desired minimum infiltrated depth = SMD

$$\text{DU} = 75\% = \frac{\text{"Minimum" infiltrated}}{\text{Average Infiltrated}} \times 100$$

Solution:

(a) Using the sketch in Figure 1 -

$$\text{Applied Depth} = \frac{\text{Ave.Inf.} \times 100}{100\% - 10\%} = \text{Ave.Inf.} \times 1.11$$

$$= \frac{\text{"Min." Inf.} \times 100\%}{75\%} \times 1.1$$

$$= \text{"Min." Inf.} \times 1.48$$
$$= \text{SMD} \times 1.48$$
$$= 4" \times 1.48$$

$$\text{Applied Depth} = 5.9"$$

(b) Using a second method of solution:

$$\text{Irrig. Efficiency with perfect timing} = \frac{\text{DU} \times (100 - \text{other losses as \% of applied})}{100}$$

$$= \frac{75\% \times (100 - 10\% \text{ seepage})}{100} = 67.5\%$$

$$\text{Gross to apply} = \frac{\text{Net}}{\text{Irrig. Eff}/100} = \frac{4''}{0.675} = 5.9''$$

Note that the percentage seepage was expressed as a percentage of the applied water. That convention must be followed to use these formulas.

The actual irrigation efficiency may be higher or lower than 67.5% in the above problem, depending upon the duration of the irrigation (assuming the same DU and % seepage). A short duration would result in under-irrigation and a higher irrigation efficiency; a longer duration would result in over-irrigation at the "minimum" point, and a lower irrigation efficiency.

Adopting the Distribution Uniformity definition using the "average of the low 1/4" as the "minimum" value will result in a slight under-irrigation at the driest point in the field. See the Efficiency chapter for a discussion on this topic.

Gross seasonal irrigation requirement. This irrigation efficiency of 67.5% in the above problem would not be the seasonal irrigation efficiency. The timing, seepage, runoff losses, and DU will vary from irrigation to irrigation. With furrows the DU is generally higher in the middle of summer than in the spring because the intake rates are lower. The formula for estimating the seasonal gross water requirements was given in the Salinity chapter as:

$$\text{Gross seasonal req.} = \frac{\text{Net Requirement} - \text{Effective Rainfall} - \text{Other}}{(\text{Irrigation Efficiency}/100) \times (1\text{-LR»}}$$

where:

Net Requirement	=	Sum of required beneficial uses such as ET, frost control, compaction, etc.
Effective Rainfall	=	Rainfall which is stored in the soil for immediate or later use by the crop. This is difficult to estimate, but Tables 8 and 9 of the Evaluation chapter can serve as one source for these numbers.
Irrigation Efficiency	=	Estimated IE accounting for non-uniformity, seepage losses , uncollected runoff, etc.
LR	=	Leaching requirement, if it is in excess of the deep percolation.
Other	=	Other contributions to seasonal water use, including a high water table or a change in soil moisture.

Table 1. ET and rainfall during a year; for a cotton crop in the Southern San Joaquin Valley of California (ET data from Westlands, 1984).

Month	ET, inches	Rainfall, inches
November	--	0.4
December	--	1.0
January	---	2.0
February	---	1.5
March	--	2.0
April	0.6	0.5
May	1.3	--
June	5.4	---
July	9.1	---
August	7.6	---
September	3.6	---
October	---	---
Totals	27.6	7.4

The problem below illustrates how to compute the gross annual irrigation requirement.

Example problem:

Given: A field of cotton is grown in the southern San Joaquin Valley of California (Use data from Table 1).
EC of the irrigation water =0.4 *dS/m*
Average DU of irrigation = 85%
Uncollected runoff, ditch seepage losses = 5% of applied.
Errors in scheduling are estimated to give a 10% extra infiltration at the "driest" point.

Find: Gross annual irrigation requirement

Formulas: The various components of LR, effective rainfall, and seasonal irrigation efficiency must be found.

Solution: (a) From Table 1, the annual ET = 27.6 inches
(b) The effective rainfall can be estimated with the use of Tables 8 and 9 of the Evaluation chapter. All of the rain falls during the "fallow winter" conditions, so Table 9 is used.

Month	Rainfall (inches)	Effective Rain (inches)
November	0.4	0.0
December	1.0	0.4
January	2.0	1.2
February	1.5	0.7
March	2.0	1.2
April	0.5	0.1
Total		3.6

(c) Leaching requirement:

From the Salinity chapter, the threshold ECe for cotton =7.7 dS/m

$$LR = \frac{ECw}{(5 \times ECe) - ECw}$$

$$= \frac{0.4}{(5 \times 7.7) - 0.4}$$

$$= 0.01$$

This means that an extra 1% water must infiltrate at the driest spot in the field to maintain salinity control. This problem states that poor irrigation scheduling will result in an extra 10% infiltration, which exceeds the 1% needed. Inefficient irrigation in this case will take care of the leaching requirement. Therefore, calculation of the gross water needed does not have to include an additional quantity for leaching in this case.

(d) This step is not needed in a standard calculation, butwill be done to clarify the next steps and Figure 2.

The depth which must infiltrate at the driest point (neglecting the LR and "other" sources) is:

Inf. depth needed at "min" point = ET - (Eff. Rain)
=27.6" - 3.6"
= 24 inches

(e) Seasonal Irrigation Efficiency estimate:

Average infiltrated = 100% - 5% seepage, etc
= 95% of applied

From the DU equation,
Minimum infiltrated = 85% of Ave. infiltrated

From estimation of poor scheduling,
Minimum infiltrated = 110% of what is needed
= 1.10 x 24 inches
= 26.4 inches

Figure 2 illustrates the destination of applied water for this problem.

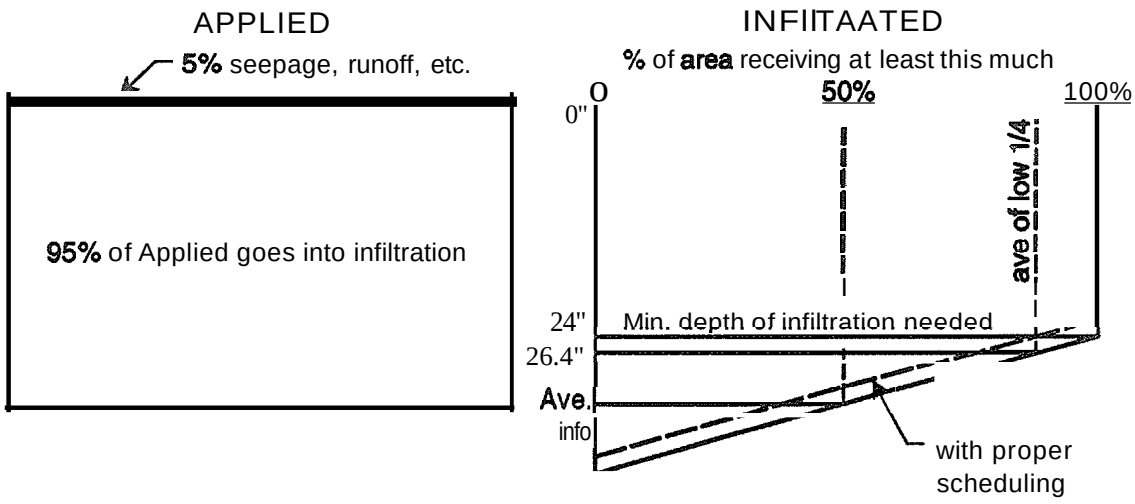


Figure 2. Destination of applied water; for example problem of annual irrigation water requirements. 85% DU. 10% excess infiltration due to poor timing.

Depth needed for infiltration

$$= \frac{\text{Minimum infiltrated}}{1.10}$$

$$= \frac{0.85 \text{ x (Average Infiltrated)}}{1.10}$$

$$= 0.773 \text{ x (Ave. Inf.)}$$

$$= 0.773 \text{ x (0.95 x Depth Applied)}$$

$$= 0.734 \text{ x Depth Applied}$$

Therefore, the Irrigation Efficiency = 73%
(disregarding the beneficial use of part of the deep percolation which compensates for LR)

(f) Gross seasonal requirement, neglecting the LR because of the predicted deep percolation

$$= \frac{\text{Net Requirement - Effective Rain}}{\text{Irrigation Efficiency/100}}$$

$$= \frac{27.6" - 3.6"}{.734}$$

$$= 32.7"$$

System flow rate requirements. For an irrigation system which completely meets the ET requirement (pius any inefficiencies) "at all times" during the growing season, the system flow rate requirement is determined by the peak ET rate. That statement leaves many options, as described below:

1. Some irrigation systems do not have enough flow rate capacity to completely replenish the water used for ET during the peak ET period. This is standard with crops such as cotton in some areas in which the soils have high water holding capacities. Such a practice requires a very dependable irrigation system and the proper use of pre-irrigations.
2. The period of peak ET rates can be defined as the hottest day, week, or month. Designers typically use ET data from the peak month. Conditions of small available water holding capacities in the root zone (e.g., shallow-rooted crops or sandy, shallow soils), or crops which need a very low MAD (Management Allowable Depletion) may require designing for the hottest one or two week period.
3. If water is not available 24 hours per day during the peak ET period, the required flow rate must be adjusted upward. In some areas it is considered good practice to plan for only 18 hour a day operation of deep well pumps in order to take advantage of off-peak-pumping rates from electrical utilities. It may also be desireable to not work on the weekends. Other designers like to plan for a less-than-24 hour a day water supply in anticipation of pipe breaks, pump repairs, etc. during the peak ET period. Too much "off time" will result in a very large flow rate and pipe size requirement.
4. The seasonal water requirement does not affect the calculations of the proper flow rate. The seasonal water requirement will, however, affect the annual power cost.

The problem below illustrates a computation of the flow rate requirement to the field of cotton of the previous problem.

Example problem: Assume the same field of cotton as in the previous problem.
Field size = 40 acres.
During the peak ET period, irrigation will only occur during the weekdays (ie., 5 days out of 7)
The pump will operate 24 hours per day during the peak ET period.
DU = 85% during the peak ET period.
Ditch seepage losses = 5% during peak ET period.
Timing of irrigation durations will be excellent.

Find: Required flow rate from source in GPM.

Formulas: $\text{Gross} = \dfrac{\text{Net}}{\text{Irrig.Eff}/100}$

$$\text{GPM} = \frac{(\text{Gross inches}) \times (\text{Area in square feet})}{96.3 \times (\text{Hours supply is available})}$$

Solution: (a) Irrigation Efficiency

$$\text{IE} = \text{DU} \times \left(100 - \frac{\%\ \text{losses other than DU}}{100}\right)$$

$$= 85\% \times \frac{(100 - 5)}{100}$$

$$= 85\% \times .95$$

$$= 80.8\%$$

(b) Gross inches

Assuming the soil is fairly deep, and knowing that cotton has a deep root zone and can be irrigated with a high MAD, the peak monthly ET can be used.

$$\text{Gross inches} = \frac{\text{ET} - \text{Effective Rainfall}}{\text{Irrig. Eff!100)}}$$

$$= \frac{9.1" - 0"}{.808}$$

$$= 11.3" \text{ per month}$$

Note that the LR is not included in the peak month, and it is assumed that there will be no over-irrigation due to poor scheduling.

(c) Hours the supply is available.

During a typical one month period, the water supply will be available:

$$= 1 \text{ month} \times \frac{31 \text{ days}}{\text{month}} \times \frac{24 \text{ hours}}{\text{day}} \times \frac{5 \text{ days!wk}}{7 \text{days!week}}$$

$$= 531 \text{ hours}$$

(d) GPM

$$\text{GPM} = \frac{\text{Gross inches} \times \text{Area in square feet}}{96.3 \times \text{Hours supply is available}}$$

$$= \frac{11.3" \times 40 \text{ acres} \times (43560 \text{ sq. } ftlac)}{96.3 \times 531 \text{ hours}}$$

$$= 385 \text{ GPM}$$

(e) Adjustment for hotter-than-average years.

The data in Table 1 is for one year. Some years may have higher peak ET rates. The 385 GPM would have to be increased in this case if the desire is to still irrigate only 5 days a week and have no underirrigation. If the values of Table 1 are about average with annual peak variations of no more than about 10%, many designers would keep the 385 GPM design flow rate and advise the grower that it may be necessary to irrigate on the weekend during hot years.

References

Westlands Water District. 1984. Water Conservation and Management Handbook. Fresno, California.

CHAPTER 16

BASIC PIPELINE HYDRAULICS

BASIC PIPELINE HYDRAULICS

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Static Conditions

Pressure. Pressure is defined as the force which is exerted upon an area. Units of pressure in the English system are:

psi (pounds per square inch), and
feet

The unit conversion is:

1 psi = 2.31 feet

The concept of static pressure is illustrated in Figure 1, which shows a column of water in a tank and pressure gauges at various elevations below the water surface.

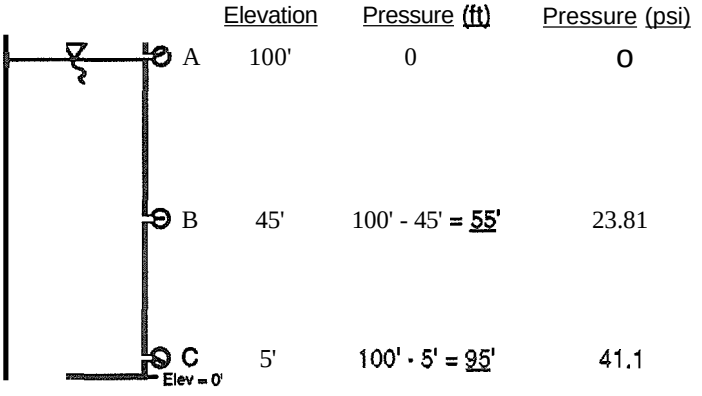


Figure 1. *Relationship between gauge pressures and vertical distance below* a *water surface. Static condition.*

Pressure gauges used to measure water pressure can be the same type as those used to measure air pressure in bicycle tires. Good gauges will be oil-filled to prevent surge damage to the internal mechanism from repeated shocks with heavy use.

Figure 1 shows that a simple pressure gauge is an open vertical tube of water which is tapped into the point of question. The water will rise in the tube a vertical distance which is equal to the pressure (measured in feet) at that point. The tube must be of large enough diameter (0.25 - 0.5 inch inside diameter) to eliminate extra rise in the tube from capillary action (force due to the attraction of the water surface to the sides of the container).

The example problem below illustrates why such open tubes are only used in very low pressure situations.

Example: A pipeline has a pressure of 50 psi.
It is desired to use a simple open vertical tube of clear plastic to measure the pressure at that point.

Find: The required height of the tube.

Formula: $\text{Feet} = \text{psi} \times \frac{2.31'}{\text{psi}}$

Solution: The water would rise in the tube a vertical distance of:

$$50 \text{ psi} \times \frac{2.31'}{\text{psi}} = 115.5 \text{ feet}$$

Obviously, this is a very high tube. Therefore, we use pressure gauges for high pressure measurements.

Figure 2 illustrates the concept that the pressure at a point in a static (no movement, but the pipe is full of water) water system is determined by the vertical height of water above that point. This measurement requires that the top of the container is open to the air (ie., not under pressure).

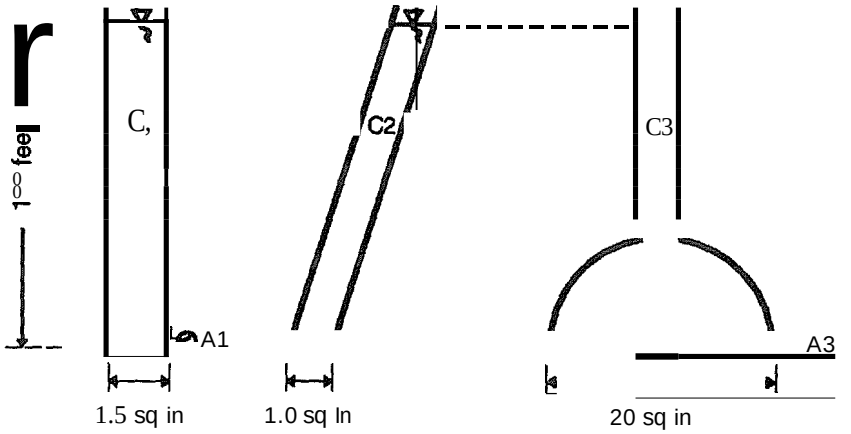


Figure 2. Three containers of different shapes and sizes filled with water to the same height. Container tops are open to the atmosphere. Pressure A1 = Pressure A2 = Pressure A3 = 100 feet

Head. Head is synonymous with "pressure". A pressure of 100 feet is often refered to as "100 feet of head".

Force. Figure 2 shows that each container, regardless of shape or volume, has the same pressure at the "A" points. However, the force exerted upon each bottom plate is not the same (although the force per unit area is).

Example problem: Find the force on the inside of the bottom plate of container C3 of Figure 2.

Formula: Force = Pressure x Area

where
. Pressure is in psi, and
. Area is in square inches

Solution:

$$\text{Pressure} = \frac{100 \text{ feet}}{2.31 \text{ ft.lpsi}} = 43.3 \text{ psi}$$

$$\text{Force} = \frac{43.3 \text{ pounds}}{\text{square inch}} \times (20 \text{ square in.})$$

$$= 866 \text{ pounds}$$

Dynamic (Moving) Conditions

Units of Flow. In the U.S., large flow rates are typically measured as Gallons per Minute (GPM) or Cubic Feet per Second (CFS).

1 CFS =449 GPM

Energy. Most situations of interest with irrigation systems are dynamic rather than static; that is, they involve moving water. Water moving through irrigation systems may change elevation, pass through valves and fittings, and increase in pressure as it flows through a pump.

Calculations of pressure at various points in the system require an understanding of equations which describe the energy of water. These equations are derived from the First Law of Thermodynamics which states that energy is conserved and changes in energy can be accounted for.

Bernoulli Equation. A simplified form of the energy equation is called the Bernoulli Equation, shown following. In this form, all of the energy components are measured in terms of "head", with units of length such as feet.

"Velocity head" is the energy component of the Bernoulli Equation which is due to water velocity.

$$\text{Velocity head} = \frac{\text{Velocity}^2}{2 \times (\text{Acceleration due to gravity})}$$

$$= \frac{V^2}{64.4} \quad \text{if}$$

Velocity head is in feet, and
Velocity is in feet/sec

The Bernouli equation shows that the energy at the upstream *(u/s)* point in a pipeline equals the energy at the downstream *(dis)* point, plus any friction and minus any pump pressure supplied between the two points.

Where,

'u/s refers to the upstream point (nearest the water source).
.dis refers to the downstream point. Water flows from upstream to downstream.

·Vu/s, *Vd/s* are the average velocities of the water at two locations, expressed as feet per second.

$$\text{Velocity} = \frac{Q}{A}$$

where,

Q = flow rate, CFS

A = cross-section area of the pipe
= 3.14 x (radius)2
and the radius is in feet

· $Elev_{u/s}$, $Elev_{d/s}$ are elevations of the centerline of the pipe at two points, measured from the same elevation datum (Le., reference elevation), expressed as feet.

· $P_{u/s}$, $P_{d/s}$ are pressures, measured in feet.

· H_f is the friction between points A and B, measured in feet.

· H_P is the pressure difference (increase) between the inlet and discharge of a pump located on the pipeline between upstream and downstream points, measured in feet.

Computations with Bernoulli's Equation. Bernoulli's Equation can be used to solve both static and dynamic conditions. It can be simplified in many situations.

Example problem with Bernoulli's Equation - static condition:

Given: Figure 1, with point "B" located 55' below the water surface.
Find: Gauge pressure at point "B"
Formulas: Bernoulli's Equation
Solution: In this case, the velocities are zero (itis a static condition), and Hf and H_p are zero because there is no movement or pump

Bernoulli's Equation reduces to:

$$Elev_A + P_A = Elev_B + P_B$$

where,

there is no "upstream" or "downstream" since it is a static condition

Solving,

$$P_B = Elev_A + P_A - Elev_B$$

PAis zero, because the pressure at any point which is in contact with the air is always zero (otherwise it would explode because no container is encapsulating it).

PB = 100 ft. - a - 45 ft. = 55 feet

Important facts which are shown by the Bernoulli Equation are:

1. For irrigation hydraulics problems, the total energy of water at any point consists of three components:
   a. Elevation
   b. Pressure
   c. Velocity
2. In a static condition, the energy at one point in a body of water or pipeline always equals the energy at any other point (neglecting temperature changes).
3. If water moves, friction occurs. The friction loss is the change in energy.
4. The rate of friction is the rate of energy change, not the rate of pressure change (unless the pipe is on flat ground).
5. Energy can be added to water with a pump.
6. Water does not move in response to a change in pressure only. This is illustrated by the previous example problem from Figure 1. Pressures at points A and B are very different, but it is a static condition. However, the energy at both points is the same.
7. In high pressure systems (greater than 20 psi) the velocity head *(V2/64.4)* is almost always ignored in the calculations because the value is typically low. For example:

Given: Pipe inside diameter = 6.30"
Flow rate = 500 GPM

Find: Velocity head, in feet

Formula: $V = \frac{Q}{A}$

$A = 3.14 \times \text{Radius}^2$

Units are square feet (A), feet (Radius), feet/sec M, and CFS (Q)

Solution: $Q = \frac{500 \text{ GPM}}{449 \text{ GPM/CFS}} = 1.11 \text{ CFS}$

$\text{Radius} = \frac{\text{Diameter}}{2} = \frac{6.30"}{12"/ft \times 2} = 0.263 \text{ ft.}$

$A = 3.14 \times (0.263 \text{ ft})^2$

$= .22 \text{ sq. ft.}$

$V = \frac{Q}{A} = \frac{1.11 \text{ } ft^3/sec}{.22 \text{ } ft^2}$

$= 5.1 \text{ feet/sec (checks } w/ \text{ Table 1)}$

$\text{Velocity Head} = \frac{V2}{64.4}$

$= \frac{[5.1 \text{ feet/sec}1^2}{64.4}$

$= 0.4 \text{ feet}$

$= 0.17 \text{ psi}$

The velocity of 5.1 fUsec in this example is close to the maximum allowable velocity in many designs. Many designers attempt to keep velocities below 5 fUsec in order to minimize water hammer problems which can occur at higher velocities (caused when the water is shut off suddenly at the end of a pipeline).

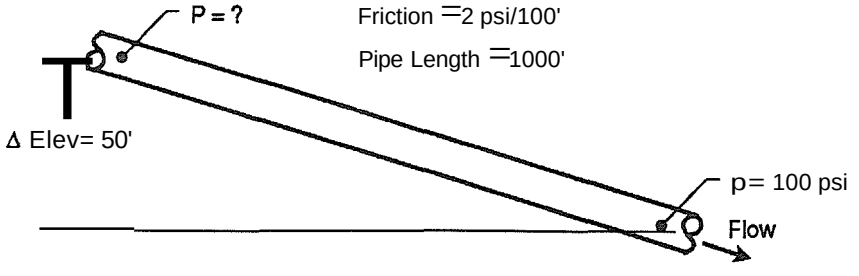
In terms of the importance of the velocity head for pressure computations, however, the 0.17 psi of this case is negligible for a high pressure system. Nevertheless, for a very low pressure system with only one or two feet of operating pressure, this would be an important energy component to include in computations.

Table 1. *Flow rate (GPM) at various velocities for various pipe or valve inside diameters. GPM* = 2.45 x V x 10^2, *where Velocity (V) = feet/sec.; Inside Diameter (10) = inches.*

PipeID, Inches	Velocity, Feet/second							
	2	3	4	5	6	7	8	9
0.62	2	3	4	5	6	7	8	8
0.82	3	5	7	8	10	12	13	15
1.20	7	11	14	18	21	25	28	32
1.75	15	23	30	38	45	53	60	68
2.20	24	36	47	59	71	83	95	107
2.70	36	54	71	89	107	125	143	161
3.30	53	80	107	133	160	187	213	240
4.20	86	130	173	216	259	303	346	389
5.60	154	230	307	384	461	538	615	691
6.30	194	292	389	486	583	681	778	875
8.20	329	494	659	824	988	1,153	1,318	1,483
10.2	510	765	1,020	1,274	1,529	1,784	2,039	2,294
12.1	717	1,076	1,435	1,794	2,152	2,511	2,870	3,228
15.0	1,103	1,654	2,205	2,756	3,308	3,859	4,410	4,961
18.0	1,588	2,381	3,175	3,969	4,763	5,557	6,350	7,144

An application of the Bernoulli equation is illustrated in the example below:

Example: Solve the figure below.



Given: Pipeline length = 1000 feet
Rate of friction loss = $\frac{2 \text{ psi}}{100 \text{ feet}}$
Downstream pressure = 100 psi
Change in elevation = 50 feet

The pipeline is classified as a "mainline"; that is, there are no intermediate outlets, and the flow at the upstream end equals the flow at the downstream end.

Pipe diameter is constant along the length.

Find: Pressure at the upstream end

Formula: Use the Bernoulli Equation, rearranged to solve for the upstream pressure.

$$P_{uis} = P_{d/s} + (E_{1evd/s} - Elevu/_S) + Hf + \frac{Vd/s^2}{64.4} - \frac{V_u/_S 2}{64.4}$$

Solution: Because the pipe diameter and flow rate are constant along the pipe length, the upstream and downstream velocities are identical. Therefore, they cancel each other in the solution,

$$\frac{(Vd/s^2 - VU/s^2)}{64.4} = 0$$

$$Pd/s = 100 \text{ psi} \times \frac{2.31 \text{ ft}}{\text{psi}} = 231 \text{ feet}$$

$$(Elevd/s - Elevu/_S) = -50 \text{ feet}$$
(*note the negative sign)

$$= \frac{2 \text{ psi}}{100 \text{ ft of length}} \times 1000 \text{ feet} \times \frac{2.31 \text{ ft}}{\text{psi}}$$

$$= 46.2 \text{ feet}$$

$$P_{uis} = 231 \text{ ft.} + (-50 \text{ ft.}) + 46.2 \text{ ft.}$$

$$= 227.2 \text{ ft.}$$

$$= 98.4 \text{ psi}$$

For most pipeline friction and pressure computations, the equation to use is a simplified form of the Bernoulli Equation as follows:

$$\text{Pdls} = \text{Puis} - \text{Hf} + \Delta\ \text{Elevation} \quad \text{I}$$

where

- Δ Elevation is "positive" if the pipeline is lower at the downstream end than at the upstream end. (Δ means "change")

. All units are in feet.

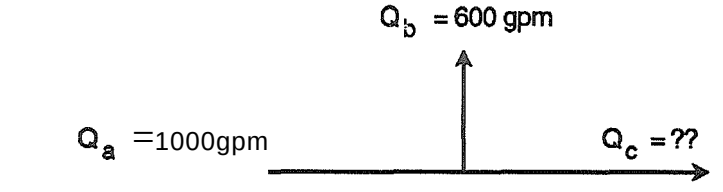
** This equation is valid for conditions of full pipeline flow only.

Using the previous problem as an example, and re-arranging sides

$$\begin{aligned} \text{Puis} &= \text{Pdls} + \text{Hf} - \Delta\ \text{Elevation} \\ &= 231\ \text{ft} + 46.2\ \text{ft} - 50\ \text{ft} \\ &= 227.2\ \text{ft.} \end{aligned}$$

Other Basic Principles

Conservation of Mass. All of the flow of water must be accounted for at a pipe junction. The flow into the junction must equal the flow exiting, unless a reservoir exists at that point. This principle is illustrated in Figure 3.



Continuity Law: $Q_c = Q_a - Q_b$

Q_c = 1000 gpm - 600 gpm = 400 gpm

Figure 3. Conservation of mass at a pipe junction

Pressure at a iunction. There is only one pressure at a pipeline junction. In Figure 4, P2 represents the pressure at the junction.

The importance of this statement is that P2 can be calculated in 3 ways, and the answer will be the same in each case.

$$P2 = P1 + \Delta\ \text{Elev}(1_2) - \text{Hf}(1_2)$$

$$P2 = P3 - \Delta\ \text{Elev}(2_3) + \text{Hf}(2_3)$$

$$P2 = P4 - \Delta\ \text{Elev}(2_4) + \text{Hf}(2_4)$$



Figure 4. Pressures in a pipe network.

Water Jet Pressure. Irrigation system hydraulics problems frequently end with a full pipe discharging into the air. The pressure of the water jet is zero, but the jet still has energy in the form of velocity head and elevation. If the jet was under pressure, it would explode in the air because there are no pipe walls to contain it. If the jet hits an object, much of the velocity head is converted to pressure.

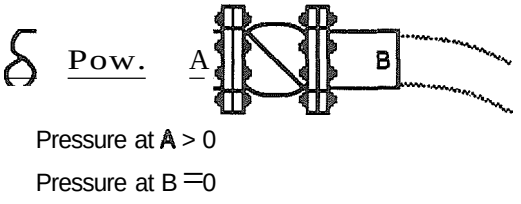


Figure 5. Pressures at a full open pipe discharge.

PVC Pipe

PVC (Poly Vinyl Chloride) is the most popular irrigation pipe material used in the U.S.. Tables 2 and 3 (on the following pages) provide typical PVC pipe dimensions.

Table 2. Typical PVC Pipe IPS Dimensions

Class 63 psi. SOR64			
Nominal Size (in)	OO (in)	ID (in)	Weight/Ft (lb)
4	4.500	4.360	.655
6	6.625	6.417	1.394
8	8.625	8.355	2.357
10	10.75	10.41	3.655
12	12.75	12.35	5.134

Class 100 psi. SOR41			
Nominal Size (in)	OO (in)	ID (in)	Weight/Ft (lb)
4	4.500	4.280	.993
5	5.563	5.291	1.518
6	6.625	6.301	2.153
8	8.625	8.205	3.633
10	10.75	10.23	5.650
12	12.75	12.13	7.954

Class 125 psi. SOR 32.5			
Nominal Size (in)	OO (in)	ID (in)	Weight/Ft (lb)
1.25	1.660	1.548	.186
1.5	1.900	1.784	.221
2	2.375	2.229	.345
2.5	2.875	2.699	.501
3	3.500	3.284	.753
4	4.500	4.224	1.238
5	5.563	5.221	1.896
6	6.625	6.217	2.693
8	8.625	8.095	4.555
10	10.75	10.09	7.091
12	12.75	11.97	9.960

CLASS 160 PSI SOR26			
Nominal Size (inches)	OO (in)	ID (in)	Weight/Ft (lb)
1.25	1.660	1.532	.211
1.5	1.900	1.754	.276
2	2.375	2.193	.429
2.5	2.875	2.655	.628
3	3.500	3.230	.934
4	4.500	4.154	1.539
5	5.563	5.135	2.354
6	6.625	6.115	3.340
8	8.625	7.961	5.661
10	10.75	9.924	8.778
12	12.75	11.77	12.352

Class 200 psi. SOR21			
Nominal Size (inches)	OO (in)	ID (in)	Weight/Ft (lb)
.75	1.050	.930	.123
1	1.315	1.189	.163
1.25	1.660	1.502	.258
1.5	1.900	1.720	.336
2	2.375	2.149	.528
2.5	2.875	2.601	.774
3	3.500	3.166	1.144
4	4.500	4.072	1.886
5	5.563	5.033	2.887
6	6.625	5.993	4.099
8	8.625	7.803	6.925
10	10.75	9.726	10.646

Schedule 40				
Nominal Size (inches)	OO (in)	ID (in)	Weight/F (lb)	Pressure Rating
.5	.840	.622	.165	600
.75	1.050	.824	.219	480
1	1.315	1.049	.325	450
1.25	1.660	1.380	.434	370
1.5	1.900	1.610	.518	330
2	2.375	2.067	.696	280
2.5	2.875	2.469	1.100	300
3	3.500	3.068	1.458	260
4	4.500	4.026	2.077	220
6	6.625	6.065	3.580	180
8	8.625	8.095	5.55	160
10	10.75	10.02	7.65	140
12	12.75	11.94	10.10	130

Schedule 80				
Nominal Size (inches)	OO (in)	ID (in)	Weight/Ft (lb)	Pressure Rating
.5	.840	.546	.203	850
.75	1.050	.742	.276	690
1	1.315	.957	.408	630
1.25	1.660	1.278	.562	520
1.5	1.900	1.500	.682	470
2	2.375	1.939	.944	400
2.5	2.875	2.323	1.440	420
3	3.500	2.900	1.930	370
4	4.500	3.826	2.820	320
6	6.625	5.761	5.390	280
8	8.265	7.625	8.180	245
10	10.75	9.564	12.100	230
12	12.75	11.38	16.700	230

Table 3. Typical PVC Pipe PIP Dimsnsions

100 ft. Head SDR93			
Nominal Size (inches)	OD (in)	ID (in)	WeightlFt (lb)
6	6.140	5.990	.929
8	8.160	7.968	1.535
10	10.20	9.962	2.299
12	12.24	11.960	3.345
15	15.30	14.954	5.159
18	18.36	17.960	7.376
20	20.40	19.950	9.129

PR 63 psi. SDR 64 PIP Dimension			
Nominal Size (inches)	OD (in)	ID (in)	WeightlFt. (lb)
15	15.30	14.789	7.69
18	18.70	18.076	11.50
21	22.05	21.311	16.00
24	24.80	23.973	20.20

PR 80 psi. SDR 51 PIP Dimension			
Nominal Size (inches)	OD (in)	ID (in)	WeightlFt (lb)
6	6.140	5.900	1.479
8	8.160	7.840	2.600
10	10.20	9.800	4.062
12	12.24	11.760	5.849
15	15.30	14.700	9.138
18	18.70	17.916	14.40
21	22.05	21.123	19.90
24	24.80	23.763	25.20

PR *100* psi. SDR 41 PIP Dimension			
Nominal Size (inches)	OD (in)	ID (in)	WeightlFt (lb)
6	6.140	5.820	1.825
8	8.160	7.738	3.217
10	10.20	9.672	5.032
12	12.24	11.61	7.251
15	15.30	14.55	11.746
18	18.70	17.73	17.80
21	22.05	20.90	24.70
24	24.80	23.51	31.30

PR 125 psi. SDR 32.5 PIP Dimension			
Nominal Size (inches)	OD (in)	ID (in)	WeightlFt (Ib)
15	15.30	14.29	14.90
18	18.70	17.47	22.30
21	22.05	20.60	30.90
24	24.80	23.17	39.20

PR *160* psi. PIP Dimension			
Nominal Size (inches)	OD (in)	ID (in)	WeightlFt (Ib)
15	15.30	14.04	18.50

PR200 psi. PIP Dimension			
Nominal Size (inches)	OD (in)	ID (in)	WeightlFt (Ib)
15	15.30	13.74	22.70

There are several points to be made from Tables 2 and 3:

1. Nominal sizes. Nominal sizes are just identifier numbers which are somewhat close to some actual dimensions. A typical 6" pipe may have no dimension which is actually 6". Forexample, a 6" Class 100 IPS pipe has a 6.625" OD and 6.301" ID. Steel pipe nominal sizes have similar differences. Perhaps the closest sizing is for the 100 foot head PIP pipes, which have an inside diameter almost identical to the nominal dimension.
2. IPS dimensions. IPS (Iron Pipe Size) refers to pipe which has been manufactured to a certain outside diameter. All 4" IPS pipe, and Schedule 40 and Schedule 80 pipe, have the same outside diameters (4.500").
3. PIP dimensions. PIP (Plastic Irrigation Pipe) refers to PVC pipe which has been manufactured to another set of specifications for outside diameter. PIP is only used for large diameter pipes, and in general the very low pressure PVC pipe is PIP dimensioned. A Class 100 PIP pipe has the same pressure rating as a Class 100 IPS pipe.
4. Pressure rating. This is the maximum internal water pressure that can be exerted continuously in a pipe without damage ata specifictemperature (73^0 F). Table 6 provides service factors for both PVC and PE pipe at different temperatures. Higher temperatures drastically reduce the pressure ratings and increase cost by requiring thicker walled pipe.

Table 4. Pressure Rating Service Factors For Temperatures For PVC and PE Pipes (from ASAE, 1990)*

Temperature			
deg.C	deg.F	PVC factor	PE factor
23	73.4	1.0	1.0
26.7	80	0.88	0.92
32.2	90	0.75	0.81
37.8	100	0.62	0.70
43.3	110	0.50	
48.9	120	0.40	
54.4	130	0.30	
60.0	140	0.22	

* *At* 90 *degrees F,* a Class *100 PVC pipe would have* a *pressure rating of (0.* 75 x *100 psi) = 75psi.*

5. Class. The "Class" of a pipe refers to its working pressure rating.
6. Schedule. Schedule 40 and Schedule 80 pipe have IPS outside diameters. As can be seen from Table 2, with Schedule pipes the working pressure decreases as the pipe size increases. Many people are familiar with the thick walled, strong Schedule pipe of small dimensions. The perception that Schedule pipe has a high pressure rating can be very misleading when using large diameter pipe.
7. SDR. The SDR (Standard Dimension Ratio) of a pipe is defined as:

$$SDR = \frac{OD}{t}$$

where

OD = Outside diameter of pipe
t = Wall thickness of pipe

The SOR's which are published by manufacturers are "standard" in thatthere are a limited numberof"standard" pressure ratings. Ail PVC pipe ofthe same material and same SOR have the same pressure rating.

8. Working Pressure. This value is not provided on the Tables, but is lower than the Pressure Rating (PR) of the pipe. The Working Pressure plus Surge Pressure should not exceed the PR (ASAE, 1990).

The PVC pipe in Tables 2 and 3 is rigid PVC. Flexible PVC is also used in some applications. Flexible materials such as flexible PVC and Polyethylene have a lower pressure rating than rigid materials, given the same wall thickness.

Friction

Introduction. Water always loses energy as it flows through pipelines. This energy loss is called friction (Hf). There are two general types offriction calculations done in hydraulic designs:

1. Friction through straight sections of pipeline.
2. Friction through valves and fittings.

For a given flow rate, the friction through a pipeline is the same regardless of whether the water is flowing uphill or downhill. As seen by the Bernoulli Equation, friction and elevation changes must be handled individually.

Friction Equations. There are two popular friction equations for pipeline materials common in surface irrigation designs:

1. The Hazen-Williams Equation is commonly used for PVC (Poly Vinyl Chloride), steel, aluminum, transite (asbestos-cement) and large diameter PE (Polyethylene) pipelines.

$$Hf = \frac{Kx\ LxQ\ 1.852}{C\ 1.852 \times 100 \times 10\ 4.87}$$

where

K	=	a constant equal to 1050 for English units, and 1.21×10^{10} for metric units.
Hf	=	total friction, feet (meters in metric units)
L	=	pipeline length, feet (meters)
Q	=	pipeline flow rate, GPM (lps)
10	=	inside diameter of pipeline, inches (mm)
C	=	a friction factor which depends upon the pipe material (see Table 5).

Table 5. Pipe Friction Factors for the Hazen-Williams Equation.

Pipe Material	C	Relative Hf
PVC, PE		
ID> 5" (127mm)	150	1.00
1.5" (38mm) <= ID <=5" (127mm)	145	1.06
ID • 1.5" (38mm)	140	1.14
Drip Hose (PE) with emitters (this includes the barb minor losses)	135-140	1.22 -1.14
Cast iron (new)	130	1.30
Cast iron (old)	100	2.12
Welded steel	120	1.51
Aluminum with couplers	120	1.51
Transite (asbestos cement)	140	1.14
Concrete, mortar jointed	102	2.00
15 year old steel	100	2.12

The Hazen-Williams equation is an empirical equation which provides a reasonably accurate estimate of friction loss in conditions of turbulent flow. Most friction tables, nomographs, and slide rules in the irrigation industry have been developed with data from medium-sized (4" -12") pipes, and the roughness factors have been extrapolated to other pipe sizes. The extrapolation is not necessarily correct, as can be seen in the PVC section of Table 5. Smaller diameter PVC pipes have more friction (Le., have lower "C" values) than will be predicted through extrapolation. For unusual hydraulic cases, the Darcy-Weisbach equation, together with the Moody diagram as found in hydraulic engineering manuals, can provide a check on accuracy.

The following points can be made about the Hazen-Williams equation if only one of the variables is changed at once:

a. By doubling the pipe length, the friction is doubled
b. By doubling the flow rate (Le., velocity), the new friction is 3.6 times greater than the original friction.

$$\text{New Hf} = \text{Old Hf} \times \frac{\text{new GPM}}{\text{old GPM}}^{1.852}$$

$$= \text{Old } \frac{\text{Hf} \times 2^{1.852}}{1^{1.852}}$$

$$= \text{Old Hf} \times 3.6$$

c. By using half the pipe diameter, the new friction is 29.2 times greater than the original friction.
d. The smaller the "C" value, the greater the friction.
e. Internal rust greatly decreases the "C" value, and increases the friction, on steel pipe.

Example problem with the Hazen Williams equation:

Given: 8" PVC, Class 125 IPS pipe
Length =350 feet
Flow rate = 1000 GPM

Find: Friction, psi

Solution: $Hf = \frac{KxLxQ\ 1.852}{C\ 1.852 \times 100 \times 10\ 4.87}$

$= \frac{1050 \times 350ft \times (1000\ GPM)\ 1.852}{150^{1.852} \times 100 \times (8.095in)\ 4.87}$

$= 4.66$ feet

$= \frac{4.66 \text{ feet}}{2.31 \text{ ft/psi}}$

$= 2.0$ psi

2. The Scobey Formula. This is used almost exclusively for concrete pipe.

$$= \frac{KCxLxQ2}{C_s\ 2 \times 10\ 5.25}$$

where,

Hf	=	friction loss, feet (meters)
KC	=	a constant, 33.54 in English units, 5.59 for metric
Q	=	flow rate, CFS (lps)
Cs	=	Roughness factor from Table 6
10	=	inside diameter of pipe, inches (em)
L	=	pipe length, feet (meters)

Table 6. Friction Factors, Cs, for Scobey's Formula with Concrete Pipe

Type of joint	Cs
Rubber Gasket joint	.370
Mortar joint	.310

Basic Pipeline Computations

Mainline. A "mainline" in typical irrigation industry jargon, when discussing friction computations, has the following characteristics:

. No open outlets in the section being studied.

