SPRINKLE AND TRICKLE IRRIGATION

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Foreword

The design text, *Sprinkle and Trickle Irrigation,* opens up a new and clear window through which to view the physics, economics, design, and management of pressurized irrigation systems. A broad array of system types and applications have been covered in detail to provide for complete understanding of systems design. Topics include soil-water-plant relations, general planning concepts, hydraulics, economics, sizing, operation, maintenance, and special uses. Pressurized irrigation system types covered include hand-line, wheel-line, solid-set, traveler, center-pivot, linear-moving and big-gun-sprinkler systems, pumping systems, and a broad array of trickle system components.

The work in this text culminates earlier major works by Jack Keller on the W. R. Ames Company *Irrigation Handbook* (1967), Rain Bird Sprinkler Manufacturing Corp.'s *Trickle Irrigation Design* (1975), and the USDA-Soil Conservation Service's National Engineering Handbook, Section 15: Irrigation—Chapter 11: *Sprinkle Irrigation* (1983) and Chapter 15: *Trickle Irrigation* (1984). These earlier works form the foundation upon which the majority of currently used design texts are based. The years of design and troubleshooting experiences of the authors and wide ranges of environments and design applications in which they have worked have resulted in the substance and robustness of this text in stated relationships and procedures.

The text takes a very good and direct approach in combining environmental demands (evapotranspiration, leaching and irrigation water requirements) with moisture and infiltration characteristics of the soil and various hydraulic, economic, and physical constraints and requirements of pressurized systems. The text gives good thought to day-to-day operations of the various types of systems and to their maintenance requirements.

Because *Sprinkle and Trickle Irrigation* covers only sprinkle and trickle design, it provides much more depth and detail than do general irrigation texts. The chapters on economic mainline design and center-pivot design are especially thorough. The chapters on trickle design address the wide array of problems which have led to the premature demises of a large number of installed trickle irrigation systems worldwide.

Sprinkle and Trickle Irrigation is full of information. Procedures are direct and toward the target. There is no ''filler'' in this book. As an engineer and designer, Jack Keller has had the rare ability to peer inside many processes which have been opaque to many designers, and to decompose these processes to add form and structure. Relationships developed combine theory, rationale, and empiricism in a balanced and thoughtful manner. The approaches to design of a number of sprinkle system types and components are often new, refreshing, and illuminating. One example of this is the text's decoupling of water application efficiency into the components of spray evaporation and wide drift, system leakage, and distribution uniformity in the soil. Distribution uniformity is further statistically decomposed into the two factors of spatial uniformity and percent adequacy of water application. This "decoupling" allows both the designer and student to identify and better understand the individual mechanisms governing various processes, and to better modify designs and estimates for different field conditions. Sections of the book which describe underlying theory are simply stated and easy to understand. They often have the feel of roomside "chats."

The text is liberally infused with equations, tables, and graphics. The graphics help to show the "form" of physics and design procedures. The equations and tables provide for proper accuracy in detailing of designs and lend themselves to computer applications.

The design concepts presented in this text are more than sufficient for education and training, yet the approaches are direct for ease of application and to ensure a high probability of a successful design. The authors present concepts and theories of irrigation and pressurized design, but fortunately, do not stop there. They push on through with empirical (when necessary) and thoughtful relationships, equations, and tables which provide the "numbers" necessary to complete well-engineered irrigation designs. Procedures presented are a healthy combination of common sense and sound theory.

The text includes numerous sections concerning operation and management of irrigation systems *after* installation, which are important parts of designs that are often neglected by designers. Sufficient information is presented in *Sprinkle and Trickle Irrigation* to allow the designer to present "operations guides" to irrigators/owners which detail proper lateral placement, scheduling, and operation.

I have used drafts of this text in undergraduate and graduate level courses in sprinkle and trickle irrigation at Utah State University for the past five years. The text has also served as a valuable reference and source of information for private work in systems design, troubleshooting, and in court cases. I wholeheartedly endorse the use of *Sprinkle and Trickle Irrigation* both in design and in education.

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Preface

The need for more careful husbandry of our planet's agricultural resources is quickening. This results from two clashing realities: the growing numbers of fellow humans with their increasing expectations for a more bountiful life, and the limitations of the planet's natural resources and ability to absorb environmental abuse. Irrigated agriculture is necessary to meet the food quantity and quality expectations of the expanding population. Improving its performance is essential in order to live within the earth's soil and water resource limitations.

Basic engineering and agronomic sciences are dominant in the curricula of practitioners in the specialized area of agro-irrigation system design and management. The academic concentration is on analysis, not on design, which requires synthesis of the analytical steps. Obtaining the specialized skills of engineering practice is left to post-graduate experience. To be effective this requires having internship opportunities. In their absence well-articulated strategies codified into bodies of knowledge or texts for selecting and designing various types of irrigation systems are needed. Neither sufficient internships or texts are available so novices are left to invent their own design procedures.

This book addresses the need for more comprehensive texts on the design process. It covers the selection and design of both sprinkle and trickle irrigation systems taking advantage of the many aspects they have in common. Together they represent the broad class of "pressurized" systems which potentially are very efficient because they discharge the irrigation water close to the plants where it will be consumed.

The major purpose of this book is to convey a system of thought patterns leading to the efficient selection, design, management, and operation of pressurized irrigation systems for agriculture. This requires being able to select and develop assemblages of individual components that will fit together to make a workable and optimized irrigation system for a given site.

To achieve this goal, the chapters are presented in the sequence used to design systems. Furthermore, most of the analytical material is presented in a brief form with limited attention given to the derivation of the standard formulas presented. This has been done in an effort to focus on the synthesis of the entire design process rather than concentrating on the analysis of the individual steps along the way.

Sequential sample calculations that involve the steps in system selection and design are extensively used. They form part of the verbal text, as an explanation is given of the logic of each step of the process. As the text progresses these calculations become more comprehensive and are linked to form complete design packages for the various types of pressurized systems.

First and foremost, this is a book for designers not researchers or students mainly interested in analytical detail. Its objective is to present and convey powerful design methodologies in a systematic way. Therefore, the emphasis is on approaches for conceptualizing, applying, and synthesizing the basic underlying system design principles and concepts. In this regard the text is unique when compared to other manuscripts on agro-irrigation. Typically a codification of important and relevant facts and analytical details about irrigation systems is presented without a systematic design approach.

For the convenience of both teachers and students there is a brief review of basic soil-plant-water relationships and other pertinent material. In addition, some very useful tables have been organized to summarize important design information. These are included to increase the value of the book as a reference for irrigation practitioners. They also provide a convenient source of data for developing practical student exercises.

This book is written from the perspective of authors with balanced professional careers which have earned them recognition in consultation and practice as well as in education and research. Most of the material represents the original thinking and formulations of the authors. They have presented parts of it in other texts such as the Sprinkle and Trickle Irrigation chapters of the *SCS National Engineering Handbook*, but not as elegantly as herein. The genesis of the text has involved a lifetime effort of grappling with how best to improve irrigated agricultural development worldwide.

Acknowledgments

This book has its roots deep in the history of sprinkle and trickle irrigation—I have been privileged to study and work with many early pioneers in this field. The intellectual debt I owe them cannot be fully measured or satisfactorily acknowledged. If individual credits were given to all who in one way or another contributed to this book the list would indeed be lengthy. However, sources of specific published information and direct contributions are referenced or footnoted to give credit where due.

The genesis of the book began in the early 1950s at Colorado State University where I was a graduate student in Irrigation Engineering under the tutelage of Dean F. Peterson. It was there that I was first exposed to the holistic nature of on-farm irrigation engineering. Later I worked for W. R. Ames Company in irrigation system design, sales, development, and management. There I learned Allan W. McCulloch's pragmatic approach to irrigation system design and management while I assisted him in writing the *Ames Irrigation Handbook*.

In 1960 Dean Peterson (Dean of Engineering) enticed me to leave Ames Company and join the Irrigation Engineering Staff at Utah State University. Dean directed my Ph.D. research and has continued to be my mentor throughout my career. At Utah State I had the opportunity to work closely with Alvin A. Bishop, Jerry E. Christiansen, Howard B. Peterson, and Gary Z. Watters. I am most grateful to them for they not only shared their invaluable knowledge related to irrigation but inspired me to be a dedicated teacher, researcher, and author.

This led to my co-authoring two books which were published in 1974. These were *Trickle Irrigation Design* with David Karmeli, whose insights were indispensable in formulating this first design text in the field; and *Irrigation System Evaluation: A Guide to Management* with John L. Merriam, whose understanding of and dedication to improving farm irrigation system management profoundly affected my own views. Important direct contributors to this text include Kenneth H. Solomon who has provided counsel and been a critical reviewer of this and my earlier texts, Richard G. Allen who has provided valuable suggestions for strengthening it, and my son Andrew A. Keller who has assisted in developing several of the concepts, tables, and figures.

Ron D. Bliesner and I have worked together for over a decade as consulting irrigation engineers focusing on setting standards for and designing pressurized irrigation systems. Ron has independently applied the design concepts and procedures presented herein in actual engineering practice. This has provided a means for realistically evaluating their usability and practicality as well as a source of fresh new ideas for improving the text.

For their assistance with developing, assembling, and reviewing the manuscript I am indebted to my former dedicated graduate students Robert E. Walker, Charles M. Burt, David W. Miller, Safa N. Hamad, Monohar M. Sawant, and Roy Steiner. For the vast amount of painstaking effort in typing the manuscript I am indebted to JoAnn Biery, Linda Fields, DeAnn Draper, and Sheridyn Stokes. The majority of the drawings represent the able computerized drafting ability of Daniel K. Fisher.

As authors know well, to bring a book from an image to reality requires a sustaining force. For me this has been the combination of the inspiration provided by Dean Peterson, my wife Sally's devotion and encouragement, and my desire to enhance irrigation performance to improve our world.