· Only one pipe diameter.

The Hazen-Williams equation, as discussed earlier, is used to compute the friction in such a pipeline.

Lateral. A "lateral" in typical irrigation industry jargon, when discussing friction computations, has the following characteristics:

- Outlets are open and evenly spaced along the pipeline.
- Each outlet has the same flow rate (even though it typically does not, since there is a different pressure at each outlet).
- The downstream end is capped, but there is one outlet at that end.
- Only one pipe diameter

Examples of laterals include:

- Hand move or side roll sprinkler pipes.
- Sections of gated pipe with water discharging from each gate.
- Some landscape and undertree sprinkler designs with only one pipe diameter and evenly spaced, identical sprinklers.

Figure 6 illustrates a lateral for a sample calculation.

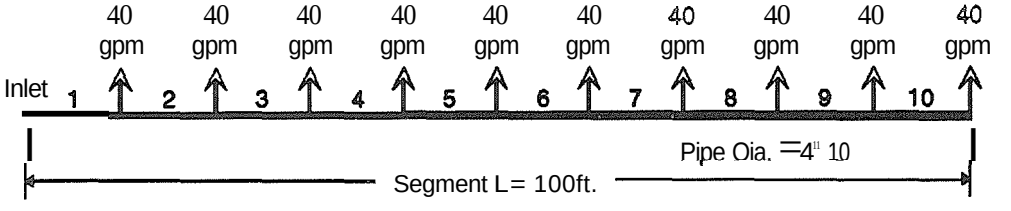


Figure 6. Lateral pipe example.

The hydraulics of a lateral can be solved by considering each pipe segment as a mainline. This is done in the following example which refers to Figure 6.

Given: 4" 10 Alum. lateral with 10 outlets onflat ground
Flow rate/outlet = 40 GPM
Pressure at dis outlet = 10 psi
Distance between outlets = 100 feet
Total length = 1000 feet

Find: Inlet pressure

Solution: Set up a calculation sheet to solve as mainline segments.

Section	dls P, feet (1)	dls out. GPM (2)	Seq. GPM (3)	pipeID inches (4)	Hf rate ftl100' (5)	Section L ft (6)	Section Hf ft (7)	Elev Chng ft (8)	PChng ft (9)	uls P ft (10)
1	23.1	40	40	4.0	.16	100	.16	0	.16	23.3
2	23.3	40	80	4.0	.58	100	.58	0	.58	23.9
3	23.9	40	120	4.0	1.22	100	1.22	0	1.22	25.1
4	25.1	40	160	4.0	2.09	100	2.09	0	2.09	27.2
5	27.2	40	200	4.0	3.15	100	3.15	0	3.15	30.3
6	30.3	40	240	4.0	4.42	100	4.42	0	4.42	34.7
7	34.7	40	280	4.0	5.88	100	5.88	0	5.88	37.6
8	37.6	40	320	4.0	7.53	100	7.53	0	7.53	45.1
9	45.1	40	360	4.0	9.36	100	9.36	0	9.36	54.5
10	54.5	40	400	4.0	11.4	100	11.4	0	11.4	65.9

Notes:
(1) For section #1, this was given. For the other sections, this equals column (10) of the previous section; Le., the dis P of one segment is the uls P of the previous segment.
(2) The outlet flow rate was given as a constant for each outlet. In some problems, this is computed at each outlet as a function of the pressure at that outlet.
(3) =(2) plus previous value of (3)
(4) Given
(5) Computed with Hazen-Williams equation. In this case, C =120 (aluminum)
(6) Given
(7) =(5) x *(6)/100*
(8) Given as zero in this case.
(9) =(7) + (8)
(10) =(1) + (9)

Answer: 65.9 psi

Lateral hydraulics can also be solved in a simplified manner using a "multiple outlet correction factor, F" as found in Table 7.

Total lateral Hf = (Total mainline Hf) x F

where the "Total mainline Hf' is the friction computed for the complete lateral length, assuming that the inlet flow traveled the complete pipe length with no outlets.

Table 7. Multiple outlet correction factor.

Number of outlets on the lateral	Correction Factor, F
1	1.000
5	.457
10	.402
20	.380
40	.364
100	.356
more than 100	.351

To compare with the previous solution, the following example is given:

Given: Same lateral problem as earlier

Find: Inlet P, by finding Total Hf' using correction factor "F"

Solution: With a flow rate of 400 GPM, the rate of Hf is

11.4 ft/1 00'

From Table 6, F = 0.402 (10 outlets)

$$\text{Total Hf} = \frac{11.4 \text{ ft of Hf}}{100' \text{ of pipe}} \times 0.402 \times 1000' \text{ length}$$

$$= 45.83'$$

Inlet P = d/s P + Hf + Elev. change
= 23.1' + 45.8' + 0
=68.9 ft.

This compares reasonably well with the 65.9 ft found by using the detailed procedure.

Minor Losses

Friction losses through fittings and valves are classified as "minor" losses because their magnitude is generally much lower than the pipeline friction. In some designs, however, these "minor" losses can be very significant, especially (i) if the valves and fittings are undersized, (ii) if the available pressure is very low, or (iii) if the valves are partly throttled (closed off).

Table 8 presents some resistance coefficients (K) for computing minor losses. These factors should be used for approximations, only. There are large differences in the smoothness, inside diameters, and construction of various valves and fittings. Whenever possible, designers should obtain a graph offlow rate versus friction loss through the device in question from the manufacturer of that device.

Table 8. General Resistance Coefficients (K) for Valves and Fittings (adapted from Crane, 1976). $Hf minor = \frac{K \times v^2}{64.4}$ where Hf = friction in feet, and V= velocity in fPS.

Valve of Fitting Description	K value for various nominal diameters						
	Alldia.	1/2"	2"	4"	8":	12"	20"
Gate valve, screwed		0.3	0.2	0.1			
Gate valve, flanged				0.2	0.1	0.1	0.1
Globe valve, screwed		13.0	7.0	5.5			
Globe valve, flanged				6.5	5.8	5.8	5.8
Angle valve, screwed		9.0	2.0	1.0			
Angle valve, flanged			2.5	2.0	2.0	2.0	2.0
Swing check valve, flanged	2.0						
Tee							
Screwed, line flow	0.9						
Screwed, branch flow		2.2	1.3	1.0			
Flanged, line flow			0.2	0.2	0.1	0.1	0.1
Flanged, branch flow			0.8	0.7	0.5	0.4	0.4
90 degree elbow							
Reg. screwed		2.0	0.9	0.6			
Reg. flanged			0.3	0.3	0.2	0.2	0.2
Long radius screwed		1.0	0.4	0.2			
Long radius flanged			0.3	0.2	0.1	0.1	0.1
Sudden enlargement	1.0						
Pipe inlets (on a wall)							
Bell-mouthed	0.1						
Square edged	0.5						
Inward projecting	1.0						
Pipe exit (all kinds)	1.0						
Butterfly valves			0.8	0.8	0.6	0.5	0.4
Mitre Bends (single)							
30 degree				0.1	0.1	0.1	0.1
45 degree				0.3	0.2	0.2	0.2
90 degree				1.0	0.8	0.8	0.7
Sudden contraction (with various ratios of small dia. to large dia.). Based on Dia. of smaller pipe							
0.8	0.2						
0.7	0.3						
0.6	0.3						
0.4	0.4						
Sudden enlargements (with various ratios of small dia. to large diameter). Based on Dia. of smaller pipe							
0.8	0.1						
0.7	0.3						
0.6	0.4						
0.4	0.7						
Valve opening elbow &valve on hand move sprinkler system (from old Ames Handbook)	7.5						

Entrance and Exit Losses

Many surface irrigation pipelines convey water from open canals with a very small head (Le., very little elevation difference between the canal and the field). In these conditions, the design of the inlet and discharge ends of the pipeline can have a significant effect upon the flow which a pipe will carry. As an example, it is often beneficial to upgrade existing culvert end conditions rather than purchase a new and larger pipe.

The total loss of energy with water flow in a short pipeline between a canal and a field is:

Total Hf = Hf entrance + Hf pipe friction + Hf exit + Hf valves

Entrance Losses. When water enters a pipe or culvert from a canal, the entrance loss is expressed as:

$$\text{Hf entrance} = \frac{Ke \times V2}{64.3}$$

where Ke is found in Table 8
V is the velocity (feet/sec) in the pipeline
Hf is the loss in feet

It must be noted that in order to get the water into a pipe from an open canal or reservoir, there is an additional component of energy which must be supplied at that point - the velocity head itself. This is not a "loss" at this point - it may be lost at the pipe exit. However, it must be supplied at the entrance. In simple terms, in order to get water into a pipe, the water must start with enough energy to flow, plus to get past any obstructions, plus to overcome friction at the entrance.

Exit Losses. Exit losses for a straight pipe discharging into the atmosphere or into a body of water are equal to the velocity head of the water at the point of discharge. For this case:

$$\text{Hf exit from a straight pipe} = \frac{Ko \times V2}{64.4}$$

$$= \frac{V2}{64.4}$$

since Ko = 1.0

If the pipe is gradually enlarged with a cone before discharge, the loss will be considerably less. It will consist of a small loss due to pipe enlargement, plus the final discharge velocity head. It is described by the formula:

$$\text{Hf exit with gradual enlargement} = \frac{Ko \times V1^2}{64.4} + \frac{V22}{64.4}$$

where Ko values are found in Table 9
V1 is the velocity in the smaller pipe (ftIsec)
V2 is the velocity at the large end of the cone (ftIsec)
Hf is the minor loss (feet)

Table 9. Ko values for gradual enlargements in pipes (King and Brater, 1963).

02101 (1)	Angle of cone, degrees (2)								
	2	4	6	8	10	15	20	30	60
1.1	.01	.01	.01	.02	.03	.05	.10	.16	.23
1.4	.02	.03	.03	.04	.06	.12	.23	.36	.53
1.8	.03	.04	.04	.05	.07	.15	.28	.44	.65
2.0	.03	.04	.04	.05	.07	.16	.29	.46	.68
Infinite	.03	.04	.05	.06	.08	.16	.31	.49	.72

Notes
(1) D2/D1 is the ratio of diameter of the larger pipe to the diameter of the smaller pipe.
(2) The angle of cone is twice the angle between the axis of the cone and its side.

Example problem. To illustrate the effect, an example problem will consider two culvert designs. One will be the original design which did not pay attention to entrance and exit losses. The second will use the original pipe but with modified end conditions. The design conditions for the original construction were:

Given:
- Elevation change between the canal water surfaces on either side of a road =0.25 feet
- Pipeline length = 40 feet
- 14" Schedule 20 steel pipe
- No modifications of either the inlet or discharge; it, both are straight pipe ends. The pipe entrance is a "protruding pipe"
- No valves in the pipeline.

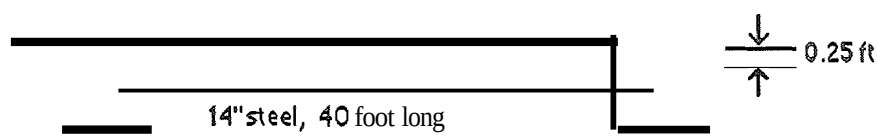


Figure 7. Original culvert design for example problem. Straight pipe ends.

Find: Flow rate

Solution: The flow rate would produce a total head loss of 0.25 feet.

Total Hf =Hfentrance + Hfpipe friction + Hfexit + Hfvalves

$$= \frac{Ke \times V2}{64.4} + \text{Hfpipe friction} + \frac{Ko \times V2}{64.4} + 0$$

$$= \frac{(Ke + Ko) \times V2}{64.4} + \text{Hfpipe friction}$$

The flow rate must be solved for by trial and error. To do this, use the form of the Hazen-Williams equation with Velocity rather than Flow Rate.

$$\text{Hfpipe friction} = \frac{10.50 \times L \times GPM^{1.852}}{10^{4.87} \times C^{1.852}}$$

$$= \frac{55.15 \times L \times V^{1.852}}{10^{1.166} \times C^{1.852}}$$

where

L	=	pipe length, feet
V	=	velocity of water in the pipe
10	=	inside diameter of the pipe, inches
C	=	Hazen-Williams C value

For this problem

Ke	=	1.0 (Table 8)
Ko	=	1.0
L	=	40 feet
10	=	13.38 inches (from steel pipe dimension manual)
Total Hf	=	0.25 feet
C	=	100 (15 year old steel pipe, Table 5)

$$\text{Total Hf} = \frac{(Ke \times Ko) \times V2}{64.4} + \text{Hfpipe friction}$$

$$\text{TotalHf} = \frac{(1.0 + 1.0) \times V2}{64.4} + \frac{55.15 \times 40\text{ft} \times V^{1.852}}{13.38\text{in}^{1.17} \times 100^{1.852}}$$

$$= (.031 \times V2) + (.021 \times V^{1.852})$$

Since the Total Hf is known,

$$0.25 \text{ feet} = (.031 \times V2) + (.021 \times V^{1.852})$$

Solving by trial and error, input several "V" values until the answer equals 0.25.

V = 2.3 fps

Therefore,

$$Q = V \times A = 2.3 \text{ fUsec} \times \frac{\pi \times (13.38,,)2}{4} \times \frac{449 \text{ GPM}}{12'/\text{ft} \quad \text{CFS}}$$

$$= 1005 \text{ GPM}$$

Solving for the individual components,

The entrance loss = $\frac{1.0 \times (2.3 \text{ ft/sec})^2}{64.4}$ = .08 feet

The exit loss = $\frac{1.0 \ (2.3 \text{ ft/sec})2}{64.4}$ = .08 feet

The pipe friction = .021 x (2.3 ft/sec) 1.852 =.10 feet

Total loss = .08 ft + .08 ft +.10 ft = .26 feet
(not .25 due to roundoff)

2. Find: Flow rate with modified pipe ends.

Assume that a 20" x 14" reducing cone is welded onto each pipe end, and that the cone has the dimensions and installation shown in Figure 8. The 20" cone gives approximately twice the area of a 14" pipe.

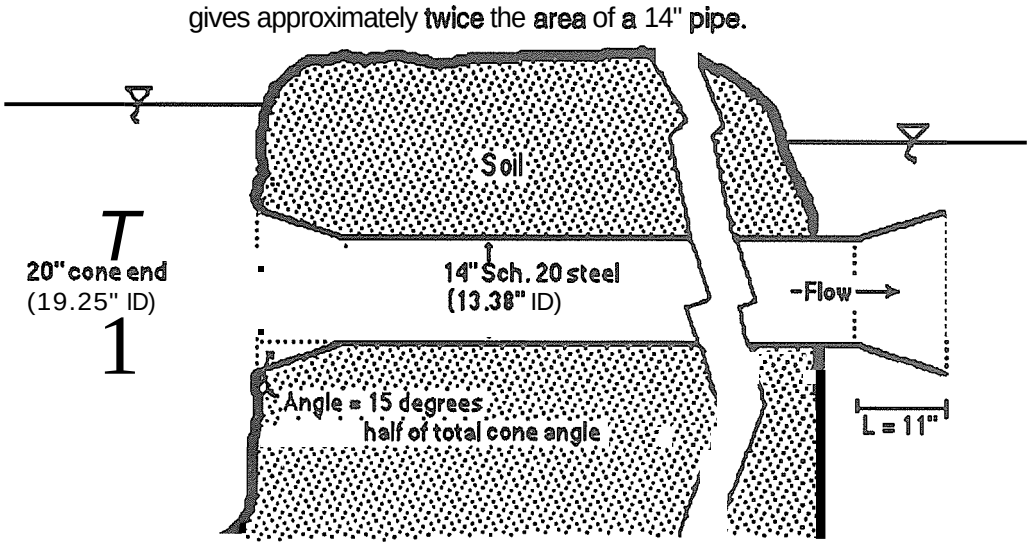


Figure 8. Cone at ends of example problem pipeline. Cone angle =30 degrees.

Solution: The end conditions can be defined as follows:

Entrance: Slightly rounded entrance, followed by a gradual reduction in diameter to 14" pipe. Gradual reductions in diameter produce almost no minor loss. Therefore, the entrance loss is due to solely to the slightly rounded entrance at the 20" opening. The *(rId)* ratio, assuming a polygon with similar angles (of the soil wall to the cone face) would be about 0.40. However, for design, itwould be better to assume an

(rld) of about 0.04 to account for unknown qualities of construction finishing. The entrance loss is therefore

$$Hf_{entrance} = \frac{Ke \times V2^2}{64.3}$$

where Ke =0.24 (from Table 9, *rld* =.04)
V2 = units of feet/sec. V2 is based upon velocity in 20" pipe.

The ratio between the two velocities is:

$$V2 = V1 \times (D1/D2)^2$$
$$= V1 \times (13.38"/19.25,,)^2$$

Therefore, for this problem,

$$Hf_{entrance} = \frac{0.24 \times (.48 \times V1)^2}{64.4}$$

$$= .000867 \times V1^2$$

Exit losses: Unlike gradually converging sections (as with the entrance), the minor loss in gradually diverging sections must be accounted for. As can be seen in Table 9, the losses increase rapidly after a cone angle of between 10 and 15 degrees (accounting forthe sum of the angles on the two sides). However, very long cones with small angles are often expensive or unavailable. Therefore, this example uses a shorter, more abrupt cone.

As described earlier,

$$Hf_{exit\ with\ gradual\ enlargement} = \frac{Ko \times V1^2}{64.4} + \frac{V2^2}{64.4}$$

where for this problem,

Ko = 0.37 (Table 9 *D2/D1* = 1.44, angle = 30 deg.)
V2 = 0.48xV1

Therefore, for this problem

$$Hf_{exit\ with\ gradual\ enlargement} = \frac{0.37 \times V1^2}{64.4} + \frac{(0.48 \times V1)^2}{64.4}$$

$$= .0093 \times V1^2$$

Pipe friction: From the earlier calculations on the original design,

$$Hf_{pipe\ friction} = \frac{55.15 \times L \times V1^{1.852}}{1D1.166 \times C1.852}$$

$$= 0.021 \times V1^{1.852}$$

Total energy loss: For this problem,

Total Hf = Hfentrance + Hfexit + Hfpipe friction

0.25 feet $= (.000867 + .0093) \times V1^2 + 0.021 \times V11.852$

0.25 feet $= (.0102 \times V1^2) + (0.021 \times V11.852)$

Solving by trial and error,

V = 2.98 ftlsec
Q = 1306 GPM

Summary - Entrance and Exit Losses. The following points can summarize facts about entrance and exit losses on culverts and irrigation pipelines:

1. Entrance and exit losses are very important in situation of almost no head loss, such as with inverted siphon and culvert designs.
2. The importance ofa well designed entrance and exit decreases, relatively, asthepipeline length increases (Le., as the pipe friction becomes a greater percentage of the total energy loss).
3. For an existing culvert or pipe entrance, it is often worthwhile to modify the existing entrance and exit to obtain an increased flow rate with the same head loss.
4. In some conditions it may be more economical to install a line gate or butterfly valve several feet after a culvert entrance rather than install a canal gate at the pipe entrance. This can allow for the use of welded reducing cones at the entrance rather than careful concrete formwork for a smooth entrance. A smaller line gate can be used because it is placed in the smaller pipe.
5. Exit conditions are more sensitive to cone angles than entrance angles. Cone angles should not exceed 10-15 degrees in orderto prevent stream turbulence and high energy losses.
6. Although not specifically stated in the example problem, the use of an expanding cone at the discharge end will reduce erosion downstream of the pipe exit because the exit velocity has been reduced. This is a very valuable point in some pump and culvert discharge designs.

Submergence **Requirements** and **Vortex** Creation.

Vortexing is well understood by most people as the "whirlpool" which is created at the drain of a bathtub when the tub empties. A simple experiment will demonstrate the effects which are discussed in this section: Fill a bathtub at least halffull. Then pull the drain plug. When the water level gets within some critical distance of the drain a vortex or "whirlpool" will begin to develop. This vortex will allow air to partially fill the drain entrance and reduce the flow rate. If the drain was a supply to a pump, the vortex would allow air into the pump, which would interrupt the normal continuous water supply. The alternating air-water supply into the pump would cause hydraulic pulsations in the pump flow and mechanical overloading of the sleeve bearings and impeller guides (Karassik et.al. 1976).

Pipe entrances and pump intakes must have a sufficient depth of water above them to prevent the creation of vortexing. In addition, pump pit walls should have proper configurations relative to the pump inlet in order to minimize the creation of vortexes. See Karassik et.al. (1976) for a description of pit design. The submergence required for vortex suppression can be approximated (ASAE EP369, 1985) by:

$$Sv = \frac{2 \times Q}{\pi \times O^2}$$

where Sv = minimum submergence in feet (m)
Q = flow rate in CFS (m^3/s)
O = inlet pipe diameter in feet (m)

As an example, consider a 12" inlet and a flow of 3 CFS from a reservoir into the pipeline. The required submergence above the pipeline, to prevent the introduction of air, is:

$$Sv = \frac{2 \times 3 \text{ CFS}}{\pi \times 0^2} = 1.9 \text{ feet}$$

For the same pipe, and assuming that the pipe has a straight, square inlet, the energy required to move the water into the pipe equals:

$$\text{Inlet energy required (Se)} = \frac{V^2}{64.4} + \frac{Ke \times V^2}{64.4}$$

where V = velocity, feet/sec (3.82 fps in this problem)
Ke = factor for entrance loss (0.5 in this problem)

Therefore, the submergence (Se) required just to get the water into the pipeline is:

$$Se = \frac{1.5 \times 3.82^2}{64.4} = 0.34 \text{ feet}$$

For the pipeline design, use the larger of the two values (Se vs. Sv). Therefore, the design sUbmergence should be 1.9 feet above the top of the pipeline. If it is physically impossible to do this, one may construct a horizontal baffle plate above the pipe inlet in order to increase the distance between the intake and the water surface.

Hydraulic Grade line and Energy line

Full Pipe Case. Some designers make frequent use of a graphic representation of energy and pressure at each point along a pipeline. The distance between a Hydraulic Grade Line (HGL) and the pipeline represents the pressure atany point. The Energy Line (EL) is slightly higher than the HGL because it also includes the velocity head.

Although the calculation and sketching of the EL and HGL are common in engineering offices, these are fairly rare tasks in irrigation dealership work for on-farm irrigation system design. For high pressure on farm irrigation systems such as drip and sprinkler (especially those using pressure regulators) there is rarely a need to draw the EL and HGL. However, for very low pressure surface systems there can be many cases where such drawings are valuable design tools. Specifically, there are situations on low pressure systems in which a vacuum may be created at an intermediate point in a pipeline if the hydraulics are solved simply by examining beginning and end conditions on the pipeline.

Construction of the HGL and EL is illustrated in the following example:

Given: A PVC pipeline carries 1300 GPM a distance of 1000 feet.
The first 600 feet of pipeline is 12" 100 foot head PIP pipe.
The last 400 feet of pipeline is 8" 100 foot head PIP pipe.
Assume a very smooth, gradual transition between the two sizes.
Slope = 0.25% downhill from the inlet.
Discharge = into the air from a plain pipe end.
Water source = a reservoir.
Assume no gates or valves in the pipeline.

Find: Required water level in the reservoir

Solution: Using the Hazen-Williams equation,
the rate of friction in the 12" (11.96" ID) pipe = $\frac{0.32 \text{ ft of friction}}{100 \text{ feet of pipe}}$

Rate of friction in 8" (7.97" ID) = $\frac{2.33 \text{ ft}}{100 \text{ ft of pipe}}$

Velocity of water in the 12" pipe = 3.7 feet/sec

Velocity head in 12" = $\frac{(3.7 \text{ ft/sec})^2}{64.4 \text{ ft/sec}^2} = .21$ feet

Velocity head in 8" = 1.08 feet

Assuming a straight edged "ordinary" entrance to the pipe from the reservoir, the entrance loss is determined as follows:

From Table 8, K = 0.5
Entrance loss = 0.5 x (velocity head)
= 0.5 x .21 feet
= .11 feet

In addition, one must consider the following:

- At the entrance some of the water energy must be converted to velocity head to get the water moving.
- At the exit all of the energy in the form of velocity head is lost.

To draw an EL and a HGL, one must begin at a pointwhere conditions are known. The only such point in this problem is at the downstream end. The following end conditions are known because it is a situation of a pipe discharging into the atmosphere:

Pressure =0
Energy =velocity head =1.08 feet.

The EL and HGL are parallel except at transitions. Both have a slope which equals the rate of friction in the pipeline.

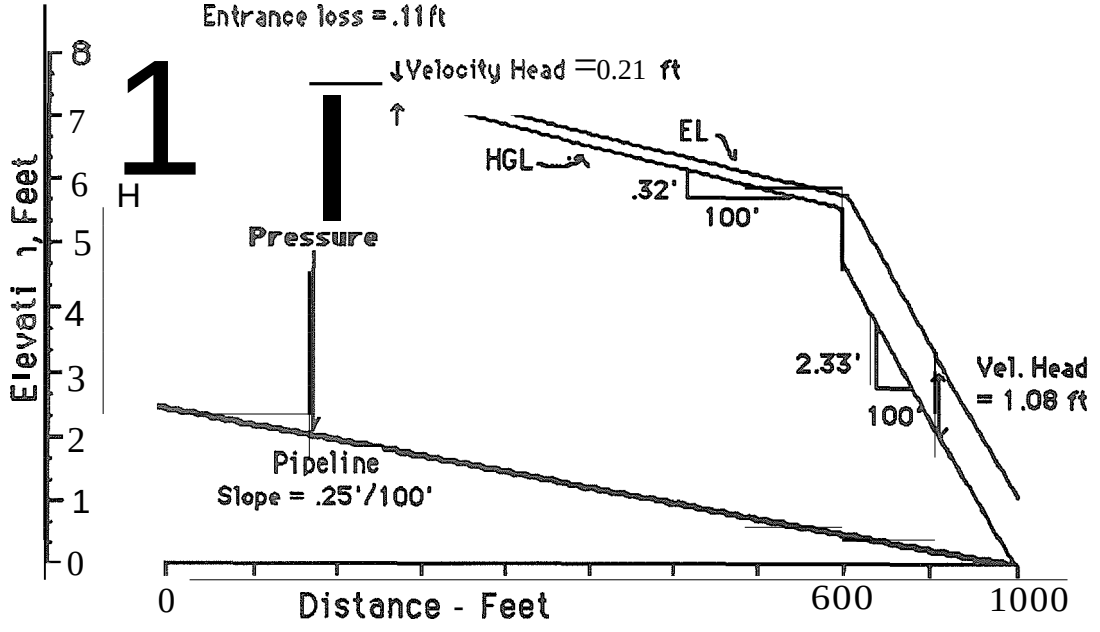


Figure 7. Hydraulic Grade Une (HGL) and Energy Une (EL) for the example problem with a full pipe.

The water height at the reservoir inlet must provide all of the energy which is expended.

The values of "H", or the height of water at the inlet, is calculated as follows:

H = *DIS* pipe exit energy (Vel. head)
+ Hf in 8"
+ Hf in 12"
+ Entrance loss
- Elevation change

Solving,

$$H = 1.08 \text{ feet} + \frac{2.33 \text{ ft}}{100 \text{ ft}} \times 200 \text{ ft} + \frac{.32 \text{ ft}}{100 \text{ ft}} \times 800 \text{ ft} + .11\text{ft} - \frac{.25\text{ft}}{100 \text{ ft}} \times 1000 \text{ ft}$$

= 8.38 ft - 2.50 ft
= 5.88 ft

Partially Full Pipes. Surface irrigation pipelines frequently have sections of pipeline which flow partially full all or part of the time. This occurs in systems with low pressures and downhill sloping ground. The use of Hydraulic Grade Lines (HGL's) is particularly useful in determining whether this condition exists. Oversight of this condition will result in less than the design flow rate and possible pipe collapse and water hammer problems.

To illustrate this problem, consider the situation below:

Given: Flow rate =1300 GPM
Pipe length = 2000 feet
Pipe specifications = 12" 100 foot head PVC pipe
First 1000 feet of pipe are on a zero slope
Last 1000 feet are on a 1% downhill slope.
Pressure required at the downstream end = 5 feet

Find: Required pressure in the pipeline at the inlet.

Solution #1 (incorrect):

Using the Bernoulli equation and examining the conditions at each end,
Formula: $P_{uls} = P_{dls} + H_f + \Delta Elev$

where

Δ Elev is positive if the *dls* end is higher than the *uls* end

and

Hf is calculated using the Hazen- Williams equation:

$$H_f = \frac{1050 \times L \times Q^{1.852}}{C^{1.852} \times 100 \times 10^{4.87}}$$

Solution:

Δ Elev = (-.01 x 1000 feet) = -10 feet

$$H_f = \frac{1050 \times (2000 ft) \times (1300\ GPM)^{1.852}}{150^{1.852} \times 100 \times 11.96\ inches^{4.87}}$$

= 6.4 feet

P_{dls} = 5 feet

P_{uls} = 5 ft. + 6.4 feet + (-10 feet) = 1.4 feet

Solution #2 (correct):
The hydraulics of pipelines with low pressures and changes in downhill slope should be examined using a hydraulic grade line, or at least by determining pressures at the various breaks in grade. If air vents and vacuum relief valves are properly installed, negative pressures (vacuums) will never actually develop at those points.

An examination of the HGL in Figure 8 shows that the required upstream pressure is 3.2 feet. not 1.4 feet. The pipe flows partially full between the grade change and a distance of 265 feet downstream. This distance would be less than 265 feet at a higher flow rate, and more at a lower flow rate.

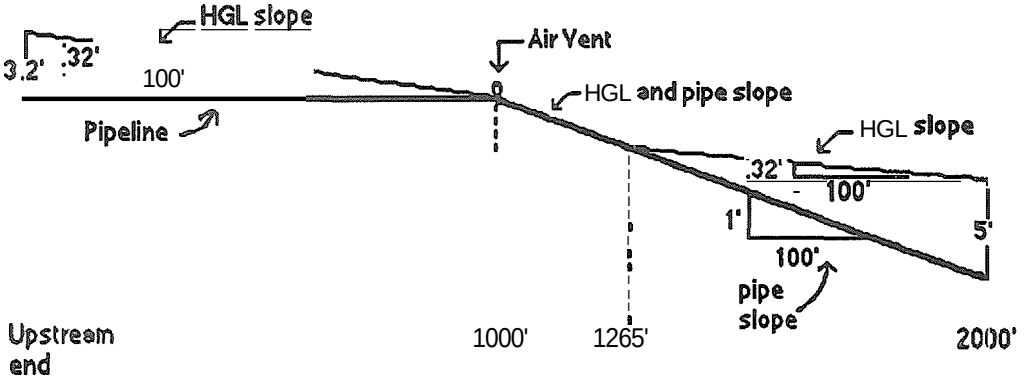
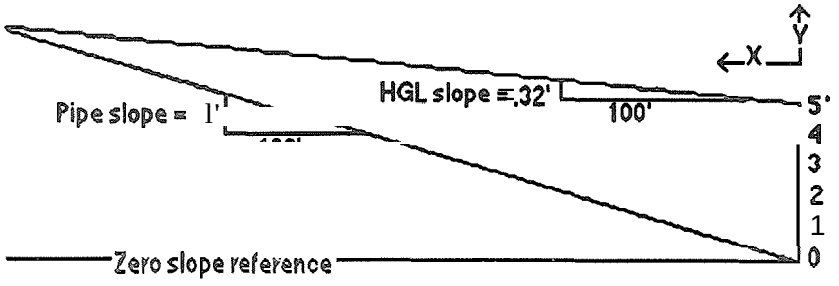


Figure 8. HGL for example problem showing vacuum at break in grade.

Solution #1 assumed a full pipe for the complete distance. This would be valid if a vacuum could be created at the break in grade. However, the vacuums might collapse the PVC pipe, and should be avoided by proper installation of air vent/vacuum relief valves at high points as shown in Figure 8.

The point of intersection of the HGL and the pipeline can be estimated by computing the distance to the intersection ofthe two slopes (HGL and pipeline), as illustrated in Figure 9. Due to the hydraulic jump at that point (at the transition between a partially full pipe and completely full pipe), there will be some additional energy loss which is not shown on the HGL in either Figure 8 or 9.

Figure 9. Calculation of distance to transition point between full and partially full flows.



Equation of pipe slope: $Y = .01X$
Equation of HGL slope: $Y = .0032X + 5$

At the point of intersection, both lines are at the same elevation. Therefore,

$$.01X = .0032X + 5$$

Solving for X,

$$.01X - .0032X = 5$$
$$.0068X = 5$$
$$X = 735 \text{ feet}$$

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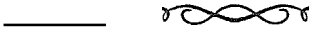
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CHAPTER 17



PIPELINE SIZING - ECONOMICS AND STRATEGIES

PIPELINE SIZING - ECONOMICS AND STRATEGIES

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PIPELINE SIZING - ECONOMICS AND STRATEGIES

Introduction

There are many techniques used by designers to determine the proper size of pipelines. Typical strategies include the following:

I. Rules of thumb.

A common rule of thumb is to design close to (yet not exceeding) a flow velocity of 5 feet per second. This is a simple rule and has some merit because

a. Higher velocities often cause water hammer problems,
b. It minimizes the required design time and
c. Friction losses at higher velocities can become very high, requiring high pump pressures.

However, by using a larger pipe diameter the water hammer potentials and friction loss will be even less. One could logically ask whether 4 fps (feet per second) is the more proper velocity (or 3.5 fps, or 3.0 fps??).

II. Fixed pressure loss available.

This is a valid strategy in many situations. For example, water may need to be delivered from a canal to a point in a field several hundred feet away. No pumps may be available, or the farmer way not want to use a pump. The problem then becomes one of determining the minimum size of pipeline which will have less energy loss (including velocity head, entrance losses, and friction) than the total energy available to lose.

III. Uniformity considerations.

Irrigation systems with multiple outlets (e.g., sprinklers ordrip) may requirethatthe pressure variation between outlets not exceed a certain amount. That pressure variation must include both elevation and friction effects.

IV. Economic pipe sizing.

In irrigation design, this term refers to a technique which selects a pipe size giving the lowest annual cost. The annual cost is a combination of both annual pumping (pump and pipeline maintenance, plus $/kw-hr charge) and annualized fixed costs (pump plus pipeline, including all initial installation costs). This strategy recognizes that for a pipeline on level ground, a flow of 1000 GPM, for example, can be pumped through a wide variety of pipe sizes. The smaller the pipe size, the lower the initial pipe cost. However, the smaller the pipe size, the larger the annual pumping cost.

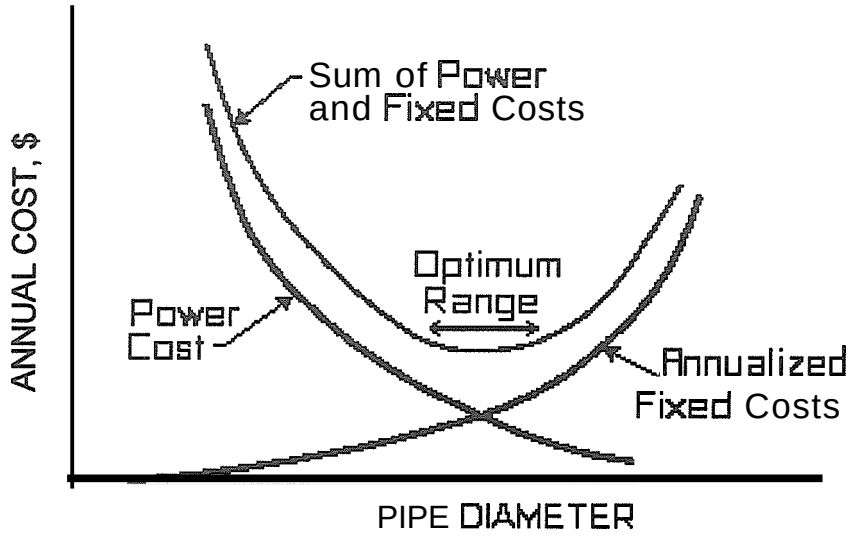


Figure 1. Effect of pipe diameter on total annual cost of ownership and operation. (Keller, 1976)

Mixed strategies. An irrigation pipe network supplied by pumps is generally sized with a combination of (a) economic pipe sizing plus (b) sizing to use up available pressure. The economic pipe sizing is to be used along the "critical hydraulic path". The "critical hydraulic path" is the route taken by a molecule of water enroute from the pump to the point in the system which requires the most pressure at the source. It is the pipe sizing along the "critical hydraulic path" which will determine the annual pumping bill. All other branches in the network can be sized to use up available pressure.

Fixed Pressure loss Available - Example

Given: - A closed pipeline must deliver water from an open canal to a point 500 feet away.
- Pipeline material = mortar-jointed non-reinforced concrete
- Flow rate = 6 CFS
- Elevation of canal water surface =3 feet above discharge end
- Discharge end = plain smooth pipe
- Level ground
- Waterman C-10 canal gate at pipe entrance

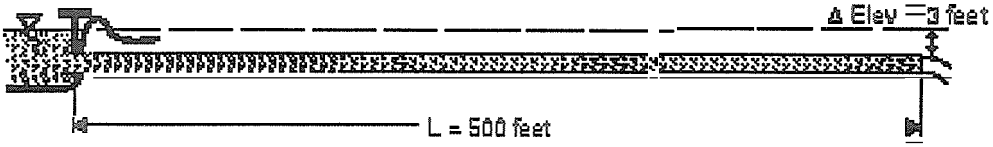


Figure 2. Fixed pressure loss available example.

Find: Minimum size of pipeline which can do the job.

Solution: The total energy loss must not exceed 3 feet

Components of energy loss = Entrance loss through C-10 valve
+ Friction in 500 feet
+ Velocity head

This problem can be solved directly using the proper mathematics. However, the method shown here is a simple trial and error solution.

Step #1 - Assume that 80% of the energy loss will be friction along the 500 feet. Determine what size concrete pipe has approximately that amount of friction.

Estimated loss in friction $= 0.80 \times 3$ feet $= 2.4$ feet $= 4.8$ ft/1000 ft.
Using the friction loss graph in the appendix for concrete pipe, with a C_S of 0.310,

an 18" pipe has a Hf of $\frac{3.2 \text{ ft}}{1000 \text{ feet}} \times 500$ feet = 1.6 ft

16" pipe has a Hf of $\frac{6 \text{ ft}}{1000 \text{ feet}} \times 500$ feet = 3 feet (too high)

20" pipe has a Hf of $\frac{1.8 \text{ ft}}{1000 \text{ feet}} \times 500$ feet $= 0.9$ ft.

Step #2 - Work through the complete problem with the 18" pipe. If the total energy loss is greater than 3 feet, then try the 20" pipe

For an 18" pipe (non-reinforced concrete, mortar jointed):

Velocity (18" ID) = 3.4 feet/sec

Velocity Head $= V^2/64.4 = \frac{(3.4 \text{ ft/sec})^2}{64.4 \text{ ft/sec}^2} = .18$ feet

For the equivalent length of pipe in determining the entrance loss, combine the values for an "ordinary entrance" and an "open gate valve", and then convert the lengths from steel to concrete pipe.

Steel pipeline equivalent lengths from the table in the appendix:
Ordinary entrance equiv. length = 25 feet
Open gate valve equiv. length = 10 feet

Equiv. length $= (25 \text{ ft} + 10 \text{ feet}) \times \frac{.85 \text{ ft of concrete}}{1.0 \text{ ft of steel pipe}} = 30$ feet of cone.

Minor loss = 30 feet of pipe x $\frac{3.2 \text{ ft of Hf}}{1000 \text{ ft of pipe}} = 0.1$ feet

Total energy loss = Veloc. Head + Minor loss + Hf
= 0.18 ft. + 0.1 ft + 1.6 ft
= 1.9 ft
Since 1.9 feet is less than the allowable 3.0 feet, select is" pipe.

Step #3 - Not needed in this case. However, once the energy losses (not including elevation changes) have been determined for one pipe size, the losses for any other pipe size and its fittings can be closely approximated by applying the formula below, which is derived from the Hazen-Williams equation:

$$(\text{ENERGY LOSS})S = (\text{ENERGY LOSS})A \times \left(\frac{ID_A}{ID_S}\right)^{4.9}$$

For example, the energy loss through a 20" pipe can be estimated based upon the results of the 18" pipe calculation.

Energy loss in a 20" pipe and fittings = 1.9 feet x *(18"/20")4.9*

= 1.9 feet x .60

= 1.1 feet

Other notes: The 18" pipe will pass more than the required 6 CFS flow rate. At a 6 CFS flow rate only 1.9 feet of the available 3.0 feet of energy are used up. In order to actually obtain precisely 6 CFS, the C-10 gate must be partially closed down. A flow meter in the pipeline is needed to know how far to close the gate.

Critical Hydraulic Path

Within an irrigation pipeline system, there is some point which is harder to reach (hydraulically) than any other point. If a pump or reservoir can provide enough pressure to that critical point, every other point in the system will have more than enough pressure available.

The critical path is the hydraulic route in the pipeline network through which a water molecule must travel enroute from the source to the critical point. Once a preliminary pipe network configuration has been selected, it is not always obvious which point is the critical point. On a simple symmetric system, it would be the point which is the most distant from the pump and highest in elevation. In real systems the critical path must be determined by a trial and error solution (which fortunately is solved rather rapidly).

Economic pipe sizing should be used along the critical hydraulic path. Therefore, in a trial and error solution, each possible critical hydraulic path is sized using economic pipe sizing. The path which has the highest pressure requirement is then designated as the critical path. The other paths are then resized (using smaller pipe sizes) to use up any available excess pressure.

Background - Economic Pipe Sizing

The costs of owning and operating a pipe, as illustrated in Figure 1, consists of:

a. Fixed costs = $ for installed pipe (material + labor + taxes)
b. Operational = $ for power and maintenance

To examine the annualized cost of owning and operating a pipe, the initial (fixed) costs, P, must be converted to an annual cost. This is done with the use of Capital Recovery Factor (CRF).

$$CRF = \frac{\text{Annual cost}}{\text{Present cost}} = \frac{i(1+i)^n}{(1+i)^n - 1}$$

where i = interest rate
n = number of years life, or the life of a loan

Some designers also use an inflation factor for annual power costs. Others feel such an adjustment is futile, based upon the fluctuations of power supply and costs. The factor to use as an adjustment (Keller, 1976) is called the "Equivalent Annualized Energy Cost Factor" (EAE), to be used as follows:

$$EAE = \frac{\text{Adjusted Annual Energy Cost}}{\text{Present Annual Energy Cost}} =$$

$$\left[\frac{(1+r)^n - (1+i)^n}{(1+r) - (1+i)}\right] \times \left[\frac{i}{(1+i)^n - 1}\right]$$

where r is the annual percentage of energy cost escalation.

For most simple pipeline economic sizing problems, the total annual cost is determined by:

Total annual cost of pipe = Annual fixed + Adjusted annual operational
= (P x CRF) + (Present annual power x EAE)

The following points should be noted or re-emphasized regarding "Economic Pipe Sizing":

1. For a given flow rate, initial cost is lowered by reducing pipe size. However, this increases friction, so the annual power cost increases.
2. An "economic pipe sizing" for lowest annual cost does not guarantee proper pressure distribution throughout a system.
3. The power bill is for (Horsepower-Hrs) in a year. Therefore, economics are very different in the Midwest than in the San Joaquin Valley of California. Crop evapotranspiration and effective rainfall rates during the irrigation season are different at every location.

4. The pump Horsepower is determined by the pressure requirements along the "critical path". All other points in the system have excess pressure. (See previous paragraphs).
5. Since the pipe sizes along the "critical path" determine the HP, these are sized by "Economic Pipe Sizing" methods. All the other pipes are sized by other criteria.
6. It may be that only a segment of the critical path is sized by "Economic Pipe SiZing". Perhaps the last lateral is sized by other criteria (allowable change in Pressure, or selection of a pipe size which allows no pressure regulators, orwhich can be moved easily).
7. Economic pipe sizing is only one piece of the economic picture. Other decisions may have a greater impact on the annual cost. For example, a decision to design for volumetric control may result in very high irrigation efficiencies and therefore less pumping peryear. That design choice will have more beneficial impact upon annual power bills than economic selection of the incorrect design layout.
8. There is a "Break-Even Flow Rate (SEQ)" at which there is no annual cost difference between using a large pipe with a low friction , versus the next smaller pipe size with a high Hf.

Example Economic Pipe Sizing Problem - Simple Mainline (Level 11

The following illustrates the most simple type of economic pipe size selection. The system consists of one mainline which has no branches or intermediate outlets. It conveys water from point A to point B.

Given:

Present energy cost = $.08/kw-hr
Interest rate =12%
Annual energy cost escalation = 5%
Years life of investment = 10
Present price of PVC =$70/100 lb of material
Tax on material = 6%
Installation costs (fittings, glue, trench, etc.)
8" = $ 90/100'
10" = $130/100'
12" =$160/100'
Impeller & shaft efficiency = 75%
Motor efficiency =90%
Crop = Deciduous orchard with clean cultivation
Location = Southern San Joaquin Valley
Irrigation Efficiency for both peak and annual ET = 85%
Field Size = 160 acres
Assume 18 hours/day operation during peak ET period
Neglect leaching requirement
Pipe length = 1320'
Need Class 100 PVC due to pressure requirements.

Find: Most economical mainline pipe size

Solution: Annual cost = Annual Power + Annualized Initial Cost

**This problem will be solved without accounting for the cost of a pump itself.

Assumption: 3" effective rainfall/year
Neglect Leaching requirement for example simplicity.

Step #1 - Find Required hours of operationlyear

From ET data published by the California Dept. of Water Resources,
Average annual ET = 39.0"
Peak monthly ET during an average year =7.76"

Net to apply = 39" - 3" = **36"/year** (neglecting Leaching Requirement)

Peak ET = $\frac{7.76"}{31 \text{ days}}$ =.25"/day

Hours of operation/year
To calculate this, use Net/Net, or Gross/Gross

Since the peak Irrigation Efficiency (IE) equals the annual IE,

$$\frac{\text{Net requiredlyear}}{\text{Net app. rate}} = \frac{36"}{(.25"/18 \text{ hrs})} = 2590 \text{ hours}$$

Step #2 - Find design GPM

$$GPM = \frac{(\text{in/hr}) \times (\text{Area})}{96.3 \times (IE/100)} = \frac{(.25"/18 \text{ hr}) \times (160 \text{ ac}) \times (43560 \text{ sq. Wac})}{96.3 \times .85} = 1183 \text{ GPM}$$

Step #3 - Determine economic multipliers

$$CRF = \frac{i(1+i)^n}{(1+i)^n-1} = \frac{.12(1.12)^{10}}{1.12^{10}-1} = .177$$

$$EAE = [(1+r)^n - (1+i)^n] \times \left(\frac{}{(1+r)-(1+i)}\right)\left(\frac{}{(1+i)^n-1}\right) = \frac{[1.05^{10} - 1.12^{10}]}{[1.05-1.12]} \times \frac{.12}{(1.12^{10}-1)}$$

$$= 1.202$$

Step #4 - Determine annual fixed cost for each pipe size

Nom. Cia. inches	Weight lb/100'	Cost $1100'	Tax $	Install $	Total Fixed $/100'	Annual Fixed $/100'
6	212.6	148.82	8.93	60	217.75	38.54
8	358.9	251.23	15.07	90	356.30	63.06
10	557.7	390.39	23.42	130	543.81	96.25
12	785.1	549.57	32.97	160	742.54	131.42

Step #5 - Find annual cost to pump water

Annual $IWHP = $ / input KW-hr x 100 / Pump Plant Eff x .746 KW / Hp x Hours / Year

Pumping plant efficiency = Impeller eff. x Motor *eff.l1* 00 = .75 x 90 = 67.5%

Annual *$1* WHP = $.08 / input KW-hr x 100 / 67.5 % x .746KW / Hp x 2590 Hours / Year

= $ 228.99/(WHP-yr)

Accounting for energy escalation,

Adjusted annual $/ WHP = $228.99/(WHP-yr) x EAE= $228.99 x 1.202

=$ **275.33**

Step #6 - Find the friction loss per 100' with 1183 GPM

$Hf = 1050 \times (1183\ GPM/150)^{1.852} \times 10^{-4.87} \times (100'/100)$

$= 48110 \times 10^{-4.87}$

Nom. Cia. inches	ActuallC inches	Hf ft/100'	WHP/100'	Annual Power $/100'	Annual Fixed $/100'	Total $/100'
6	6.301	6.15	1.84	506.12	38.54	544.66
9	8.205	1.70	.508	139.90	63.06	202.96
10	10.226	0.58	.174	47.88	96.25	144.13
12	12.128	0.25	.075	20.86	131.42	152.28

Explanation of calculations for various columns in the table above:

Hf - from the Hazen-Williams equation, C=150

$WHP = \frac{Hf \times GPM}{3960} = \frac{Hf \times 1183\ GPM}{3960}$ where Hf is in feet

Annual Power = WHP x *($1* WHP-yr) = WHP x ($275.33/[WHP-yr])

Annual Fixed - From Step #4

Total = Total annual cost = Annual Power + Annual Fixed

Economic Pipe Size Analysis - Limits to Level I

There are difficulties with the method of analysis for the simplified mainline illustrated in the previous section. First, for each problem with a different flow rate, all of the calculations must be redone. As an example, consider the system in Figure EC3



Figure 3. Example A for illustration of simple economic pipe size analysis limits

For the pipeline illustrated in Figure 3, a reduction in pipe size in the 1000 GPM section, resulting in an increase of friction by "X" psi, means that the pressure for the whole 2000 GPM must be increased by "X" psi. Therefore, the friction would be calculated for that section using 1000 GPM, but the annual HP cost should be calculated assuming a pump GPM of 2000 GPM.

Second, this does not adequately handle a branching mainline unless analyzed properly. For a branching mainline such as illustrated in Figure 4, the pressure may be increased just once for friction requirements on one branch. Simultaneously, the pressure is increased on the other side. However, pipe sizes in two segments (one on each branch) may be decreased as a result of the single pressure increase rather than just in one segment for Example A.

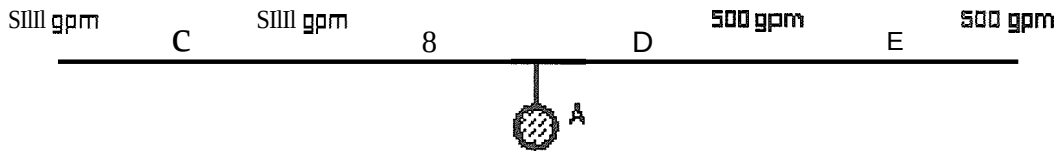


Figure 4. Example B for illustration of simple economic pipe size analysis limits.

Described another way, consider Example B in Figure 4. Pipe segments C & E are identical, and pipe segments B & D are also identical. If pipe segment "C" is reduced in size, this will increase the pump pressure. However, the size of "E" can also be reduced. Therefore, the logic shown below should be used in an economic analysis:

Pipe	Hf calculated with Q of:	HP cost calculated with pump Q of:
A	2000GPM	2000 GPM
B	1000 GPM	1000 GPM
C	500 GPM	1000 GPM

At this point it becomes apparent that it can be time consuming and monotonous to do all the calculations required for each pipe segment. A typical irrigation system with many branches would have numerous "pump flow rate" calculations, in addition to friction calculations, for the economic selection process.

Economic Pipe Size Analysis -Level II

Keller (1976) developed a logic which yields a graphical solution for this type of problem. Once the economic "Break Even Flow Rates (BEQ's)" have been determined for the available pipe sizes, a graph can be constructed and then used for a wide range of pump and section flow rates.

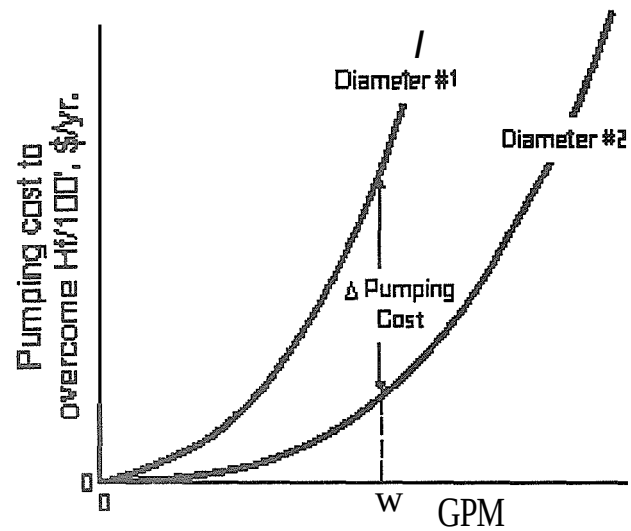


Figure 5. At some break even flow rate (BEQ), a comparison between two pipe diameters shows that the difference in annual pumping costs equals the annualized fixed cost difference.

Keller's method is based around the concept that once the break even flow rates (BEQ's) are known, the most economical pipe size for any intermediate flow rate is also known. This is illustrated in the example below.

Example: The break even flow rate has been determined for a specific set of economic conditions as the following:

BEQ between 6" and 8" is 500 GPM
BEQ between 8" and 10" is 800 GPM
SEQ between 10" and 12" is 1100 GPM

The flow in a pipe along the critical path in a design is 950 GPM.
What is the most economical pipe size to select?

Answer: 10", because the flow falls between 800 GPM and 1100 GPM

Determination of BEQ's - level II

The following procedure is used for determining the BEQ (Break Even Flow Rate) for various sizes of pipes. The economics, pump size, and pumping hours of the Levell example are used.

The procedure compares two pipe sizes at a time. The purpose is to determine what difference in friction would give a difference in Annual Power Cost which is equivalent to the difference in Annual Fixed Cost. Then the flow rate which provides that friction difference is solved for.

Steps #1 - 5 - Complete steps 1-5 of the Levell economic analysis

Step #6 - Calculate the difference in annual fixed costs for adjacent pipe sizes.

For example, the Annual Fixed cost for 6" pipe = $ 38.54 per 100 feet
the Annual Fixed cost for 8" pipe =$ 63.06 per 100 feet

Annual Fixed Cost difference = $ 63.06 - $ 38.54
= $ 24.52 per 100 feet

Step #7 - Use the horsepower equation and the cost of pumping, to determine how many feet of friction the answer to Step #6 is equivalent to.

Step#7a - Instead of spending an additional "Annual Fixed Cost difference" in purchasing a larger pipe, that same amount could be used to pay for the extra horsepower needed to pump through a smaller pump. This extra horsepower, based upon the adjusted power cost, is equivalent to:

$$\text{Equivalent WHP} = \frac{\text{Annual Fixed Cost difference}}{\text{Annual cost per WHP}}$$

In this example,

$$\text{Equivalent WHP} = \frac{\text{Annual Fixed Cost difference}}{\$275.33 \text{ per WHP}}$$

Step#7b - The pump flow rate is known from Step #2. Therefore, the "Equivalent WHP" can be converted to an "Equivalent Change in Friction" (Equivalent Δ Hf)

Solving for FEET in the WHP equation,

$$\text{Equivalent } \Delta \text{ Hf (in feet)} = \frac{(\text{Equivalent WHP}) \times (3960)}{1183 \text{ GPM}}$$

$$= \frac{(\text{Annual } \Delta \text{ Fixed \$}) \times 3960}{(\$275.33/\text{WHP}) \times (1183 \text{ GPM})}$$

$$= .01216 \times (\text{Annual } \Delta \text{ Fixed \$})$$

Step#7c - Determine BEQ's for the different pipe sizes under the economic and annual pumping conditions considered. When considering two pipe sizes, at the BEQ the difference in friction will be equal to the" Equivalent Δ Hf (in feet)" calculated in Step #7b.

The BEQ's can be solved for directly with an equation developed as follows:

For Diameter #1 : $Hf1 = 1050 \times (Q1/C)^{1.852} \times 01^{-4.87}$

For Diameter #2: $Hf2 = 1050 \times (Q2/C)^{1.852} \times 02^{-4.87}$

Q1 = Q2 at the BEQ
The Hazen-Williams "C" is the same (C=150)

Therefore,

$$\Delta Hf = Hf1 - Hf2$$

$$= \frac{1050}{C^{1.852}} \times (Q^{1.852}) \times [01^{-4.87} - 02^{-4.87}]$$

Solving for the BEQ at a certain Δ Hf

$$Q = \left[\Delta Hf \times \frac{C^{1.852}}{1050} \times \frac{1}{(01^{-4.87} - 02^{-4.87})}\right]^{.54}$$

$$BEQ = \left[\frac{10.21}{01^{-4.87} - 02^{-4.87}} \times \Delta Hf\right]^{.54}$$

where 01<02
01,02 are inside diameters in inches
BEQ is the Break-Even Flow rate in GPM
Δ Hf is in feet

Pipe Dia., Nominal	ID Inches	Annual Fixed *$/100'*	Annual Fixed *$/100'*	Equiv. Hf Ftl100'	BEQ GPM
6	6.301	38.54	-	-	
			24.52	0.2982	275.
8	8.205	63.06	-	-	
			33.19	0.4036	683.
10	10.226	96.25	-	-	
			35.17	0.4278	1365.
12	12.128	131.42	-	-	

The SEQ between 10" and 12" is 1365 GPM, which exceeds the system flow rate of 1183 GPM calculated in Step #2. Although it would be impossible to have a flow greater than the actual pump flow rate, this is not an error; the value of 1365 GPM is used in the Level III economic sizing procedure.

Another interpretation of the SEQ table above is as follows:

Pipe Diameter, inches	Economic Range of Flow Rates
6"	less than 275 GPM
8"	275 -683 GPM
10"	683 - 1365 GPM
12"	greater than 1365 GPM

The Level II economic analysis example can be used for a single mainline with multiple segments, but only with a pump flow rate of 1183 GPM. The pipeline illustrated in Figure 6 is sized as follows:

Pipe Segment	GPM	Economic Pipe Size
A	1183	10"
S	883	10"
C	583	8"
D	233	6"

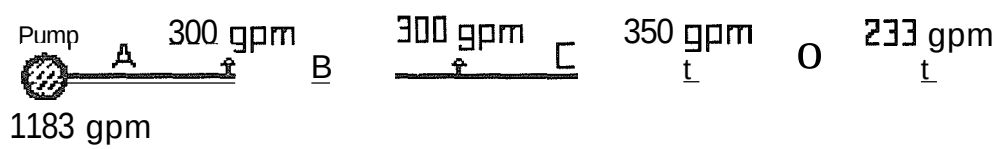


Figure 6. Example mainline for Level II analysis.

With this economic pipe sizing technique, the friction loss is not calculated along the critical path as a means of sizing the pipelines. The technique does not guarantee the lowest friction loss, nor a proper pressure distribution. It does guarantee the lowest annual cost for a simple conveyance function. Friction losses must be calculated after the pipe sizes are known.

Economic Pipe Size Analysis - Limits to Level II

The level II analysis is only valid for a single tapered mainline. It cannot be used "as is" on a branching system such as shown in Figure 9.

The logic of level II assumes that if there is one extra increment of pressure available for friction loss, only one pipe segment can have a reduction in diameter. It is a one-for-one tradeoff. In the system of Figure 9, the tradeoff is better than one-for-one. If there is one extra increment of pressure available, several identical, symmetrical pipeline segments may be reduced in diameter without further increasing the HP requirement.

In addition, the results of the level II analysis are only valid for the single pump flow rate which formed the basis of the calculations (in the example, 1183 GPM). If the flow rate is changed, the calculations must be redone.

Economic Pipe Size Analysis - level III

Keller (1976) developed the concept of Sreak Even Flow Rates (SEQ's) further into what is here called a third level of economic analysis. He found that the results from a level II analysis, if plotted properly on log-log paper, could be used to determine economic pipe sizes for any pump flow rate, not just the original pump flow rate (eg., 1183 GPM in the example). The logic was then further expanded to apply to branching systems.

Economic Pipe Size Selection Graph Development

Once the SEQ's for a single pump flow rate have been calculated, a universal graph for the same economic conditions can be developed. Figure 7 is an economic pipe size selection graph developed with the following SEQ's for a system flow rate of 1000 GPM:

Pipe size break	SEQ,GPM
2"- 2.5"	7.3
2.5" - 3"	9.5
3" - 4"	36
4" - 6"	105
6" - 8"	315
8" -10"	805

The graph is developed as follows:

Step #1. Obtain logarithmic (log-log) graph paper. For this example, 4 cycle logarithmic paper is needed because the design problem flow rates will cover the following cycles:

1-10; 10-100; 100-1000; 1000-10,000

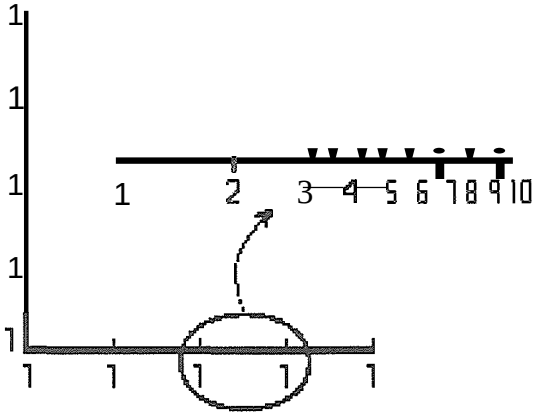


Figure 7a. Scales *on* 4 *cycle logarithmic paper.*

Step #2. Label the vertical axis as "Pump or Pipe Entrance GPM". Label the horizontal axis as "Section GPM"

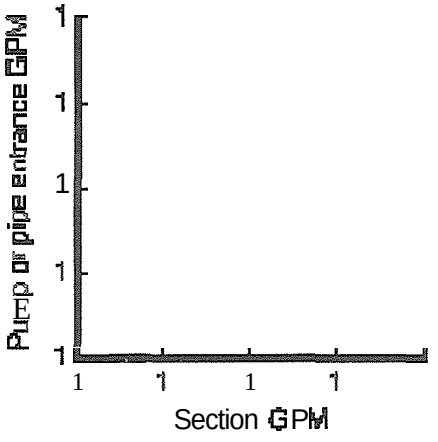


Figure 7b. Labeling the axis of the logarithmic paper.

Step #3. Assign values to the axis. There is no zero on logarithmic paper. Each increment of "1" is actual 10 times greater than the previous "1" value. For this problem, start both axis at 1.0 GPM.

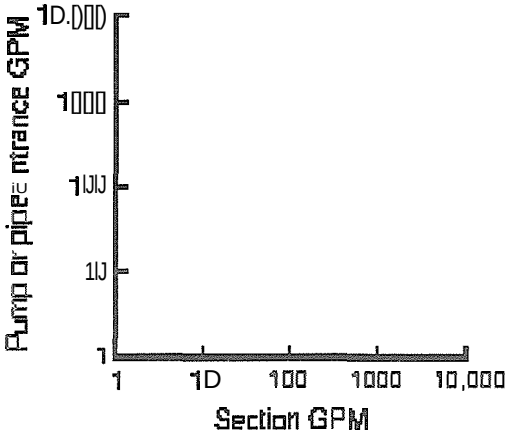


Figure 7c. Unit placement on logarithmic paper.

Step #4. Plot the BEQ's versus the pump flow rate for which they were calculated.

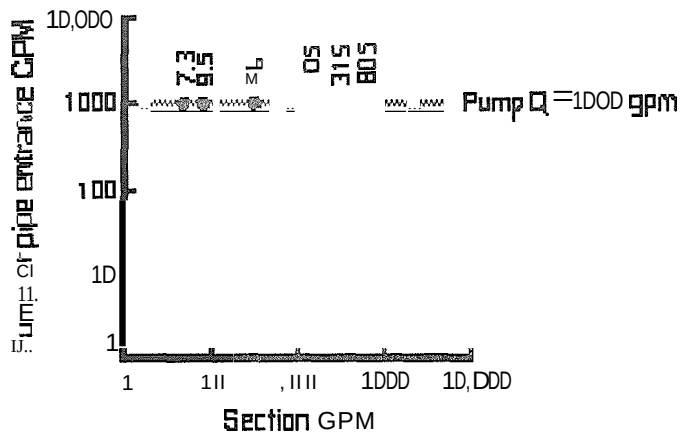


Figure 7d. Plot of *the* SEQ's *for* a *pump flow rate* of *1000 GPM.*

Step #5. Draw a straight 45 degree line between (1, 1) and (10000,10000). This is useful as a guide only, because a "Section Flow Rate" cannot exceed the "Pipe Entrance GPM". Some of the original points plotted in Step #4 may fall below this line.

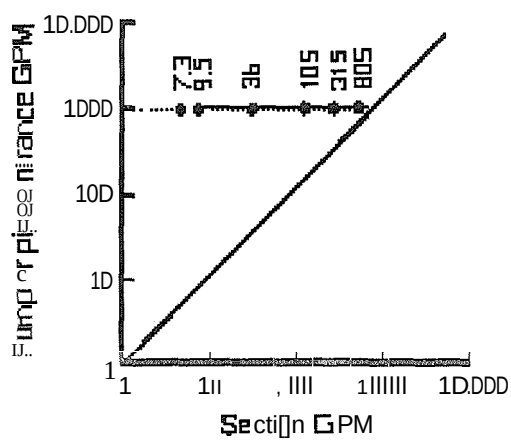


Figure 7e. Insertion of 45 *degree line.*

Step #6. Draw a straight line through each ofthe BEQ's. The lines must have a slope of 1.85 inches vertical drop for every inch horizontal change. The slopes must be drawn with the use of a ruler; do not use the scales on the logarithmic paper, as they are not linear. The line segments above the 45 degree line will be used to determine proper pipe sizes.

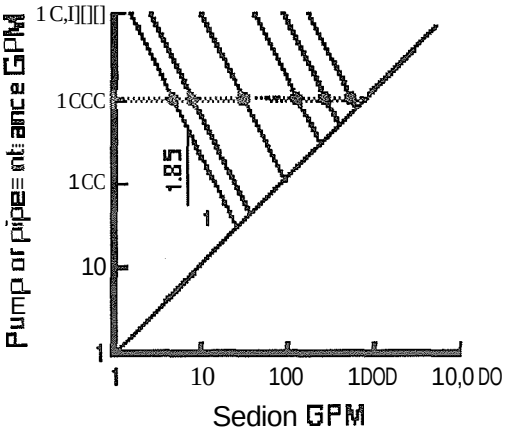


Figure 7f. Construction of economic zone boundaries for each pipe size diameter.

Step #7. Label each "zone" on the graph with the pertinent pipe sizes.

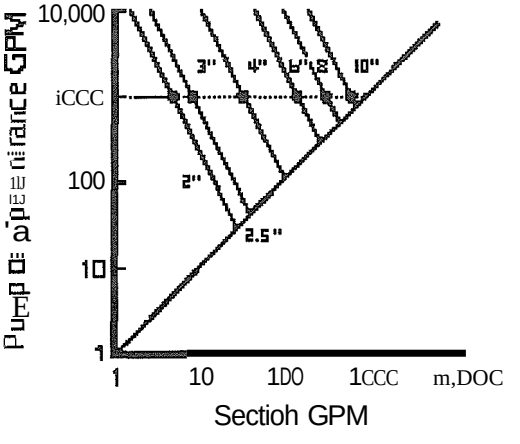


Figure 7g. Designation of pipe diameter zones. Final step in graph construction.

Use of Economic Pipe Size Selection Graph

The graph shown in Figure 7g can be used for any other irrigation system having the same economic constraints as used in the development of the 1000 GPM BEQ's. For example, assume an irrigation system in the same area has a pump flow rate of 2000 GPM and a single mainline with multiple segments. The segment in question has a flow rate of 600 GPM,

As illustrated in Figure 7h, the most economic pipe size is determined to be 8".

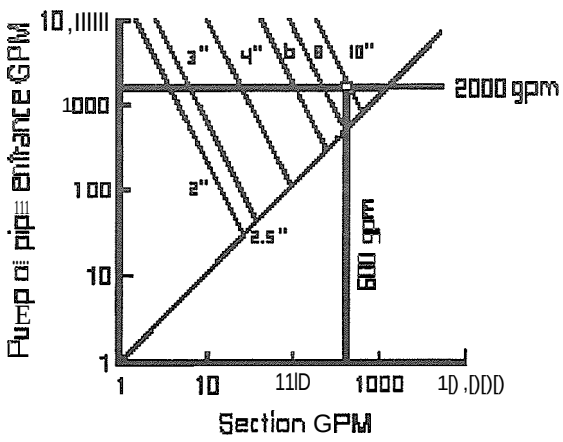


Figure 7h. Example problem. Single, tapered mainline. 2000 GPM pump, 600 GPM in pipe section.

The graph can also be used to solve for branching mainline sizes. Consider the example below (Figure 8) in which a 2366 GPM mainline splits into two symmetrical submains, with 1183 GPM going in each direction. Each submain is further tapered.

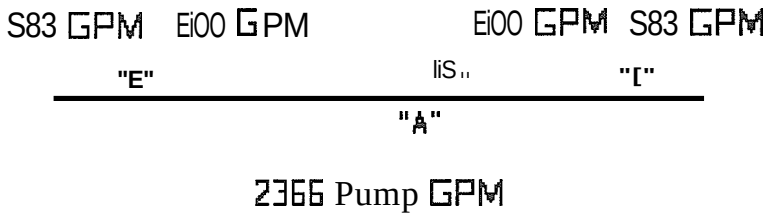


Figure 8. Pipeline for illustration of proper entry points on economic pipe size selection graph. Level ground. Symmetric submains.

When sizing pipe segment "A", the "Pump or Pipe Entrance GPM" is 2366 GPM. However, when sizing segments "B" and "e" (assuming those are on the critical hydraulic path), the "Pump or Pipe Entrance GPM" on the graph is 1183 GPM. By entering the graph in this manner, the solution recognizes that if pipe segment "B" is downsized, pipe segment "D" can also be downsized with no additional power requirement.

The solution requires developing a table as follows for the pipe segments along the critical path:

Pipe segment	SegmentGPM	"Pump" GPM	Economic diameter
A	2366	2366	12" **
B	1183	1183	10"
C	583	1183	8"

**The economic analysis used to develop Figure 7g did not consider pipe diameters larger than 12". Larger diameters, such as 15" and 18", should be analyzed to determine the next SEQ's. The proper diameter for segment "A" may be 18" rather than 12".

Example Design Problem -level III

Given: The system shown in Figure 9

level ground.

Spacing between outlets along a lateral =500'

Spacing between laterals = 2000'

Flow rate/outlet = 30 GPM

Required outlet pressure = 40 psi

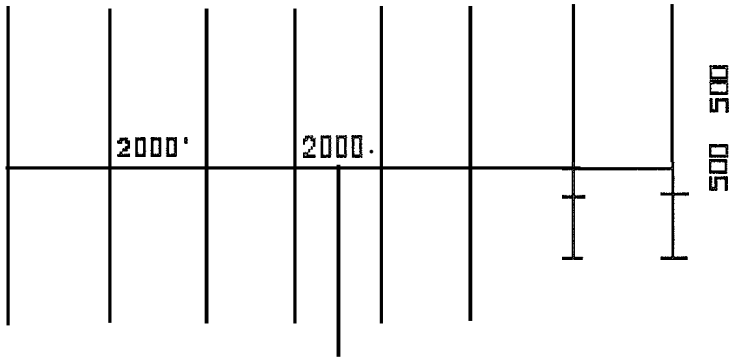


Figure 9. Branched pipe network requiring Level III economic analysis.

Find:

a. All of the pipe sizes along the critical path, using economic pipe sizing techniques.
b. The pressure at each point along the critical path.
c. The required pump pressure, psi
d. The pipe sizes on the lateral closest to the pump.

Solution:

Step #1 - Because the system is symmetrical, the work only has to be done on 1/4 of the system, as shown in Figure 10. Since there is no change in elevation, the critical path is the path which leads to the most distant outlet (Le. path Pump-H-D-A).

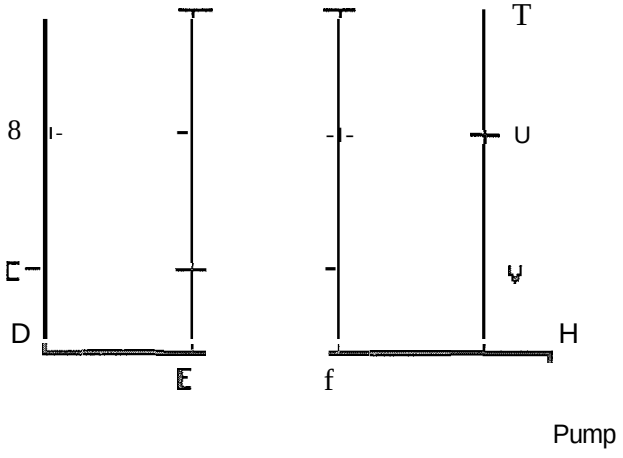


Figure 10. One fourth of the symmetrical pipe network shown in Figure 9.

Step #2. Set up a table to solve for economic pipe sizes (by reference to Figure 7g) along the critical path.

Section u/s of **Point #**	**Outlet** GPM	Section GPM	"Pump" GPM	**Diameter Inches**
A	30	30	90	2.5
B	30	60	90	3.0
C	30	90	90	3.0
D	90	180	720	6.0
E	180	360	720	6.0
F	180	540	720	8.0
G	180	720	720	8.0
H	720	1440	1440	10.0

Step #3. Expand the table to determine the pressures at each point. This information will be useful in sizing the laterals which do not lie on the critical path.

Section u/s of **Point #**	**Outlet** GPM	Section GPM	"Pump" GPM	Dia. inches	ID inches	l **ft.**	Hf **ft.**	Elev **ft.**	**P ft.**	u/s **P ft.**
										92.4
A	30	30	90	2.5	2.70	500	2.10	0	2.10	94.5
B	30	60	90	3.0	3.28	500	2.96	0	2.96	97.5
C	30	90	90	3.0	3.28	250	3.13	0	3.13	**100.6**
D	90	180	720	6.0	6.30	2000	3.77	0	3.77	104.4
E	180	360	720	6.0	6.30	2000	13.60	0	13.60	118.0
F	180	540	720	8.0	8.21	2000	7.90	0	7.90	125.9
G	180	720	720	8.0	8.21	1000	13.50	0	13.50	139.4
H	720	1440	1440	10.0	10.20	4000	33.90	0	33.90	173.3

Step #4. Size pipeline lateral T-G. This is the beginning of the use of a "mixed strategy" in sizing pipelines for the system. Pipeline T-G is not sized using "economic selection", but rather is sized to use up all the available pressure which exists at the entrance to the lateral.

The design criteria is to have 40 psi (92.4 ft) remaining at point "T". From the table above, the inlet pressure to lateral T-G is 125.9 ft. Therefore, the allowable friction loss (because there is no elevation change) is: 125.9 ft - 92.4 ft = 33.5 ft.

This translates into an **average rate** of Hf of 33.5 *ftl1250'* = 2.7 ft/1 00'. Knowledge of this rate is useful as a guideline to selecting the proper pipe diameters.

Section u/s of **Point #**	**Outlet** GPM	Section GPM	**Dia.** inches	**ID** inches	Rate of Hf *Wi00'*	l **ft.**	Hf **ft.**	Hf **ft.**
T	30	30	1.5	1.78	3.2	500	16.0	16.0
U	30	60	2.5	2.70	1.5	500	7.50	23.5
V	30	90	2.5	2.70	3.2	250	8.80	31.5
""Note that the Hf = 31.5 ft < allowable 33.5 ft.								

The pressure at point "T" is therefore 2 feet above the minimum allowable of 92.4 ft. The pressure at point "T" = 94.4 ft.

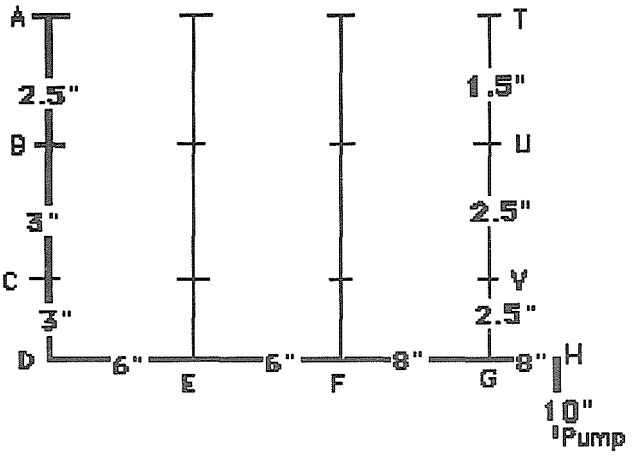


Figure 11. *Final pipe sizes along the critical path and lateral* G-T.

This completes an example of the Level III of economic pipe sizing.

Economic Pipe Size Analysis - Limits to Levell III

Although the Level III analysis is fairly rigorous, there are situations in which it may or can not provide the most "economical" selection. For example, many irrigation systems have multiple water sources. The Level III analysis is designed for a single pump station. Systems with many branching subunits may also require special treatment.

Nevertheless, the Level III method is suitable for many situations. In addition, in the process of learning the method, a designer develops a better awareness for proper economicsizing. The designer may not decide to pursue a rigorous economic analysis, but strive for a solution which is close to the optimum low point shown on Figure 1.

Economic Pipe Size Analysis - Summary

Designers must always be cognizant of the fact that if they become buried in any type of analysis, they may overlook the big picture. Designers who are only interested in the "most economic pipe size" may not spend enough time on the initial layout, on pressure control, or in making sure the design is actually manageable.

The most important element in any surface irrigation design is the ultimate manageability. This will influence the economics of pumping, crop production, and labor convenience much more than a detailed analysis of economic pipe sizing. Once the manageability question has been adequately addressed, economic pipe sizing should enter the picture. Good designers address both issues.

Fortunately, the long manual calculations presented in this chapter are adaptable to microcomputer spread sheet programs. In addition, there are a number of excellent interactive microcomputer design packages commercially available for irrigation designers which include this logic.

References

Keller, Jack. 1976. Energy Economics in Pipe and System Selection. Proceedings of the Sprinkler Irrigation Association Technical Conference. Kansas City, MO.

CHAPTER 18

PIPELINE PROTECTION

PIPELINE PROTECTION

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PIPELINE PROTECTION

Introduction

Airvents, vacuum relief valves, pressure relief valves, and/or pressure regulators are found on almost all irrigation pipelines. They are often used for pipeline protection, and may be used to reduce or eliminate problems such as water hammer or flow blockage. In spite of the widespread use of this hardware, pipes are frequently blown up and partial flow blockage is common on low pressure surface irrigation systems. This chapter will serve as a guide to the understanding and proper selection of the various protection devices.

First, some definitions must be established:

1. Water hammer. A pressure surge in a pipeline caused by a sudden change in water velocity. Typical causes include the sudden starting or stopping of a pump, sudden valve movement, or air movement in a pipeline. The surge may damage or destroy pipelines and pumps if severe enough. The word "water hammer" originates from the hammering sound which plumbing makes inside house walls when faucets are turned on or off quickly.

2. Pressure relief valve. An automatic on-line valve which discharges water into the atmosphere if the pipeline pressure exceeds some specified limit.

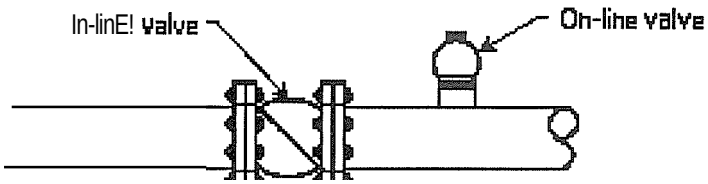


Figure 1. *In-line vs. on-line valve.*

3. Air vent. An on-line valve which exhausts air from the pipeline into the atmosphere. Some air vents only discharge air while the pipe is filling and not yet under pressure; other air vents can continuously discharge air even after the pipeline is under pressure and mostly full of water. Many airvents are also designed to act as vacuum relief valves.

4. Vacuum relief valve. An on-line valve which opens to allow air into the pipeline if the pipeline pressure becomes negative (Le., if a suction or vacuum develops). The relief of the vacuum prevents the pipe from collapsing inward.

5. Check valve. An in-line valve which allows flow in only one direction in a pipeline. If the flow begins to reverse direction, a flap will close and prevent backflow.

6. Pressure regulator. An in-line valve which automatically adjusts its opening (Le., automatically throttles) to deliver a preset downstream pressure. The upstream pressure must be greater than the downstream pressure; a pressure regulator throttles (dissipates) excess pressure.

Air in Pipelines

The presence of air in pipelines is undesirable for the following reasons:

1. Flow restriction. Air bubbles tend to accumulate at high points. This air accumulation forms a restriction in the pipe similar to a throttled valve. The increased friction reduces the flow rate. "High" points do not have to be very high. Gradual undulations on gently rolling land are often the biggest culprits because they are easily neglected. Most irrigation pipelines are not laid on a constant slope; rather, the trenchers maintain a constant depth below the ground surface. As a result, irrigation pipelines tend to have many small high points.

2. Complete air locks. Air bubbles at high points in a line can completely stop the flow on a low pressure system. Many irrigation installers have seen a low lift pump spinning without delivering any water out the end of the pipeline because of an air lock. Figure 2 depicts a simple case in which a pipe outlet is below a reservoir water surface, yet no flow will occur.

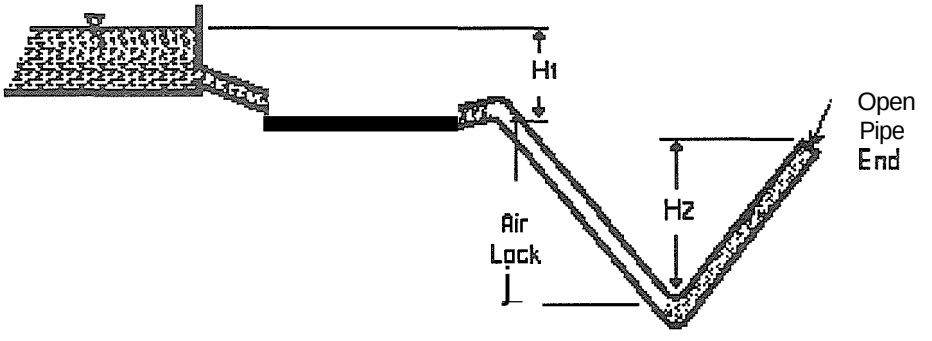


Figure 2. An airlock in a pipeline will occur if H2 is greater than H1, stopping all flow.

3. Surging as a compressible fluid. large air bubbles moving in pipelines will compress at low points (high pressure) and expand at high points (low pressure locations), causing surging action.

4. Sudden surging with dislodging. Air bubbles which accumulate near high points may increase in size until the tangential force ofthe moving water causes them to sUddenly move downstream. This results in large pressure surges. The installation of properly located air vents will eliminate this cause of water hammer.

Air enters pipelines in the following ways:

1. line filling and drainage. When an irrigation is stopped the water drains away from the high points in the pipeline and is replaced by air. With surface irrigation systems, the majority of the pipeline is generally full of air before a pump is started. This air must be removed when the system is started up.

2. Deep well pump startup. When deep well pumps are started, the column pipe is full of air down to the water level (often several hundred feet deep). The impellers push water up the column, forcing the air out ahead of the water. If the pump discharge is directly connected to a pipeline without an air vent, this air goes straight into the pipeline.

3. Overflow stands. Water which cascades through the air in overflow stands, or from an open pipe pump discharge into a pump stand, mixes large quantities of air into the water.

4. Low pressure points. On sloping ground, low pressure surface irrigation pipelines may alternatively run full or partially full, depending upon the flow rate, slope, and location. In the sections of partially full flow, air will enter through vacuum relief vents.