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SPRINKLE AND TRICKLE IRRIGATION

I

INTRODUCTION

1

Approaching Agro-Irrigation System Design

A major purpose of the text is to convey a system of thought patterns leading to the comprehensive selection and design of various types of sprinkle and trickle irrigation systems for agriculture. To achieve this goal, the chapters are presented in the sequence used to design systems. Most of the analytical material is presented in a brief form; only limited attention is given to the derivation of the standard formulas presented. This has been done in an effort to focus on the synthesis of the entire design process, rather than concentrating on the analysis of the individual steps along the way. It is assumed that students and practitioners are already familiar with basic soil-plant-water, economic, and hydraulic principles.

The text is not entirely devoid of analytical detail, as some derivations and details are presented for two types of situations. One is for material that is not common to the standard hydraulic, agronomic, or economic perceptions. The other is for situations where a detailed analysis is needed to link the steps in the synthesis process.

Rather than concentrating on analytical detail in an abstract sense, sequential sample calculations that involve the steps in the design of typical irrigation systems are extensively used. The sample calculations themselves form part of the verbal text, as an explanation is given of the logic of each step of the design process.

Sprinkle and trickle irrigation together represent the broad class of "pressurized" irrigation methods, in which water is carried through a pipe system to a point near where it will be consumed. This is in contrast to surface irrigation methods, in which water must travel over the soil surface for rather long distances before it reaches the point where it is expected to infiltrate and be consumed. Thus, surface irrigation methods depend on critical uncertainties associated with water infiltration into the soil while being conveyed, as well as at the receiving site.

With sprinkle irrigation, water is jetted through the air to spread it from the pipe network across the soil surface. This adds a degree of uncertainty to sprinkle irrigation, as wind and other atmospheric conditions affect the application efficiency. The usual goal of sprinkling is uniform watering of an entire field.

With trickle irrigation the distribution of the water after it leaves the pipe network depends only on localized lateral movement above or on the soil surface or in the soil profile. Thus, water is conveyed through the pipe system almost directly to each plant, and only the soil immediately surrounding each plant is wetted. This leads to the potential high application efficiency associated with trickle irrigation.

The material in this book was developed over many years and represents a considerable amount of original thinking and formulation by the authors. The creativity required depended upon and was stimulated by many other irrigation professionals. However, the exact genesis of much of the material is unknown (by the authors) or without formal reference, so credit citations are limited accordingly. Citations are given where the material presented could be uniquely pinpointed and credited to individual authors and where they would be most useful for reference and additional study.

CONCEPTUALIZING THE DESIGN PROCESS

Irrigation system design is like putting a puzzle together. Paraphrasing from Keller (1980), the purpose of system design is to develop assemblages of individual components that will fit together to make a workable and optimized irrigation system for a specific site. The components include: hardware items, such as pipes and emitters; machinery, such as pumps and motors; processes, such as trenching and assembling; and ideas, such as moving sprinklers and cleaning filters. The *irrigation designer's art* is to know the systems that are appropriate for a given site and the order in which selected components fit together to make a system. This takes experience and a multidisciplinary approach, for there are numerous system variations to select from and the site includes both the natural and social resources.

The engineering design process involves selecting the size and shape of components both to make the system workable and to produce the least cost and greatest gain. Both the *art* and *engineering* of irrigation systems require a clear mental image of what is to be accomplished and how the end results (i.e., the system in operation) will appear.

There is not a blueprint for the design process, but the following suggestions should be helpful in the search for an image of the system and the engineering solution for achieving it. The final solutions are usually quite simple—after they have been developed—but developing them may be complicated.

The first order of business is to become acquainted with working irrigation systems, and it is surprising what can be learned by careful "looking" at both good and bad systems. Getting acquainted with the systems is important because of the need for images and the need to not waste time reinventing what already exists. After getting the picture in mind, one needs to study how each system works and how its components are related and fitted together. With all this in mind, it is time to start thinking about selecting, modifying, and tailoring systems for various site conditions.

Next comes the site analysis and the necessity of creative data gathering to understand what pertinent physical and social resources are at hand and decide what can and should be accomplished. It is here that visualizing what is to happen and focusing on images of the irrigation systems are important.

At this point, the art of irrigation design comes into play; this involves selecting appropriate systems that will meet acceptable goals and fit the available resources. The objective is a "good" design. With care and practice designers should be able to select the good designs from the bad ones until the suitable ones are found.

One way to explain how this last part is done is through mediation. Paraphrasing Pirsig (1974), as though he were talking about irrigation system design instead of motorcycle maintenance: when you first approach a design getting stuck is normal, but this stuckness and a blank mind precede inventiveness. Stuckness should not be avoided, because the harder you try to hold on to it, the faster your mind will naturally and freely move toward finding a good design. Just concentrate on what you want to accomplish—live with it for a while. Study it as you study a line when fishing, and before long you will get a little nibble, a system design idea, asking in a timid way if you are interested.

Synthesis and Analysis

With the image of a suitable system in mind, one can apply classic, structured, dualistic subject-object knowledge. Here is where the engineering techniques come into play, as designers endeavor to structure the system so it will work in the best way possible. Doing this involves the two basic categories of engineering problem solving, which Rubenstein (1975) has nicely defined:

> Problem solving can be viewed as a matter of appropriate selection. When we are asked to estimate the number of marbles in a jar, we go through a process of selecting an appropriate number. When asked to name an object, we must select the appropriate word. In performing an arithmetic operation as simple as 8×7, we must select from our store of numbers the appropriate one.
>
> We can distinguish two basic categories of problems. One consists of a statement of an initial state and desired goal in which the major effort is the selection of a solution process to the desired explicit goal, but for which the process as a whole (i.e., the complete pattern of the solution) is new to us, although the individual steps are not. In such a case, we verify the acceptability of the solution by trying various process for a solution and eliminating progressively (reducing to zero) the misfits between the desired goal and the results obtained from the trial processes. This kind of problem may be con-

sidered as a problem of *design or synthesis* in which a complete solution process is synthesized from smaller steps.

The second type of problem focuses more on the application of known transformation processes to achieve a goal. The goal may not be recognized as the correct solution immediately, but can be verified by the process in such a way that no misfit exists between the conditions of the problem (initial state) and the solution. This kind of problem may be considered as a problem of *analysis* in which the solution consists of a transformation or change in representation of given information so as to make transparent the obscure or hidden.

Irrigation System Design

Designing an irrigation system is a synthesis problem, and determining the friction loss in a pipeline is an analysis problem. Most of the engineering curriculum is concentrated on analysis. The program begins with basic science courses, followed by the engineering science and analysis courses. By the time the more complex problems of design or synthesis are reached, both students and professors are conditioned to think most problems can be solved with nice neat formulas that will produce "correct" answers.

To get into a better frame of mind for designing systems, try working with a tangram puzzle. Cut out seven pieces of cardboard so they have the same shape and relative dimensions as the shapes shown in Fig. 1.1. Now assemble all the pieces to make a large square or triangle, recording the thought process along the way; then write a two-page essay telling how you went about working the puzzle.

The heart of the engineering technique can best be described as a design synthesis process to achieve an objective end goal. Preliminary designs are examined via a means-ends analysis by subjecting them to a model of the environment that is most representative of the one in which the real system will operate then noting how close the system behavior fits the goal behavior. The detection of misfits leads to modifications of the components and possibly to complete changes in the system. Once an acceptable system is synthesized, alternative acceptable models are conceived. From these feasible systems, one is selected as "best" in terms of some criteria, such as least cost or maximum production.

Successful designers avoid getting set on any prescribed procedure, they explore many routes, maintain an open mind and a flexibility to abandon and return to various routes. Once the total picture of the system in operation has been formed, the most important guide in the search for a design solution is to work backward. Begin with the crop to be irrigated, and design the system back to the water supply.

Figure 1.2 is a map of the preliminary design process that leads to a set of

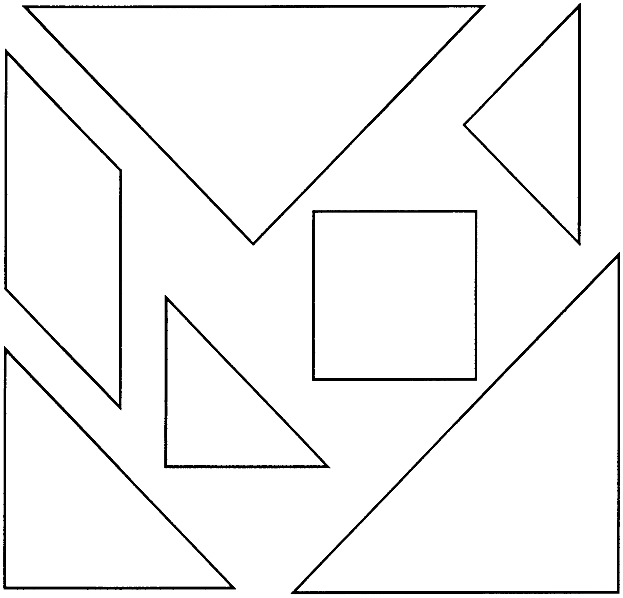


FIG. 1.1. Unassembled Tangram.

potentially suitable system types and layouts for further consideration. The next step is to select one of them at a time and complete a detailed design for it. Figure 1.3 shows a map of this process for a classic sprinkle irrigation system with hand-moved sprinkler laterals. The resulting designs for all the potentially suitable systems form the set from which the best one can be selected.

DESIGN STRATEGY

This text covers the selection, design, and some management aspects of both sprinkle and trickle irrigation systems for agricultural crop production. It takes advantage of the many features they have in common and presents the step-by-step procedures necessary to design complex as well as simple systems. It also provides basic background material related to their attributes and a rational process for selecting the best system type and configuration.

Objective

First and foremost, this is a text for designers, not researchers or students mainly interested in analytical detail. The objective is to present and convey powerful design methodologies in a systematic way. Therefore, the emphasis is on approaches for conceptualizing, applying, and synthesizing the basic underlying principles and concepts to develop system design (or operation) packages. In

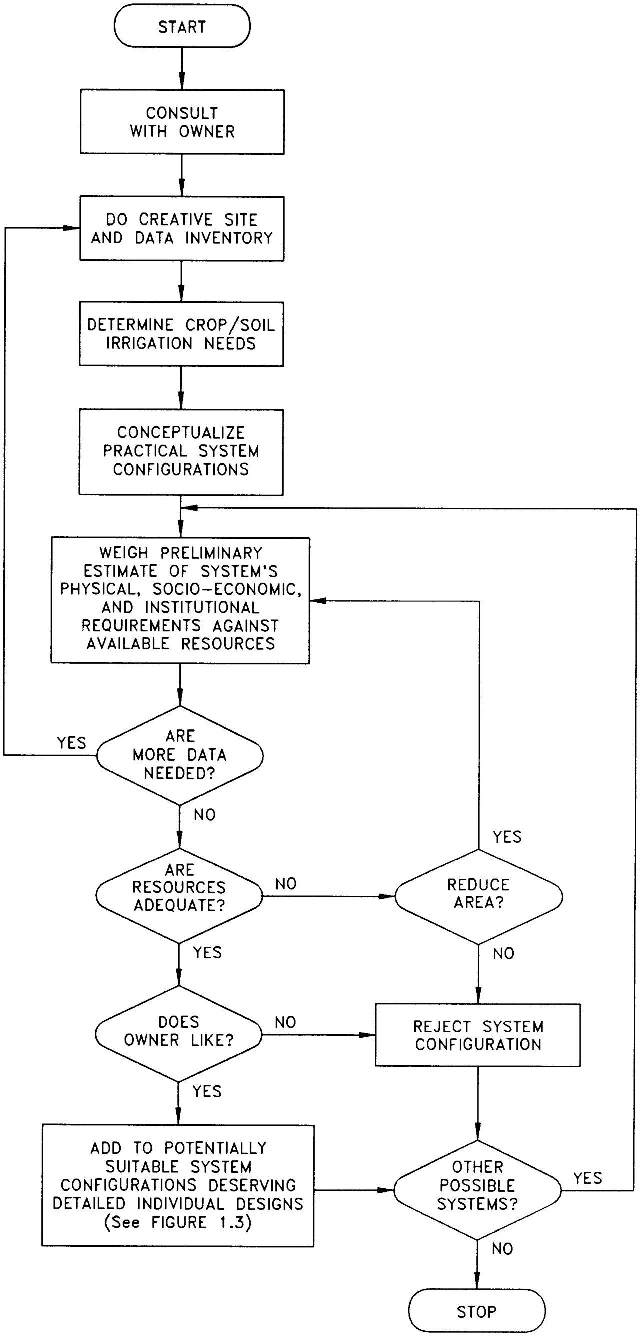


FIG. 1.2. Preliminary Design Flow Map for Selecting Potentially Suitable Farm Irrigation System Configurations (Types and Layouts).

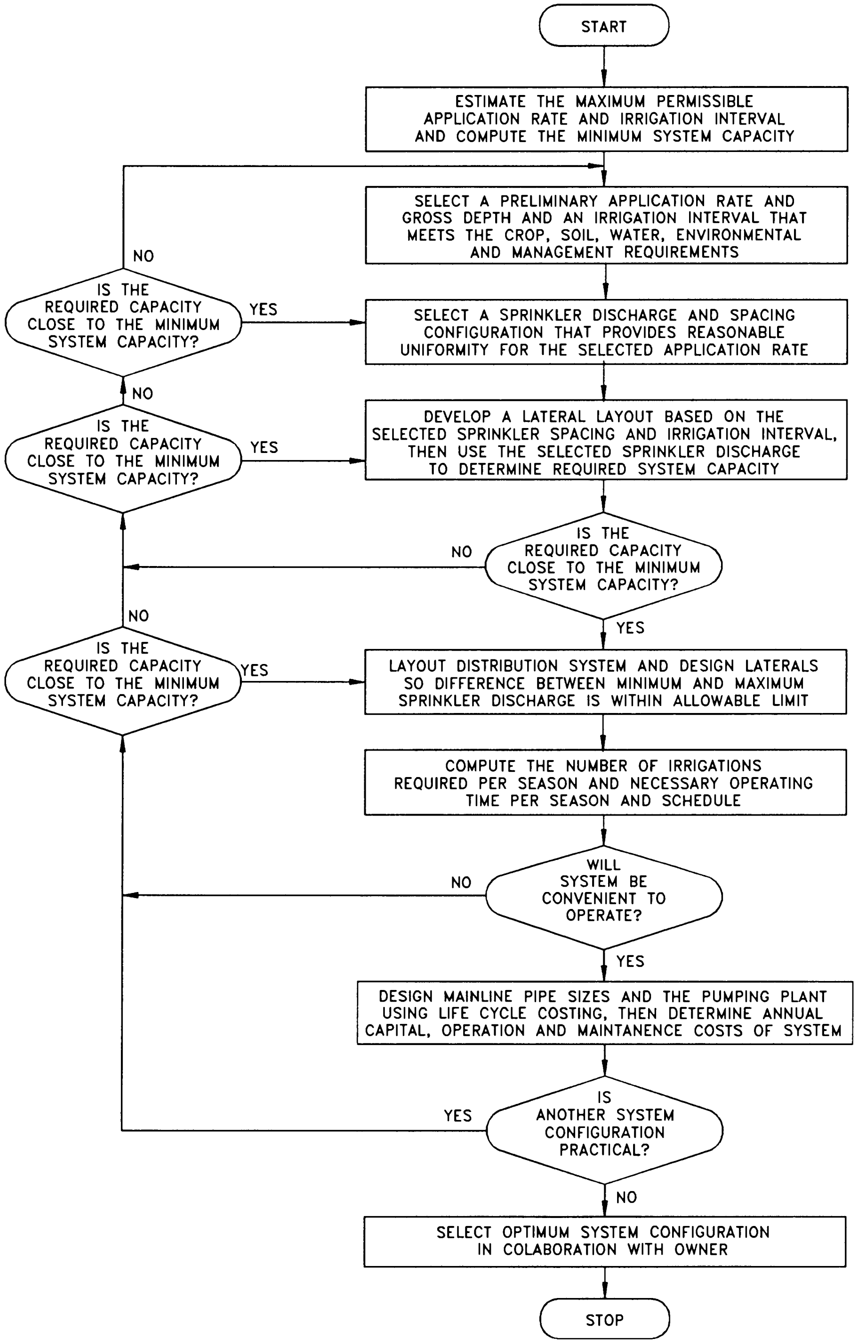


FIG. 1.3. Design Process for a Classical Sprinkle Irrigation System with Hand-moved Laterals.

this regard the text is unique compared with other manuscripts that present a codification of important and relevant facts and analytical detail, but not a systematic approach for designing irrigation systems.

It is assumed the reader already knows or has access to fundamental background material associated with irrigation system design and the physical aspects of their management; therefore, only limited space is devoted to such material. However, for the convenience of both teachers and students, there is a brief review of basic soil-plant-water relationships and other pertinent material. In addition, some very useful tables have been organized to summarize important design information. These are included to increase the value of the text as a reference for irrigation practitioners. They also provide a convenient source of data for developing practical student exercises.

Extensive use is made of comprehensive sample calculations. These are used to demonstrate the concepts and provide direct guidance for realistically applying the design procedures. The early sample calculations are, by necessity, quite limited in scope. But as the text progresses in the respective sprinkle and trickle irrigation sections, they become more and more comprehensive. For example, Chapter 14 has 10 sample calculations that are all linked to form a complete and comprehensive design package for a center-pivot system. Furthermore, these calculations are tied directly back to the information, data, and concepts presented in most of the earlier chapters.

Practicing Design

Practicing design can be thought of in two ways. One is practicing by doing ''homework'' exercises to learn how to design irrigation systems. The other is the ''engineering practice'' of designing irrigation systems for implementation. Conceptually, the two are not far apart, and the reference data, analytical material, design procedures, and sample calculations serve them both well. The sample calculations not only provide guidance for retrieving and using data to design system components, they also serve as a means for checking computational procedures that are carried out independently.

Computer-assisted design procedures are typically used in practice. However, no computer programs are given in the text, nor are they referred to or used to solve the sample calculations. It is expected that students and practitioners will use computer spreadsheet and graphics programs to sharpen and test their designing skills. These programs should then be used to test design sensitivity against the various input variables.

The analytical equations and associated sample calculations were formulated with the above thought in mind. They provide the necessary algorithms and procedures for using the spreadsheet programs. In addition, the sample calculations are designed to provide a means for checking or testing program output.

Analytical Methods

There are three ways, or methods, to analyze many of the design relationships. The most intuitive way is often to follow a stepwise set of computations. For example, to determine the pressure head loss along a pipe with uniformly spaced outlets, the analysis can be done by determining the loss between each outlet and then adding the elemental losses together. Doing this manually is both tedious and time-consuming, but with a computer it is simple and quick.

A second way to carry out the analysis is to first do it stepwise for a standard design situation and then to use regression analysis to find a direct dimensionless numerical solution that fits the results. A third way is to find a direct numerical solution that is based on the theoretical or empirical relationships involved.

Where practical, this text covers the analytical methods that are based directly on the theoretical relationships. This is done to enhance understanding of the subject. But in practice, stepwise solutions may be more straightforward and practical, assuming a computer and the skill to use it are at hand.

System Selection

Making rational decisions about which system type and configuration will best serve the goals and objectives of an irrigated agricultural development is of primary importance. No matter how well a given system is designed, if another system type or configuration would serve the goals and objectives better, it would not be the best selection.

Knowledge of the attributes and characteristics of the pressurized systems and developing optimal designs are essential for making rational system selection decisions, but not sufficient. What must also be done is to select the "best type and configuration" of irrigation system for each site situation.