5. Falling water in wells. Some ground water aquifers have water-bearing strata interspersed by impervious layers. When a well pump is started, the pumping water level may drop below several of these layers. Water continues to enter the well from the upper strata, but it enters the well casing perforations and then falls down through the air before hitting the water surface. This mixes air in the water, and the air-water mixture may be pumped to the surface, looking like a white milky liquid. Air entrainment of this type is very difficult to effectively remove.

6. Miscellaneous. Air can be introduced through vortexing at pump intakes, fertilizer injectors which run out of fertilizer, and other points. The only rational assumption with agricultural irrigation systems is that air will be introduced to the system, and that hardware must be installed to remove that air.

Vacuum Relief

Most air vents used on low pressure irrigation pipelines serve a dual purpose of air release and vacuum relief. The following conditions commonly create vacuums in pipelines:

1. Sudden valve closure. When a valve is closed, the water downstream of the valve is still flowing and will create a vacuum immediately downstream of the valve. If the vacuum is not relieved (by allowing air into the pipeline through a vacuum relief valve), two things can happen:

   a. The pipe downstream of the valve may collapse (flatten out).

   b. If the pipe is strong enough to prevent the collapsing, column separation (Le., a large vacuum space of air and water vapor is created) may occur between the valve and the downstream water column. The flow may then reverse and slam back into the closed valve with a very high pressure surge.

2. Pump shutoff. The action is similar to a sudden valve closure. Water in the pipeline continues to move downstream, plus the water in the column soon reverses flow direction.

3. Drainage from a high point. Once the flow has stopped in a pipeline, drainage will begin to occur from high points. A vacuum is created at the high points.

Types of Air Vents

There are four types of air vents commonly used on irrigation pipelines. There are large differences in both cost and function. The four types are:

1. Large orifice, non-continuous air release plus vacuum relief (LAY). The Waterman AV-150 and AVP-1 series fall under this category. These are high volume vents with float balls or cylinders that seal under pressure. They do not release air from pipelines once the pipelines are under pressure. They will not alleviate problems which are the result of air entrainment during system operation such as falling water, water spilling over overflow stands, or injection through fertilizer injectors. They are designed to expel large volumes of air during system start-up, and to provide protection against vacuum development.

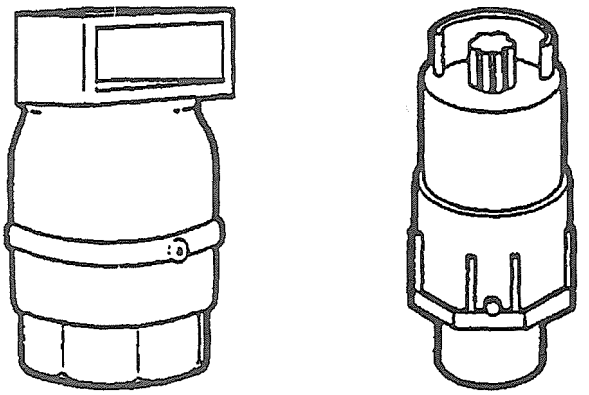


Figure 3. Waterman AV-150 and AVP-1 used for large orifice, non-continuous air release plus vacuum relief (LAV)

2. Continuous acting air release valves (CAY) (Sometimes called automatic air vent valves). The Waterman CAV-6 performslhis function. The valve has a small venting orifice and is used wherever air is entrained in water under pressure. It will not discharge the large volumes of air on start-up, but will protect against subsequent air entrainment problems.

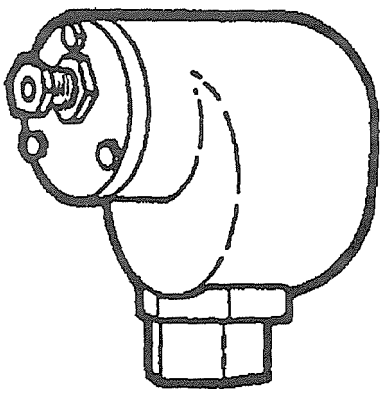


Figure 4. Waterman CAV-6. A continuous acting air release valve (CAV).

3. Combination large orifice/continuous air release/vacuum relief valves (COMB). The Waterman models AVR-2, CR-100 and CRP-8 are used for this combination function.

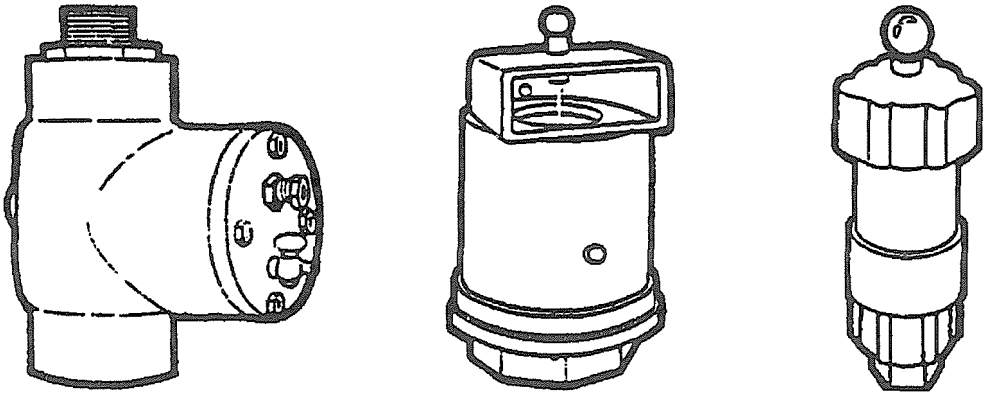


Figure 5. Waterman AVR-2, CR-100 and CRP-8. Combination air valves (COMB).

4. Non-mechanical open vents. Open vents are the most simple and effective method of controlling air problems. They provide a combination of large orifice exhaust/continuous air release/vacuum relief, plus act as a pressure relief valve. However, their use is limited to systems of very low pressure (or else the water will flow out the vent), and they obstruct farm machinery operation due to their height.

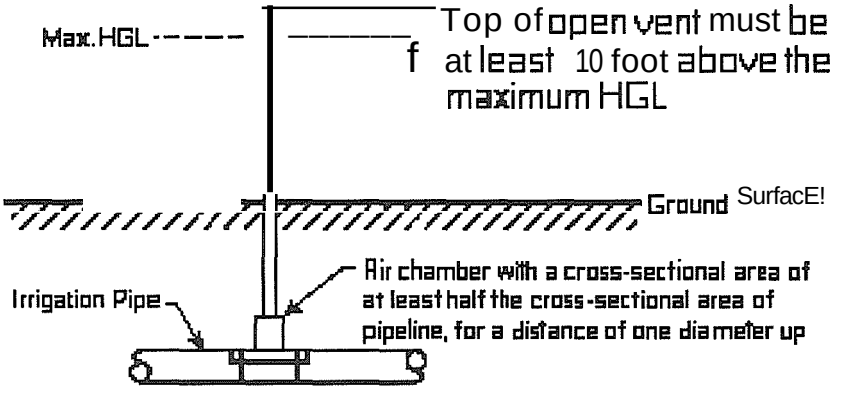


Figure 6. Open air vent.

Large Orifice, Non-Continuous Air Vents (LAV'S)

Determination of the proper size of an LAV depends upon the location. On flat ground the air release mechanism governs the sizing, and Table 1 should be used. At high points, where a line break or power failure may cause rapid line drainage from the high point, vacuum relief may be the governing factor in determining the proper orifice diameter. For that condition, Table 2 should be used.

Table 1 and Table 2 provide the minimum orifice diameter required, which is not necessarily the same as the nominal diameter. For example, a 2" Waterman AV-150 (2" is the nominal

size) has a 2" orifice for venting air, but an effective 1.75" orifice for vacuum relief. Such information is provided in the Waterman Industries product catalogs. When selecting the proper air vent size, use the larger of the sizes recommended by both Table 1 and Table 2.

Table 1 assumes a maximum water velocity of 5 feet/second in the pipeline while filling. Table 1 designs for no more than a 1 psi pressure at the air vent during air release. By designing for this low pressure, water hammer from sudden air vent closure is prevented, and problems of air vents blowing closed are minimized.

Table 1. Minimum orifice sizes for LAV's on flat ground.

Pipe Diameter, Inches	Min. Orifice Diameter, Inches
6	1.0·
10	2.2
15	3.2
20	4.0
30	5.0
36	6.5
·2" minimum recommended by some	

Table 2 is based upon a maximum vacuum of 1.0 psi. If there is a possibility that water will flow in both directions from the high point, multiply the size by 1.4. Capacities of both Table 1 and Table 2 are based upon information from APCO (1981).

Table 2. Minimum orifice diameter for vacuum relief at high points, inches.

Slope ftl100 ft.	Pipe Diameter, Inches				
	6	10	15	20	30
	Minimum air inlet orifice diameter, inches				
1	0.6	2.3	3.5	4.6	6.8
2	1.0	2.4	3.9	5.2	8.0
3	1.2	2.8	4.2	5.7	8.6
4	1.6	3.0	4.4	6.1	9.2
6	2.0	3.3	4.8	6.3	10.2
10	2.2	3.6	5.3	7.0	12.5
20	2.4	4.0	6.3	8.4	14.5

Placement of large Orifice, Non-Continuous Air Vents (LAV'S)

LAV's are necessary for all irrigation pipelines. The rules for proper placement are:

1. Immediately downstream of any in-line shutoff valve, such as a gate valve or butterfly valve. This will provide vacuum relief upon valve closure.

2. At the entrance to all downhill sloping pipelines. The air vent will release air as the water fills the pipe from the bottom (the pipe will flow partially full at first and fill from the bottom towards the top, and must displace air).

3. At all high points, for vacuum relief and for air release during line filling.

4. Twenty feet upstream of all pipe ends (not at the very downstream end). This provides twenty feet of air "cushion" at the end of the pipe if it is filled rapidly.

5. Upstream of pump check valves. This releases the air from the column when the pump is started, and also provides vacuum relief in both directions when the pump is shut off. In some cases where a very deep well pump discharges into a previously pressurized pipeline, a very small air vent is needed (rather than a large air vent) in conjunction with a slow opening pump control valve. That combination will prevent water hammer on the check valve which might be caused by very rapid air release, followed by a sudden wall of water hitting the check valve.

6. Every quarter of a mile along the pipeline, in conjunction with continuous acting air vents.

7. Do not place a LAV between a deep well pump and a booster pump if the booster pump has any chance of exerting a suction on the air vent.

Continuous Acting Air Release Valves (CAV's)

CAV's can remove small flow rates of air continuously and at high pressures (i.e., after the pipeline is full) through small orifices. Air, unlike water, is compressible and a large flow rate can escape rapidly through a small orifice if it is under pressure.

Under average conditions water holds about 2% dissolved air by volume. The equation below defines the relationship (derived from Lescovich, 1972) between the required orifice diameter and the pipeline pressure. For low pressure surface irrigation systems, the required orifice diameters are considerably larger than in high pressure drip and sprinkler systems.

CAV orifice diameter (inches) = K x GPMO.5
= K x [SORT (GPM)]

where K = .011212xP-0.4515
and P is the pressure in psi

For example,
assume a pipeline pressure of 50 psi and a flow rate of 500 GPM.

K = .011212 x 50-0.4515
= .011212 x .171
=.00192

Dia = .00192 x 5000.5
= .00192 x 22.36
==.043inches

Table 3. Minimum inside diameters required for Continuous Acting Air Vents (CAV's), in inches.

Pressure, PSI	Flow Rate, GPM				
	100	300	600	1000	2000
10	.040	.069	.097	.125	.177
40	.021	.037	.052	.067	.095
60	.018	.031	.043	.056	.079
80	.016	.027	.038	.049	.069
100	.014	.024	.034	.044	.063

Typical continuous acting airvent (CAV) connections on non-reinforced concrete irrigation pipelines have included air chambers, as shown in Figure 6. Similar fittings are available as saddles for low head PVC pipe, but are not generally available on higher pressure PVC (Class 100 and greater). The purpose of an air chamber is to provide a large area to give a greater opportunity for air bubbles to enter the vent.

There are not nearly enough continuous acting air vents on most irrigation pipelines. With surface irrigation, one reason is that many older open vents (which functioned as CAV's) have been replaced by air vents of the LAV design. Common pipeline installation specifications state that "air vents" are needed without designating CAV's specifically. CAV's typically cost five to six times more than an LAV, which often gUides the choice between a CAVversus an LAV.LAV's, however, provide absolutely no air removal once the irrigation system is even slightly pressurized. Both designs are needed on systems.

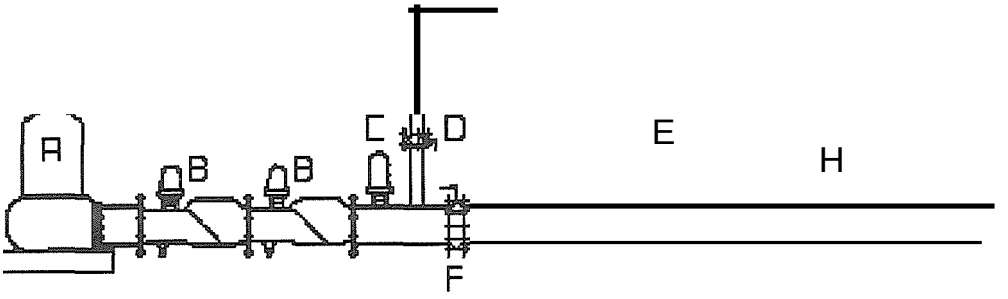
Placement of Continuous Acting Air Release Valves (CAV's)

The proper placement of CAV's is more important than proper sizing. Installation too close to other pipe fittings will be ineffective due to the turbulence offlow. In turbulent conditions, the air will be mixed up with the water and will flow right past the air vent. The following formula provides an estimate of the minimum distance downstream of fittings at which CAV's should be installed:

$$L = 0.15 \times V \times O$$

where L = distance downstream of fittings or air entraining point, feet.
V = velocity of water in the pipeline, feet/second
O = inside diameter of the pipeline, inches

For example, a well pump discharge may have entrained air. There are generally several fittings such as check and gate valves at the pump discharge. The CAV should be located downstream of those fittings a distance of "L". The LAV should be located upstream of the check valve. The proper relative configuration of various valves is shown in Figure 7.



Legend:
A - Pump
B - Waterman CPC-30 backflow prevention system
C - Waterman AA-9 pressure relief
D - Waterman VBEE butterfly valve on pipe to fill spray rigs
E - Waterman CR-100 (0.15 x Y x D *dIs* of "F")
F - Waterman VBEE butterfly valve
G - Fertilizer injector point
H - Flow meter (10 diameters *dIs* of "F")

Figure 7. Proper locations of various valves at a well pump discharge which supplies a buried PVC pipeline.

The following locations require CAV's:

1. Downstream of any air entrainment point. Examples of such points are:
   a. Pumps
   b. Fertilizer injectors
   c. Overflow stands
   d. Points of transition between partially full flow and completely full flow in pipelines.
2. Every quarter mile (and preferably every 660 feet) on pipelines. The LAV's never exhaust all of the air in the empty pipelines on system startup, and the CAV's are needed to remove the air once the system is under pressure.
3. At the high point and slightly downstream of all high points. In a static condition the air would accumulate exactly at the high point; with moving water the air may accumulate slightly downstream. It is possible to install a manifold using several saddles spaced on the top of the pipe and connected to one CAV to provide the venting. The exact location of the air accumulation will vary depending upon the downhill slope and water flow rate.

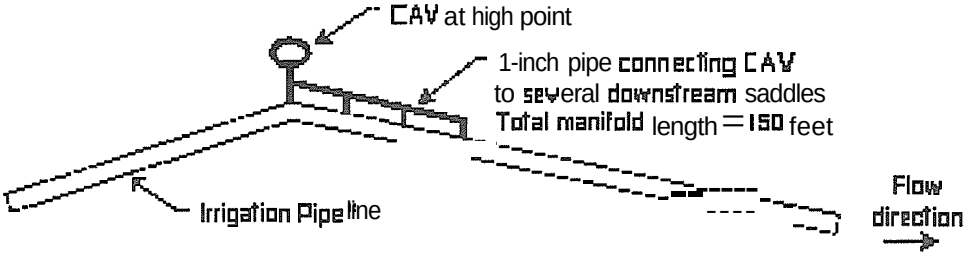


Figure 8. Profile view of 1-inch manifold assembly on downstream side of high point connecting to a single CAV.

High Pressure Problems in Pipelines - Pressure Surges (Water Hammer)

Pipelines and their fittings are designed to operate safely up to certain specified pressures. Excessive high pressures, encountered as either a sudden shock wave or as a continuous high pressure, will damage the irrigation hardware. Prevention of high pressure damage requires an understanding of both the causes of high pressure and the hardware used to prevent damage. The discussion below should shed some light upon the common rule of thumb which states that water velocities in pipelines should not exceed 5 feet/sec.

Pressure surges are caused by a sudden velocity change of the water in a pipeline. The additional pressure caused by a surge due to sudden velocity changes is:

$$P = .01345aV$$

where
- P = surge pressure, psi
- a = pressure wave velocity, fps
- V = initial velocity of the stopped water, fps

The wave velocity (a) is not the same as the water velocity (V). The wave velocity is the speed at which a pressure surge is transmitted by the pipe walls.

$$a = \frac{4660}{\left[1 + \frac{k}{E}(SDR - 2)\right]^{0.5}}$$

where
- a = wave velocity, ftlsec
- k = fluid bulk modulus (300,000 psi for water)
- SDR = Standard Dimension ratio of the pipe (pipe CD/wall thickness)
- E = Modulus of elasticity of the pipe
  - 400,000 psi for PVC
  - 15,000,000 psi for cast iron
  - 29,000,000 psi for ductile iron
  - 30,000,000 psi for steel

Example: Find water hammer wave velocity in Class 100 PVC pipe

Solution:
$$a = \frac{4660}{\left[1 + \frac{300000}{400000}(41 - 2)\right]^{0.5}} = 841 \text{ ft/sec}$$

KnOWledge ofthe wave velocity is used to compute the pressure surge; it is also helpful in determining how slowly valves should be closed. If a valve is closed in less than the time required for a wave to travel to the inlet and back (wave travel time), the maximum pressure surge will occur. This round-trip travel time is:

$$\text{Time} = \frac{2L}{a}$$

where
- Time = seconds
- L = length of pipe, feet
- a = pressure wave velocity, fps (from above)

The following illustrates the use of the wave velocity and surge pressure equations:

Given: The velocity of water in a Class 100 PVC pipe is 5 fps.
Pipe length = 1320 feet
The flow is suddenly stopped.

Find a) The increase in pressure (i.e., surge pressure), in PSI
b) The maximum valve shutoff time which would still cause the maximum surge pressure.

Solution:

a) $P = .01345 \times a \times V$
$= .01345 \times 847 \text{ fps} \times 5 \text{ fps}$
$= (11.39 \text{ psi/fps}) \times 5 \text{ fps}$ (check against Table 4)
$= 57 \text{ psi}$

This is the increase in pressure due to the fast valve closure immediately upstream of the valve. Assuming an initial line pressure upstream of the valve of 70 psi, the maximum pressure would be:

Maximum pressure = Regular pressure + surge pressure
= 70 psi + 57 psi
= 127 psi

b) The surge pressure of 57 psi was calculated for instantaneous valve closure. However, if the valve is closed in less time than it takes a wave to move to the water source and back again, the surge pressure will still be 57 psi.

The wave travel time $= 2 \times L / a$
= 2 x 1320 *feetl847* fps
= 3.1 seconds.

If the valve was closed in 3.1 seconds, the pressure rise would by 57 psi. If the valve was closed in 5.0 seconds, the pressure would still rise, but not by an additional 57psi.

Table 4. Surge Pressures (psi) vs. SDR for PVC pipe, based upon 1.0 fps instantaneous velocity change.

SDR	Pressure Rating, PSI	Surge, PSI
21.0	200	16.0
26.0	160	14.4
32.5	125	12.8
41.0	100	11.4
64.0	63	9.2
93.0	43	7.5

Actual surge analysis is generally much more complicated than the simple example above indicates. Precise surge analysis can be complicated and expensive. For most irrigation designers, the most practical method of solution is (i) to understand the available protection hardware and (ii) to install that equipment at enough locations and with reasonable settings. A fact of life which makes this approach almost necessary is that irrigation systems tend to be added to or changed with time until they may only vaguely resemble the original design. A detailed theoretical analysis based upon the initial design may not be valid in a few years or even immediately after installation.

For a more detailed discussion of the theory of hydraulic transients (Le., pressure surges), an excellent reference is available by Watters (1984). The book includes examples and computer programs for analysis. There are also a number of computer software packages available commercially for the solution of water hammer problems. A good source of information is the advertising section of the magazine "Civil Engineering", published monthly by the American Society of Civil Engineers.

High Pressure Problems in Pipelines - Steady High Pressure

The most common causes of this category are:

1. Large changes in elevation, combined with a flow rate at something other than the "design" condition. The most simple case is with a reservoir supply, as illustrated in Figure 9.

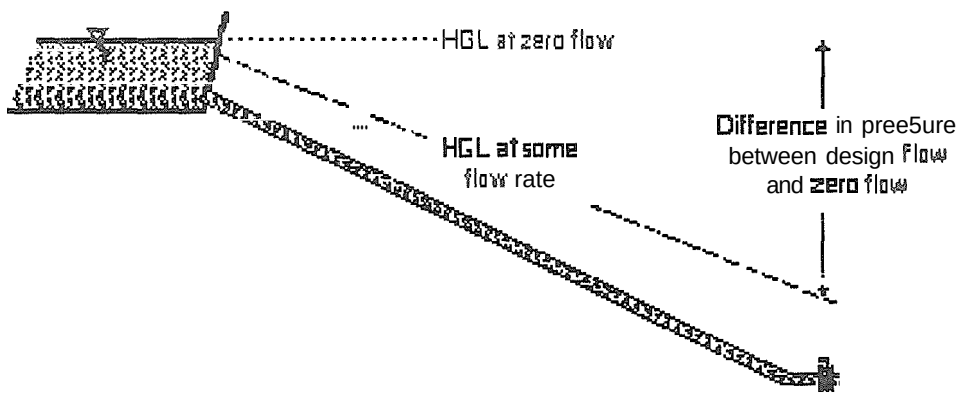


Figure 9. Profile view of a simple pipeline with a reseNoir supply. At zero flow rate, the pressure at the downstream end of the pipeline is considerably higher than at design flow.

2. A pumped water supply, with the flow rate being throttled at a valve within the field. Consider the following example:

Given: Well pump
1500 GPM
Drawdown at 1500 GPM = 40 feet
Standing water level in the well = 200 feet
Column friction loss at 1500 GPM =5 feet
Desired surface discharge pressure =20 feet.

Find: a) TDH (total dynamic head) required.
b) TDH if the pump still runs but the valve is shut off

Solution: a) TDH required = 40' + 200' + 5' + 20'
= 265 feet

Assume the selected well pump has the following performance:

GPM	TDH(feet)
0	325
500	310
1000	290
1500	265

b) At zero flow rate there is no drawdown or column friction. Also, the pump impellers deliver more pressure at no flow than at 1500 GPM.

At zero flow rate,

Surface discharge pressure = Impeller pressure - Depth to standing water
= 325' - 200'
= 125'

The pressure rating of 100 feet for the pipe, which was more than sufficient for the design flow rate, is insufficient for the condition of no flow rate. This pressure of 125 feet assumes very slow valve closure; a fast valve closure would increase the pressure even more due to surging.

Maximum Allowable Pressures

The various standards for PVC pipe are not in total agreement regarding the "safety factor" for PVC pipe regarding pressure surges. Even within a single standard the instructions are not clear. For example, ASAE Standard A5AE 5376.1 (ASAE, 1990) contains all three statements below:

1. "The pipeline shall have a pressure class rating greater than the static or working pressure plus surge at any point in the system. Surge pressures should not exceed 28 percent of the pipe's pressure class rating; therefore, if surge is not known, the working pressure shall not exceed the maximum allowable working pressure as given..." (which is 72% of the pressure rating, or in this case 0.72 x 100 feet = 72 feet)

2. For low pressure systems, "Pressure relief valves shall be large enough to pass the full pump discharge with a pipeline pressure no greater than 50 percent above the permissible working head of the pipe."

3. For high pressure systems, "Pressure relief valves shall be set to open at a pressure no greater than 34.5 kPa (5 psi) above the pressure rating of the pipe or the lowest pressure rated component in the system."

The Handbook of PVC Pipe (Uni-bell, 1977) states that "In general, system operating pressure (static pressure plus surge pressure) should not exceed 150% of the pipe

pressure rating or pressure class." With the abundance of varied recommendations it is not surprising that irrigation designers have been at a loss to know the proper selection method for pressure relief devices.

For the low pressure pipelines typical of surface irrigation systems, there are several rules which can be used to gUide the proper selection of protection devices:

1. Low pressure pipelines often have higherpressure ratings than the fittings used on them.

2. Fitting are often under stress due to temperature changes or improper installation. They may be quite sensitive to surge pressures.

3. It is impossible to accurately predict every potential water hammer cause or magnitude in an irrigation system. Factors such as air entry and farmers attaching new pipelines act as wild cards during design.

4. Based upon (1-3) above, it is important to understand how each pressure relief or pressure regulation device works so that a design estimate can be made of its performance after installation.

5. In general, it is desirable to design hardware with the intent of preventing the sum of all the pressures from exceeding the pressure rating of the pipe.

6. PVC and PE pipe are pressure rated at 73.4 degrees Fahrenheit. The pressure rating will vary with the temperature.

Protection From High Pressures - Overview

Protection from damage due to high pressures is available in two forms. Some irrigation systems use both types. These are:

1. Devices or methods which prevent excessive pressures from ever building up in a system. These may be classified as "preventative" or "pressure regulation" measures/ devices.

2. Devices which do not prevent the initial development of higher pressures, but which prevent the increasing pressures from reaching their potential. These are "pressure relief' devices.

Pressure Regulation and Preventative Methods

The following are commonly found in irrigation systems:

1. Use of gear operators on line valves rather than notched plates and handles. Gear operators help prevent very fast closure, but even the time necessary to close a gear operator may be less than the wave travel time. In addition, most of the flow restriction

occurs during the last 20% of a valve closure. The key concept is to close valves very slowly.

2. Throttling a pump discharge with a manual gate valve or butterfly valve at the pump itself to lower the line pressure. This will not protect against pressure surges and will not automatically compensate where flow rate reductions cause an increased pump discharge pressure.

3. Use of a pump control valve to automatically deliver a pre-set downstream pressure. The use of these valves is common and quite valuable in protecting against high steady pressures, but the following should be understood about their use:
   a. Pump control valves typically have a 4-8 psi minor loss across them. This increased pressure requirement may represent significant annual power costs. One exception is an electrically controlled butterfly valve which has almost no head loss. Some newer diaphragm control valves can be operated with a 2-3 psi loss.
   b. Pump control valves need routine maintenance. They have movable parts, flexible materials, and small tubes or screens which can plug. Many of the installed pump control valves do not operate properly because of a lack of maintenance.
   c. Pump control valves are designed to deliver a constant pressure at their discharge side. Due to changing hydraulic grade lines (HGL's) within the system, the pressures within the irrigation system further downstream and upstream of the valve will change as the flow rate changes, even if the pressure stays constant at one point. Pump control valves do not protect against water hammer due to sudden valve closure out in the field.
   d. There are many options available on pump control valves. One particularly valuable option in preventing water hammer is the slow opening and slow closing feature.

4. Spring loaded check valves. Spring loaded check valves are preferable to simple flap valves. Flap valves may allow reverse flow to begin and then suddenly slam shut, causing water hammer. Spring loaded checks shut immediately, preventing backward flow.

5. Concrete pipe stands. When a pump begins operating against a full pipe it takes time for the water in the pipe to start moving. For a short time the pump may operate at or near the shut-off head. By providing a large diameter open pump stand between the pump and the line the pump stand will fill up as the water in the pipeline is just beginning to move, providing a buffer. This is only applicable on very low pressure systems.

6. Slow filling of pipes. It is a standard recommendation that pipelines be filled with a flow rate that develops a velocity of one foot per second or less. This is written in many specifications, but is almost unheard of with irrigation system operation. Slow filling allows air to escape through vents and prevents water from crashing into valves and ends of pipes. During irrigation system startup a pump is operating against a very low pressure. Therefore, the pump flow rate is higher than the normal design flow rate, and filling velocities may be much higher than during regular irrigations. In order to fill a

pipeline slowly a person must manually control a valve near the pump, or a slow fill automatic pump control valve must be used.

Pressure Relief Devices

Pressure relief valves are automatic on-line devices which release water from a pipeline (usually into the air) when the pressure builds up to a preset level. There are four common types of pressure relief devices: spring loaded valves, hydraulically actuated diaphragm valves, air chamber surge tanks, and open vents.

Spring loaded Pressure Relief Valves

This design, typified by the Waterman AA-6 and Waterman AA-96 and AA-60 valves, is the most common device used on irrigation systems for pressure relief. It is often the most economical surge protection device.

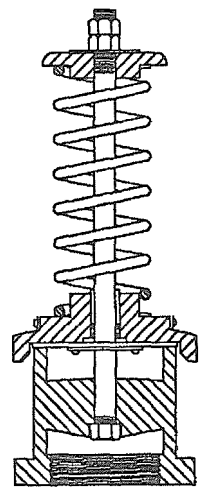


Figure 10. Watennan AA-96 pressure relief valve.

For proper design and application, the following points must be considered:

1. The opening or "cracking" pressure (the pressure stamped on the valve at the factory) must be considerably less than the maximum allowable pressure for the pipeline. This statement is in direct conflict with the recommendations provided in ASAE Standard 8376.1
2. Adequate sizing is critical; valves installed on irrigation systems are often too small to provide sufficient protection. ASAE Standard S376.1 states that the ratio of the nominal size pressure relief valve to pipeline diameter shall be no less than 0.25. This is often a reasonable estimate.
3. Both the opening of the valve and the discharge depend upon the pressure at the relief valve.

Consider a conveyance pipeline with the following conditions:

Given: 10" 1DO-foot head PVC
Pump flow rate during normal operation =1000 GPM
Water temperature =73.4 degree. F
Design pressure upstream ata point during normal operation =55feet

Find: a) Is there a need for a pressure relief device?
b) If a pressure relief device is needed, solve for the proper size and setting of the spring-loaded pressure relief valve with the catalog performance curve on Figure 11.

Solution:

a) The velocity of the water in the pipeline is 4.1 feet/second. Using Table 4, the maximum surge pressure is:

Surge P = 7.5 psi x 4.1 ftIsec x 2.31 ftIpsi
= 71 feet

The maximum pressure in the line, not accounting for air movement or the pump curve effect at a reduced flow rate, but solely due to fast valve closure is:

Max. Pressure = Surge P + Dynamic P
= 71 feet + 55 feet
= 126 feet.

This pressure exceeds the pressure rating of the pipe and fittings (100 feet) by a considerable amount, so a pressure relief valve is needed.

b) To properly size and set a pressure relief valve, the performance curve must be obtained from the manufacturer (for example, curves for the AA-96 and AA-6 are found in the Waterman Industries Red Top Irrigation catalog). The "catalog curve" on Figure 11 is characteristic of those for spring loaded pressure relief valves. The curve will be different for each size and spring setting.

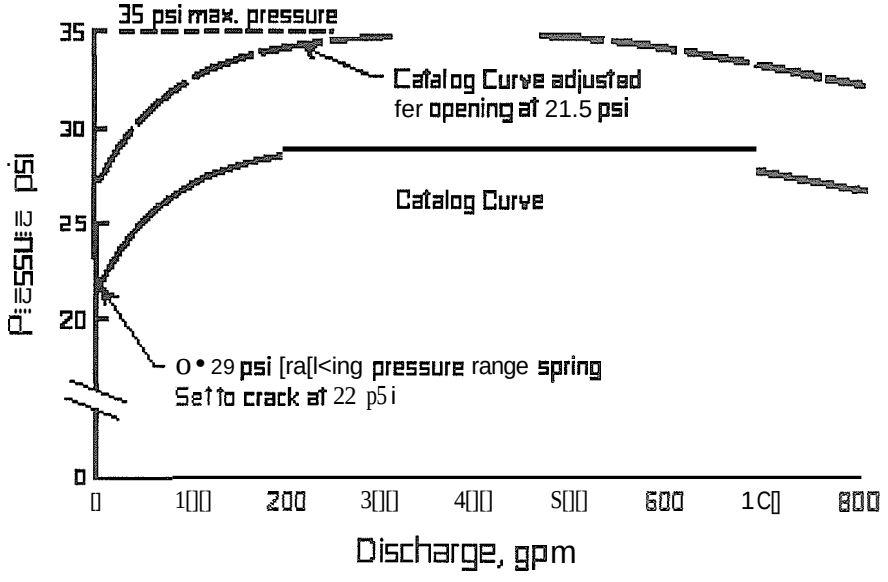


Figure 11. Typical characteristic curves for a 4" spring loaded pressure relief valve of a specified spring type.

The sizing procedure is provided in the steps below:

i) The initial sizing criteria for the pressure relief valve in this example are:
- Opening (cracking) pressure greater than 55 feet (24 psi)
- Maximum pressure less than 100 feet (43 psi)
- Discharge velocity in the riser not to exceed 15 feet/second (this last specification is used as a general rule to minimize the restriction between the pipeline and the valve.)

ii) After selecting the minimum cracking pressure and maximum line pressure, the required discharge flow rate should be estimated. For this example, a surge pressure of 71 feet resulted from a flow rate change of 1000 GPM. The allowable surge pressure (to not exceed the 100 foot pressure rating) is:

Max. allowable surge pressure = Press. rating - operating press.
= 100 ft - 55 feet
= 45 feet

This 45 feet surge pressure (S.P.) would develop from a flow rate change of:

$$\text{Flow rate change} = \frac{\text{Max S.P.}}{\text{S.P. from 1000 GPM}} \times 1000 \text{ GPM}$$

$$= (45 \text{ feet} / 71 \text{ feet}) \times 1000 \text{ GPM}$$

$$= 634 \text{ GPM}$$

Assuming that a gate valve in the line completely shuts off the flow, this means that the remaining flow must be discharged through a pressure relief valve immediately upstream of the in-line valve.

Press. relief discharge =
= 1000 GPM - 634 GPM
= 366 GPM

iii) Using the specification of a maximum velocity of 15 fps through the pressure relief valve, the minimum satisfactory diameter is:

$$D = 13.6 \times (QN)0.5$$

where
D = diameter of riser or valve, in inches
Q = discharge, CFS
V = max. velocity, (15 feet/sec)

In this example, D = 3.15 inches

The catalog curve of Figure 11 is for a hypothetical 4" valve, which is the minimum nominal size which can be used to satisfy the 15 fps rule.

iv) To determine whether the 4"valve represented with the catalog curve on Figure 11 will work in this situation, a new curve is drawn beginning at the desired cracking pressure. Forthis example, a desired cracking pressure of27.5 psi is selected. This is 3.7 psi higher than the normal operating pressure at the point, so the valve will not crack open frequently due to the normal operating pressure variations in the pipeline. In some higher pressure designs this cracking pressure should be 10-20 psi higher than the ordinary operating pressure at a point to prevent nuisance opening and closing of the valve near the normal pipeline operating pressure.

v) The new curve should be examined to determine the maximum line pressure which will occur before the desired discharge rate is achieved. In Figure 11, the maximum pressure will be 35 psi, which is less than the maximum allowable of 43 psi.

vi) The ordering specifications for the pressure relief valve are:
(Example only, blanks indicate where numbers would be)

- Model #___ (pertaining to the model represented by Figure 11)
- 4" diameter
- Spring # , with a cracking range of 0-29 psi
- Cracking pressure of 27.5 psi

vii) This design provides a factor of safety at both the upper and lower ranges. At the lower range, an extra 3.7 psi is provided for normal operation. At the upper range, the pipeline pressure will stay at least 8 psi below the pressure rating for this condition. As noted previously, a conservative design is warranted because of unknown or unexpected causes of pressure surges. The springs used in these pressure relief valves also may vary by 2-3 psi in their performance.

In another example, the 4" valve might have been larger or smaller than needed. In such a case, the curves for other sizes must be examined. Most irrigation systems have more than one pressure relief valve. A factor of safety is provided if each valve is sized to act independently, as was done in the example above.

Hydraulically Actuated Diaphragm Valves.

Hydraulically operated actuated diaphragm, globe or angle pattern valves are another common design for pressure relief. These valves are generally considerably more expensive and complex than the spring loaded pressure reliefvalves, but offer one definite advantage. This advantage is that the valve is completely open or completely closed.

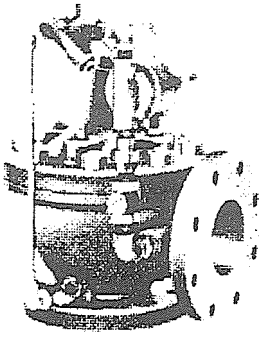


Figure 12. Hydraulically operated, diaphragm actuated pressure relief valve. CLA-VAL Co.

A spring loaded pressure relief valve will not open fully until the pressure has risen considerably higher than the cracking pressure. A hydraulically operated diaphragm valve will open fully at the cracking pressure. For a situation in which the operating pressure is close to the pipe pressure rating, a diaphragm actuated valve may be the best selection.

For situations in which the operating pressure is considerably lower than the pipe pressure rating, the extra pressure rise (beyond the cracking pressure) required by the spring loaded valves is usually not a problem.

Air Chamber Surge Tanks

Air chamber surge tanks are common at the pumping plants for city water systems and irrigation district pipelines, but are seldom seen in on-farm irrigation system designs. This is probably due to both the high cost (relative to spring-loaded pressure relief valves) and an unfamiliarity with the proper design of surge tanks.

An air chamber surge tank, as shown in Figure 13, acts as a large spring within the system. Water is allowed to flow into and out of a pressure tank which is partly filled with air. The air will compress during a pressure surge, allowing water to enter the tank and thus relieve the surge. This water will flow out as the pipeline pressure reduces. Such a surge tank also acts as a vacuum relief valve in the case of a decrease in line pressure.

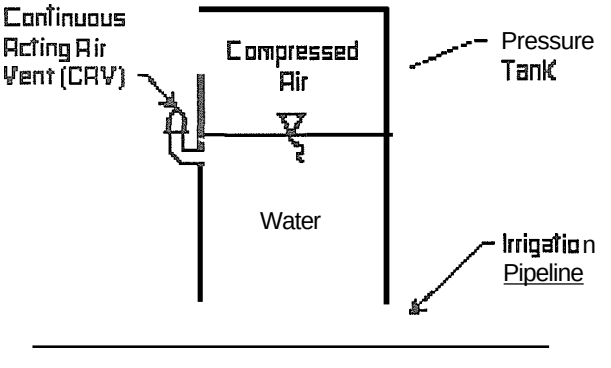


Figure 13. Air chamber surge tank. Tank may be vertical or horizontal, and above or to the side of the pipeline.

Air chamber surge tanks require a source of compressed air. This air may be supplied during well pump startup by allowing air from the pump column to go into the tank. Large tanks almost always have a permanently filled pneumatic bladder or an air compressor to maintain air pressure. A good source for information on the design of surge tanks and the required volumes of air and water can be found in a book by Parmakian (1963).

Open Vents

A simple open air vent will function as a dependable and effective pressure relief device. This type of vent, described earlier in the chapter, is only used on very low pressure systems in which the hydraulic grade line is less than 15-20 feet above the ground surface. For very low pressure systems such as non-reinforced concrete pipelines, these vents are the least expensive and most dependable pressure relief mechanism available.

Effects of Pipe Temperature on Pipeline Performance

Temperature effects must be considered for thermoplastic pipe (Polyethylene and PVC) used in surface irrigation designs. This section is placed in the chapter of "pipeline protection" because a good understanding of temperature effects will prevent pipeline damage.

There are five considerations regarding temperature:

1. Freezing damage.
2. Temperature expansion and contraction, with effects on pipe cutting and marking during installation.
3. Temperature expansion and contraction, with effects on stress at fittings.
4. Reductions in pressure rating of pipe at high temperatures.
5. Ease of installation during cold weather.

The first 4 items pertain to pipeline damage or protection.

Freezing damage. In areas of freezing weather, pipelines must be below the frostline, or be easily drainable. If pipelines must be drained, the number of drainage valves can be reduced by installing the pipeline on a constant slope rather than at a specific depth below the ground surface.

Installation - thermal expansion. Thermoplastic pipe expands and contracts with temperature changes. The end of a long pipeline which is left in an open trench may move several inches or feet during a hot summer day. Therefore, all locations of outlet saddles and locations of final pipe cuts should be marked on the pipe during the early morning during installation in the summertime. The actual drilling or cutting can be done during the day. Shading of the pipe with a thin (1-2") layer of soil during the daytime reduces the temperature changes. Final connections and backfill should be done within plus or minus 5-10 degrees Fahrenheit of the final operating temperature.

The coefficients of thermal expansion for thermoplastic pipe are:

PVC - .000031 (foot of expansion per foot of pipe) per degree F
PE - .000083 - .00016 per degree F

Consider a common irrigation installation of low head PVC. A pipeline is 1000 feet long. The temperature in the early morning is 60 degree F, and in the mid-afternoon is 100 degrees F. The pipe is longer in the mid-afternoon by:

Expansion = (Coeff. of expansion) x (Pipe length) x (Change in degree F)
= .000031 x 1000 feet x (100- 60 deg. F)
= 1.24 feet

Stress - thermal expansion. Many low head PVC pipelines and fittings are solvent welded (ie, glued), and buried at temperatures other than the operating temperatures. The pipe length is then fixed, but large stresses develop as the pipe temperatures change. This is probably a major cause of failures on fittings, which are exposed to large tangential forces. The use of gasketed pipe can reduce the stress in some cases, but in other cases gasketed pipe has been known to separate at discrete joints. The best solution is prevention by burial of the pipe at the proper temperature. If water temperatures are different from soil temperatures, a possible solution is the periodic use of expansion joints and fittings, plus a design with multiple short pipe lengths (ie, 300 feet) rather than long lengths (i.e., 2,000 feet).