To carry out this essential function, an efficient procedure for selecting the most promising pressurized irrigation system or systems for meeting development goals is given. To be efficient the procedure includes a strategy for prescreening to select a set of the most promising adaptable systems for a given site situation. This is followed by more detailed designs and economic analysis of them to arrive at the final system selection.

REFERENCES

Keller, Jack. 1980. Irrigating for rainbows. Paper presented for 61st Faculty Honor Lecture at Utah State University, Logan, Utah.

Pirsig, R. M. 1974. *Zen and the Art of Motorcycle Maintenance*. New York: Bantam Books.

Rubenstein, M. F. 1975. *Patterns of Problem Solving*. Englewood Cliffs, New Jersey: Prentice-Hall, Inc., pp. 6–7.

2

Sprinkle and Trickle Agro-Irrigation Overview

Sprinkling as an important method of agricultural irrigation had its beginnings in the early part of this century. The earliest agricultural sprinkler systems were an outgrowth of lawn sprinkling. Before 1920, agricultural sprinkling was limited to orchards, nurseries, and intensive vegetable production, In the 1930s, the cost of sprinkle systems was reduced by the development of impact sprinklers and lightweight steel pipe with quick couplers. With these improvements, sprinkle irrigation began to spread and to be used on a wide range of specialty and filed crops throughout the world.

By 1950, better sprinklers, aluminum pipe, and more efficient pumping plants further reduced the cost and increased the usefulness of sprinkle irrigation and accelerated the expansion of this method of irrigation. More recently, the self-propelled center-pivot, which gained popularity in the 1960s, has provided a means for relatively low-cost, high-frequency automatic irrigation with a minimum of labor (see Fig. 2.1). Worldwide, about 8 million hectares (ha) [20 million acres (A)] are equipped with center-pivot systems, and about 75% of them are in the United States.

Additional innovations are continually being introduced to reduce labor and increase the efficiency of sprinkling. Today sprinkling is a major means of irrigation on all types of soils, topographies, and crops.

The first experiments leading to the development of trickle irrigation were introduced in Germany in 1860, where short clay pipes with open joints were used to combine subsurface irrigation with drainage. In the 1920s, perforated pipe was introduced, and subsequent experiments centered on development of perforated pipe made of various materials and on control of flow through the perforations.

The early use of trickle irrigation was confined to greenhouses. The technique as we know it today was not practical for field crops until the introduction of low-cost plastic tubing in the early 1940s. Another significant step in the evolution of trickle irrigation took place in Israel in the later 1950s, when long-path emitters were greatly improved. In the late 1950s and early 1960s, much research and many pilot field demonstrations of trickle irrigation were undertaken. Now trickle irrigation is used on about 1.2 million hectares (3 million

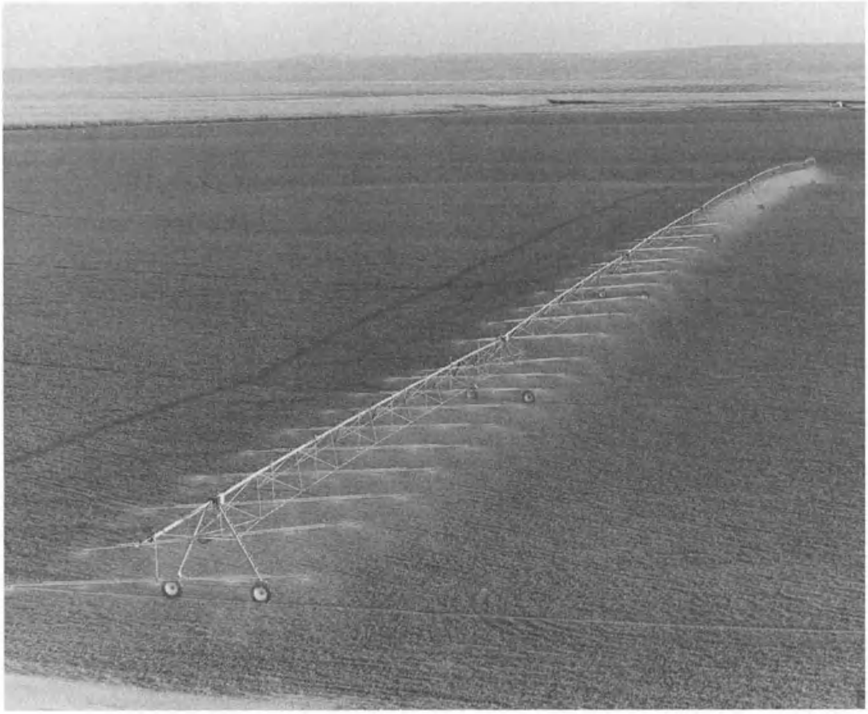


FIG. 2.1. Center Pivot Irrigation System in Operation (Source: Nelson Irrigation Corp.).

acres) in fields, orchards, and greenhouses throughout the world, with about one-third of it in the United States.

SPRINKLE

Sprinkle irrigation systems can be broadly divided into set and continuous-move systems. In set systems, the sprinklers remain at a fixed position while irrigating, whereas, in continuous-move systems, the sprinklers operate while moving in either a circular or a straight path. The set systems include systems moved between irrigations, such as hand-move (see Fig. 2.2) and wheel line laterals, hose-fed sprinkler grid, perforated pipe, orchard sprinklers, and gun sprinklers. These will be referred to as *periodic-move systems*. Set systems also include such systems as solid-set sprinklers, which will be referred to as *fixed systems*. The principal continuous-move systems are center-pivot and linear moving laterals and traveling sprinklers (see Fig. 2.3).

Figure 2.4 shows schematically the basic components and layout of a typical periodic-move sprinkle system. With carefully designed periodic-move and fixed systems, water can be applied uniformly at a rate based on the intake rate of

FIG. 2.2. Hand-move Sprinkler Lateral in Operation.

FIG. 2.3. Hose-fed Traveling-gun Sprinkler in Operation (Source: Nelson Irrigation Corp.).

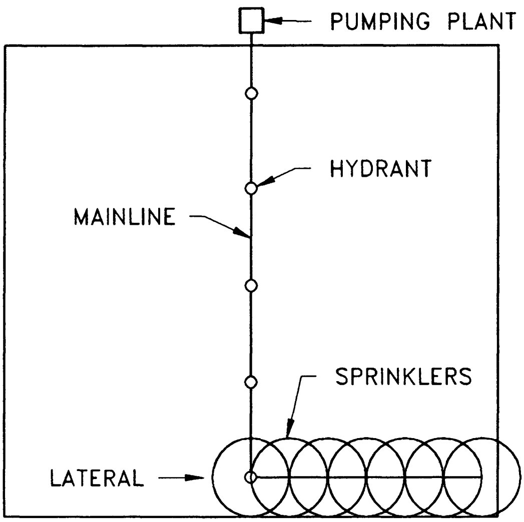


FIG. 2.4. Schematic of Basic Periodic-move Sprinkle Irrigation System.

the soil, thereby preventing runoff and consequent damage to land and to crops. Continuous-move systems can have even higher uniformity of application than periodic-move and fixed systems. Also the travel speed of these systems can be adjusted to apply light watering that reduce or eliminate runoff.

Adaptability of the Sprinkle Method

Sprinkle irrigation is suitable for most crops. It is also adaptable to nearly all irrigable soils, because sprinklers are available in a wide range of discharge capacities. With proper spacing, water may be applied at any selected rate above 3 mm/hr (0.12 in./hr) for periodic-move systems. On extremely fine-textured soils with low intake rates, particular care is required when selecting proper nozzle size, operating pressure, and sprinkler spacing to apply water uniformly at low rates.

Periodic-move systems are well-suited for irrigation in areas where the crop-soil-climate situation does not require irrigations more often than every 5 to 7 days. Where soils having low water-holding capacities and shallow-rooted crops are to be irrigated, lighter, more frequent irrigations are required. Fixed or continuously moving systems are more adaptable for such applications; however, where soil permeability is low, some of the continuously moving systems, such as the center-pivot and traveling-gun, may cause runoff problems. In addition to being adaptable to all irrigation frequencies, fixed systems can also be designed and operated for frost and freeze protection, blossom delay, and crop cooling.

The flexibility of present-day sprinkle equipment and its efficient control of water application make the method almost universally applicable. Its usefulness on most topographic conditions is subject only to limitations imposed by land-use capability and economics. It can be adapted to most climatic conditions where irrigated agriculture is practical. However, extremely high temperatures and wind velocities and low humidities present problems in some areas, especially where irrigation water contains large amounts of dissolved salts.

Salinity Problems. Such crops as grapes, citrus, and most tree crops are sensitive to relatively low concentrations of sodium and chloride. Under conditions of low humidity, such crops may absorb toxic amounts of these salts from irrigation water falling on the leaves.

Because water evaporates between rotations of the sprinklers, salts accumulate on leaves more during this alternate wetting and drying cycle than if sprayed continuously. These salts are then absorbed by the plant and may damage it. Toxicity shows as leaf burn (necrosis) on the outer leaf edge and can be confirmed by leaf analysis. Such injury sometimes occurs when either the sodium or chloride concentration in the irrigation water exceed 70 or 105 parts per million (ppm), respectively. Irrigating during periods of higher humidity, as at night, often greatly reduces or eliminates this problem.

Annual and forage crops, for the most part, are not sensitive to low levels of sodium and chloride. Recent research indicates, however, that they may be more sensitive to salts taken up through the leaves during sprinkling than from the soil when irrigated by any method.

Under extremely high evaporative conditions, more tolerant crops, such as alfalfa, have suffered some damage when sprinkled with water having on electrical conductivity, EC_w, of only 1.3 dS/m and containing 140 ppm sodium and 245 ppm chloride. In contrast, little or no damage has occurred from the use of waters having an EC_w as high as 4.0 dS/m and respective sodium and chloride concentrations of 550 and 1295 ppm when evaporation conditions were low.

Under semiarid conditions of California, vegetable crops have been sprinkle-irrigated and found fairly insensitive to foliar effects at very high salt concentrations. In general, local experience is necessary to set guidelines for a crop's salt tolerance under local conditions.

Damage can occur from the spray drifting downwind from sprinkler laterals discharging poor-quality water. Therefore, *in arid climates where saline waters are being used, for periodic-move systems, the laterals should be moved downwind for each successive set.* Thus, the salts accumulated from the drift will be washed off the leaves. Sprinklers that rotate at 1 revolution per minute (rpm) or faster are also recommended under such conditions.

If overhead sprinklers must be used, it may not be possible to grow certain sensitive crops, such as beans or grapes. A change in irrigation method to furrow, flood, basin, or trickle may be necessary. Under-tree sprinklers have been used in some orchards, but lower leaves, if wetted may still show leaf burn symptoms due to foliar absorption.

Similar soil salinity guidelines should be used for all irrigation methods except trickle. Therefore, use the standard procedures when determining allowable levels of soil salinity and leaching requirement for various crops, water qualities, and soils.

Advantages

Sprinkle irrigation is an adaptable means of supplying all types of crops with frequent and uniform applications of irrigation over a wide range of topographic and soil conditions. Sprinkle irrigation can be partly or fully automated to minimize labor costs, and systems can be designed to minimize water requirements.

Adaptability. Some of the more important objectives that can be attained by sprinkling are:

- Effective use of small, continuous streams of water, such as from springs and small tube or dug wells;
- Proper irrigation of problem soils with intermixed textures and profiles or the irrigation of shallow soils that cannot be graded without detrimental results;
- Irrigation of steep and rolling topography without producing runoff or erosion; and
- Effective, light, frequent waterings whenever needed, such as for germination of a crop like alfalfa or lettuce, which may later be surface-irrigated.

Labor Savings. Following are some features of the sprinkle method relative to labor and management requirements:

- Periodic-move sprinkle systems require labor for only one or two relatively short periods each day to move the sprinkler laterals in each field. Labor requirements can be further reduced by utilizing mechanically moved, instead of hand-moved, laterals. Furthermore, unskilled labor can be used, because irrigation decisions are made by the manager, rather than by the irrigators.
- Most mechanized and automated sprinkle systems require very little labor and are simple to manage.
- Fixed sprinkle systems can eliminate field labor during the irrigation season and be fully automated to simplify management.

Special Uses. Some of the more important special uses of sprinkle irrigation include:

- Modifying weather extremes by increasing humidity, cooling crops, and alleviating freeze damage to buds and leaves by use of special systems designs;
- Using light, intermittent irrigation to supplement erratic or deficient rainfall, or to start early grain or pasture so that other inputs can be planned with assurance of adequate water; and
- Leaching of salts from saline soils, which is more efficient under sprinkle than under surface irrigation methods (because the soil is less saturated), but it takes more time.

Water Savings. High application efficiency can be achieved by properly designed and operated sprinkle irrigation systems. Properly engineered systems are easy to manage or automate to achieve overall seasonal irrigation efficiencies of 75% or greater. It is because much of the finesse needed to operate them can be designed into the systems hardware, thus reducing the management and labor inputs and training needed.

Disadvantages

The disadvantages of sprinkle systems are mainly in the areas of high costs, water quality and delivery problems, and environmental constraints. Systems should be designed by a competent specialist giving full consideration to irrigation, efficiency, cost, and convenience of operation.

High Costs. Both initial and pumping costs for sprinkle irrigation systems are higher than for surface irrigation systems on uniform soils and slopes. However, surface irrigation may be potentially more efficient. General initial and pumping costs for sprinkle systems are:

- Based on mid-1980s prices, the cost ranges of the various types of sprinkle systems, complete with mainlines and pumping plants, are: for simple systems, from \$450 to \$700 per ha (\$180 to \$280 per A); for mechanized and self-propelled systems, form \$800 to \$1200 per ha (\$320 to \$480 per A) and for semi-and fully automated fixed systems from \$2000 to \$3500 per ha (\$800 to \$1400 per A).
- The pump operating cost for pressurizing water is a continuous expense (unless water is delivered to the farm under adequate pressure). It costs about \$0.25 per ha-mm of water per 100 kPa of pressure (\$0.20 per A-ft per 1.0 psi) based on \$0.20 per liter (L) (\$0.75/gal) for diesel or \$0.06/kwh for electricity. Typical sprinkle lateral inlet pressure requirements range from about 200 to 400 kPa (30 to 60 psi) depending on the sprinklers used.

Water Quality and Delivery. The sprinkle method is restricted by the following water-related conditions:

- Large flows intermittently delivered are not economical to use without a reservoir, and even minor fluctuations in rate cause difficulties.
- Saline water may cause problems because salt is absorbed by the leaves of some crops and high concentrations of bicarbonates in irrigation water may spot and affect the quality of fruit when used with overhead sprinklers.
- Certain waters are corrosive to the metal pipes typically used in many sprinkle irrigation systems.

Environmental and Design Constraints. Some important constraints that limit the applicability of the sprinkle method are:

- Sprinkling is not well-adapted to soils having an intake rate of less than about 3 mm/hr (0.12 in./hr).
- Windy and excessively dry conditions cause low sprinkle irrigation efficiencies.
- Field shapes other than rectangular are not convenient to handle, especially for mechanized sprinkle systems.

TRICKLE

The trickle irrigation systems in common use today can be classified in a number of ways. However, for this text, the following four categories will be employed, because each requires a different layout or hydraulic design procedure:

- *Drip irrigation,* where water is slowly applied through small emitter openings to the soil surface;
- *Spray irrigation,* where water is sprayed over the soil surface near individual trees;
- *Bubbler irrigation,* where a small stream or fountain of water is applied to flood small basins or the soil surface adjacent to individual trees; and
- *Subsurface irrigation,* where water is applied through emitters below the soil surface. (Subsurface irrigation is not the same as subirrigation, which is done by controlling the water table.)

General Operation

For trickle irrigation, water is delivered by a pipe distribution network under low pressure in a predetermined pattern. Figure 2.5 shows a typical trickle lateral hose supplying water to a row of trees. Emitters are affixed to the hose,

FIG. 2.5. Lateral Hose for Trickle Irrigation in a Young Orchard.

which lies on the soil surface alongside the row of young trees. The emitters dissipate the pressure in the pipe distribution network by discharging water through narrow nozzles or long flow paths. The discharge rate is only a few liters or gallons per hour to each tree.

Upon leaving an emitter, water flows through the soil profile by capillarity and gravity. Therefore, the area that can be watered from each emission point is limited. Choosing a duration and frequency of application and emission point spacing that meet both the evapotranspiration demands of the crop and the infiltration and water-holding characteristics of the soil is important.

For wide-spaced permanent crops, such as trees and vines, emitters are manufactured individually as units that are attached by a barb to a flexible supply line called an *emitter lateral*, *lateral hose*, or simply *lateral*. Some emitters have several outlets that supply water through small-diameter "spaghetti" tubing to two or more emission points. These are used in orchards to wet a larger area with a minimum increase in costs.

For seasonal row crops, such as tomatoes, sugar cane, and strawberries, the lateral with emitter outlets is manufactured as a single disposable unit. These disposable laterals may have either porous walls from which water oozes or single or double-chambered tubing with perforations spaced every 0.15 to 1.0 m (0.5 to 3.3 ft).

For all types of trickle systems, the laterals are connected to supply pipe lines called *manifolds*. Figure 2.6 shows schematically the basic components and layout of typical trickle irrigation systems.

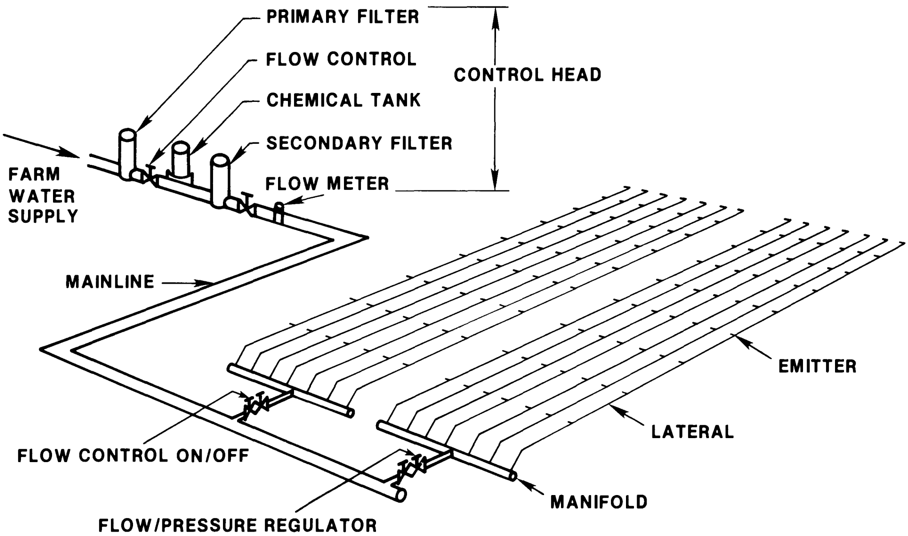


FIG. 2.6. Basic Components of a Trickle Irrigation System.

Advantages

Trickle irrigation is a convenient and efficient means of supplying water directly to the soil along individual crop rows or surrounding individual plants, such as trees and vines. A trickle irrigation system offers special agronomical, agrotechnical, and economical advantages for efficient use of water and labor. Furthermore, it provides an effective means for efficiently utilizing small continuous streams of water.

Water and Cost Savings. The high interest in trickle irrigation is because of its potential to reduce water requirements and operating costs. Trickle systems can irrigate some kinds of crops with significantly less water than is required by the other irrigation methods. For example, young orchards irrigated by a trickle system may require only one-half as much as orchards irrigated by sprinkle or surface irrigation. As orchards mature, the savings diminish, by they still may be important to growers who need to irrigate efficiently because of the scarcity and high price of water.