Pressure rating - thermal effects. PVC and PE pipe is pressure rated at a temperature of 73.4 degrees F (23 deg. C). Higher temperatures drastically reduce the pressure ratings and increase cost by requiring thicker walled pipe.

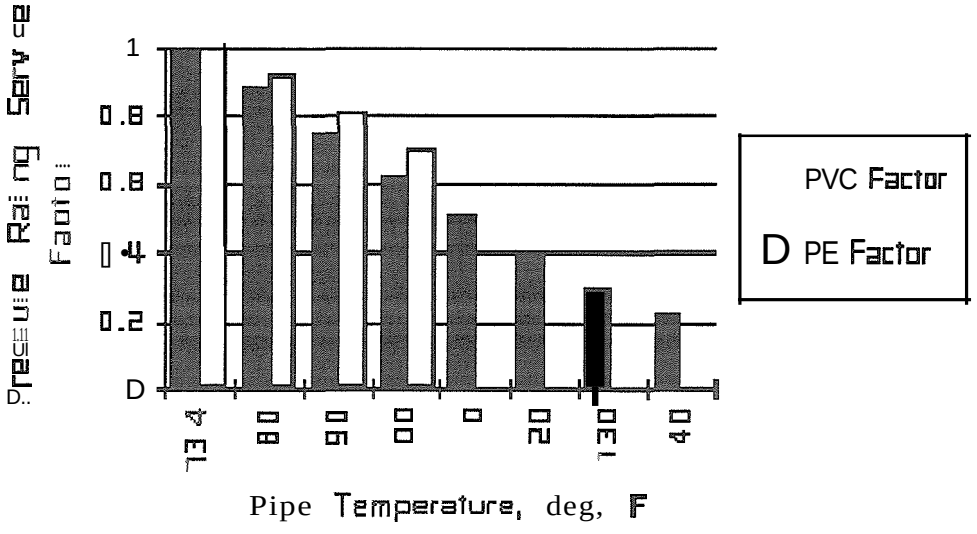


Figure 14. Pressure Rating Service Factors For Temperatures For PVC and PE Pipes (from ASAE, 1990)*

* At 90 degrees F, a Class 100 PVC pipe would have a pressure rating of (0.75 x 100 psi) = 75 psi.

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CHAPTER 20

PUMPS

PUMPS

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PUMPS

Introduction

This chapter will cover some basic points encountered by irrigation designers. For more detail one should consult pump manufacturer literature or standard reference books on pumps found in any university engineering library.

What most people call a "pump" actually consists of a combination of a power source, the pump itself, a drive shaft connecting the power source to the pump, and a housing to mount various components and direct the water to a discharge end. The function of a pump is to use mechanical energy to move water. The discharge of a pump is rated as a combination of flow rate and pressure.

Common power sources for surface irrigation system pumps are tractors with PTO's, diesel or natural gas engines, and electric motors.

A basic pump consists of one or more impellers plus a housing (bowls) for the impellers. The impellers are connected to a shaft which rotates; the bowls are stationary and surround the impellers.

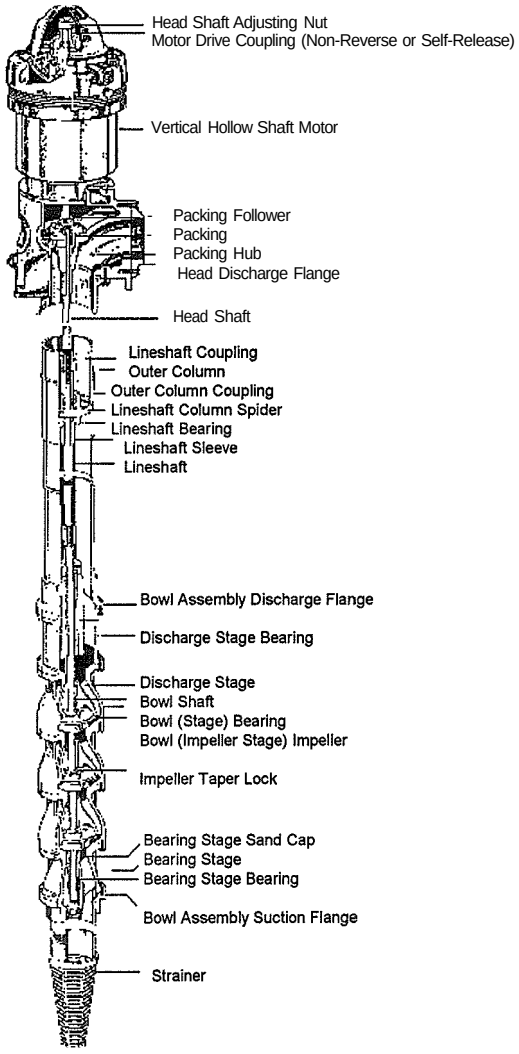


Figure 1. Cutaway of a well pump showing components.

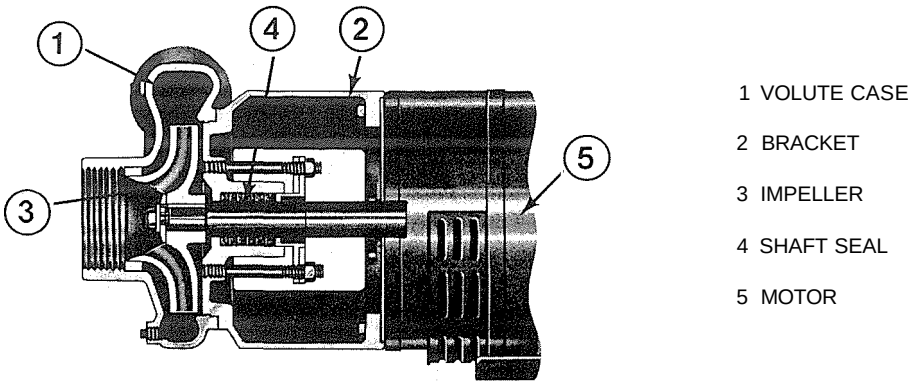


Figure 2. Cutaway of a horizontal end suction pump.
Source: BERKELEY PUMPS

Manufacturers provide performance curves for pumps at specific RPM's (revolutions per minute). Engines can operate at a variety of RPM's. Gear heads, which connect engines to the shaft of vertical pumps, can be sized to provide a pump RPM which is greater than the engine RPM. Motors will spin a shaft at an almost constant RPM.

Motors

A motor is wired and insulated to perform within an optimum horsepower (HP) range. A 50 **HP** motor is designed to produce a maximum of 50 **HP** output with the possibility of a 5-15% overload safety factor depending upon the air temperature, load, and manufacturer. A 10% overload means that a 50 HP motor can safely deliver 55 HP at 40°C (104°F) ambient temperature without burning up the insulation on the Windings or having the motor bearings fail. In general, older motors (pre 1970's) can be overloaded more than newer motors because the older motors were of heavier construction.

In any case, high temperatures are bad for motors. Motor cooling fins should be kept clean, shade should be provided in hot climates, and adequate air circulation around the motor should be guaranteed. Some people have wrapped hoses around the outside of motors and allow some of the cool discharge water to pass through the hose and back to the source in an effort to keep motors cool in very hot conditions.

A 50 **HP** motor will not necessarily deliver 50 **HP** to a pump shaft. A motor draws power in proportion to the applied load. A pump will change the **HP** load to the shaft as the flow changes in response to irrigation system demands. A pump/motor combination which was originally installed to require 50 **HP** may later be operated in a condition which pulls 60 **HP** from the motor, causing motor overheating and burnout. Electrical control panels generally have "heaters" which automatically disconnect the power if a constant high current (Le., overload) is pulled through the panel. This saves the motor.

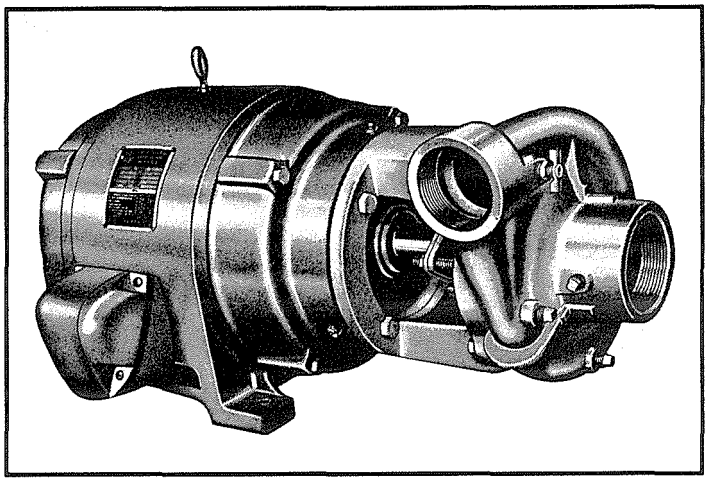
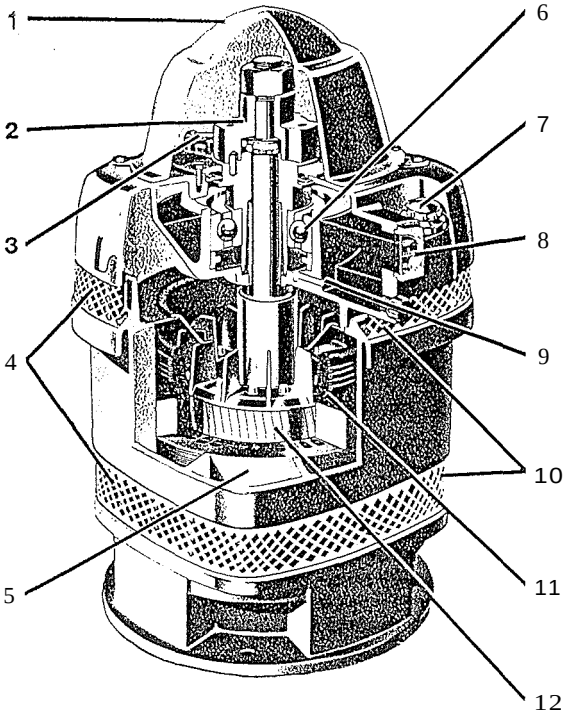


Figure 3. 3 *phase motor on an end suction pump.*
Source: BERKELEY PUMPS



1. Top cover
2. Non-reverse rachet or self-release coupling
3. Lock bar
4. Dual air flow system
5. Stator
6. Bearing
7. Plug
8. Window
9. Metered oil flow
10. Stainless steel screens
11. Class F Insulated System
12. Rotor with integral fan blades

Figure 4. *Cutaway of* a 3 *phase hollow shaft motor for* a *well pump.*
Source: U S Motors

Most agricultural pumping motors are NEMA Type B, 3 phase induction motors with 240/480 volts. Common motor RPM's are: 1150, 1750, and 3500 RPM. A motor will spin the shaft at almostthe same speed regardless ofthe HP load. Standard motor sizes, with their rated output HP and typical efficiencies, are listed in Table 1. Motor efficiency stays relatively constant regardless of age or applied load as long as the load is greater than 75% of the rated HP. As can be seen from Table 1, large motors have a higher efficiency than smaller motors.

Table 1. *Approximate Efficiencies of Standard* ** *Three-Phase Electric Motors*

HP rating of Motor	Full Load Efficiency (%)
0.5	72
1	76
1.5	77
2	79
3	80
5	82
7.5	83
10	84
15	85
20	86
25	86
30	87
40	88
50	89
60	90
75	91
100	92

** *Most motor manufacturers now offer special, improved efficiency motors for an extra cost in the higher HP ranges.* See *section on high efficiency motors.*

High Efficiency Motors

Due to increased electrical power costs, motor manufacturers have produced new lines of high efficiency motors. The payback period for the higher initial cost of these new motors will depend upon the cost of electricity and hours of pumping, but can often be 2 to 3 years or less. The payback period can be calculated using the same procedures as those used in simple economic pipe sizing.

For a calculation of annual energy savings using present costs, use the following equation:

$$ES = 0.746 \times HP \times C \times N \times \left[\frac{100}{EB} - \frac{100}{EA}\right]$$

where,

ES = Annual energy savings, $/year
HP = Brake Horsepower required from the motor
C = Energy cost, $/(kw-hr)
N = Hours/year of pumping time
Es = Standard motor efficiency, %
EA = Premium motor efficiency, %

There are several methods of estimating motor efficiency, some of which involve assumptions for part of the calculations. As a result, a motor efficiency measured by Japanese standard JEC-37 will give an 85% efficiency for a 7.5 HP motor, but the US standard IEEE-112, Method B (NEMA Standard MG1-12.53a) will give an 80.3% efficiency for the same motor (Montgomery, 1981). The National Electrical Manufacturers Association (NEMA) standard should be used for comparisons.

The percentage savings (not $/year) due to converting from a standard motor to a premium motor are generally much greater for smaller HP motors. Large HP motors must be fairly efficient in most cases anyway because of the problems of heat dissipation.

Pump I"peller Curves

Manufacturers supply performance curves for pump impeller/bowl assemblies. The following information is obtained as a function of flow rate from the curve:

a. Pressure (Total dynamic head, TDH).
b. Horsepower required at the impeller (Brake HP).
c. Impeller efficiency.
d. Net Positive Suction Head (NPSH)

Figure 5 illustrates a general performance curve for a radial (centrifugal) pump design which is typical of well and booster pumps.

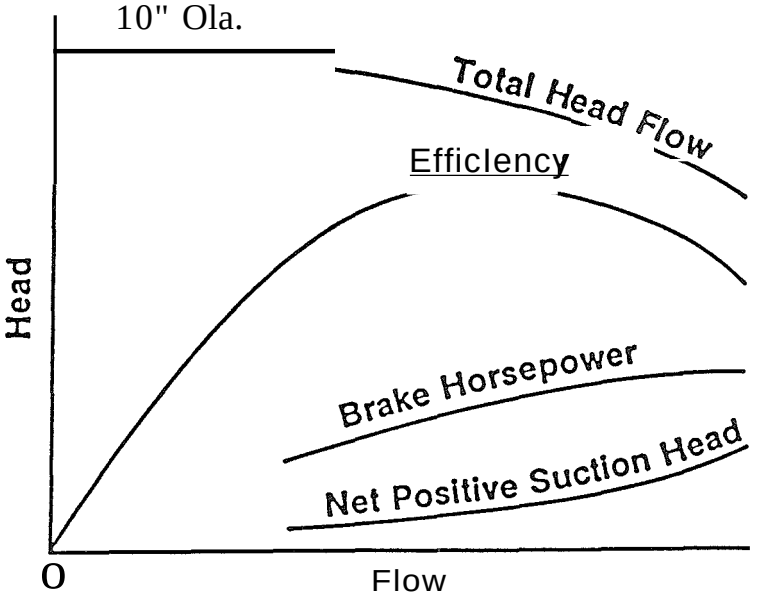


Figure 5. Typical relationships of pressure (head), efficiency, BHP, and NPSH for a radial pump design.

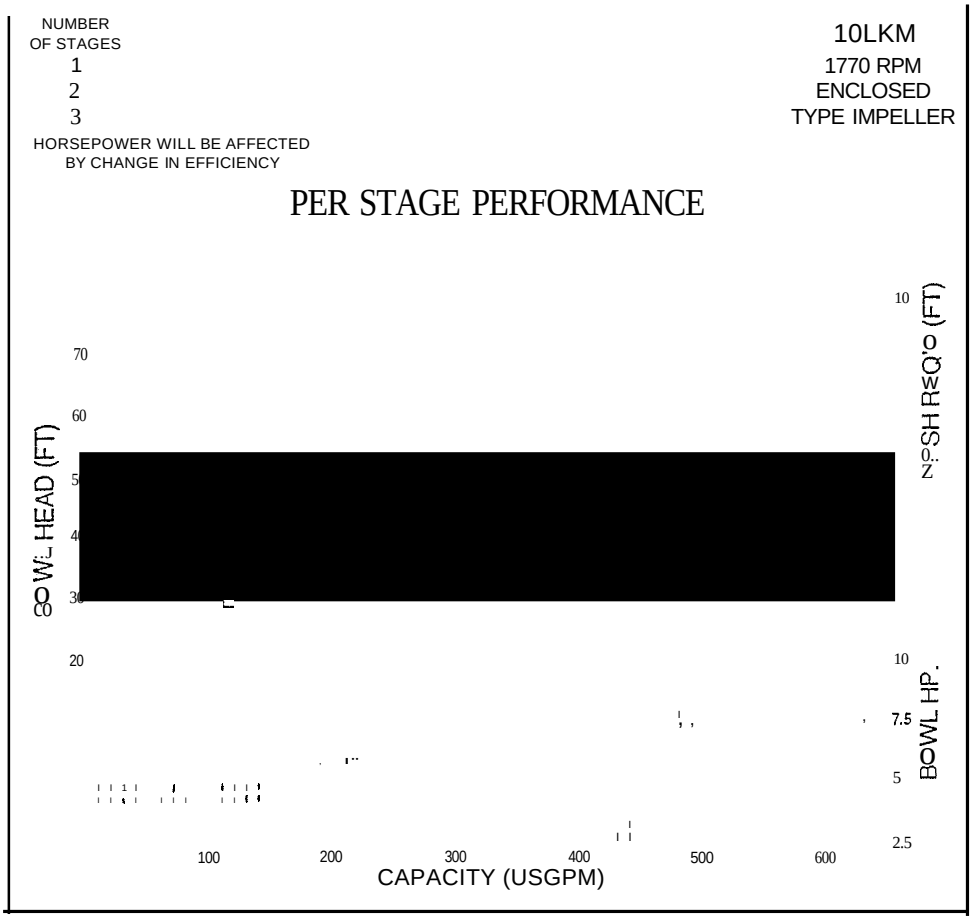


Figure 6. *Performance of several impeller diameters within* a *single bowl design. Courtesy Peabody Floway Pumps.*

As can be seen in Figure 6, a range of impeller diameters can work satisfactorily in the 10LKM bowl design. If an irrigation system required a TDH of 59.5 feet at 400 GPM, the "A" impeller would do the job at a 79.2% impeller efficiency. The BHP required by the impeller for that flow rate would be approximately 7.6 HP.

Another irrigation system may be designed to require a TDH of 54 feet at 400 GPM. In that case, the 1OLKM pump would be ordered with an impeller trimmed to a diameter which is between the "A" and "8" impellers. The "A" impeller would provide too much pressure for that application. A good pump dealer will specify the flow rate and pressure so that the manufacturer can trim the impellers appropriately for each application.

If a pump on an existing installation delivers too much pressure, a relatively inexpensive way of reducing the power bill is to remove the pump and trim the impellerto the appropriate diameter. This will reduce the required BHP. The same pump motor and panel can be used without changes. This recommendation assumes that the trim would not be too great. On Figure 6 it is seen that the smaller impeller "G" has a lower efficiency than impeller "A". If the efficiency is too low, it might be better to select a new pump design rather than trim the old impeller.

Figure 6 shows that the 10LKM pump is designed to operate most efficiently in a range of 350-500 GPM at a TDH of 50-60 feet. The pump will operate under many other conditions, but at a lower efficiency. For different operating conditions another pump may be a better selection.

System Curves

When an irrigation system is designed, the pressure (TDH) requirement is generally determined for a specific flow rate. Then a pump is selected to provide that flowlTDH combination at a good impeller efficiency.

Figure 6 shows that a pump can provide a wide range of flowlTDH combinations. A common question is: "How does the pump know what it is supposed to deliver on an irrigation system?" The answer lies in the irrigation system curve, and how it is combined with the pump curve.

An irrigation system will be installed and operated with a certain flow rate passing through it. The required TDH and flow rate in that condition represent one point along a system curve illustrated in Figure 7. A higher flow rate can be moved through the irrigation system if the pump pressure is also increased to account forthe extra friction. The irrigation system curve sweeps upward and to the right.

Figure 7 shows that a pump curve and a system curve meet at only one point. To have a higher flow rate the system needs more pressure, but the pump can only put out more flow at a lower pressure. For a given system comprised of pipes, elevation changes, and valve settings, a pump will only deliver one flow rate.

The irrigation system curve of Figure 7 begins at zero flow rate with a pressure requirement greaterthan zero. In this case the elevation of the discharge point is greater than the water supply. A pump must first overcome the elevation difference before flow will begin.

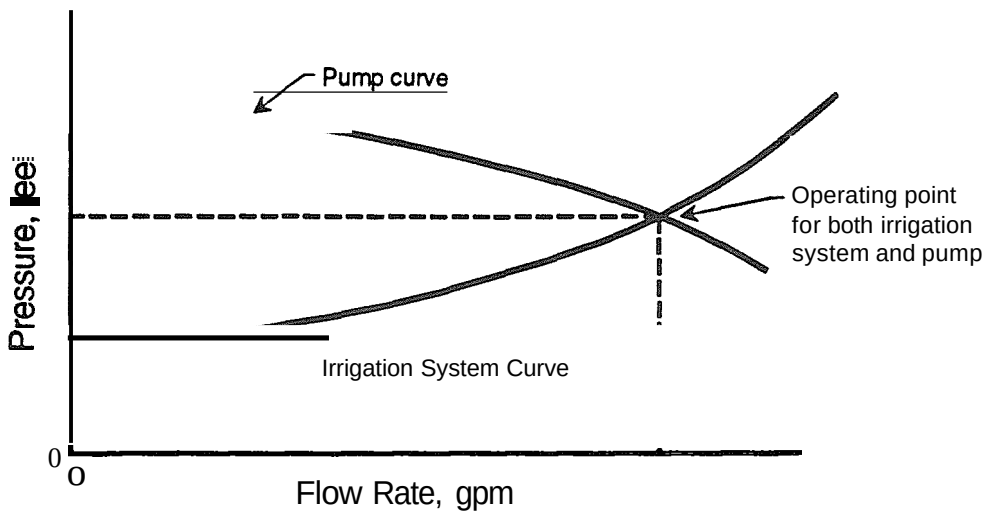


Figure 7. Combination of a pump curve and irrigation system curve.

Of course, the flow rate through an irrigation system can be changed by closing and opening valves. This valve adjustment creates a new system curve, and the pump curve will meet it at another point, as illustrated in Figure 8. The pump BHP, flow rate, TDH, and efficiency will be different at the new point.

Irrigation pumps must often deliver water at different times to locations with different pressure requirements. This is really a situation with two system curves. In that case, a pump should be selected to closely match both desired operating points at a reasonable efficiency. It is rare that both points will fall on the selected impeller curve, but no operating points can lie above the curve. For those operating points which fall below the pump curve, the pump will produce a slightly higher flow rate than needed.

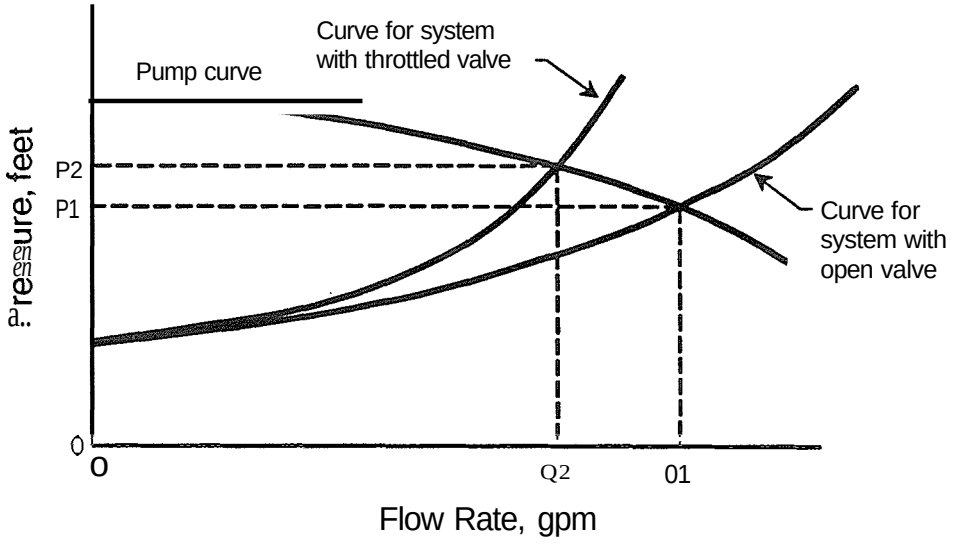


Figure 8. General effect of throttling a valve on an irrigation system supplied by a pump. When the valve is throttled the flow rate is reduced and the pump pressure is increased.

Irrigation designers frequently attempt to select a pump for which the design point will fall slightly to the right of the peak impeller efficiency. If the water level in a well drops, or if the pump has some slightwear, the flow rate will decrease but the pump performance will move into a zone of improved efficiency. If the initial design point was selected to the left of the maximum efficiency point, the efficiency would decrease if the water level in the well dropped.

In summary, a pump will be able to provide a wide range of flows and pressures, but it can only provide a limited range of flow/pressure combinations with a high efficiency. When an irrigation system is designed the TDH and flow rate should first be selected, after which a pump should be selected to operate efficiently at that design condition. Pumps are manufactured to provide virtually any necessary combination of flow rate and pressure.

Radial Flow (Centrifugal) Pumps

Pumps which are commonly referred to as split case booster, end suction booster, and deep well turbines are generally radial (centrifugal) pump designs. Figure 9 illustrates the

action of the impeller on the water. Water is forced from the eye of the impeller (the center inlet) radially out to the impeller edges, after which it is forced through specially designed bowls to create pressure.

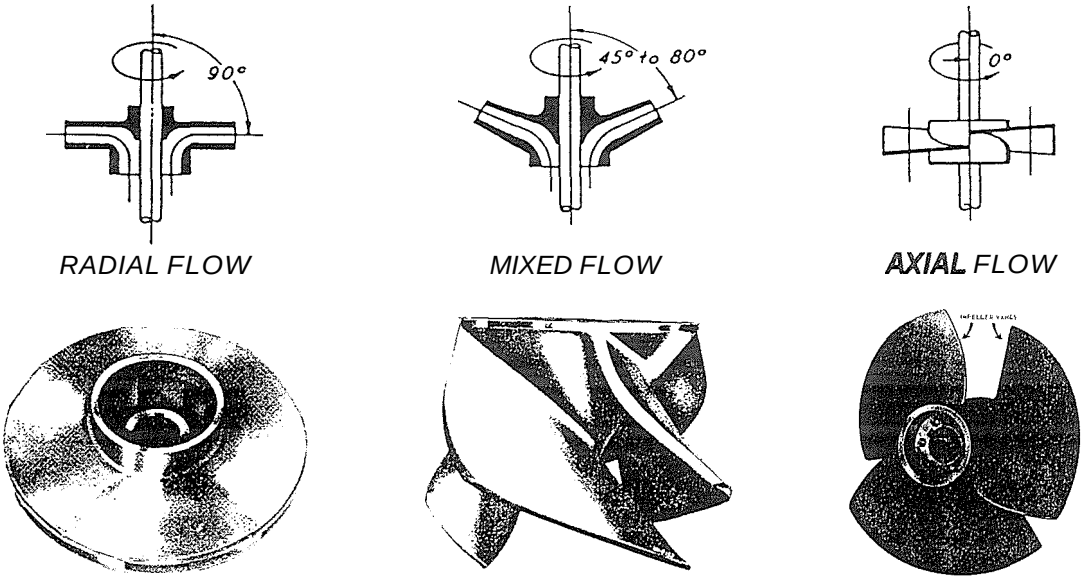


Figure 9. Impellers of various designs

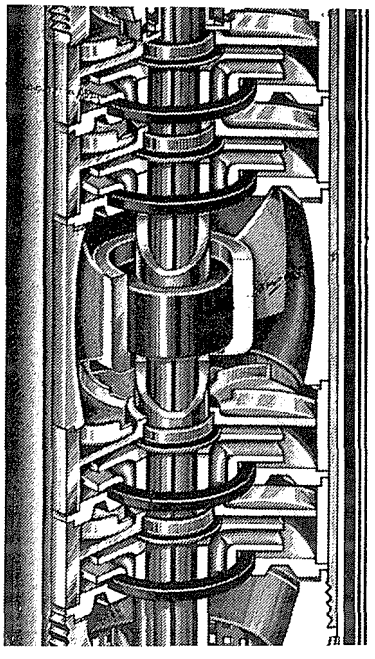


Figure 10.
Radial flow impellers with bowls.

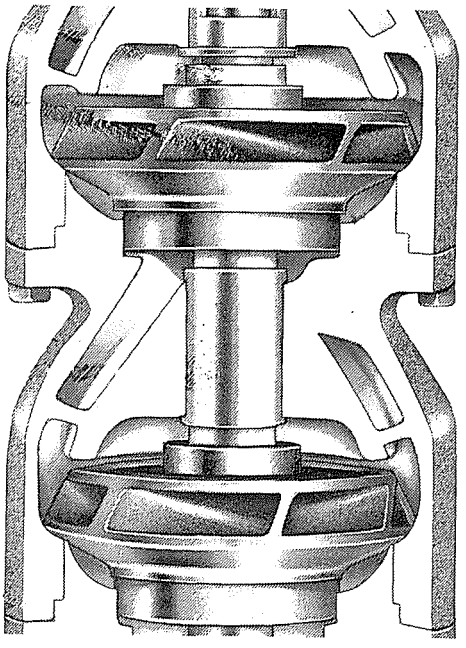


Figure 11.
Mixed flow impellers with bowls.

Figure 6 illustrates a typical centrifugal pump performance. It is characterized by a rather flat curve (Le., a small change in pressure with a certain change in flow rate) over the beginning two thirds of the curve. The brake HP (shown as "Bowl HP" on the curve) increases as the flow rate increases. The highest pressure occurs at or near zero flow rate. Centrifugal pumps are commonly used for medium and high pressure installations.

In general, radial (centrifugal) pumps can be throttled for a period oftime without causing any damage to the pump or motor. At shutoff, almost all of the BHP of a centrifugal pump is used to heat the water in the impeller/bowl assembly. For the large flow rate pumps generally used in surface irrigation, the pumps can sometimes be operated at zero flow rate for up to a few minutes before damage will result to the pump. The pump manufacturer should always be consulted if there is a possibility of running a pump at zero flow rate for more than 10-20 seconds, however.

The recommended procedure for starting centrifugal irrigation pumps is to start them with the discharge valve closed, and then to very slowly open the valve. By filling the irrigation lines slowly, many water hammer problems will be avoided (see the section of pipeline protection and surge in another chapter).

Operation at the best efficiency, orwithin the 30-percent span, allows the pump to operate with the least strain mechanically and hydraulically. At the best efficiency (as determined from the manufacturer's impeller performance curve), the mechanical or radial loads acting on the shaft, wear rings, and bearings are at their minimum. By operating the pump in this zone, the wear factor is greatly reduced; and sleeves, wear rings, shafts, and bearings will yield their longest life and need the least maintenance (Salvina, 1987).

All irrigation pumps must be primed (ie, the bowl/impeller assemblies must be full of water) before they are turned on. This is accomplished by either having the bowls below the water surface (as in a well installation) or by removing all the air from the suction pipe and the pump with a vacuum pump (as with a booster pump situated above a canal).

A case of overloading the motor can be illustrated with Figure 6 (ignoring any HP losses due to shaft bearings). Assume that the initial design condition for an irrigation system was 350 GPM at 64 feet of TDH. Impeller "A" would be chosen. A logical selection of motor size would be 7.5 HP. Later the irrigation system might be modified and operated at 600 GPM. The 7.5 HP motor would be supplying 9.0 HP, which is a 20% overload and more than recommended.

Axial Flow (Propeller) Pumps

Axial flow pumps are used in high flow rate, low lift situations. A need to lift several CFS to a canal 5-10 feet above a river would be a typical case.

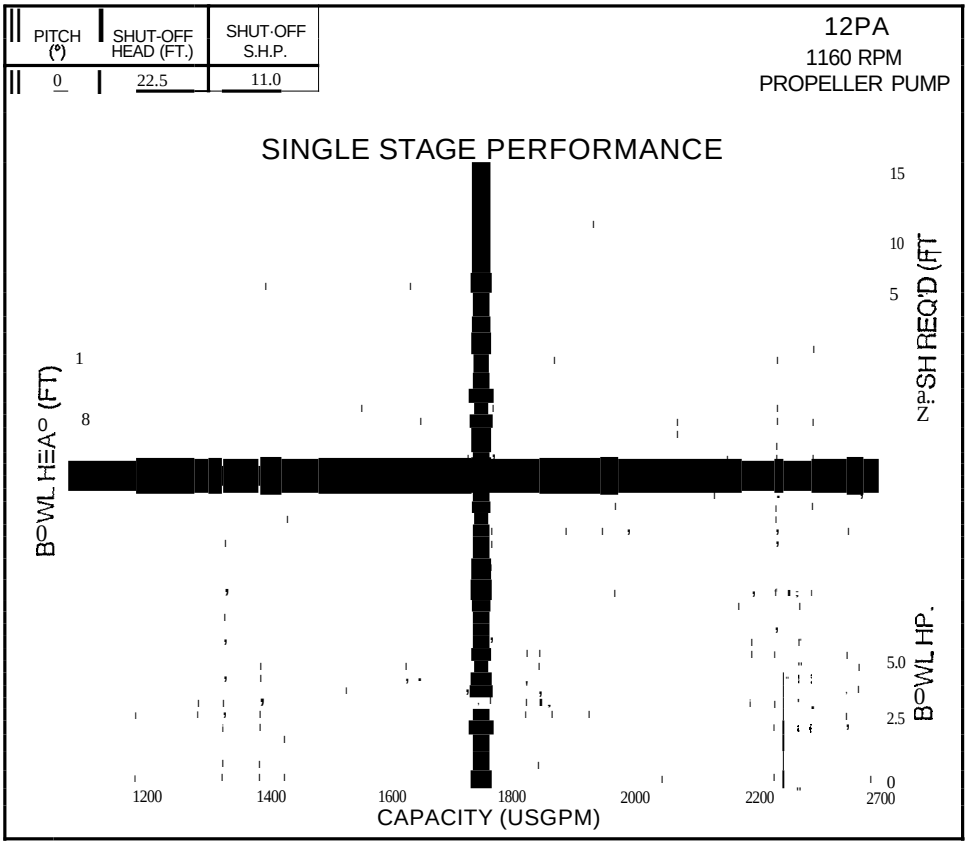


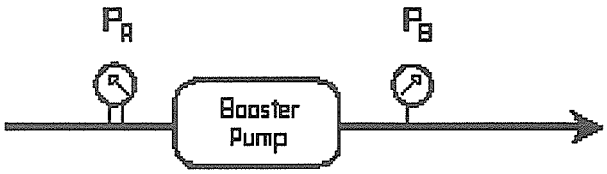
Figure 12. *Propeller pump performance curve. Courtesy of Peabody Floway.*

The most notable difference between the operating characteristics of a radial versus axial flow design is how the BHP varies with flow rate. With a propeller pump the BHP increases as the flow rate decreases. Therefore, propeller pumps are not started with the same rules as for centrifugal pumps; the general rule is to never throttle a propeller pump.

Calculating Pump TDH

The Total Dynamic Head (TDH) is the pressure which must be supplied by the pump. This is not the same as the total irrigation system pressure. For example, a booster pump may only provide 50 feet of a total of 150 feet required by an irrigation system.

For a booster pump, calculation of the TDH is illustrated in Figure 13. Pressure available at the inlet to a booster pump is not lost; it is accounted for in the TDH requirement calculation. For a well pump, calculation of the TDH is illustrated in Figure 14. Notice that the setting of the bowls is not a part of the TDH calculation.



$$TDH = P_B - P_A$$

Figure 13. Calculation of the TDH required by a booster pump.

Example #1. Calculation of the required TDH of booster pump.

In this example, assume that the booster pump is an end suction centrifugal pump located above a canal. Therefore, PA is negative (ie, a vacuum).

Assume the following:

PA = -6 feet
PB = 60 feet
TDH = PB - PA = 60 feet - (-6 feet)
= 60 feet + 6 feet
= 66 feet

Example #2. Calculation of the TDH required by a booster pump.

In this example, a booster pump is used to increase the pressure coming from a well pump. The booster is located at the well discharge.

Assume the following:

Discharge pressure of the well pump = 20 feet
Pressure needed at the discharge of the booster = 80 feet
PA = 20 feet
Ps = 80 feet
TDH = Ps - PA = 80 feet - 20 feet
= 60 feet

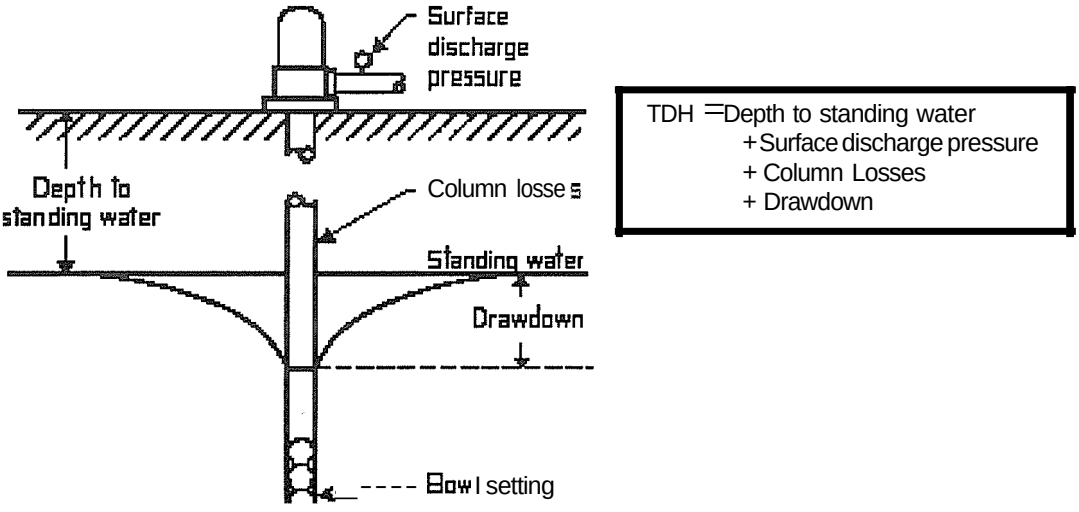


Figure 14. Calculation of the TDH required by a well pump

The "pumping water level" in a well is the sum of the depth to standing water and the drawdown. The drawdown depends upon the flow rate, and the depth to standing water may vary from year to year and from season to season. In a new installation it is essential to develop the well properly and to evaluate the pumping water level in order to calculate the yield of the well and the required TDH.

Example #3. Calculation of the TDH required by a well pump.

Assume the following conditions:

Depth to standing water level = 100 feet
Estimated column losses = 3 psi
Estimated drawdown = 20 feet
Pump has an open discharge (into air)

TDH = Standing depth + column loss + drawdown + surface discharge P
TDH = 100 ft + 3 psi x (2.31 ftlpsi) + 20 ft + 0 ft
=127 ft.

NPSH and Cavitation

The NPSH is an especially important consideration for irrigation system designers in pump installations in which the pump impellers may be close to or above the supply water level, such as with a booster pump taking water from a canal. The following explanation, taken from Peerless (1977) describes more about pump suctions and NPSH:

> "At any given temperature all liquids have a definite pressure atwhich they boil. Every day we witness the fact that a liquid boils at atmospheric pressure when it reaches a sufficiently high temperature. It is important also to remember that a liquid will boil at any temperature if the pressure is reduced sufficiently. It is the problem of the system designers and pump application engineers to make certain that there is sufficient pressure on the fluid being fed to the pump so that the liquid does not boil in the suction system of the pump.
>
> The term "suction lift" is very misleading. No reciprocating, gear, turbine vane, or centrifugal pump can lift any liquid. It is imperative that the liquid be forced into the pump for it to operate properly. The weight of air in the earth's atmosphere is very frequently used to force liquid into the pump suction and creates the illusion that a pump can lift a liquid. In a closed system where atmospheric or other pressure in excess of the liquid vapor pressure is not available, the liquid level must always be maintained above the pump centerline.
>
> ... In order for a pump to operate, the pressure pushing the liquid into the pump must overcome the static elevation, thefriction, and the turbulence losses in the suction line, accelerate the fluid, and maintain sufficient pressure to keep the fluid in the suction line from boiling (Le., to prevent the pressure from dropping as low as the vapor pressure of the fluid).

If the NPSH available in an installation is insufficient, the pump will cavitate and serious operational difficulties may develop. These troubles can include serious reduction in capacity and efficiency, excessive vibrations, reduced life of pump parts due to cavitation erosion, and damage to the pump from possible vapor lock and running dry.

A centrifugal pump has a minimum required NPSH to prevent cavitation, which varies with capacity and speed. This characteristic is inherent in the design of a pump and is just as much a performance characteristic as its head-capacity relation. In order for a pump to operate cavitation-free, the system NPSH available in an installation must exceed the NPSH required of the pump for operating conditions."

The NPSH required is the sum of (a) the drop in pressure inside the pump plus (b) the velocity head at the pump inlet. The NPSH required is not included in the calculation of TDH. This drop in pressure inside the pump inlet is recovered at the discharge of the pump.

The system NPSH available at an installation can be calculated as:

NPSH available = Patm - Δ Elev - Hf - P_V

where all units are in feet and

Patm = Atmospheric pressure, from Table 2.

Δ Elev= Height of the pump inlet above the water source

Hf = Friction in the suction line, including entrance losses

P_V = Vapor pressure, from Table 3.

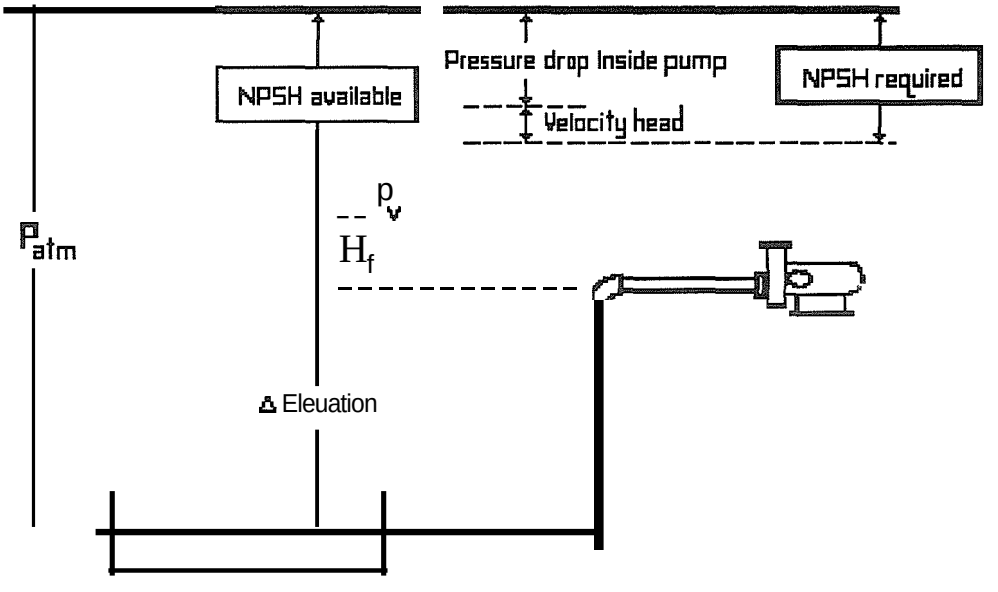


Figure 15. Determination of NPSH available for a booster pump above water source. To avoid cavitation, the NPSH available must be greater than the NPSH required.

*Table 2. Altitude vs. Standard Atmospheric Pressure**

Altitude above Sea Level		Atmospheric Pressure at 75 degrees F (24 degrees C)
Feet	Meters	Feet
0 (sea level)	0	34.0
500	152	33.4
1000	305	32.8
2000	607	31.6
3000	914	30.5
4000	1219	29.4
5000	1524	28.3
8000	2438	25.2
10000	3048	23.4

**Note -Atmospheric pressure changes daily due to weather conditions. These are standard values.*

Table 3. Vapor pressure of water at various temperatures

Water Temperature		Vapor Pressure Feet
Degrees F	Degrees C	
32	0	0.21
50	10	0.41
70	21	0.84
90	32	1.61
110	43	2.93

The maximum elevation difference between a pump and the water source can be estimated by rearranging the NPSH equation to:

Max. height of pump above water surface, feet

$$Patm - NPSH_{required} - Hf - P_V - \text{Safety factor}$$

where

Patm = from Table 2, feet
NPSHrequired = from pump curve, feet
Hf = All minor losses and friction on suction side, feet
P_V = Vapor pressure of water, feet, from Table 3
Safety Factor = at least 2-3 feet to account for weather changes, plugged inlet screens, etc.

Pumps in Series and Parallel

Pumps are frequently combined in series or in parallel. Figure 16 illustrates a typical well pump which is actually a combination of three identical pumps in series. In a series installation, all the water from one pump passes through the next pump. Pumps are placed in series to gain additional pressure. Each pump adds an additional increment of pressure to the incoming water.

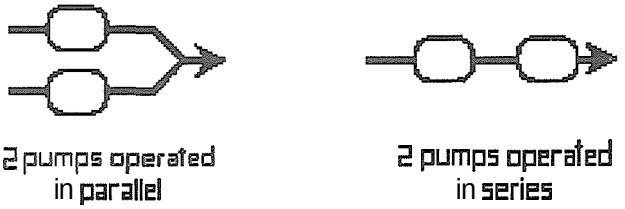


Figure 16. *Definition of parallel and* series *installations.*

Parallel installations are used when several pumps are used side-by-side to provide additional flow. A small pump may be used for small flows, a medium pump used for medium flow rates, and the two pumps operated together (in parallel) for large flows. This is common at canal entrances and reservoir discharges.

When two or more pumps are operated together in series and/or in parallel, they act as a single pump. A combined pump curve can be developed to predict performance. A well pump may be operated very efficiently in series with a booster pump if they are appropriately matched.

Series combination. To develop a combined pump curve for two impellers in series, a table such as Table 4 must be computed. The TDH of each impeller must be computed for several flow rates, and the TDH's summed.

TDH of impellers in series = Sum of individual TDH's at that flow rate

$$\text{Combined impeller efficiency} = \frac{100 \times (TDHa + TDHb) \times GPM}{3960 \times (BHPa + BHPb)}$$

where
TDHa = TDH of one impeller at the specified GPM, feet
TDHb = TDH of second impeller at same GPM, feet
BHPa = Brake HP of the impeller at TDHa and the specified GPM
BHPb = Brake HP of the second impeller at TDHb at the same GPM

Table 4. *Computation of combined curve with two impellers in* series.
Impellers are curve "A" and curve "B" from the 10LKM pump on Figure 6.

GPM	Impeller "A"		Impeller "B"		Combined Curve	
	TDH	BHP	TDH	BHP	TDH, ft	Efficiency, %
0	66	5.1	56	4.5	122	0
100	66	5.1	56.2	4.5	122.2	32
200	65.1	5.5	56	4.8	121.1	59
300	63.7	6.5	54	5.7	117.7	73
400	59.2	7.5	49.5	6.5	108.7	78.4
500	51.1	8.3	41.8	7.2	92.9	75.7

Parallel combination. Table 5 was used to develop a combined curve for two impellers operated in combination. The GPM of each impeller must be computed for several pressures, and the GPM's summed.

GPM of impellers in parallel = Sum of individual GPM's at the specified TDH

$$\text{Combined impeller efficiency} = \frac{100 \times (GPMa + GPMb) \times TDH}{3960 \times (BHPa + BHPb)}$$

Table 5. Computation of combined CUNe with two impellers in parallel. Impellers are CUNe "A" and CUNe "B" from the 10LKM pump on Figure 6.

TDH	Impeller "A"		Impeller "B"		Combined Curve	
	GPM	BHP	GPM	BHP	GPM	Efficiency, %
66	0	5.1	0	4.5	0	---
64	265	6.1	0	4.5	265	40
60	385	7.3	0	4.5	385	49
56	445	7.8	0	4.5	445	51
56	445	7.8	180	4.7	625	70.7
54	470	8.0	290	5.6	760	76
52	490	8.2	350	6.2	840	76.6
50	513	8.4	390	6.5	903	76.5
48	535	8.6	420	6.7	955	75.7
46	555	8.7	445	6.8	1000	75
44	575	8.9	470	7.0	1045	73

Table 5 shows at a TDH of 54 feet to 46 feet the efficiency of this particular impeller combination is relatively high. However, the combined efficiency drops off rapidly at TDH's of greater than 56 feet. Impeller "B" cannot pump any water at a TDH greater than 56 feet; the impeller simply spins. If a foot valve is not provided on the pump with impeller "B", impeller "A" could actually pump water backwards through the spinning impeller "A" at the higher pressures. At high pressures impeller "B" should be shut off and isolated from impeller "A".

Original Pump Selection and Effects of Changing Conditions

The examples below, using Figure 17, will illustrate the procedure of selecting a proper impeller combination for a well pump, and how the performance would vary if conditions were changed after installation.

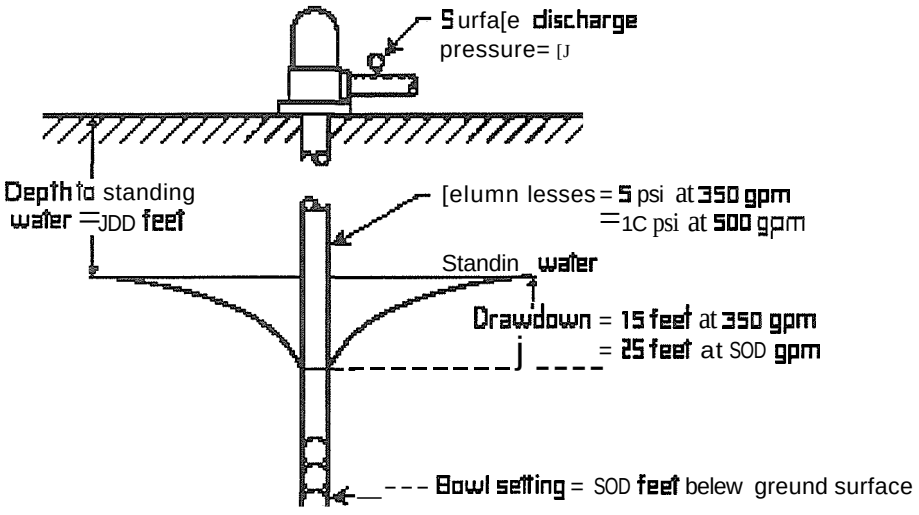


Figure 17. Conditions for example problem illustrating proper impeller selection.

Example problem #1: Original design situation with an open discharge

Given: Conditions of Figure 17, with a flow rate of 500 GPM

Find: Assuming that the only pump available is the one of Figure 6, select one impeller size that can satisfy the conditions of Figure 17.

Specify:

a. Impeller diameter
b. Number of impellers
c. Combined efficiency

Solution:

1. Determine the TDH requirement.

TDH=	Depth to standing water	=	300'
	+ Column losses (10 psi x 2.31'/psi)	=	23'
	+ Drawdown	=	25'
	+ Pressure at ground surface	=	0'
		TDH =	348'

2. Set up a table to determine the most efficient impeller to place in series for a total of 500 GPM and TDH of 348'

Possible # of Impellers	FtIlmpeller	Impeller Size	Impeller Efficiency
1	348	none available	
5	69.6	none available	
6	58	none available	
7	.49.7	slightly less than "A"	77%
			<----- Choose
8	43.5	slightly greater than "8"	74%
9	38.7	slightly less than "8"	72%

A total of 7 impellers will be used in series to provide the required total TDH. It is the logical choice because (a) it has the least number of impellers which can do the job, so has the lowest initial cost, and (b) it has the highest efficiency.

Example problem #2 - The pump selected in example problem #1 is in the well. Now a pipeline is connected to the pump discharge, so it no longer discharges into the atmosphere. The new flow rate is 350 GPM.

Find: The pressure at the ground surface discharge.

Solution:

1. Determine the TDH delivered by one impeller at 350 GPM. This can be found on the pump CUNes.
   From the pump *CUNe,* the TDH of one impeller (ofthe size selected in example problem #1) at 350 GPM is 61 feet. This is found by sketching an intermediate *CUNe* (between "A" and "8") which passes through (500 GPM, 49.7 feet) on the 10LKM *CUNe* sheet.

2. Determine the TDH delivered by all 7 impellers in series at 350 GPM.

   TDH = 7 impellers x 61'/impeller
   = 427 feet

3. Calculate the pressure remaining after the water has been raised to the ground surface and also overcome the column friction.

P at ground surface =		
TDH	=	427.00'
- column losses		- 11.60'
- depth to standing water		- 300.00'
- drawdown		- 15.00'
P atG.S.	=	100.40'

References

Montgomery, D.C. 1981. How to specify and evaluate energy-efficient motors. General Electric Company. Hendersonville, Tenn.

Peerless Pump Co. 1977. NPSH - Net Positive Suction Head. Peerles Pump Co. Indianapolis, Indiana 46206.

Salvina, L.C. 1987. Whatto look for in centrifugal pump maintenance. Opflow. Vol. 13, NO.2. American Water Works Assoc.

CHAPTER 21

CANALS AND DITCHES

CANALS AND DITCHES

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CANALS AND DITCHES

Introduction

Canals and ditches are often used for water conveyance rather than buried pipelines. The advantages of canals are:

1. Lower initial cost for large flows on very flat ground
2. Simple construction in some situations.

For large flows such as with an irrigation district conveyance system, canals may be the only economically feasible solution. However, pipelines are generally preferrable for permanent installations for small distribution systems and conveyance facilities within a field. Disadvantages of canals and ditches include:

1. Land is taken out of production for rights of way.
2. Maintenance requirements are greater than for pipelines.
3. On on-farm conveyance systems a "dead" area must be provided between the canal and the field to provide room for tractors to turn around.
4. Canals provide obstructions for equipment in the field.
5. Small canals are more difficult to control than small pipelines; unless level basins or furrows are used, it is difficult to implement some of the more modern surface irrigation management techniques with small canal supplies.
6. Canals must be placed uphill from the "command" area, yet can only flow downhill. Pipelines can run both up and downhill.
7. Canals typically have large seepage losses which can be avoided with pipes.

Canal sizing forthe maximum anticipated steady flow rate is relatively simple. The hydraulic solution is straightforward, and designers have rules stating the maximum side slope and velocities. The design aspect which is most frequently overlooked is control of the water surface when the flows vary.

All irrigation canals usually operate under unsteady (Le., changing with time) flow rate. Any time the flow rate is changed at one turnout, the water levels and flows downstream will be affected. In addition, the changes arrive gradually at the various points downstream, making the problem more complicated than with a pipeline.

Channel Design

Manning equation. The Manning equation (sometimes referred to as the "Chezy-Manning equation") is most commonly used to solve steady and uniform open channel flow problems.

$$Q = \frac{1.49}{n} \times A \times Rh^{0.67} \times SO.5$$

where

A = Cross section area of flow, square feet
Q = Flow rate, CFS
n = roughness coefficient from Table 1.

Rh =Hydraulic radius, feet; the ratio of the area to the wetted perimeter
S =Slope of the canal bottom, feet/feet

The Manning equation takes the form below for trapezoidal channels.

$$Q = \frac{1.49 \times S^{.5} \times [(b \times y) + (m \times y^2)]^{1.67}}{n \times [b + (2 \times Y \times \sqrt{\{m^2 + 1\}})]^{.67}}$$

where

Q =CFS
n =roughness coefficient
b =bottom width of the channel bottom, feet
m =channel side slope (ratio of horizontal to vertical)
For a rectangular channel, m=O
S =hydraulic gradient, feet/feet (slope of channel bottom)
Y =water depth, feet

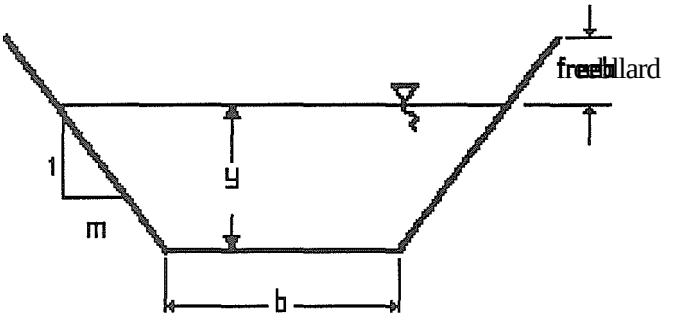


Figure 1. *Symbols for trapezoidal canal dimensions used in Manning equation.*

If the depth of water is unknown but the flow rate, slope, and geometric properties of the channel are known, the depth cannot be solved for directly mathematically. The solution must be obtained by trial and error, with a graphical solution, or using a numerical procedure such as Newton's method. Figure 2 contains a computer program using Newton's method.

```
 5 PRINT "THIS PROGRAM SOLVES THE MANNING EQUATION FOR UNIFORM FLOW"
10 PRINT"        IN A TRAPEZOIDAL CHANNEL"
15 PRINT
20 PRINT"        SOLVE FOR ONE UNKNOWN VARIABLE"
25 PRINT
30 PRINT " ******* "
35 INPUT "WHEN READY TO PROCEED, INPUT ANY NUMBER AND PRESS (RETURN)"; ZZ
40 HOME
45 PRINT "THE VARIABLES ARE:"
50 PRINT"        1. DEPTH OF WATER, FEET"
55 PRINT"        2. FLOW RATE, CFS"
60 PRINT "       3. ROUGHNESS, MANNING'S N"
65 PRINT "       4. BOTTOM WIDTH, FEET"
70 PRINT "       5. SLOPE OF CHANNEL BOTTOM, FEET/FEET"
75 PRINT"        6. SIDE SLOPE (HORIZONTALNERTICAL), NOT (VERT/HORIZ)
```

```
80  PRINT "  *******  "
85  PRINT: PRINT
90  PRINT  "IF BOTTOM WIDTH IS UNKNOWN,"
95  PRINT  "          THE ANSWER BELOW IS  4"
100 PRINT
105 PRINT  "AFTER GIVING THE ANSWER, PUSH THE (RETURN) KEY"
110 PRINT
115 INPUT  "THE UNKNOWN VARIABLE IS NUMBER        ";NP
120 HOME
125 PRINT
130 PRINT  "FOR THE UNKNOWN VARIABLE, PUT IN A REASONABLE FIRST ESTIMATE"
135 PRINT
140 PRINT  "PRESS THE (RETURN) KEY AFTER EACH ENTRY"
145 INPUT  ", DEPTH IN FEET      =         "; Y
150 INPUT" Q, FLOW IN CFS      =         "; Q
155 INPUT  "  MANNING'S N        =         "; N
160 INPUT  "  BOTTOM SLOPE, FT/FT  =  "; SO
165 INPUT  "  BOTTOM WIDTH, FEET      =  "; B
170 INPUT  "  SIDE SLOPE, HORNER      =" ; M
174  REM  P IS WETTED PERIMETER
175  P = B + 2*Y*(MI\2 + 1)^0.5
179  REM  A IS CROSS SECTIONAL AREA
180  A = B*Y + M*YI\2
184  REM  R IS HYDRAULIC RADIUS
185  R = AIP
190  DE = 1.49*A*(RI\.66667)
195  IF NP = 1 THEN GOTO 255
200  IF NP = 2 THEN GOTO 225
205  IF NP = 3 THEN GOTO 235
210  IF NP = 4 THEN GOTO 255
215  IF NP = 5 THEN GOTO 245
220  GOTO 255
225  Q = DE * SQR(SO)/N
230  GOTO 500
235  N = DE * SQR(SO)/Q
240  GOTO 500
245  SO = (Q*N/DE)1\2
250  GOTO 500
255  MN = 0
260  A = (B + M*Y)*Y
265  SZ = SQR(MI\2 + 1)
270  P = B + 2*Y*SZ
275  SS = 1.49 * SQR(SO)
280  QN = Q * N
285  PP = PI\.666667
290  AP = AI\.666667
295  F = QN * PP - SS * A * AP
300  IF NP = 4 THEN GOTO 315
305  IF NP = 1 THEN GOTO 330
310  IF NP = 6 THEN GOTO 345
315  PX=1
320  AX= Y
325  GOTO 360
330  PX = 2 * SZ
```

```
335 AX = B + 2*Y*M
340 GOTO 360
345 PX = 2 * M * Y/SZ
350 AX=Y*Y
360 OF = 0.666667 * ON * PX * PPIP - SS * AP * AX * 513
365 DT = FIDF
370 IF NP = 1 THEN Y = Y-DT: GOTO 395
385 IF NP = 4 THEN B = B - DT: GOTO 395
390 IF NP = 6 THEN M = M - DT
395 MN = MN + 1
400 IF MN < 20 AND ABS (DT) > 0.00001 THEN GOTO 260
450 REM
455 REM END OF NEWTON-R LOOP WITH 0 AND OF
460 REM
500 FOR NO = 1 TO 10: PRINT: NEXT NO
505 PRINT" SOLUTION"
510 PRINT: PRINT
515 PRINT" 0 (CFS) = "0
520 PRINT" MANNING'S N = "N
525 PRINT" CHANNEL BOTTOM SLOPE = "SO
530 PRINT" BOTTOM WIDTH (FEET) = "B
535 PRINT" WATER DEPTH (FEET) = "Y
540 PRINT" SIDE SLOPE (HORIZNERTICAL) = "M
545 PRINT: PRINT
550 PRINT "IF YOU WANT TO RUN THE PROGRAM AGAIN,"
555 PRINT " TYPE IN THE WORD RUN"
560 END
```

Figure 2. Computer program written in Basic for the solution of Manning's equation.

For designers without small computers, an assortment of graphical solutions are available for common concrete slipform and other sections.

Table 1. Typical values of Manning's roughness coefficient, n (Jeppson, 1965)

Channel Material	Mannin n
Smooth brass, glass, lucite	0.010
Planed timber, unpainted steel, finished concrete	0.012
Unfinished concrete, asphalt vitrified clay, brick in mortar, cast iron	0.015
Riveted steel, rubble masonry	0.017
Rough concrete, small firm gravel	0.020
Corrugated metal, clean excavated earth	0.023
Earth, good condition, rock excavation	0.025
Clean natural streams	0.030
Weedy streams and channels	0.050

Side slopes. Side slopes for typical on-farm slip form concrete canals range from 1:1 (horizontal:vertical) to 1:1.5. The allowable side slopes on earth lined canals varies with soil stability.

Table 2. Maximum canal slopes, general guidelines

Soil Type	Side Slope (horiz:vertical)
Sandy Loam	3.0: 1
Silt Loam	2.0: 1
Clay Loam	1.5: 1
Hard Clay	1.0: 1

Seepage losses. Seepage losses from both lined and unlined canals can be a major problem, especially at the beginning of an irrigation season when the intake rates are high. Table 3 gives an approximate (at best) estimate of canal seepage losses depending upon soil texture for unlined ditches.

Proper soil compaction at optimum moisture content can almost eliminate seepage on some soils. This is certainly obvious from the fact that many dams are made of compacted earth. Inadequate attention is generally given to soil compaction on ditches constructed on the farm level.

Table 3. Approximate canal seepage losses (Withers and Vipond, 1980). Values are given for relative comparison purposes only, and can vary widely.

Type of soil	Seepage Loss	
	m3/m2lday	ft3/ft2lday
Impervious clay loam	0.07 - 0.10	0.23 - 0.33
Clay loam, silty soil	0.15 - 0.23	0.49 - 0.75
Clay loam with gravel, sandy clay loam	0.23 - 0.30	0.75 - 0.98
Sandy loam	0.30 - 0.45	0.98 - 1.48
Sandy soil	0.45 - 0.55	1.48-1.80
Sandy soil with gravel	0.55 - 0.75	1.80 - 2.46

Sediment and Erosion. Water with a high sediment load is very difficult to manage. When the water velocity decreases, the sediment drops out and fills up canals. This has a beneficial effect in that it reduces seepage losses, but it also reduces the flow capacity of the canals.

Ideally a delivery system should be managable so that flow rates can be varied in response to farmer demands. With silt-laden water this is only possible if the water levels in the canals are simultaneously allowed to vary in order to maintain high velocities. This complicates outlet design; water on the main canals is generally checked at the outlets to provide enough water depth to deliver water to the ground surface.

Ditches and canals with high sediment loads should be operated with a velocity between two limits. The upper limit is the point of erosion; the lower limit is that of sedimentation. The two limits are often not compatible, and either sedimentation or erosion will occur.

Table 4. Maximum flow velocities to prevent erosion (Booher, 1974).

Unlined Ditch Soil	Maximum velocity, feet/sec
Sand	1.0 - 2.5
Sandy Loam	1.7 - 2.5
Clay Loam	2.0 - 3.0
Clays	3.0- 5.0
Gravel	3.0 - 5.0

Table 5. Critical velocities for the prevention of sift deposits (Zimmerman, 1966)

Depth of water (feet)	Minimum velocity (feet/sec)
1	0.8
2	1.3
3	1.7
4	2.1
5	2.4
6	2.75
7	3.0

Table 5 shows that sediment accumulation is not a major problem on small canals, as compared to large irrigation district canals. On-farm canals rarely have depths greater than 3 feet, and only require low velocities to prevent sedimentation.

Miscellaneous. The following design points are provided by Kruse, Humpherys, and Pope (1981):

1. The minimum slope for an unlined farm ditch should be 0.0004 (.04 feet per 100 feet).
2. On-farm ditches which provide water to furrows must be capable of being checked so that the water surface is 4-6 inches above the ground surface. This provides adequate head for the operation of siphon tubes and spiles.
3. On-farm ditches which provide water to border strips may need to be checked higher than 6 inches above the ground surface because the water level in the border strips may be 4-6 inches deep at the head end.
4. Permanent on-farm ditches should have a freeboard of about 1 foot to provide for checking the water and to allow for settling. Minimum freeboard for ditches carrying 1-5 CFS should be 4-6 inches, and for ditches carrying 5-10 CFS, 6-8 inches.

Diverting Water from a Ditch to a Furrow or Border Strip

The shovel. The most simple method of diverting water from a ditch is to dig a temporary hole in the ditch bank, allow the water to flow through, and then refill the hole when the furrows or border strip at that location is irrigated. Although this is widely practiced in some areas, there is very little flow control ability with this method, it is labor intensive, and there are better ways to accomplish the same purpose.

Permanent Turnouts. Concrete lined ditches are often constructed with Waterman SureSeal Ditch Gates as turnouts for groups of furrows or individual border strips. The design includes a positive one-quarter turn cam lock, and a long-life gasket to provide a tight seal against leakage.

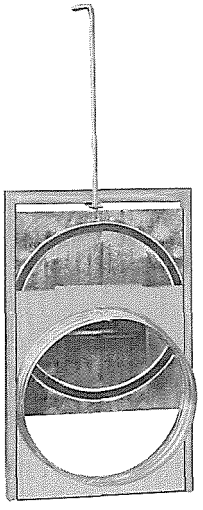


Figure 3. *Waterman SureSeal Ditch Gate.*

Other designs use short section of pipe supplied by the canal, with a pasture valve or alfalfa valve at the downstream end.

Of primary importance in design is the fact that the turnout be an underflow, or orifice type of design. The SureSeal Ditch Gate and alfalfa valves fall under this classification. A weir or overflow design is not desirable for a turnout, because the turnout flow will vary considerably with only a small change in water depth in the canal.

Siphon Tubes and Spiles. Siphon tubes are lightweight, inexpensive, and easy to use. They transport the water over the ditch banks while keeping the banks undamaged. They are commonly used in a variety of sizes, ranging from 1 inch diameter and up.

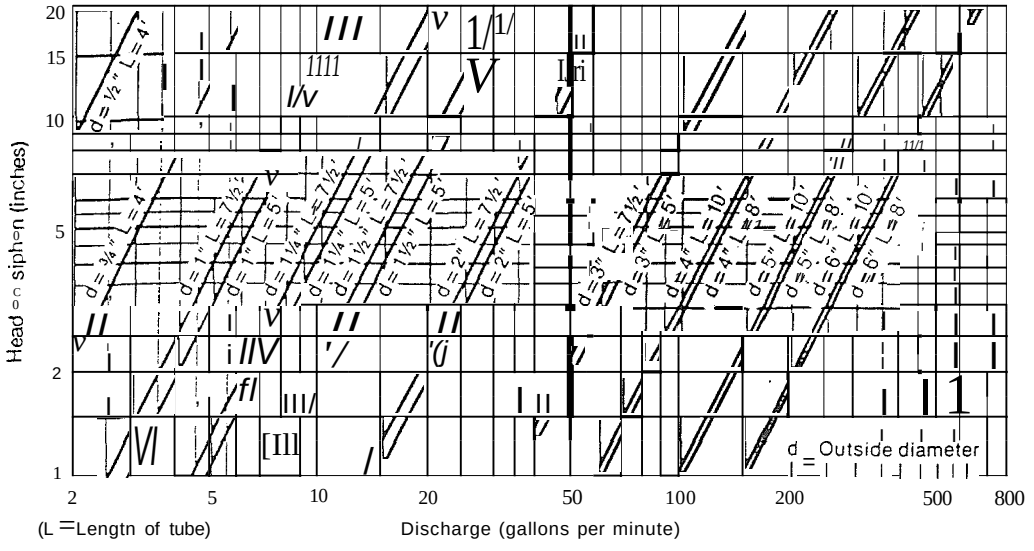


Figure 4. *Discharge of aluminum siphon tubes at various heads. (USDA, 1983)*

Figures in English Units

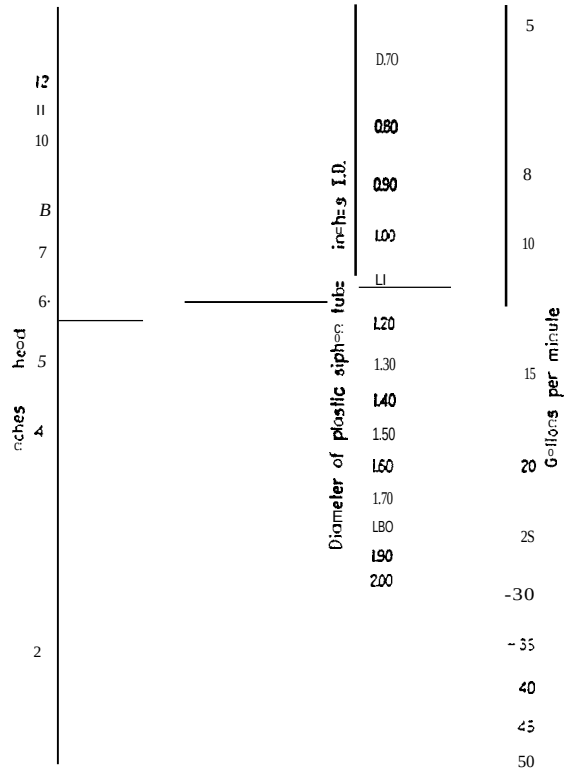


Figure 5. *Discharge of plastic siphon tubes at various heads (Hansen et al., 1980)*

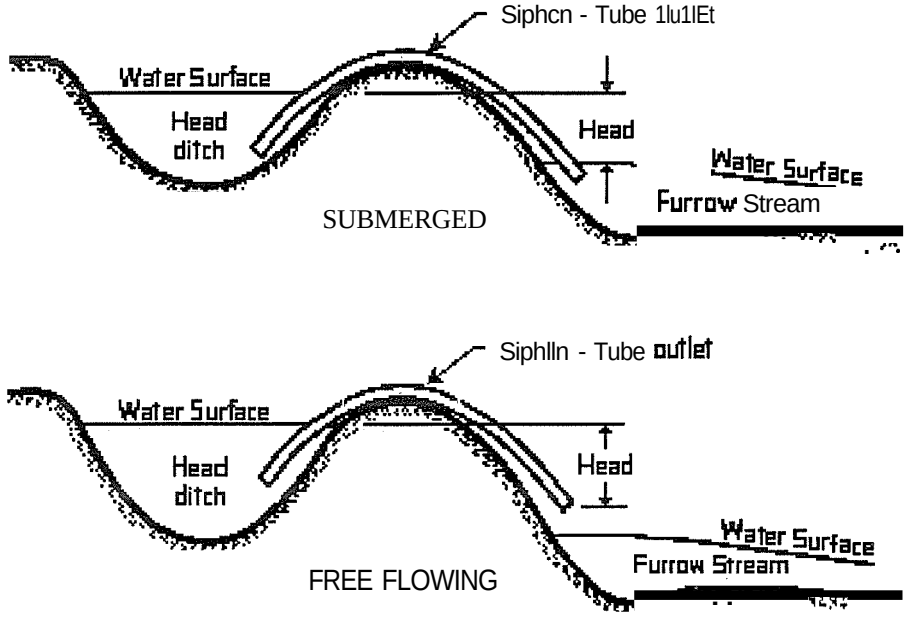


Figure 6. *Determination of the head* across a *siphon.*

Spiles are short sections of pipe which are temporarily placed in the banks of a ditch bank. They are often used in the situation shown in Figure 7, in which a few large siphons supply an intermediate ponding basin, from which water is supplied to individual furrows with the spiles.

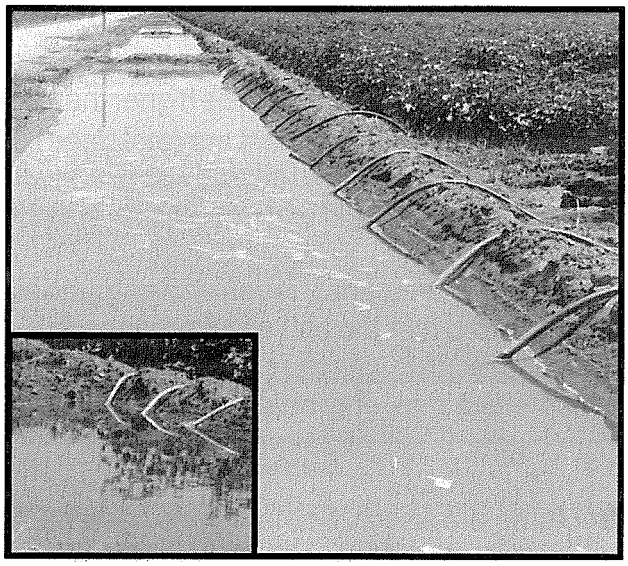


Figure 7. The use of an intermediate ponding basin with both siphons and spiles.

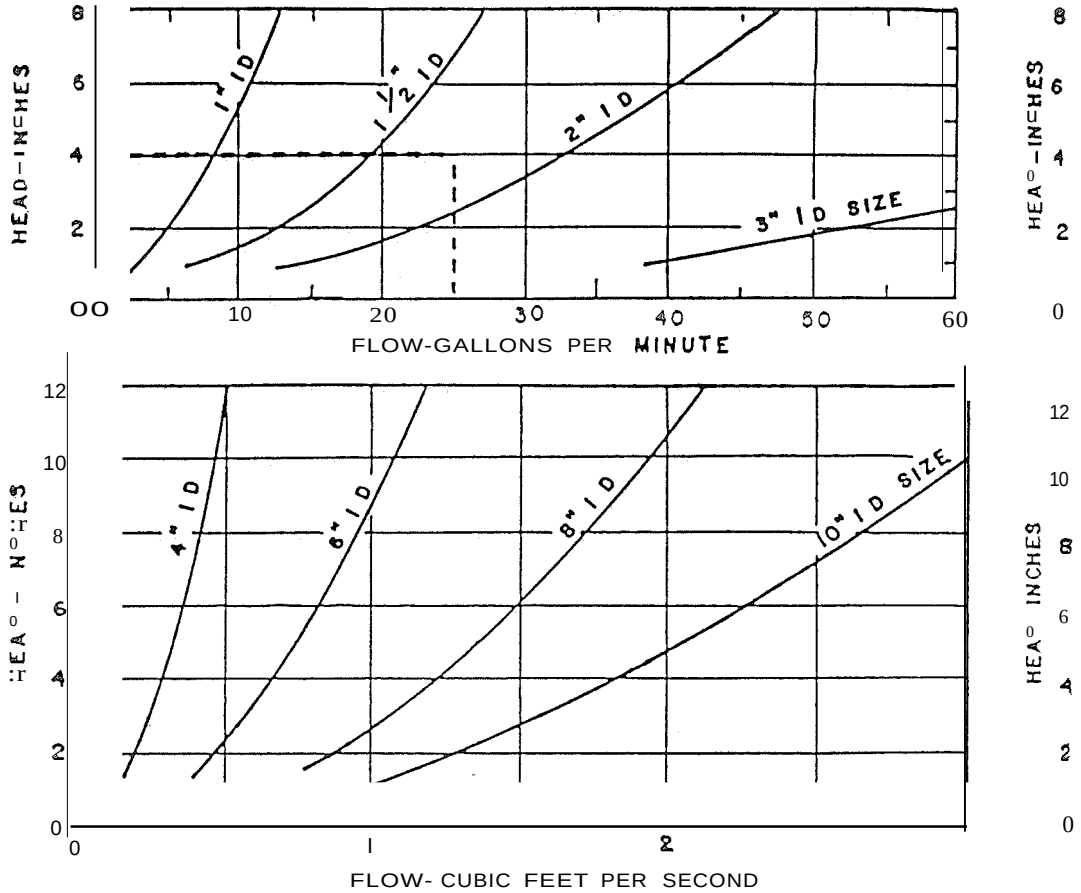


Figure 8. Capacities of spile-tube outlets (Ma", 1980)
TOP: Flow through spile-tube outlets.
BOTTOM: Flow through spile-tube turnouts.

Water Level Control

Lateral Ditches and Canals. In order to control the water flow from lateral canals into individual furrows or border strips, two requirements must be met:

1. The water level in the lateral canal must be higherthan the water level in the furrow or border strip.
2. The water level in the lateral canal should not change with time, so that the flow into the furrows or strips will not change unless intentionally.

The normal flow depth in many lateral canals is below the ground surface. Check structures are used to dam up the water so that the first requirement can be met. On lateral canals with a steep slope, several check structures may be needed for each set of siphon tubes, because the damming up effect may only be sufficient for 10-20 furrows upstream of the dam. The upstream check dams will allow most of the flow to pass over them.

A variety of check structures are used. Some canals have permanent structures; earth ditches usually use canvas dams. An overflow check structure is preferrable to an underflow structure, because the water level can be maintained more constant.



Figure 9. Canvas dam on an earthen ditch.



Figure 10. Check structure on a slip form canal.

Main Canals. Main canals supply laterals on large farms. Often the flow rates in the main canals will vary with time, but the flows down specific laterals should remain the same. With canals, the water levels will change if the flow rate changes. This water level change will cause the flow through the turnout to also change.

Check structures (cross regulators) are installed on main canals immediately downstream of turnouts which supply the lateral canals. If manually controlled, these main canal check structures should be of an overflow design, and the turnouts should be of an orifice design.

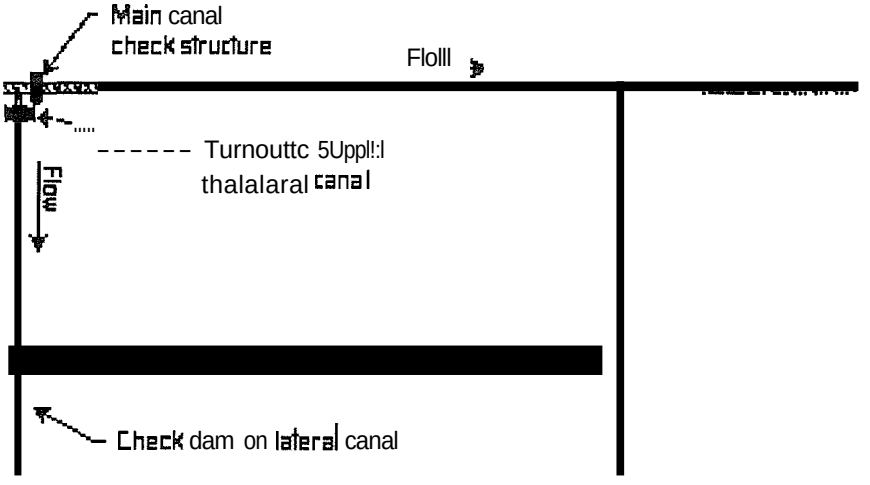


Figure 11. *Configuration of* a *main canal and lateral turnout. Plan view.*

The use of an overflow design on manually operated main canal check structures will pass the water level fluctuations to the very downstream section of the main canal. A buffer sump should be constructed just before the last lateral to capture any excess flow from the upstream sets. Water can be pumped from this sump to supply the most downstream sets. The result is a very small fraction of the water is pumped, and only a few feet. However, the complete canal system can be operated without the standard problems of varying flows into the lateral'canals.

To minimize the change in head on the turnouts to the laterals, the overflow, or weir designs, on the main canal can have the following design features:

1. Pass the majority of the main canal flow through an underflow gate, with some of the flow always passing over the gate also as a weir. This design is illustrated in Figure 12. The combination overflow and underflow helps prevent sediment deposition upstream of the check structure.

   The most common error in managing this check structure design is that the irrigators allow all of the water to pass under the gate rather than always having some flow pass over the gate. That will defeat the purpose; the gate will act hydraulically as an underflow gate rather than as an overflow gate.

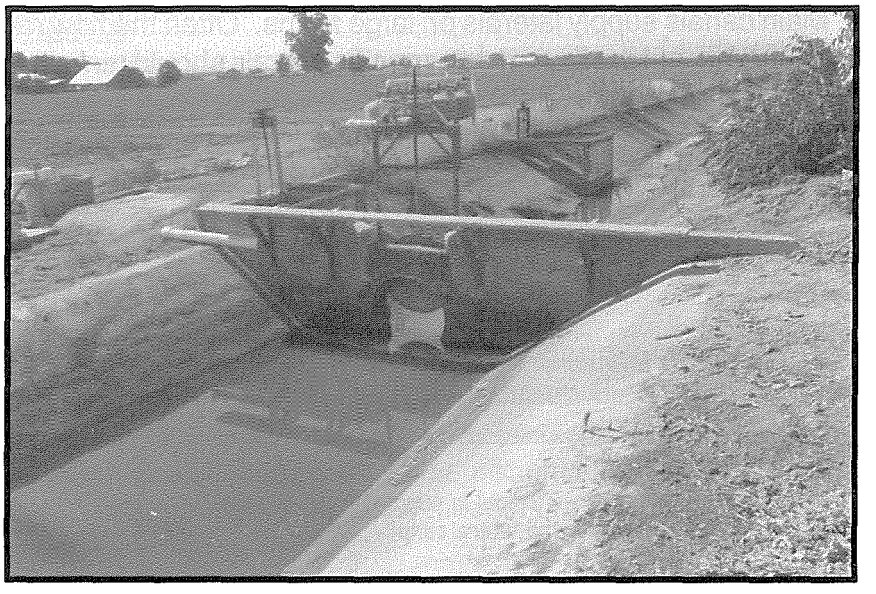


Figure 12. *Check structure in* a *main canal. Combination overflow and underflow.*

2. Use a very long crested weir as the main canal check structure. This requires an "oblique" or "duckbilled" weir design, as illustrated in Figure 13. With a long crested weir, a large change in flow rate across the weir will only give a small change in head. A large f1ushout gate should be installed close to the downstream point of the weir, to flush out silt which will accumulate upstream of the weir.