Trickle irrigation can reduce the cost of labor, because the water needs only to be regulated, not tended. The regulation is usually accomplished by automatic timing devices, but the emitters and system controls should be inspected frequently.

Easier Field Operations. Trickle irrigation does not stimulate weed growth, because much of the soil surface is never wetted by irrigation water (see Fig. 2.7). This reduces costs of labor and chemicals needed to control weeds. Also,

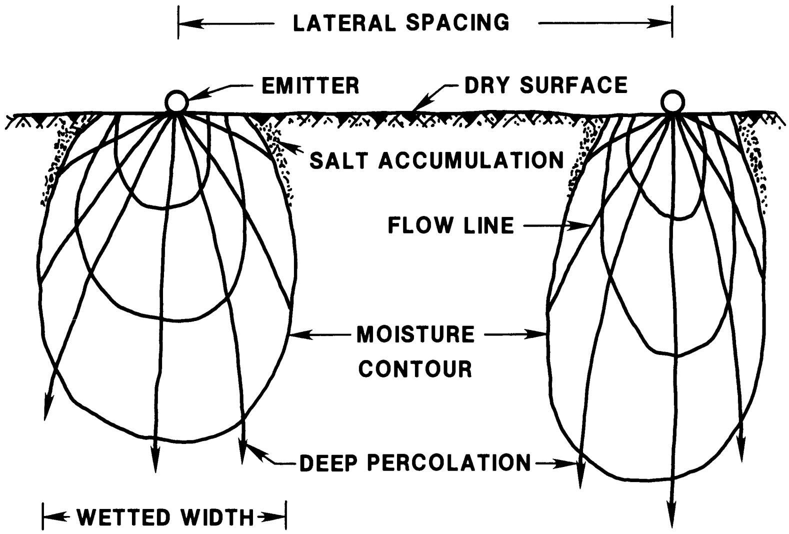


FIG. 2.7. Typical Pattern of Soil Wetting under Trickle Irrigation Showing Salt Accumulation (Source: Karmeli and Keller, 1975 (Fig. 1.5)).

because a trickle system wets less soil during an irrigation, uninterrupted orchard or field operations are possible. With row crops on beds, for example, the furrows in which farm workers walk remain relatively dry and provide firm footing.

Injecting fertilizers into the irrigation water can eliminate the labor needed for ground application. Several highly soluble fertilizers are available for this purpose, and newly introduced products widen the choice. Greater control over fertilizer placement and timing through trickle irrigation may improve its efficiency.

Use of Saline Water. Frequent irrigations maintain most of the soil in a well-aerated condition and at a soil moisture content that does not fluctuate between wet and dry extremes. Less drying between irrigation keeps the salts in the soil more dilute, making it possible to use more saline water than can be used with other irrigation methods.

Use on Rocky Soils and Steep Slopes. Trickle irrigation systems can be designed to operate efficiently on almost any topography. In fact, some trickle systems are operating successfully on avocado ranches that are almost too steep to be harvested (see Fig. 2.8). Because the water is applied close to each tree, rocky areas can be irrigated effectively by a trickle system even when the spacing between trees is irregular and tree sizes vary. Furthermore, problem soils with intermixed textures and profiles and shallow soils that cannot be graded can be efficiently irrigated by a trickle system.

FIG. 2.8. Trickle Irrigated Avocado Trees Growing on Steep Slopes.

Disadvantages

The main disadvantages inherent in trickle irrigation systems are their comparatively high initial cost, their susceptibility to clogging, their tendency to build up local salinity, and, where improperly designed or maintained, their spotty distribution of water.

High Costs. Based on mid-1980s prices the cost ranges for the various type of trickle system, complete with mainlines and pumping plants, are: for drip, spray or bubbler systems in orchards, from $2200 to $4000 per ha ($880 to $1600 per A); for drip or subsurface systems with disposable lateral tubing in row crops, from $1800 to $3000 per ha ($720 to $1200 per A); and for drip systems with reusable laterals in row crops, from $3000 to $5000 per ha ($1200 to $2000 per A).

Clogging. Because emitter outlets are very small, they can easily become clogged by particles of mineral or organic matter. Clogging reduces emission rates and the uniformity of water distribution, which causes damage to plants. To guard against clogging, particles of mineral or organic matter present in the irrigation water must be removed before the water enters the pipe network. However, particles may form within the pipes as water stands in the lines or evaporates from emitter orifices between irrigations. Iron oxide, calcium carbonate, algae, and microbial slimes are problems in many trickle systems. Chemical treatment of the water is necessary to prevent or correct most of these causes of clogging.

Distribution Uniformity. Most trickle irrigation emitters operate at pressures ranging from 2 to 14 m (6 to 45 ft) of head. If a field slopes steeply, the individual emitter discharges may differ by as much as 50% from the volume intended. Furthermore, the lines will drain through lower emitters after the water is shut off; hence, some plants receive too much water and others too little.

Soil conditions. Some soils may not have sufficient infiltration capacity to absorb water at the usual emitter discharge rate. Under these conditions, ponding and runoff can be expected. For example, with a 4 L/hr (one gph) discharge, the soil must have an infiltration capacity of 13 mm/hr (0.5 in./hr) to keep the pool of free water around the emitter from exceeding 60 cm (2 ft) in diameter.

Sandy soils are usually well adapted to trickle irrigation, especially those that have horizontal stratification. Stratification aids trickle irrigation, because it promotes lateral water movement, which wets a greater volume of soil. Experience has shown that medium-textured soils are usually also well suited for trickle irrigation, but runoff is likely to develop on some fine-textured soils.

Salt Accumulation. Salts often concentrate at the soil surface (see Fig. 2.7) and become a potential hazard. This is because light rains can leach them downward into the root zone. Therefore, when rain falls after a period of salt accumulation, irrigation should continue on schedule unless about 50 mm (2 in.) of rain have fallen. This is necessary to ensure leaching of salts below the root zone.

During trickle irrigation, salts also concentrate below the surface at the perimeter of the volume of soil wetted by each emitter (see Fig. 2.7). Too much drying of the soil between irrigations may reverse the movement of soil water and transfer salt from the perimeter of the wetted volume back toward the emitter, which can cause crop damage. To avoid this type of salt damage, water movement must always be away from the emitter.

Hazards. Some of the more prevalent hazards associated with trickle irrigation are:

- If uncontrolled events interrupt an irrigation, crop damage may occur rather quickly. The ability of roots to forage for nutrients and water is limited to the relatively, small *volume of soil wetted*, which *should be at least 33% of the total potential root zone*.
- Rodents sometimes chew polyethylene laterals. Rodent control or use of polyvinylchloride (PVC) laterals may be necessary to prevent this damage.
- Should a main supply line break or the filtration system malfunction, contaminants may enter the system. This could result in plugging up a large number of emitters that would need to be cleaned or replaced; therefore, safety screens should be provided and maintained at the lateral inlets.

TEXT LAYOUT

The text is divided into four sections: introductory concepts, sprinkle irrigation, trickle irrigation, and system selection.

Part I. Introduction

This is a short section made up of Chapters 1 through 3. Chapter 1 presents the basic approach to agroirrigation systems design used in this text. Chapter 2, "Sprinkle and Trickle Agroirrigation Overview," gives a brief history and description of sprinkle and trickle irrigation systems and some of their relative advantages and disadvantages. Chapter 3 "Soil-Plant-Water Relations," presents needed data and begins the design approach for sprinkle and trickle irrigation systems.

Part II. Sprinkle Irrigation

This section contains Chapters 4 through 16. Chapter 4 gives a comparative description of the various types of sprinkle systems, and Chapters 5 through 12 are presented in a logical sequence to address the design of "set" sprinkler systems. For set systems, sprinklers with nearly equal discharges are uniformly spaced along pipes to irrigate rectangular strips. The lines of sprinklers are periodically shifted through a series of moves (either by hand or mechanically) to irrigate the entire field. But they are not moving and sprinkling at the same time.

Chapters 6, 8, 10, 11, and 12 cover application efficiency, hydraulics, pipe network design, pressure requirements, and pumping. Therefore, they are also basic to the continuously moving types of sprinkle systems, as well as to trickle systems.

In Chapter 13 "Traveling Sprinkler Design," the concept of an irrigation machine with a very large, continuously moving sprinkler is introduced. This requires providing a flexible water supply hose and a mechanical means for moving the sprinkler and the hose. It also requires changes in the way of looking at application uniformity and the edge effects around the irrigated area.

Chapter 14 "Center-Pivot System Design," is the most comprehensive of the individual chapters. Center-pivots are the most elegant of irrigation machines, and they are used much more extensively than any of the other pressurized irrigation methods. (In the United States center-pivots account for over half of the pressurized irrigation.)

A center-pivot machine is made up of a line of sprinklers pivoting around a point to irrigate a large circular field. To handle the continuously moving and circular aspects involved, practically all of the steps for set sprinkler system design require some modification. In addition, there are a number of design concepts that are entirely specific for center-pivots. Many people think that, because a center-pivot is a factory-made machine, there is little design work to be done by the field engineer. However, this is not true, and Chapter 14 contains a considerable amount of new and unique material for optimizing the design of center-pivots.

Chapter 15 "Linear-Moving System Design," covers the design and operation of machines that irrigate while moving linearly. Practically all the basic design concepts relevant to a moving lateral are covered in the earlier chapters. The hydraulics are the same as for set systems, and the watering characteristics under the moving row of sprinklers are the same as for center-pivots. The major new points in this chapter are related to the management of the moving process.

Chapter 16 covers special uses of sprinkler systems and the application of fertilizers through the water, or fertigation. Because fertigation is important to both trickle and sprinkle irrigation, this serves as somewhat of a transition between Parts II and III.

Part III. Trickle Irrigation

There is no real difference between a tiny sprinkler and a spray-type trickle emitter, but the design approach is different. This is because, under sprinkle irrigation, the normal objective is to uniformly wet the entire surface and thus each plant. But with trickle irrigation the objective is not to uniformly wet the surface, but only to get a uniform amount of water to each plant.