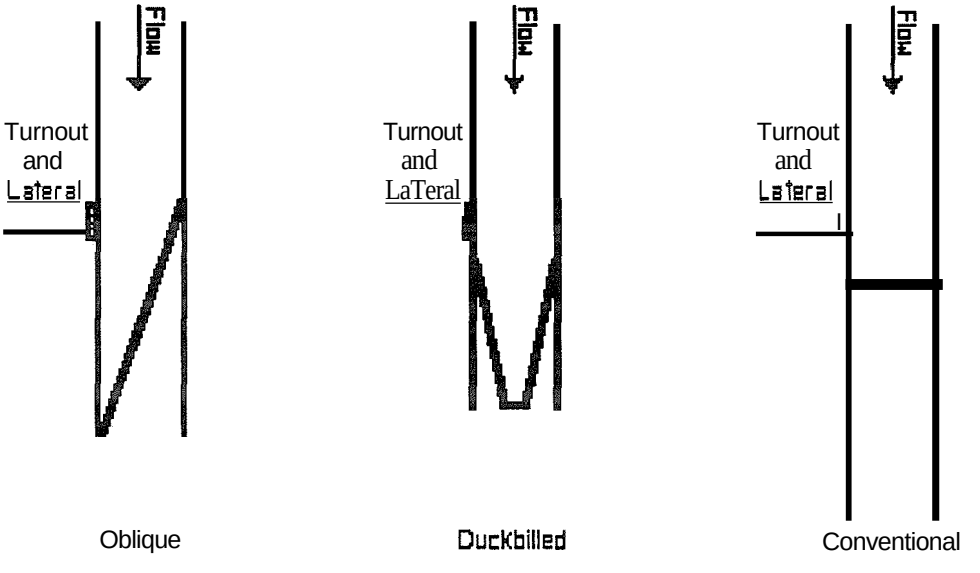


Figure 13. *Plan view of various weir designs.*
Oblique and duckbilled configurations give a *longer crest width than conventional designs.*

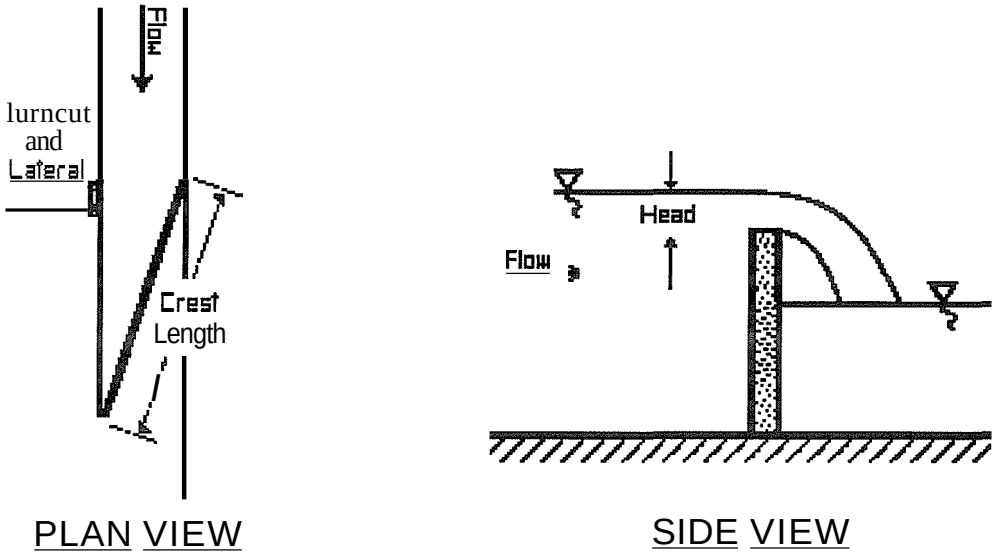


Figure 14. *Definition of terms on* a *weir.*

The approximate equation for long crested weirs is:

$$Q = 3.3 \times L \times F \times H1.5$$

where

Q = flow rate, CFS
L = weir crest length, feet
H = upstream water level above crest, feet
F = a de-rating value because a weir at an angle to the flow cannot pass quite as much water per foot of crest as a perpendicular weir.

There is little hydraulic benefit in having a total weir length greater than about 8 times the width of the water surface in the canal. At this length, the "F" value is about 0.8 for heads (H) of about 1.0 feet, and 0.9 for heads (H) of about 0.5 feet. In order for this equation to be valid, the downstream water level must be lower than the weir crest. The exact factors depend upon the precise shapes of the weirs, the type of crest, and the heads. However, these factors provide an approximate idea of the values to use in a simple on-farm design.

A long crested weir can be designed with a crest length sufficiently long to pass the maximum flow rate with a head of 6". At zero flow rate, the head would be 0". Therefore, there would never be more than a 6" variation in head on the weir (or the lateral turnout). The required weir length can be estimated from the data in Table 6.

As an example, assume that a main canal flow was a maximum of 10 CFS. In order to have no more than a 6" water depth over the weir crest at peak flow, the required crest length is:

$$L = \frac{10 \text{ CFS}}{1.05 \text{ CFS/foot of length}}$$

$$= 9.5 \text{ feet}$$

Head, inches	Flow rate per foot of weir length, CFS
1	0.08
2	0.21
3	0.39
4	0.60
5	0.86
6	1.05
8	1.53
12	2.64

Table 6. *Approximate capacities of long crested weirs.*

More Sophisticated Canal Control. A wide variety of automation techniques are available to control water levels and flow rates in canals. Readers should refer to the Proceedings of the July 1987 Specialty Conference on Water Delivery System Automation of the Irrigation and Drainage Division of the American Society of Civil Engineers, held in Portland, Oregon for a state of the art summary of such techniques. Another good reference is by Burt and Plusquellec (1990). The Irrigation Training and Research Center (ITRC) at California Polytechnic State University, San Luis Obispo, Calif., has a large outdoor facility to teach canal automation and control techniques.

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CHAPTER 22

FERTIGATION

FERTIGATION

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FERTIGATION

Introduction

Fertigation is the practice of simultaneously conducting Fertilization and Irrigation. Fertilizer is applied to the irrigation water and enters the soil with the water.

Fertigation can be an efficient and cost effective method offertilizing. With some irrigation methods such as micro-irrigation (drip and microsprinklers) all of the fertilizer must be applied through the irrigation water. Application of both fertilizer and pesticides is standard practice among center pivot and linear move irrigators.

When fertilizer is injected into the irrigation water, the fertilizer molecules are in solution and move with the water as an identical liquid. The fertilizer goes where the water goes; if more water goes into the soil at one point than at another, more fertilizer also goes into the soil at that point. Once the solution has entered the soil, some fertilizers will move through the soil with the water; others will stay near the soil surface.

Fertigation with surface irrigation has some aspects which make it different from fertigation with other methods. Disadvantages of chemical application through surface systems versus other irrigation methods are:

1. Chemicals cannot be applied to the leaves. Therefore, most insecticides and fungicides cannot be applied with surface irrigation.
2. Surface irrigation systems cannot apply very small dosages of water; usually a minimum of 2-4" of water must be applied simultaneously with the fertilizer. Other irrigation methods may be able to apply much less water.

Advantages of surface irrigation fertigation compared to other methods of chemical application are:

1. Chemicals do not touch the leaves. Therefore there is less probability of salt burn on the leaves.
2. There is no wind drift of chemicals.
3. There is no evaporation of spray droplets. Therefore, there is less volatilization of ammonia and ammonium compounds with surface irrigation.
4. The chemicals are spread out over a greater surface area than with drip irrigation, so the soil does not see a drastic chemical change as with drip irrigation. Many soils under drip irrigation will seal up when ammonium fertilizers are applied.

The practice of fertigation makes sense for a variety of reasons:

1. Chemicals can be applied when needed.
2. Fertilizer can be applied through the irrigation system when a crop is tall and a tractor would not be able to move through the field.
3. The equipment required for effective fertigation is very inexpensive; it costs much more to run a tractor through the field. The tractor also causes additional soil compaction with each pass.

4. By using the proper injection equipment, no additional power beyond the normal irrigation pumping is required to injecter fertilizers.
5. There is less groundwater pollution of nitrates with proper fertigation as compared to broadcasting of fertilizers.
6. Fertilizers can be applied very uniformily with a properly designed and managed irrigation system if the proper fertilizers are used.

Common Errors with Surface Irrigation Fertigation

Fertigation through surface irrigation must be conducted while observing the basic rules of proper surface irrigation management. Three common errors are listed below.

Error NO.1. Stop fertilizer injection when the water on furrows has just advanced to the end ofthe field. This practice will result in large quantities of fertilizers at the head (top) end of the field, and insufficient fertilizer in the middle and lower end. The fertilizer distribution uniformity across the field can only be as good as the water distribution uniformity.

Error NO.2. Inject at a variable rate. With surface irrigation and other moving systems such as center pivots or linear moves, the fertilizer must be injected at a constant rate (constant ppm fertilizer into the incoming irrigation water). With other irrigation methods such as drip or undertree sprinkler, the injection rate may vary with time during a set. The partial exception is with furrow (not border strips) irrigation, in which it may be desirable to withhold fertilizer application on non-cracking soils until the water has advanced 1/3 the distance down the furrow. After that, fertigation can begin at a constant rate. The practice of withholding the fertilizer at the beginning of a set will prevent excess fertilizer application to the top end of the furrow, even though excess water will infiltrate.

Error NO.3. Inattention to water chemistry. As explained later, various nitrogen fertilizers may **volatilize** (go into the air as a gas) or cause calcium scale to precipitate, depending upon the type of nitrogen fertilizer and water chemistry. Some simple checks coupled with chemical amendments can avoid these problems.

Nitrogen Fertilizers

The four different forms of nitrogen found in fertilizers for fertigation are:

1. Ammonia (NH3) with a neutral electrical charge.
2. Ammonium (NH4+) which is a cation and is attracted to the soil particles which are negatively charged due to the soil's cation exchange capacity
3. Nitrate (N03-) which is an anion and is repelled by the soil clay particles.
4. Urea [(H2N)2CO] which is organic and has a neutral electrical charge.

The common nitrogen fertilizers used for fertigation are (adapted from Burt and Ruehr, 1991):

1. Anhydrous ammonia constitutes the major nitrogen fertilizer which is used in the U.S. Anhydrous ammonia is pure ammonia gas (NH3) under high pressure. No

pump is needed to inject it into gated pipe or canal water; the pressure of the solution in the anhydrous ammonia tank is sufficient for injection.

The reaction of anhydrous ammonia with water is as follows:

NH3	+	H20	<==>	NH4	+	OH-
Ammonia		Water		Ammonium		Hydroxide

This reaction is instantaneous and converts the ammonia form (which is otherwise a gas and can escape from the water or soil) into the ammonium form which will be held on the soil cation exchange capacity which is composed of the soil clay minerals and organic matter colloids.

As can be seen above, the reaction of ammonia with water causes the pH of the water to rise as a result of the formation of the hydroxide (OH-) ion. The pH of the water can approach 9 or higher depending upon the concentration of nitrogen added to the water. The high pH of the water has two consequences:

a. The reaction of ammonia in water is reversible. Thus, the more ammonia that is added to the water, the higher the pH and the greater the probability that the reaction will reverse and cause the ammonia to volatilize out of the water and be blown away with the wind. Not only is the nitrogen then lost as a fertilizer; the ammonia gas can be toxic to plants. This volatilization problem can be prevented or controlled in one of two ways:
   1. Do not apply large rates of ammonia fertilizer at one time. Maintain the application rate at less than 10 pounds of nitrogen per acre-inch of irrigation water.
   2. Acidify the water.

b. The reaction of ammonia in water causes the formation of hydroxide which can then react with the calcium and magnesium in the water to form lime. An example of this reaction is given below:

Ca^{2+}	+	20H-	+	C02	-->	CaC03	+	H_2O
Calcium		Hydroxide		Carbon Dioxide		Calcium Carbonate		Water

The calcium carbonate and magnesium carbonates are generally referred to as lime. This lime forms a very fine colloidal material which precipitates or settles out of the irrigation water. Pipelines and siphon tubes can become encrusted with the precipitated salts. Small cracks in buried concrete pipelines have actually been sealed by injecting ammonia and allowing this precipitation process to occur. More frequently it is a problem because of increased friction and heavier aluminum pipe weights due to the heavy scaling.

The calcium and magnesium precipitation problem is commonly avoided by injecting the precipitation inhibitor "Calgon" (Sodium hexametaphosphate or Rose Stone $[NaPO_3]_{13}$) into the irrigation water upstream of the fertilizer injection point. Table 1 gives recommended inhibitor dosages. Precipitation can also be prevented by acidifying the water.

Table 1. *Rates of Anhydrous Ammonia and Inhibitor Injection per 1000 GPM of water (Ag. Ammonia Institute,* 1961)

Irrigation ppm	Water **Hardness**** Grains/gal	Max. Ammonia Applications lbs/hr.	Inhibitor lbs/hr
35 -120	2-7	37	0.25
120 - 170	7 - 10	37	0.38
170 - 425	10 - 25	37	0.63
425 - 850	25 - 50	28	0.75
•• Hardness is defined as the sum of the Calcium plus Magnesium concentrations in the water.			

2. Aqua Ammonia. Aqua ammonia is anhydrous ammonia which has already been dissolved in water. Because it is more dilute, it has approximately 25% the nitrogen content by weight as anhydrous. The problems with volatilization and calcium precipitation are the same as for anhydrous. The advantage of using aqua ammonia rather than anhydrous is that it is safer to work with because it is under very low pressure ratherthan in the highly pressurized state of anhydrous. It is also not cold, as anhydrous is.

   Both aqua ammonia and anhydrous ammonia should always be handled with extreme caution. If the NH3 fumes are breathed, they combine with water in the lungs to create a very basic (high pH) solution which destroys the body tissue. The anhydrous is also very cold, which can freeze and dessicate body tissue.

3. Urea. Urea fertilizers can be dissolved in water easily and do not have the volatilization problems of ammonia fertilizers.

4. Nitrate fertilizers. There are many fertilizer salts composed partly of nitrates. Nitrates are often combined with other forms of nitrogen or other nutrients for special needs. Examples are potassium nitrate and ammonium nitrate.

5. Ammonium fertilizers. As with nitrates, some fertilizers are composed partly of ammonium. Examples are urea ammonium nitrate, and ammonium sulfate.

Nitrogen Movement with Irrigation Water Over and Into the Soil

Nitrate compounds. The nitrate form of nitrogen is mobile and will move with the water as if it was another water molecule. If a uniform soil devoid of nitrogen was fertigated with a nitrate fertilizer and the irrigation stopped when the water just reached the bottom of the root zone, the complete soil profile would have a uniform water and nitrate concentration. The same ppm nitrate would be found at the surface as at the bottom of the root zone.

In the subsequent irrigation, some ofthe remaining nitrate (which had not been used by the plants in the meantime) would be picked up by the percolating irrigation water and carried down toward the bottom of the root zone. At the end of the next irrigation the nitrate concentration would be lower at the surface than at the middle or bottom of the root zone.

In other words, the concentrations of nitrate would be very similarto concentrations of other salts found in irrigation water. The difference is that nitrogen compounds can change form and concentrations depending upon microbial activity, aeration, and plant uptake.

Ammonium compounds. Ammonia fertilizers (anhydrous and aqua) convert to ammonium in the irrigation water as described earlier. Ammonia fertilizers are the most common type used in surface irrigation fertigation because of the relative low cost ofthis form of nitrogen. Ammonium ions (NH4+) are positively charged, and therefore attach or "fix" to the outside of the negatively charged soil particles. Therefore, the ammonium will not move more than a few inches into the soil, although the water may infiltrate to a depth of several feet. There is no deep percolation of ammonium ions below the root zone. However, much of the ammonium is converted to nitrates between irrigations and those nitrates will move with the water during the next irrigation.

The fixation of ammonium ions is generally a disadvantage for the following reasons:

1. With border strips, which have extensive water-soil surface/organic matter contact, a significant amount of the injected fertilizer may be pulled out of the water and "fixed" to the soil surface as the water is moving to the lower end of the field. The amount which is fixed in this fashion will depend upon the concentration of ammonium and the cation exchange capacity of the soil surface. The author has observed that the ammonium concentration in a furrowwater at a particular distance will increase from almost zero up to the injection rate with time as the upstream surface exchange sites become saturated with ammonium. Leavitt (1966) noted that with contour irrigation of rice, all ammonia injected into the water was adsorbed ("fixed") in the first 2 to 3 kilometers of travel. The border strips and rice contours have extensive water/soil contact; the problem of surface fixation at the head of a furrow is probably minimal.
2. The ammonium which is carried to a point in the field and then infiltrates, will only infiltrate a few inches. That makes it unavailable to the majority of the root system. It is sometimes recommended to apply the ammonium the irrigation before it is needed because of this problem; that allows time for conversion to nitrate and subsequent movement into the root zone during the next irrigation.
3. The ammonium, being fixed close to the soil surface, may volatilize and be lost once the soil surface dries out.

Urea compounds. Urea has a neutral electrical charge and therefore will move across and downward into the soil as does nitrate to a point. Urea will move downward in the soil with the water until it encounters the urease enzyme, which is produced by a variety of soil microorganisms. The upper 6" to 18" of soil may contain all of the urea which infiltrates during an irrigation. The urease enzyme hydrolyzes the urea as follows:

$$\underset{\text{Urea}}{[H_2N]_2CO} + \underset{\text{Water}}{H_2O} \text{ ------> } \underset{\text{Ammonium carbonate}}{[NH4]2C03}$$

Ammonium carbonate decomposes to ammonia, ammonium, and carbonate. Within a few hours after application the urea will therefore act as an ammonium ion.

Selecting the Best Nitrogen Fertilizer for Fertigating with Surface Irrigation

Plants can uptake both the nitrate (N03-) and ammonium (NH4+) forms of nitrogen. In warm, well-aerated, slightly acid to slightly alkaline soils the nitrate form will predominate. Some plants seem to respond better to ammonium than nitrate, and visa versa (Tisdale and Nelson, 1970).

The first recommendation is to use a combination nitrate/urea fertilizer. This will minimize the volatilization and surface fixation causes associated with ammonium and ammonia fertilizers. However, the urea is converted to ammonium rapidly, so that nitrogen in the ammonium form is almost immediately available to plants.

Fertigation with only nitrate will result in increased nitrate leaching. Because the nitrate moves as a water molecule, large amounts of nitrate will deep percolate if the irrigation water deep percolates. For an irrigation with a DU of 75% in which the timing coincides precisely with filling the "minimum" point of the root zone with water, 25% of the infiltrated water will deep percolate. This says that at least 25% of the applied nitrate is immediately lost to deep percolation if only nitrate is used in fertigation.

The use of a urea/nitrate mix eliminates part of this problem. Assuming a 50/50 mixture of nitrate and urea, the 25% loss is reduced to 12.5% (the urea will stay in the upper middle layers of the soil), and can be reduced even further by improving the irrigation distribution uniformity. A urea/nitrate mix provides both plant-usable forms of nitrogen (ammonium and nitrate) to the soil.

The second recommendation is to apply the fertilizer over several irrigations rather than in only one or two irrigations. This will also help to minimize the deep percolation losses, result in a lower expenditure for nitrogen fertilizer, and improve plant response to nitrogen.

leaching on Sandy Versus Clay Soils

References are often made to the high nitrogen leaching on sandy soils. With surface irrigation there are higher nitrate leaching losses on sandy soils than on clay soils, but this is only because the irrigation systems are not properly designed and/or managed. In general, people tend to have excessive run lengths and use very low furrow stream sizes, both of which contribute to very poor uniformities on sandy soils. For properly designed and managed surface irrigation systems, the nitrate leaching loss should be similar on both sandy and clay soils.

Nutrients Other Than Nitrogen

Sulfur (as sulfate, 504-) and potassium (K+) can be easily applied through fertigation as a combination salt with forms of nitrogen. The rules of uniformity of water distribution apply to them the same as for nitrogen.

Phosphorus is more difficult to apply because of the soil fixation problems similar to ammonium. Some newer formulations such as polyphosphates reduce that problem.

Fertigating and Handling Return Flow Water

Proper management of sloping furrows requires the use of return flow systems. Neglecting any surface fixation of ammonium ions during the travel down the surface of the furrow, the fertilizer concentration of the water in the return flow system should have the same concentration (expressed as ppm) as the original water.

Assume that the original source has a constant flow rate of 1000 GPM, and fertilizer is injected at a rate to give 100 ppm fertilizer in the 1000 GPM. Assume that when the return flow pump turns on, an additional 400 GPM is mixed with the original 1000 GPM, and the 1000 GPM is not changed.

In this case, the returned 400 GPM already has 100 ppm fertilizer. The fertilizer should continue to be injected at the original rate in orderto add 100 ppm fertilizer to the remaining 1000 GPM.

In summary, continue injecting fertilizer at the same rate into the original source water, but the source water must continue arriving at the original flow rate.

Safety and Backflow Prevention

Common techniques include the use of an air gap between the injector and the well, and double check valve assemblies.

The following description is from the American Society of Agricultural Engineering EP409 (ASAE, 1986). It pertains to injection systems for pipelined systems, and specifically to injection pumps.

> *For an irrigation pump driven by an intemal combustion engine, an easy and adquate interlocking mechanism is to have the chemical injection pump belted to the drive shaft or an accessory pulley* of *the engine. This will ensure that both the irrigation pump and chemical injection pump will be tumed* off *at the same time. Also,* a *mechanical, nonnally-closed check valve is required in the chemical injection line to stop backflow from the irrigation system into the chemical supply tank when the injection pump is stopped. This will prevent the chemical supply tank from overflowing. Interlocking is shown in Figure 1 for an internal combustion engine.*

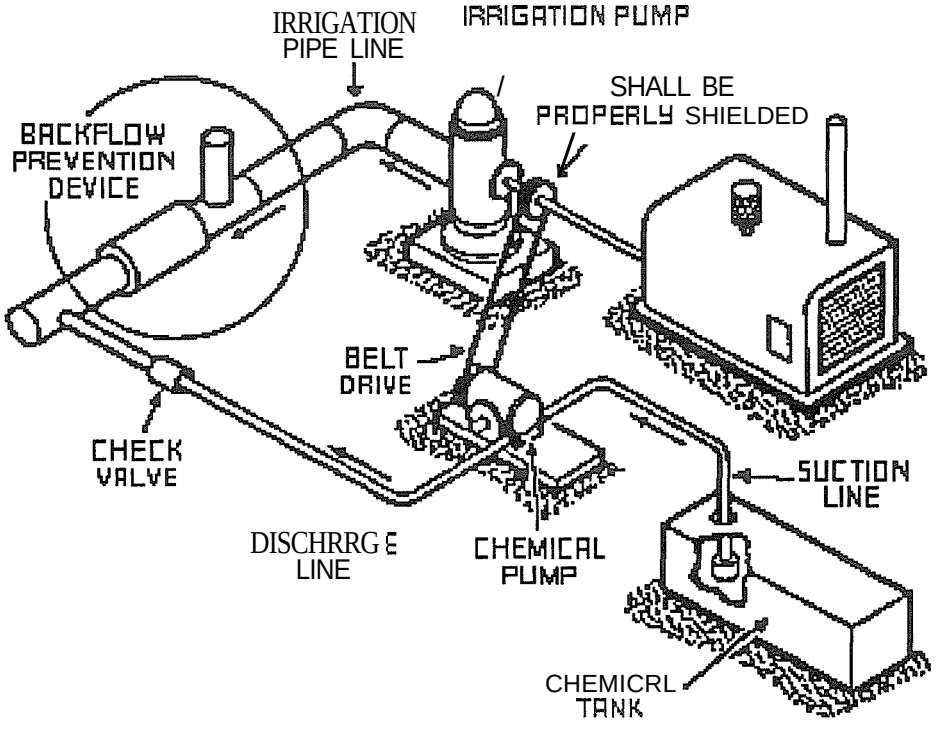


Figure 1. *Safety devices for injection of chemicals in irrigation systems using engine power units.*

For an electrical motor driven irrigation pump, a *separate electric motor* is *usually needed to power the chemical injection pump. The electric controls for the two electric motors should be interlocked* so *that the injection motor will stop when the electric motor on the irrigation pump stops.* A *check valve, vacuum breaker (atmospheric vacuum breaker, pressure vacuum breaker, or similar antisiphon device) or normally closed solenoid valve can be placed in the injection line after the injection pump.* A *solenoid valve will provide* a *positive shutoff on the chemical injection line* so *that neither the chemical nor the water can flow in either direction if the chemical injection pump stops. An electric solenoid valve must be electrically interlocked with the irrigation pump. The interlocking procedure using electric motors* is *shown in Figure 2.*

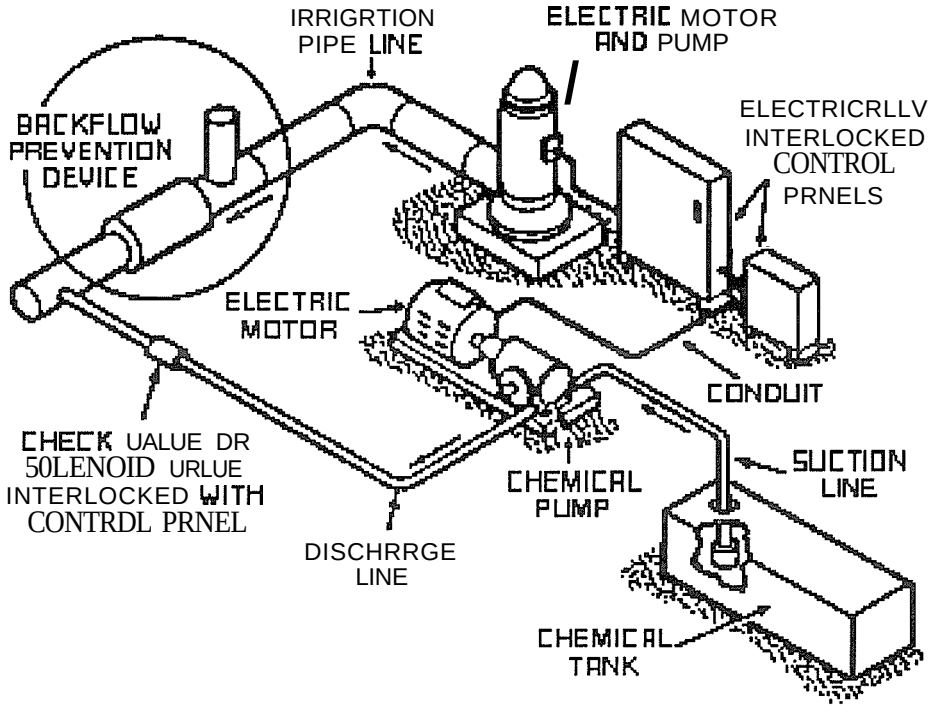


Figure 2. Safety devices for injection of chemicals in irrigation systems using electric motors.

In addition to providing devices for the injection of liquid chemicals, safety precautions should be provided to protect workers against accidental discharge or spillage of chemicals. A water source should be provided near the chemical supply tank and injection pump for washing off any chemicals that contact the skin. Protective goggles, face shields, and clothing should also be work when making chemical dilutions. concentrated chemicals should always be added to the water in preparing dilutions in a chemical supply tank. Water should not be added directly to a small volume or quantity of concentrated chemicals under any circumstances.

Bulk chemicals, in particular, and diluted solutions shold be stored in secured areas. Information, such as label directions, on chemical storage, temperature, life, handling, etc. should be posted and followed. Also, local safety regulations and codes for worker protection should be adhered to under all situations. (ASAE, 1986).

Hardware for Iniecting Fertilizers into low Pressure Irrigation Systems

There is a wide variety of hardware to be used in injecting fertilizers. The injection systems shown here require no outside power, and are simple and reliable to use. They also are capable of injecting fertilizer at a constant rate.

Anhydrous ammonia tank. Anhydrous ammonia comes in a high pressure tank, and is injected under its own pressure. Figures 3 and 4 show connections for pipelines and ditches.

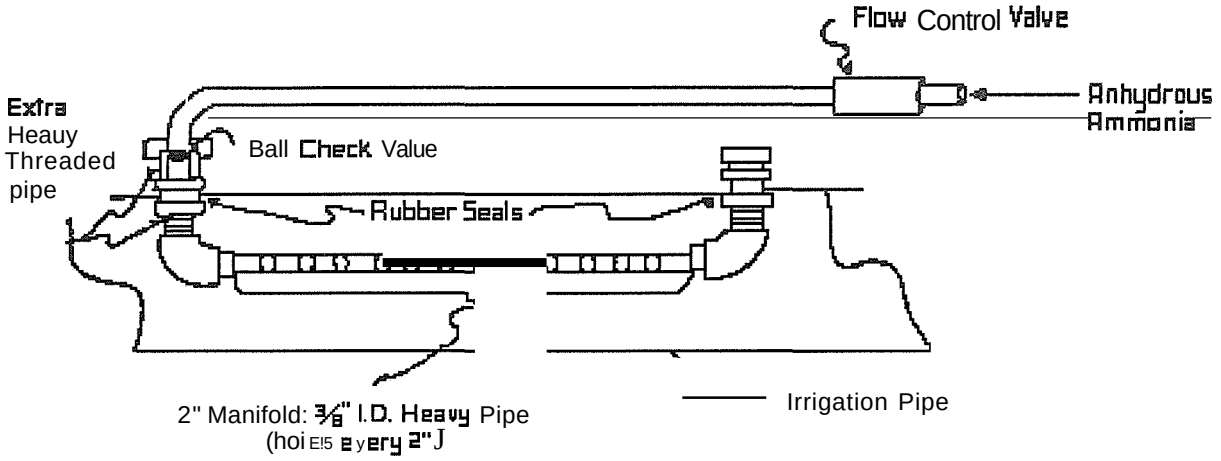


Figure 3. Gated pipe installation for anhydrous ammonia (A&L Mid West, 1985)

Install a manifold inside the last 2 feet of the first section of irrigation pipe. The manifold should be extra heavy pipe, 3/8" diameter, 2 feet long with drilled holes every 2 inches.

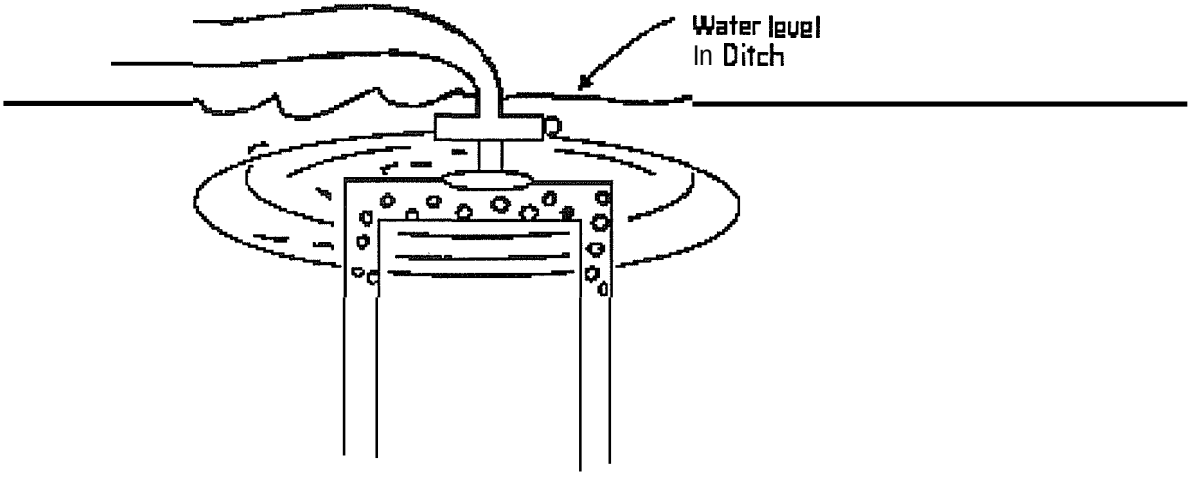


Figure 4. Open ditch installation for anhydrous ammonia (A&L Mid West, 1985)

Install a metering device in the irrigation ditch.

Low pressure gas. Liquid fertilizers are often transported to the field in non-pressure tanks mounted on trailers. It is common to seal the fertilizer container and pressurize it (at 3-10 psi) with a bottle of nitrogen gas. This low pressure will force the fertilizer out at a constant rate. The flow rate is adjusted with a flow control valve or a sized orifice.

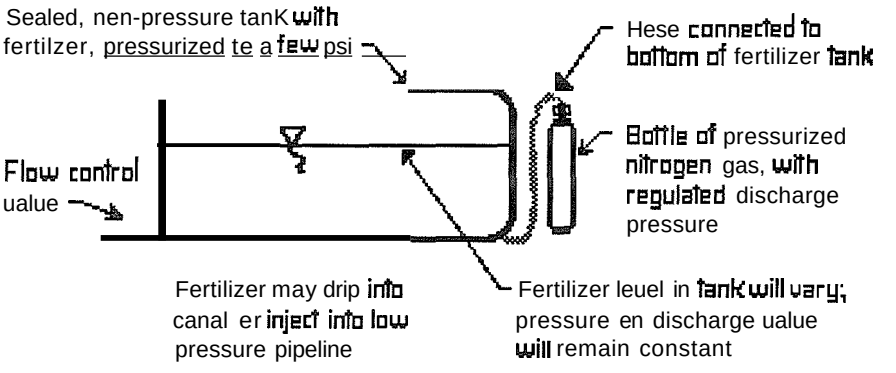


Figure 5. Use of low pressure gas to inject liquid fertilizer.

Float valves. Float valves are ideal for controlling the injection rate of liquid fertilizers from a non-pressurized container into an open ditch. The float valve, similar to the float valve on a toilet, maintains a constant pressure on the discharge control valve, even though thepressure in the fertilizer tank will vary as the fertilizer level changes. This assures a constant flow rate of fertilizer.

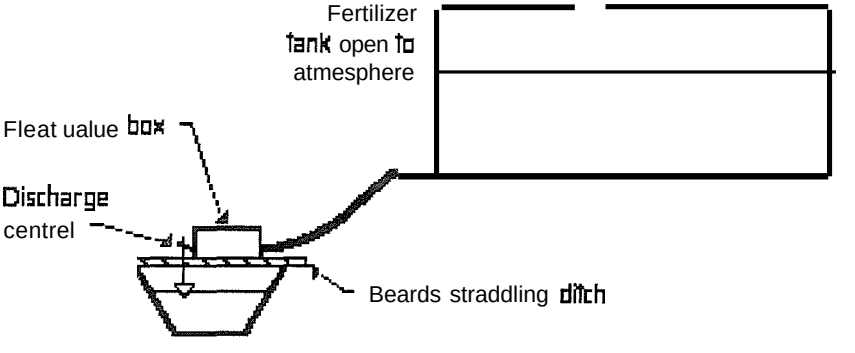


Figure 6. Float valve and unpressurized fertilizer tank.

Chicken feeder. Chicken feeders are used in injecting "Rose Stone" water conditioner and other chemicals requiring small dosages into pipe stands or canals.

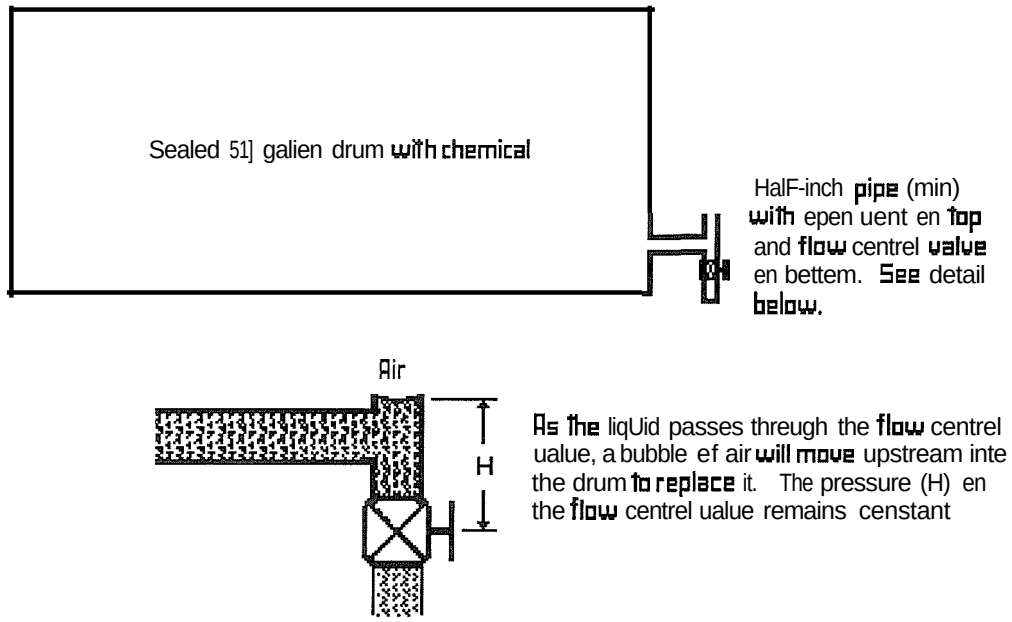


Figure 7. Chicken feeder for gravity injection.

Constant head tank. Airtight tanks can be modified for a constant head by installing a tube vertically through the container to within an inch or so of the bottom. If the tube does not extend below the liquid surface, the pressure on the valve will vary with the liquid level.

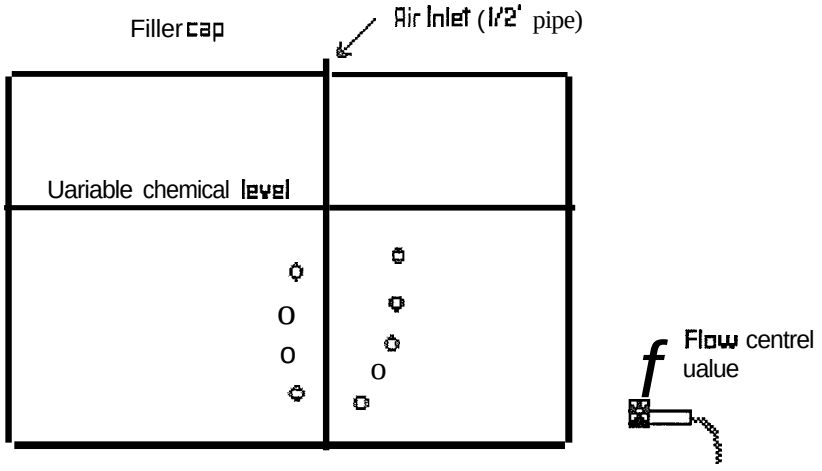


Figure 8. Constant head tank.

Venturi Injector. Venturi injectors are applicable for surface irrigation if they are capable of injecting fertilizers from an open tank (not a differential pressure tank). In other words, they must be capable of exerting a vacuum on a fertilizer supply line and injecting this fertilizer under pressure into the irrigation line.

Venturi designs require some of the irrigation water to be forced through the venturi itself. Typical flows of irrigation water through the venturi will be 5-40 GPM. A pressure drop of about 20% is needed to create a suction on most commercial units. Good designs are

capable of injecting up to 1 GPM offertilizer. Venturi operation is explained by the Bernoulli equation, which states that if the velocity is increased, the pressure will drop. Venturis have a neck of a small diameter to accomplish this velocity increase. The injection port is at the venturi neck.

Venturi designs require a pressure differential, and should be used if that pressure differential is always built into the system design. A typical installation would use the pressure drop across a pump control valve to force some water through the venturi.

Differential pressure tanks. Differential pressure tanks are not recommended for surface irrigation, because the fertilizer injection rate varies with time. These pressure tanks are plumbed into high pressure lines. Some irrigation water is forced through the tanks, and this picks up fertilizer in the tank. As time progresses, the fertilizer concentration in the tank decreases, as does the injection rate of fertilizer.