Part III includes Chapters 17 through 24, which cover the design process for trickle irrigation systems in a logical sequence. Chapters 17 and 18 cover the different types and components of spray- and drip-type trickle systems and the problems associated with clogging the emitters and filtration.

Because water is delivered almost directly to each plant under trickle irrigation, the design process focuses on getting the optimum amount of water discharged at the emission points and emission uniformity. Chapters 19 through 21 cover the aspects related to planning concepts, emitter selection and design criteria, and system design strategies. In these chapters the technical aspects of the emitters themselves and the part they play in the overall emission uniformity are amplified. In addition, how to best use saline water is also covered.

Chapters 22 through 24 cover three levels of hydraulics: Hydraulics of laterals, which are hoses with uniformly spaced emitters, hydraulics of manifolds, which are the pipelines the laterals are connected to, and the hydraulic synthesis of the total pipe network. For the most part, the design process presented uses numerical solutions rather than requiring graphical and interpolation from charts, as in the past.

Part IV. System Selection

This section contains only Chapter 25 "Pressurized Irrigation System Selection," which is the final chapter. It gives and demonstrates a rational procedure for selecting the most promising pressurized irrigation system or systems for meeting specified development goals.

The procedure focus on economic and institutional, as well as physical, parameters associated with different system types and configurations. It involves choosing a set of adaptable systems that could potentially meet the development goals and then weighing their capitol, labor, land, water, power, and management input costs against potential output benefits. Consideration is also given to the relative importance, scarcity, and reliability of the inputs and outputs.

Appendixes

The appendixes contain three important sections. Appendix A is a list of the more comprehensive reference and textbooks on agroirrigation. Access to a few of these would be useful for obtaining more details on the basic background and analytical materials and expanded bibliographies for them. Appendix B is a glossary of the symbols used repetitively within the text. Appendix C is an annotated listing of the American Society of Agricultural Engineers standards and engineering practices related to pressurized irrigation systems.

3

Soil-Water-Plant Relations

Understanding basic soil-water-plant relations is central to the ability to design and manage trickle and sprinkle irrigation systems. It is assumed that the reader has already acquired a familiarity with the general concepts underlying these interactions, so the text will address this subject only briefly by reviewing a few important terms.

SOIL WATER

The tables related to soil water that are presented in this section will be useful as preliminary design information.

Water-Holding Capacity

Soils of various textures have varying abilities to retain water. Except for required periodic leaching, any irrigation beyond the field capacity of the soil is an economic loss. Table 3.1, which gives typical ranges of available water-holding capacities (field capacity minus permanent wilting point) of soils of different textures, was adapted from Chapter 1, Section 15, of the Soil Conservation Service's (SCS) *National Engineering Handbook* and is presented here for convenience. Where local field data are not available, the listed averages may be used as a guide for preliminary designs, but final designs should be based on actual field data.

Root Depth

The total amount of soil water available for plant use in any soil is the sum of the available water-holding capacities of all horizons occupied by plant roots.

Typical plant feeder root and total root depth are given in many references; however, the actual depths of rooting of the various crops are affected by soil conditions. Therefore, the actual depth at any site should be checked. Where local data are not available and there are no expected root restrictions, Table 3.2 can be used as a guide to estimating the effective root depths of various crops.

Table 3.1. Range in available water-holding capacity of soils of different texture

Soil texture	Water-holding capacity Range mm/m	Water-holding capacity Average mm/m
1. Very coarse texture—very coarse sands	33 to 62	42
2. Coarse texture—coarse sands, fine sands, and loamy sands	62 to 104	83
3. Moderately coarse texture—sandy loams	104 to 145	125
4. Medium texture—very fine sandy loams, loams, and silt loams	125 to 192	167
5. Moderately fine texture—clay loams, silty clay loams, and sandy clay loams	145 to 208	183
6. Fine texture—sandy clays, silty clays, and clays	133 to 208	192
7. Peats and mucks	167 to 250	208

NOTE: 1 mm/m = 0.012 in./ft.

The values given are averages selected from several references. They represent the depth at which crops will obtain the major portion of their needed water when grown in a deep, well-drained soil that is adequately irrigated.

CONSUMPTIVE USE AND DESIGN

Deciding how much water a system should be able to deliver to a crop over a given period is ultimately a question of selecting a capacity that, over the life of the system, will maximize profits to the farmer. To begin to address this question of system capacity, it is necessary to know how much water a crop will use, not only over the entire growing season, but also during the part of the season when water use is at its peak. It is the rate of water use during this peak consumptive period that is the basis for determining the rate at which irrigation water must be delivered to the field. Examples of typical seasonal and peak daily crop water requirements are given in Table 3.3.

Percentage of Area Shaded and Wetted

Another factor determining system capacity for trickle irrigation is that trickle systems need supply water only to the immediate vicinity of each plant being irrigated, unlike sprinkle systems, which are designed to wet an entire field. Thus, trickle systems can satisfy crop water requirements without an unnecessarily large amount of water being evaporated from the soil surface. Because of this practice, trickle system capacity is not a function of water consumption over an entire field, but only over that portion of the field actually receiving irrigation water.

Table 3.2. Effective crop root depths that would contain approximately 80% of the feeder roots in a deep, uniform, well-drained soil profile (1 m = 3.28 ft)*

Crop	Root depth (m)	Crop	Root depth (m)
Alfalfa	1.2 to 1.8	Lettuce	0.2 to 0.5
Almonds	0.6 to 1.2	Lucerne	1.2 to 1.8
Apple	0.8 to 1.2	Oats	0.6 to 1.1
Apricot	0.6 to 1.4	Olives	0.9 to 1.5
Artichoke	0.6 to 0.9	Onion	0.3 to 0.6
Asparagus	1.2 to 1.8	Parsnip	0.6 to 0.9
Avocado	0.6 to 0.9	Passion fruit	0.3 to 0.5
Banana	0.3 to 0.6	Pastures	0.3 to 0.8
Barley	0.9 to 1.1	Pea	0.4 to 0.8
Bean (dry)	0.6 to 1.2	Peach	0.6 to 1.2
Bean (green)	0.5 to 0.9	Peanuts	0.4 to 0.8
Bean (lima)	0.6 to 1.2	Pear	0.6 to 1.2
Beet (sugar)	0.6 to 1.2	Pepper	0.6 to 0.9
Beet (table)	0.4 to 0.6	Plum	0.8 to 1.2
Berries	0.6 to 1.2	Potato (Irish)	0.6 to 0.9
Broccoli	0.6	Potato (sweet)	0.6 to 0.9
Brussels sprout	0.6	Pumpkin	0.9 to 1.2
Cabbage	0.6	Radish	0.3
Cantaloupe	0.6 to 1.2	Safflower	0.9 to 1.5
Carrot	0.4 to 0.6	Sorghum (grain & sweet)	0.6 to 0.9
Cauliflower	0.6	Sorghum (silage)	0.9 to 1.2
Celery	0.6	Soybean	0.6 to 0.9
Chard	0.6 to 0.9	Spinach	0.4 to 0.6
Cherry	0.8 to 1.2	Squash	0.6 to 0.9
Citrus	0.9 to 1.5	Strawberry	0.3 to 0.5
Coffee	0.9 to 1.5	Sugarcane	0.5 to 1.1
Corn (grain & silage)	0.6 to 1.2	Sudan grass	0.9 to 1.2
Corn (sweet)	0.4 to 0.6	Tobacco	0.6 to 1.2
Cotton	0.6 to 1.8	Tomato	0.6 to 1.2
Cucumber	0.4 to 0.6	Turnip (white)	0.5 to 0.8
Egg plant	0.8	Walnuts	1.7 to 2.4
Fig	0.9	Watermelon	0.6 to 0.9
Flax	0.6 to 0.9	Wheat	0.8 to 1.1
Grapes	0.5 to 1.2		

*Approximately 80% of the feeder roots are in the top 60% of the soil profile.
Soil and plant environmental factors often offset normal root development; therefore, soil density, pore shapes and sizes, soil-water status, aeration, nutrition, texture and structure modification, soluble salts, and plant root damage by organisms should all be taken into account.

SOIL MOISTURE MANAGEMENT

A general rule of thumb for many field crops in arid and semiarid regions is that the soil moisture deficit, SMD, within the root zone should not fall below 50% of the total available water-holding capacity. This is a management-al-

Table 3.3. Typical peak daily and seasonal crop water requirements in different climates

	Type of climate and water requirements, mm									
	Cool		Moderate		Hot		High desert		Low desert	
Crop	Daily	Seas	Daily	Seas	Daily	Seas	Daily	Seas	Daily	Seas
Alfalfa	5.1	635	6.4	762	7.6	914	8.9	1016	10.2	1219
Pasture	4.6	508	5.6	610	6.6	711	7.6	762	8.9	914
Grain	3.8	381	5.1	457	5.8	508	6.6	533	5.8[1]	508[1]
Beets	4.6	584	5.8	635	6.9	711	8.1	732	9.1	914
Beans	4.6	330	5.1	381	6.1	457	7.1	508	7.6	559
Corn	5.1	508	6.4	559	7.6	610	8.9	660	10.2	762
Cotton	-	-	6.4	559	7.6	660	-	-	10.2	813
Peas	4.6	305	4.8	330	5.1	356	5.6	356	5.1[2]	356[2]
Tomatoes	4.6	457	5.1	508	5.6	559	6.4	610	7.1	660
Potatoes	4.6	406	5.8	457	6.9	553	8.1	584	6.9[2]	533[2]
Truck vegetables	4.1	305	4.6	356	5.1	406	5.6	457	6.3[2]	508[2]
Melons	4.1	381	4.6	406	5.1	457	5.6	508	6.4[2]	559[2]
Strawberries	4.6	457	5.1	508	5.6	559	6.1	610	6.6	660
Citrus	4.1	508	4.6	559	5.1	660	-	-	5.6	711
Citrus (w/cover)	5.1	635	5.6	711	6.4	813	-	-	6.9	889
Dec orchard	3.8	483	4.8	533	5.8	584	6.6	635	7.6	762
Dec orchard (w/cover)	5.1	635	6.4	711	7.6	813	8.9	914	10.2	1016
Vineyards	3.6	356	4.1	406	4.8	457	5.6	508	6.4	610

[1]Winter planting.
[2]Fall or winter planting.
NOTE: 1 mm = 0.039 in.

lowed deficit; MAD = 50%. Because it is also desirable to bring the moisture level back to field capacity with each irrigation, the depth of water applied at each irrigation is constant throughout the growing season. This means that the duration of each irrigation is also constant, although the frequency of application varies as a function of changes in the rate of water use over the growing season.