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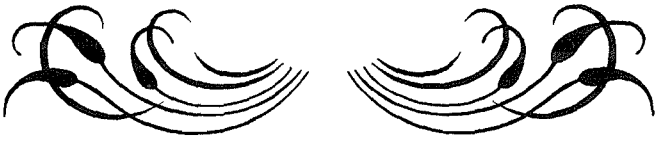
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APPENDIX



CAPACITY OF UNLINED HEAD DITCHES

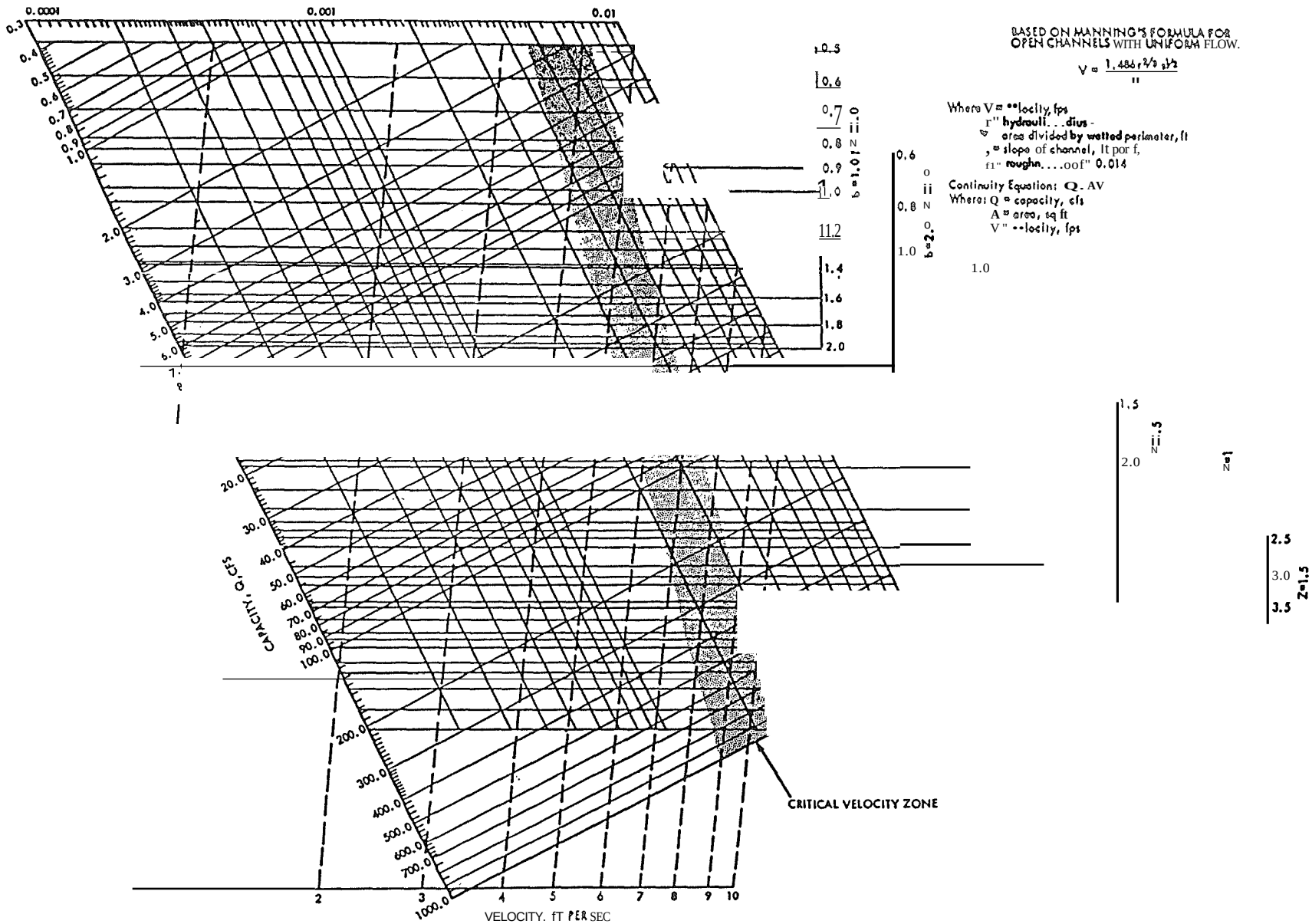
Characteristics	Fallin ft per 100	Depth of flow in feet													
		0.4		0.6		0.8		1.0		1.2		1.4		1.6	
		V	Q	V	Q	V	Q	V	Q	V	Q	V	Q	V	Q
Bottom width-1.5' Side slopes—1½:I	0.1	.51	.43	.67	.96	.79	1.71	.91	2.73	1.04	4.11	1.15	5.80	1.26	7.86
	0.2	.72	.60	.96	1.38	1.13	2.44	1.32	3.96	1.48	5.86	1.64	8.27	1.79	11.17
	0.3	.89	.75	1.t8	1.70	1.39	3.00	1.63	4.89	1.82	7.2t	2.02	10.18	2.20	13.71
	0.4	1.03	.87	1.37	1.97	1.61	3.48	1.88	5.64	2.11	8.36	2.33	11.74	2.54	15.85
	0.5	1.16	.97	1.53	2.20	1.80	3.89	2.10	6.30	2.35	9.31	2.61	13.15	2.84	17.72
Bottom width-1.5' Side slopes—1:1	0.1	.51	.39	.67	.84	.79	1.45	.90	2.25	1.03	3.34	1.13	4.59	1.22	6.05
	0.2	.72	.55	.96	1.21	1.13	*2.08*	1.30	3.25	1.46	4.73	1.60	6.50	1.74	8.63
	0.3	.89	.68	1.18	1.49	1.39	2.56	1.56	3.90	1.80	5.83	1.97	8.00	2.14	10.61
	0.4	1.03	.78	1.37	1.73	1.61	2.96	1.86	4.65	*2.08*	6.74	2.28	9.26	2.47	12.25
	0.5	1.16	.88	1.53	1.93	1.80	3.31	2.01	5.03	2.29	7.55	2.55	10.35	2.77	13.74
Bottom width-2' Side slopes—1½:1	0.1	.52	.54	.70	1.22	.83	2.12	.97	3.40	1.09	4.97	1.20	6.89	1.32	9.29
	0.2	.75	.78	1.00	1.74	1.20	3.07	1.38	4.83	1.55	7.07	1.71	9.82	1.87	13.16
	0.3	.92	.96	1.23	2.14	1.46	3.74	1.69	5.92	1.91	11.71	2.10	12.05	2.30	16.19
	0.4	1.07	1.11	1.42	2.47	1.69	4.33	1.96	6.86	2.20	10.03	2.42	13.89	2.65	18.66
	0.5	1.19	1.24	1.59	2.77	1.89	4.84	2.19	7.67	2.47	11.26	2.72	15.61	2.97	20.91
Bottom width-2' Side slopes—1:I	0.1	.52	.50	.70	1.09	.85	1.90	.97	2.91	1.08	4.15	1.19	5.66	1.28	7.37
	0.2	.75	.72	tOO	1.56	1.20	2.69	1.38	4.14	1.53	5.88	1.69	8.04	1.82	10.48
	0.3	.92	.88	1.23	1.92	1.46	3.27	1.69	5.07	1.89	7.26	*2.08*	9.90	2.24	12.90
	0.4	1.07	1.03	1.42	2.22	1.69	3.79	1.96	5.88 .	2.18	8.37	2.40	11.42	2.58	14.86
	0.5	1.19	1.14	1.59	2.48	1.89	4.23	2.19	6.57	2.44	9.37	2.69	12.80	2.89	16.65
Bottom width—3' Side slopes—1½:1	O.t	.84	.78	.74	1.73	.87	2.92	1.04	4.68	1.17	6.74	1.29	9.21	1.41	12.18
	0.2	.81	1.17	1.07	2.50	1.30	4.37	1.48	6.66	1.67	9.62	1.84	13.14	2.00	17.28
	0.3	.99	1.43	1.32	3.09	1.60	5.38	1.82	8.19	2.06	11.87	2.26	16.14	2.46	21.25
	0.4	1.14	1.64	1.53	3.58	1.86	6.25	2.13	9.59	2.38	13.71	2.61	18.64	2.84	24.54
	0.5	1.29	1.86	1.71	4.00	2.07	6.96	2.38	10.71	2.66	15.32	2.92	20.85	3.17	27.39
Bottom width—3' Side slopes—1:1	0.1	.55	.75	.75	1.62	.90	2.74	1.05	4.20	1.17	5.90	1.29	7.95	1.40	10.30
	0.2	.81	1.10	1.07	2.31	1.30	3.95	1.48	5.92	1.67	8.42	1.84	11.33	1.99	14.65
	0.3	.99	1.35	1.32	2.85	1.60	4.86	1.82	7.28	2.06	10.38	2.26	13.92	2.44	17.96
	0.4	1.14	1.55	1.53	3.30	1.86	5.65	2.13	8.52	2.38	12.00	2.61	16.08	2.81	20.68
	0.5	1.29	1.75	1.71	3.69	2.0.	6.29	2.38	9.52	2.66	13.41	2.92	17.99	3.15	23.18



	Depth of flow in feet									
	.5		.75		1.0		1.25		1.5	
0.1	.45	.20	.59	.51	.72	1.01	.85	1.76	.97	**2.77**
0.2	.64	.29	.83	.71	1.04	1.46	1.21	2.50	1.38	3.95
O.3	.78	.35	1.02	.88	1.2.	1.78	1.49	3.08	1.69	4.83
0.4	.91	.41	1.19	1.02	1.48	2.0.	1.72	3.56	1.95	5.58
0.5	1.02	.46	1.33	1.14	1.65	2.31	1.92	3.97	2.19	6.26

Courtesy USDA.

Source: MarT, J.C. 1980. Furrow Irrigation



BASED ON MANNING'S FORMULA FOR OPEN CHANNELS WITH UNIFORM FLOW.

$$V = \frac{1.486 r^{2/3} s^{1/2}}{n}$$

Where V = velocity, fps
r = hydraulic radius - area divided by wetted perimeter, ft
s = slope of channel, ft por f,
n = roughness coef = 0.014

Continuity Equation: Q = AV
Where: Q = capacity, cfs
A = area, sq ft
V = velocity, fps

CAPACITY CHART FOR CONCRETE·LINED DITCHES

LEGEND: Z = side slope
b = canal bottom width, fl.

USEFUL DATA

CONVERSION REFERENCE					
METERS		INCHES	KILOMETE		MILES
1	=	39.370	1	=	0.6214
0.0254	=	1	1.6093	=	1
		FEET	LITER		CUBIC FEET
1	=	3.281	1	=	0.0353
0.3048	=	1	28.316	=	1
		YARDS			CUBIC INCHES
1	=	1.0936	1	=	61.024
0.9144	=	1	0.0164	=	1
SQUARE METERS		SQUARE FEET			QUART
1	=	10.764	1	=	1.0567
0.093	=	1	0.9464	=	1
CUBIC METER		GALLON			GALLON
1	=	264.17	1	=	0.2642
0.00376	=	1	3.785	=	1
		CUBIC FEET	HECTARE		ACRES
1	=	35.314	1	=	2.741
0.0283	=	1	0.40469	=	1
CUBIC METER PER SECOND		CUBIC FEET PER SECOND	KILOGRAM		OUNCES
1	=	35.314	1	=	53.274
0.0283	=	1	0.0284	=	1
CUBIC CENTIMETE		CUBIC INCHES			POUND
1	=	0.061	1	=	2.2046
16.387	=	1	0.4536	=	1
METER PER SECOND		FEET PER SECOND	KILOGRAM PER CUBIC METER		POUNDS PER CUBIC FOOT
1	=	3.281	1	=	0.0624
0.3048	=	1	16.018	=	1
PASCAL		POUNDS PER SQUARE INCH			
1	=	0.000145			
6894.8	=	1			
		POUNDS PER SQUARE FOOT			
1	=	0.0209			
47.88	=	1			

METRIC CONVERSION FOR GATE OPENINGS			
INCHES TO METERS		METERS TO INCHES	
6	0.152	0.15	5.9
8	0.203	0.20	7.9
10	0.254	0.25	9.8
12	0.305	0.30	11.8
14	0.356	0.35	13.8
15	0.381	0.37	14.8
16	0.406	0.40	15.7
18	0.457	0.45	17.7
20	0.508	0.50	19.6
24	0.610	0.60	23.6
30	0.762	0.75	29.5
36	0.914	0.90	35.4
42	1.07	1.00	39.4
48	1.22	1.25	49.2
54	1.37	1.50	59.1
60	1.52	1.80	70.9
72	1.83	2.00	78.7
84	2.13	2.50	98.4
96	2.44	2.75	108.0
108	2.74	3.00	118.0
120	3.05	3.25	128.0
126	3.20	3.50	137.8
130	3.30	4.00	157.5
132	3.35		
144	3.66		
156	3.96		

USEFUL DATA

STANDARD FLANGE DIMENSIONS

SIZE IN INCHES	FLANGE DIAMETER IN INCHES		BOLTCmCLE IN INCHES		NUMBER OF BOLTS		DIAMETER OF BOLTS	
	25LB.	125 LB.	25 LB.	125 LB.	25 LB.	125 LB.	25 LB.	125 LB.
I		41/4		31/8		4		1/2
I 1/4		45/8		31/2		4		1/2
I 1/2		5		37/8		4		1/2
2		6		43/4		4		5/8
21/2		7		51/2		4		5/8
3		71/2		6		4		5/8
31/2		81/2		7		8		5/8
4	9	9	71/2	71/2	8	8	5/8	5/8
5	10	10	81/2	81/2	8	8	5/8	3/4
6	11	11	91/2	91/2	8	8	5/8	3/4
8	13 1/2	13 1/2	11 3/4	113/4	8	8	5/8	3/4
10	16	16	141/4	141/4	12	12	5/8	7/8
12	19	19	17	17	12	12	5/8	7/8
14	21	21	183/4	183/4	12	12	3/4	I
16	231/2	231/2	211/4	21 1/4	16	16	3/4	I
18	25	25	223/4	223/4	16	16	3/4	I 1/8
20	271/2	271/2	25	25	20	20	3/4	I 1/8
24	32	32	291/2	291/2	20	20	3/4	11/4
30	383/4	383/4	36	36	28	28	7/8	I 1/4
36	46	46	423/4	423/4	32	32	7/8	I 1/2
42	53	53	491/2	491/2	36	36	I	I 1/2
48	591/2	591/2	56	56	44	44	I	I 1/2
54	66114	661/4	623/4	623/4	44	44	I	13/4
60	73	73	691/4	691/4	52	52	I 1/8	13/4
66	80	80	76	76	52	52	I 1/8	13/4
72	861/2	86112	821/2	821/2	60	60	I 1/8	13/4
84	993/4	993/4	951/2	951/2	64	64	I 1/4	2
96	113 1/4	113 1/4	1081/2	1081/2	68	68	I 1/4	21/4

STANDARD ORIENTATION OF HOLES IS STRADDLE VERTICAL CENTER LINE

AREA OF CIRCULAR OPENINGS FOR GATES

Diameter in Inches	Area in Sq. Inches
1	.79
1 1/2	1.77
2	3.14
3	7.07
31/2	9.62
4	12.6
5	19.6
6	28.3
61/2	33.2
8	50.3
10	78.5
12	113
14	154
15	177
16	201
18	254
20	314
21	346
24	452
27	573
30	707
36	1018
42	1385
48	1809
54	2289
60	2826
66	3419
72	4069
78	4776
84	5539
90	6359
96	7235
102	8167
108	9156
120	11,304

USEFUL DATA

CLEAR OPENING AREAS FOR RECTANGULAR GATES					
GATE SIZE IN INCHES	AREA OF CLEAR OPENING IN SQUARE FEET	GATE SIZE IN INCHES	AREA OF CLEAR OPENING IN SQUARE FEET	GATE SIZE IN INCHES	AREA OF CLEAR OPENING IN SQUARE FEET
6X6	0.25	36X36	9.00	72X42	21.0
8X8	0.45	36X42	10.5	72X48	24.0
lOX 10	0.70	36X48	12.0	72X60	30.0
12 X 12	1.00	36X60	15.0	72X72	36.0
12X 18	1.50	36X72	18.0	72X84	42.0
12X20	1.67	36X84	21.0	72X96	48.0
12X24	2.00	42X24	7.00	78X78	42.3
14X 14	1.36	42X30	8.75	78X96	52.0
15 X 15	1.56	42X36	10.5	84X36	21.0
16X 16	1.78	42X42	12.3	84X60	35.0
16X24	2.67	42X48	14.0	84X72	42.0
18 X 12	1.50	42X60	17.5	84X84	49.0
18 X 18	2.25	42X72	21.0	84X96	56.0
18X24	3.00	48X30	10.0	84 X 108	63.0
18X36	4.50	48X36	12.0	96X48	32.0
20X20	2.78	48X48	16.0	96X60	40.0
20X24	3.33	48X54	18.0	96X72	48.0
20X36	5.00	48X60	20.0	96X 84	56.0
24X 12	2.00	48X72	24.0	96X96	64.0
24 X 18	3.00	48X 84	28.0	96X 120	80.0
24X24	4.00	48X96	32.0	96X 144	96.0
24X30	5.00	48 X 108	36.0	108 X 48	36.0
24X36	6.00	54X48	18.0	108 X60	45.0
24X48	8.00	54X54	203	108 X 108	81.0
24X60	10.00	54X72	27.0	120X42	35.0
30X24	5.00	60X36	15.0	120X 72	60.0
30X30	6.25	60X48	20.0	120 X 84	70.0
30X36	7.50	60X60	25.0	120X96	80.0
30X48	10.00	60X72	30.0	120 X 120	100.0
30X60	12.50	60X84	35.0	120 X 192	160.0
36 X 18	4.50	60X96	40.0	144 X 144	144.0
36X24	6.00	66X66	30.3		
36X30	7.50	66X72	33.0		

USEFUL DATA

DISCHARGE COMPARISONS
CUBIC FEET PER SECOND TO GALLONS PER MINUTE TO GALLONS PER DAY

Cubic Ft. per Second	Gallons per Minute	Gallons per Day	CubicFt. per Second	Gallons per Minute	Gallons per Day	Cubic Ft. per Second	Gallons per Minute	Gallons per Day
0.1	44.88	64,627	4.3	1,930.2	2,778,969	8.5	3,816.0	5,493,311
0.2	89.76	129,254	4.4	1,975.1	2,843,596	8.6	3,860.9	5,557,938
0.3	134.64	193,882	4.5	2,020.0	2,908,223	8.7	3,905.8	5,622,565
0.4	179.52	258,509	4.6	2,064.9	2,972,850	8.8	3,950.7	5,687,192
0.5	224.40	323,136	4.7	2,109.8	3,037,477	8.9	3,995.6	5,751,819
0.6	269.28	387,763	4.8	2,154.7	3,102,104	9.0	4,040.5	5,816,446
0.7	314.16	452,390	4.9	2,199.6	3,166,731	9.1	4,085.4	5,881,073
0.8	359.04	517,017	5.0	2,244.5	3,231,358	9.2	4,130.3	5,945,700
0.9	403.92	581,645	5.1	2,289.4	3,295,985	9.3	4,175.2	6,010,327
1.0	448.80	646,272	5.2	2,334.3	3,360,612	9.4	4,220.1	6,074,954
1.1	493.68	710,899	5.3	2,379.2	3,425,239	9.5	4,265.0	6,139,581
1.2	538.56	775,526	5.4	2,424.1	3,489,866	9.6	4,309.9	6,204,208
1.3	583.44	840,153	5.5	2,469.0	3,554,493	9.7	4,354.8	6,268,835
1.4	628.32	904,780	5.6	2,513.9	3,619,120	9.8	4,399.7	6,333,462
1.5	673.20	969,407	5.7	2,558.8	3,683,747	9.9	4,444.6	6,398,089
1.6	718.08	1,034,034	5.8	2,603.7	3,748,374	10.0	4,488.0	6,462,720
1.7	762.96	1,098,662	5.9	2,648.6	3,813,001	12.0	5,386.0	7,755,260
1.8	807.84	1,163,290	6.0	2,693.5	3,877,628	14.0	6,283.0	9,047,810
1.9	852.72	1,227,917	6.1	2,738.4	3,942,255	16.0	7,181.0	10,340,340
2.0	897.60	1,292,544	6.2	2,783.3	4,006,882	18.0	8,078.0	11,632,900
2.1	942.48	1,357,171	6.3	2,828.2	4,071,509	20.0	8,976.0	12,925,440
2.2	987.36	1,421,798	6.4	2,873.1	4,136,136	25.0	11,220.0	16,156,800
2.3	1,032.2	1,486,425	6.5	2,918.0	4,200,764	30.0	13,464.0	19,388,140
2.4	1,077.1	1,551,052	6.6	2,962.9	4,265,392	35.0	15,708.0	22,619,520
2.5	1,122.0	1,615,679	6.7	3,007.8	4,330,020	40.0	17,952.0	25,850,890
2.6	1,166.9	1,680,306	6.8	3,052.7	4,394,648	45.0	20,196.0	29,082,230
2.7	1,221.8	1,744,933	6.9	3,097.6	4,459,276	50.0	22,440.0	32,313,580
2.8	1,256.7	1,809,560	7.0	3,142.5	4,523,904	60.0	26,928.0	38,776,280
2.9	1,301.6	1,874,187	7.1	3,187.4	4,588,532	70.0	31,416.0	45,239,040
3.0	1,346.5	1,938,814	7.2	3,232.3	4,653,160	80.0	35,904.0	51,701,760
3.1	1,391.4	2,003,441	7.3	3,277.2	4,717,787	90.0	40,392.0	58,164,460
3.2	1,436.3	2,068,068	7.4	3,322.1	4,782,414	100.0	44,880.0	64,627,200
3.3	1,481.2	2,132,696	7.5	3,367.0	4,847,041	110.0	49,368.0	71,089,900
3.4	1,526.1	2,197,324	7.6	3,411.9	4,911,668	120.0	53,856.0	77,552,600
3.5	1,571.0	2,261,952	7.7	3,456.8	4,976,295	130.0	58,344.0	84,015,300
3.6	1,615.9	2,326,580	7.8	3,501.7	5,040,922	140.0	62,832.0	90,478,000
3.7	1,660.8	2,391,207	7.9	3,546.6	5,105,549	150.0	67,320.0	96,940,700
3.8	1,705.7	2,455,834	8.0	3,591.5	5,170,176	160.0	71,808.0	103,403,400
3.9	1,750.6	2,520,461	8.1	3,636.4	5,234,803	170.0	76,296.0	109,866,200
4.0	1,795.5	2,585,088	8.2	3,681.3	5,299,430	180.0	80,784.0	116,329,000
4.1	1,840.4	2,649,715	8.3	3,726.2	5,364,057	190.0	85,272.0	122,791,700
4.2	1,885.3	2,714,342	8.4	3,771.1	5,428,584	200.0	89,760.0	129,254,400

The calculations above are based on:the following:
1 cu. ft. of water = **7.48** gallons
1 cu. fl. per sec. = (7.48X60) = 448.80 8al. per minute
1 cu. fl. per sec. = (7.48X60X24) = 646,272 gal. per day

USEFUL DATA

CONVERSION OF FEET OF HEAD TO POUNDS PER SQUARE INCH

HEAD IN FEET	PRESSURE IN LBS.PER SQUARE INCH	HEAD IN FEET	PRESSURE IN LBS.PER SQUARE INCH	HEAD IN FEET	PRESSURE IN LBS.PER SQUARE INCH	HEAD IN FEET	PRESSURE IN LBS.PER SQUARE INCH
1	0.433	41	17.77	81	35.10	210	91.01
2	0.867	42	18.20	82	35.53	220	95.34
3	1.300	43	18.64	83	35.97	230	99.68
4	1.733	44	19.07	84	36.40	240	104.01
5	2.167	45	19.50	85	36.83	250	108.35
6	2.600	46	19.94	86	37.27	260	112.68
7	3.034	47	20.37	87	37.70	270	117.01
8	3.47	48	20.80	88	38.14	280	121.35
9	3.90	49	21.24	89	38.57	290	125.68
10	4.33	50	21.67	90	39.00	300	130.01
11	4.77	51	22.10	91	39.44	320	138.68
12	5.20	52	22.54	92	39.87	340	147.35
13	5.63	53	22.97	93	40.30	360	156.02
14	6.07	54	23.40	94	40.74	380	164.69
15	6.50	55	23.84	95	41.17	400	173.36
16	6.93	56	24.27	96	41.60	420	182.02
17	7.37	57	24.70	97	42.04	440	190.69
18	7.80	58	25.14	98	42.47	460	199.36
19	8.23	59	25.57	99	42.90	480	208.03
20	8.67	60	26.00	100	43.34	500	216.70
21	9.10	61	26.43	105	45.50	550	238.37
22	9.53	62	26.87	110	47.67	600	260.04
23	9.97	63	27.30	115	49.84	650	281.71
24	10.40	64	27.73	120	52.00	700	303.38
25	10.83	65	28.17	125	54.17	750	325.05
26	11.27	66	28.60	130	56.34	800	346.72
27	11.70	67	29.03	135	58.50	850	368.39
28	12.13	68	29.47	140	60.67	900	390.51
29	12.57	69	29.90	145	62.84	950	411.73
30	13.00	70	30.33	150	65.00	1,000	433.40
31	13.43	71	30.77	155	67.17	2,000	866.79
32	13.87	72	31.20	160	69.34	3,000	1,300.19
33	14.30	73	31.63	165	71.51	4,000	1,733.58
34	14.74	74	32.07	170	73.67	5,000	2,166.98
35	15.17	75	32.50	175	75.84		
36	15.60	76	32.93	180	78.01		
37	16.04	77	33.37	185	80.17		
38	16.47	78	33.80	190	82.34		
39	19.90	79	34.23	195	84.51		
40	17.34	80	34.67	200	86.68		

CONVERSION OF POUNDS PER SQ. INCH TO FEET OF HEAD							
Lbs. per Sq. in.	Head in Feet	Lbs. per Sq. in.	Head in Feet	Lbs. per Sq. in.	Head in Feet	Lbs. per Sq. in.	Head in Feet
1	2.309	16	36.95	31	71.59	46	106.2
2	4.619	17	39.26	32	73.90	47	108.5
3	6.928	18	41.57	33	76.21	48	110.8
4	9.238	19	43.88	34	78.52	49	113.2
5	11.55	20	46.19	35	80.83	50	115.5
6	13.86	21	48.50	36	83.14	60	138.6
7	16.17	22	50.81	37	85.45	70	161.7
8	18.48	23	53.12	38	87.76	75	173.2
9	20.78	24	55.43	39	90.07	80	184.8
10	23.09	25	57.74	40	92.38	90	207.8
11	25.40	26	60.05	41	94.69	100	230.9
12	27.71	27	62.36	42	97.00	125	288.6
13	30.02	28	64.66	43	99.31	150	346.4
14	32.33	29	66.97	44	101.6	200	461.9
15	34.64	30	69.28	45	103.9	500	1155.

CONVERSION TABLE FOR UNITS OF FLOW				
Cubic ft.lsec.	Gallons/ min.	Million gallons/day	Acre.Inches/24 hrs.	Acre-feetf24 hrs.
1.0	448.8	0.646	23.80	1.984
0.00223	1.0	0.00144	0.053	0.00442
1.547	694.4	1.0	36.84	3.07
0.020	8.98	0.0129	0.476	0.0397
0.025	11.22	0.0162	0.595	0.0496
0.042	18.86	0.0271	1.0	0.0833
0.504	226.3	0.3259	12.0	1.0

The following approximate formulas may be conveniently used to compute the depth of water applied to a field:
Cu. ft.fsec. x hrs. divided by Acres =acre-inches per acre, or average depth in inches.
Gal./min. x hrs. divided by 450 x acres =acre-inches per acre, or average depth in inches.

• Equivalent values are given in the same horizontal line.

LIST OF EQUIVALENTS

The following equivalents may be used for converting from one unit to another and in computing volumes from flow units:

1. One cubic foot per second
   = 448.83 (approximately 450) gallons per minute
   = 1 acre-inch in 1 hour and 30 seconds (approximately 1 hour), or 0.992 (approximately 1) acre-inch per hour
   = 1 acre-foot in 12 hours and 6 minutes (approximately 12 hours), or 1.984 (approximately 2) acre-feet per day (24 hours)

2. One gallon per minute
   = 0.00223 (approximately 1/450) cubic foot per second
   = 1 acre-inch in 452.6 (approximately 450) hours, or 0.00221 acre-inch per hour
   = 1 acre-foot in 226.3 days, or 0.00442 acre-foot per day

3. Million gallons per day
   = 1.547 cubic feet per second
   = 694.4 gallons per minute
   = 1 acre-inch in 39.1 minutes, or 36.84 acre-inches per day
   = 1 acre-foot in 7 hours and 49 minutes, or 3.07 acre-feet per day

4. One acre-inch
   = 3,630 cubic feet
   = 27,154 gallons
   = 1/12 acre-foot

5. One acre-foot
   = 43,560 cubic feet
   = 325,851 gallons
   = 12 acre-inches

6. One cubic foot
   = 1,728 cubic inches
   = 7.48 (approximately 7.5) gallons weighs approximately 62.4 pounds (62.5 for ordinary calculations)

7. One gallon
   = 231 cubic inches
   = 0.13368 cubic foot weighs approximately 8.33 pounds

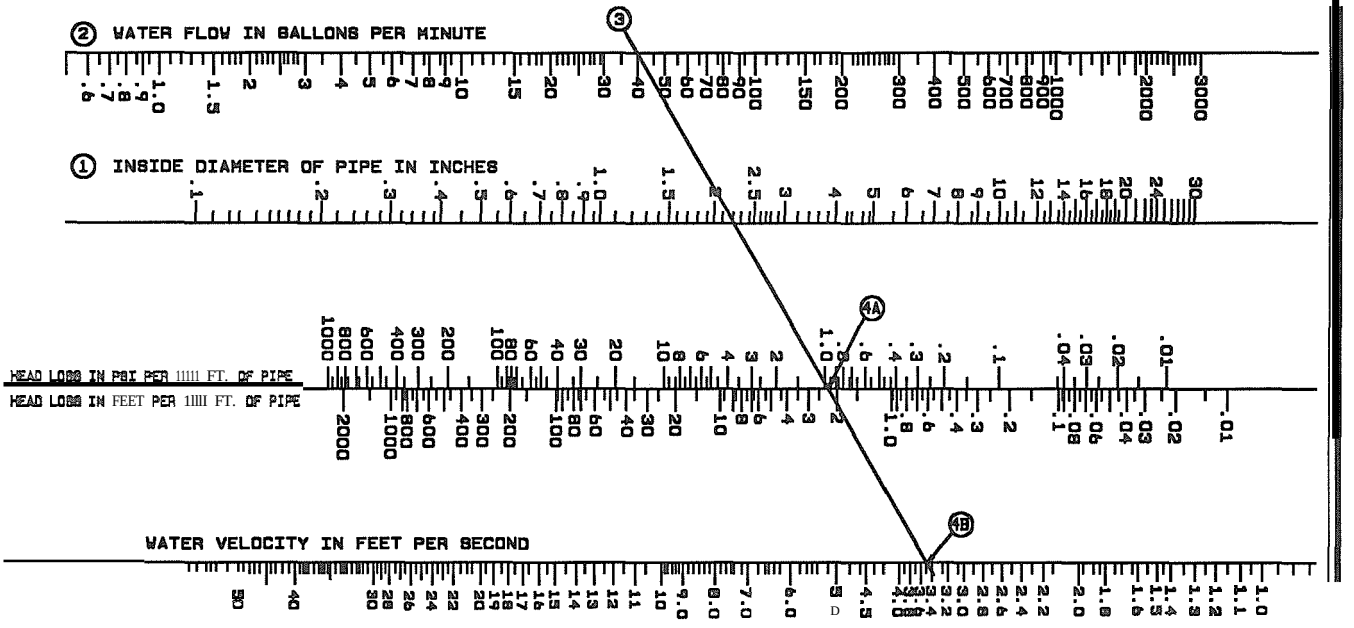
8. One acre
   = 43,560 square feet
   = 4,840 square yards
   = 160 square rods is equivalent to a square apprOXimately 209 feet on each side

9. One rod
   = 16.5 feet
   = 5.5 yards

10. One southern California miner's inch
    = 0.02 (1/50) cubic foot per second
    = 8.98 (approximately 9) gallons per minute
    = 0.80 (4/5) California statute miner's inch
    = 1 acre-inch in 50 hours and 25 minutes, or 0.0198 (approximately 1/50 acre-inch per hour)
    = 1 acre-foot in 605 hours (approximately 25 days), or 0.397 (approximately 1/25) acre-foot per day

11. One California statute miner's inch
    = 0.025 (1/40) cubic foot per second
    = 11.22 (approximately 11¼) gallons per minute
    = 1.25 southern California miner's inches
    = 1 acre-inch in 40 hours and 20 minutes, or 0.0248 (approximately 1/40) acre-inch per hour
    = 1 acre-foot in 484 hours (approximately 20 days), or 0.0496 acre-foot (approximately 1/20) per day

FRICTION IOSS CHARACTERISTICS OF WATER FLOW THROUGH PLASTIC PIPE



HOW TO USE THIS NOMOGRAPH:

(1) Select the desired pipe size (inside diamter, see price list).
(2) Determine the amount of water to flow through the pipe.
(3) Place a straight-edge on these two points.
(4) The points at which the straight-edge intersects the head-loss line and the velocity line give these two values under the given conditions.

EXAMPLES:
2 PV-160 (to O. 2.193) ①
40 Gal. Per Min. Service ②
(1) Line up these points with a straigt-edge ③
(2) Read 95 (or 2.2 ft.) from the Head-Loss line 4A
(3) Read 3.49 Ft. Per Sec. from the Velocity Line 4B

THE VALUES ON THIS GRAPH ARE BASED ON THE WILLIAMS AND HAZEN FORMULA:

$$f = \frac{.0985 \; q^{1.85}}{d^{4.866}} \qquad f = .2083 \left(\frac{100}{c}\right)^{1.85} \times \frac{q^{1.85}}{d^{4.8655}}$$

f = Friction head in feet of water per 100 feet of pipe
d = Inside diameter of pipe in inches
q = Flow in gallons per minute
c = Constant for inside roughness of the pipe. (150)

FRICTION LOSS IN CONCRETE IRRIGATION PIPE IN FEET PER 1000 FEET

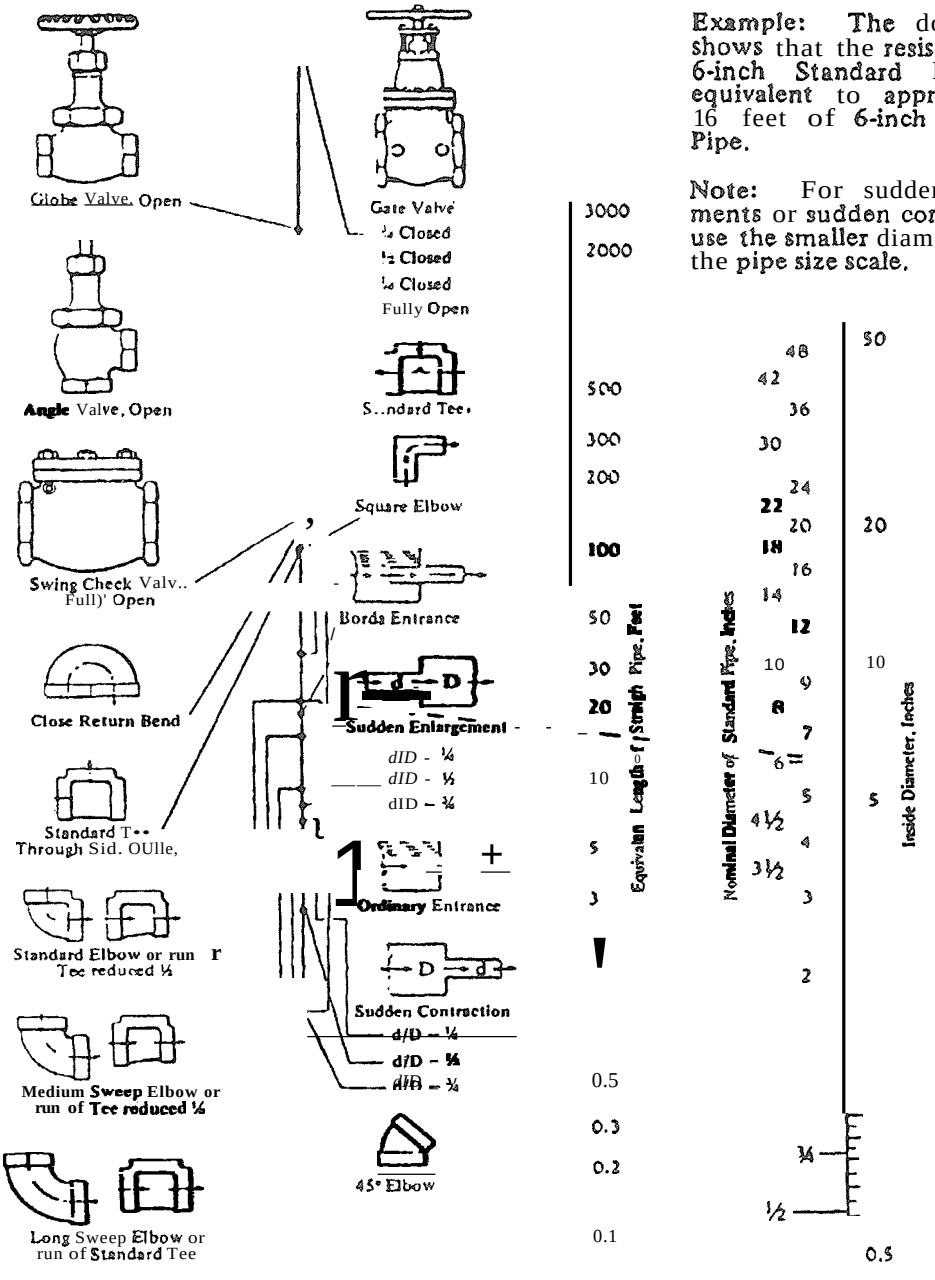
Computed from Scobey's Formula $V \sim C_s$ HO.5 $d^{1.25}$, where C_S=.310

FLOW			PIPE DIAMETER IN INCHES (el)												
Cu. Ft. per Sec.	Miner's Inches*	Gallons per Min.	6	8	10	12	14	16	18	18	20	21	24	30	36
0.1	5	45	.3	0.1											
0.2	10	90	1.1	0.2											
0.3	15	135	2.6	0.6											
0.4	20	160	4.6	1.0											
0.5	25	225	7.2	1.6											
0.6	30	270	10.4	2.3	0.7	0.3									
0.7	35	315	14.0	3.2	1.0	0.4									
0.8	40	360	18.4	4.1	1.3	0.5	0.2								
0.9	45	405	23.4	5.2	1.6	0.6	0.3								
1.0	50	449	28.8	6.4	2.0	0.8	0.4	0.2							
1.2	60	539	42.0	9.2	2.8	1.1	0.5	0.3	0.2						
1.4	70	628	56.0	12.5	3.8	1.5	0.7	0.5	0.3						
1.6	80	718	74.0	16.3	5.1	2.0	0.8	0.6	0.4						
1.8	90	808	93.0	20.7	6.5	2.4	1.1	0.8	0.5	0.3					
2.0	100	898	115.0	25.4	8.0	3.0	1.4	0.9	0.7	0.4	0.2				
2.2	110	987	140.0	30.8	9.5	3.7	1.6	1.1	0.8	0.4	0.3	0.2			
2.4	120	1.077	165.0	36.5	11.4	4.4	1.9	1.3	1.0	0.5	0.3	0.2			
2.6	130	1,167		43.0	13.3	5.1	2.3	1.6	1.1	0.6	0.4	0.3			
2.8	140	1,257		50.0	15.5	5.9	2.6	1.8	1.3	0.7	0.4	0.3			
3.0	150	1,346		57.3	17.8	6.8	3.0	2.1	1.5	0.8	0.5	0.4	0.2		
3.2	160	1,436		65.3	20.2	7.7	3.4	2.4	1.7	0.9	0.5	0.4	0.2		
3.4	170	1,526		73.5	22.8	8.8	3.9	2.7	1.9	1.0	0.6	0.5	0.2		
3.6	160	1.616		82.5	25.6	9.8	4.4	3.0	2.2	1.2	0.7	0.5	0.3		
3.8	190	1,706		92.2	28.5	10.8	4.9	3.4	2.4	1.3	0.8	0.6	0.3		
4.0	200	1.795			31.5	12.2	5.4	3.8	2.7	1.5	0.9	0.6	0.3		
4.5	225	2,020			39.7	15.3	6.8	4.7	3.4	1.9	1.1	0.8	0.4		
5.0	250	2,244			49.1	18.8	8.4	5.9	4.2	2.3	1.3	1.0	0.5		
5.5	275	2,469			59.6	22.8	10.2	7.1	5.0	2.7	1.6	1.2	0.6		
6.0	300	2,693			70.7	27.1	12.1	8.4	6.0	3.2	1.9	1.4	0.7		
6.5	325	2,917			82.7	31.8	14.2	9.9	7.1	3.8	2.2	1.7	0.8		
7.0	350	3,142				36.9	16.5	11.5	8.2	4.4	2.5	2.0	1.0	0.3	
7.5	375	3,366				42.4	18.9	13.2	9.4	5.1	2.9	2.3	1.1	0.4	
8.0	400	3,591				48.2	21.5	15.0	10.7	5.8	3.3	2.6	1.3	0.4	
8.5	425	3.815				54.4	24.3	16.9	12.1	6.5	3.7	2.9	1.4	0.5	
9.0	450	4,039				61.0	27.2	19.0	13.5	7.3	4.2	3.3	1.5	0.5	
9.5	475	4,264				68.0	30.3	21.1	15.1	8.2	4.7	3.6	1.8	0.6	0.2
10.0	500	4.488				75.3	33.6	23.4	16.7	9.0	5.2	4.0	2.0	0.6	0.2
11.0	550	4,937					40.7	28.3	20.2	10.8	6.3	4.8	2.4	0.7	0.3
12.0	600	5,386					48.4	33.7	24.0	12.9	7.4	5.8	2.9	0.9	0.3
13.0	650	5.835					56.8	39.6	28.2	15.2	8.8	6.8	3.4	1.0	0.4
14.0	700	6,284					65.9	45.9	32.7	17.7	10.2	7.8	3.9	1.2	0.5
15.0	750	6,732						52.6	37.5	20.3	11.7	9.0	4.5	1.4	0.5
16.0	800	7.181						60.0	42.7	23.0	13.2	10.2	5.1	1.6	0.6
17.0	850	7,630						67.7	48.2	26.0	14.9	11.6	5.8	1.8	0.7
18.0	500	8.079							54.0	29.3	16.8	13.0	6.5	2.0	0.8
20.0	1,000	8,977							66.7	35.9	20.7	16.0	7.9	2.5	0.9
22.0	1.100	9,874							80.7	43.3	25.1	19.3	9.6	3.0	1.1
24.0	1,200	10,772								51.9	28.8	23.0	11.4	3.6	1.4
26.0	1.300	11,689								60.8	34.9	27.0	13.4	4.1	1.6
28.0	1,400	12,567								70.1	40.4	31.4	15.6	4.8	1.9
30.0	1,500	13,468									46.2	36.0	17.9	5.6	2.1
32.0	1.600	14.363									53.3	41.0	20.4	6.3	2.4
34.0	1,700	15,250									59.9	46.2	22.9	7.1	2.7
36.0	1.800	16,158									67.1	51.8	25.7	8.0	3.1
38.0	1,900	17.056										57.7	28.6	8.9	3.4
40.0	2,000	17,953										64.0	31.7	9.9	3.8
42.0	2,100	18,851										70.5	35.0	10.9	4.2
46.0	2,300	20,646											41.9	13.0	5.0
50.0	2,500	22,442											49.5	15.4	5.9

Customary. or Southern California miner's Inch -1/50 cu. ft. per sec. = 9 gallons per minute.

RESISTANCE OF VALVES AND FITTINGS TO FLOW OF FLUIDS

Note: Head loss through check valves varies with types manufactured Consult manufacturer for correct values.



Example: The dotted line shows that the resistance of a 6-inch Standard Elbow is equivalent to approximately 16 feet of 6-inch Standard Pipe.

Note: For sudden enlargements or sudden contractions, use the smaller diameter. d, on the pipe size scale.

Note: Data in the above chart is satisfactory for most applications; for more detailed information contact the fittings manufacturer.

Source: Crane Engineering Data Catalog
Flow of Fluids Through Valves, Fittings, and Pipe
Technical Paper No. 410