In humid regions, it is necessary to allow for rains during the irrigation period. However, the 50% limitation on soil moisture depletion should be followed as a general guide for field crops.

Soil management, water management, and economic considerations determine the amount of water used in irrigating and the rate of water application. The standard design approach has been to determine the amount of water needed to fill the entire root zone to field capacity, and then to apply at one application a larger amount to account for evaporation, leaching, and efficiency of application. The traditional approach to the frequency of application has been to take the depth of water in the root zone reservoir that can be extracted assuming MAD = 50%, and, using the daily consumptive use rate of the plant, determine

Table 3.4. Guide for selecting management-allowed deficit, MAD, values for various crops

MAD, %	Crop and root depth
25–40	Shallow-rooted, high-value fruit and vegetable crops
40–50	Orchards,* vineyards, berries and medium-rooted row crops
50	Forage crops, grain crops, and deep-rooted row crops

*Some fresh fruit orchards require lower MAD values during fruit finishing for sizing.

how long this supply will last. Such an approach is useful only as a guide to irrigation requirements, as many factors affect the volume and timing of applications for optimal design and operation of a system.

Table 3.4 is presented as a guide for selecting the appropriate maximum soil moisture depletion, or management-allowed deficit, for near optimum production of various crops. As indicated, for crops having high market values, it is often profitable to irrigate before 50% of the soil moisture in the root zone has been depleted.

Irrigation Depth

The maximum net depth of water to be applied per irrigation, d_x, is the same as the maximum allowable depletion of soil water between irrigations. It is computed by:

$$d_x = \frac{\text{MAD}}{100} W_a Z \tag{3.1}$$

where

d_x = maximum net depth of water to be applied per irrigation, mm (in.)
MAD = management-allowed deficit, which can be estimated from Table 3.4, %
W_A = available water-holding capacity of the soil, which can be estimated from Table 3.1, mm/m (in./ft)
Z = effective root depth, which can be taken from Table 3.2, mm (ft)

Irrigation Interval

The appropriate irrigation interval, which is the time that should elapse between the beginning of two successive irrigations, is determined by:

$$f' = \frac{d_n}{U_d} \tag{3.2}$$

where

f' = irrigation interval or frequency, days
d_n = net depth of water application per irrigation, to meet consumptive use requirements, mm (in.)
U_d = conventionally computed average daily crop water requirement, or use rate, during the peak-use month, which can be estimated from Table 3.3, mm/day (in./day)

The value selected for d_n will depend upon system design and environmental factors, and it should be equal to or less than d_x. When d_n is replaced by d_x in Eq. 3.2, f' becomes the maximum irrigation interval, f_x.

GENERAL DESIGN CONCEPTS

Design of both trickle and sprinkle irrigation systems is a synthesis process, where such properties as a soil's intake rate and crop water requirements, such items as pipes and pumps, and such processes as trenching or moving pipe must be integrated to form a good irrigation system. The irrigation designer's art is to know the kinds of techniques appropriate for a given site and to have a clear mental image of what the system can accomplish and how the completed system will appear.

The first order of business in sharpening design skills is to become acquainted with working irrigation systems, to inspect both good and bad systems, and see how and why they succeed or fail to deliver good results. Also, in examining existing systems, it is important to think of ways to improve a system's performance at its present site, as well as ways to tailor the system to satisfy other site conditions.

Fitting an irrigation system to a site not only demands knowledge of irrigation systems, it also requires an ability to analyze sites. Careful site analysis provides data that lead to an understanding of the physical and social resources that determine what can and ought to be accomplished by a proposed irrigation system. At this point, the art of irrigation design comes into play, as the designer selects an appropriate system to fit the available resources and tunes the system to fit the project goals. If this first design is not satisfactory, the designer should simply try something else until a design emerges that suits the farmer's resources and needs, as well as other project goals.

One of the keys to successful design is the ability to make realistic assumptions in areas where solid data are lacking. With experience and sound judgment, it is possible to formulate good designs even if detailed soils, operating costs, or other data are unavailable. Nonetheless, the greater the amount of reliable information used to formulate a design, the less will be the risk of the design failing to live up to expectations.

Before getting into the details of irrigation design, let us examine a simple hand-move sprinkle system and a simple trickle system, to see how climate, crop, and soil data are used in design. Each system irrigates a level, square field 400 m (1320 ft) on a side.

Sample Calculation 3.1. *Design a simple sprinkle system.*

GIVEN: A level, square field 400 m (1320 ft) on a side. The field is planted in alfalfa with a consumptive use rate for design purposes of 6.0 mm/day (0.24 in./day). The available soil water-holding capacity is W_a = 100 mm/m (1.2 in./ft), and the root depth of the alfalfa is assumed to be Z = 1.7 m (5.6 ft).

FIND: How climate, crop, soil, and spatial information are used in design.

CALCULATIONS: From Table 3.4, the target management-allowable deficit for the sprinkle system (see Fig. 3.1) is MAD = 50%, and by Eq. 3.1 the maximum net depth, d_x, of water that may be withdrawn from the root zone between irrigations is:

$$d_x = \frac{50}{100} 100 \times 1.7 = 85 \text{ mm } (3.35 \text{ in.})$$

For a daily consumptive use rate of U_d = 6.0 mm/day (0.24 in./day), by Eq. 3.2 the maximum irrigation interval should not exceed:

$$f_x = \frac{85}{6} \approx 14 \text{ days}$$

or using English units:

$$f_x = \frac{3.35}{0.24} \approx 14 \text{ days}$$

Because workers in this region like to take Sundays off and because downtime due to mechanical failure is always possible, this system will be designed to apply the water required for the 14-day irrigation interval in 11 days of operation. Assuming that laterals are moved twice a day, this gives a total of 22 moves during the 11 days of irrigation, with each move covering 400 m/22 = 18.2 m (1320 ft/22 = 60 ft), as shown in Fig. 3.1.

The above calculations have given us the schedule of operation for the system during the peak consumptive use period. More information can be had by considering the design consumptive use rate of the crop and the area to be irrigated. By multiplying these two values together, we arrive at the rate of consumptive use for the design area, and by adjusting this figure to account for inefficiencies in irrigation, we find the rate at which the irrigation system must supply water

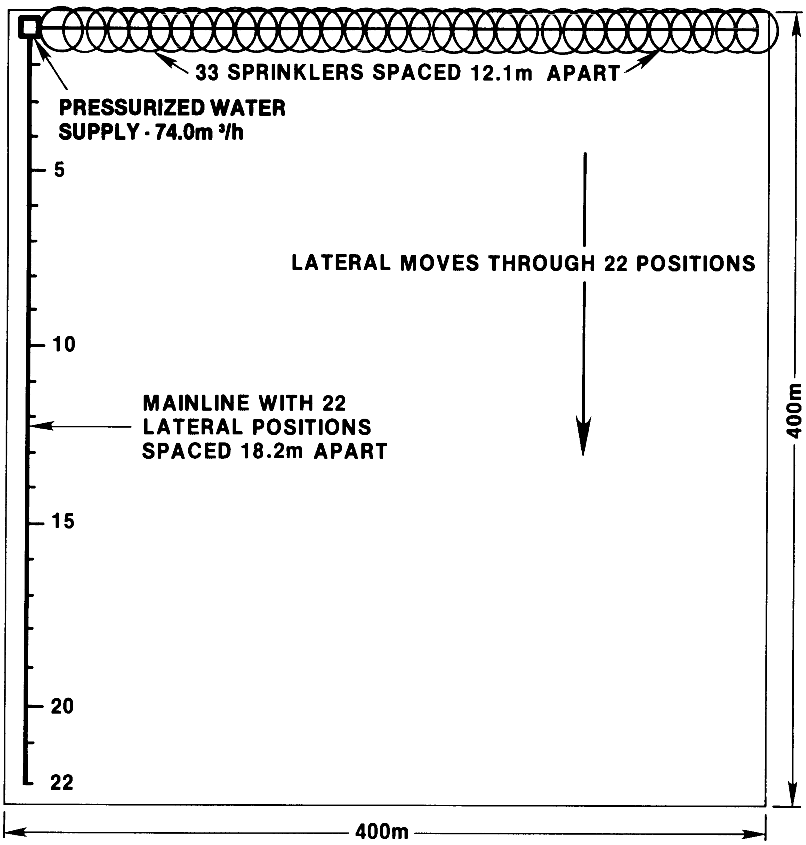


FIG. 3.1. Simple Periodic-move (Set) Sprinkle System Design Layout, with Sprinklers Spaced 12 m Apart Along a Portable Lateral Pipe That is Moved 18 m After Each Set.

to the field. In this case, by assuming an overall water delivery and application efficiency of 75%, if operated continuously, the system flow would be:

$$\frac{(400\ \mathrm{m})^2 \times 6\ \mathrm{mm/day}}{0.75} = 1280\ \mathrm{m^3/day}$$

$$= 53.3\ \mathrm{m^3/hr}\ (235\ \mathrm{gpm})$$

Because it takes 1 hr to move the lateral pipe after each irrigation set, the system operates for only 22 hr per day during the 11 days of operation or $22 \times 11 = 242$ hr out of the total $24 \times 14 = 336$ hr in the 14-day irrigation interval. This reduction in actual operating time increases the required flow rate to:

$$\frac{336\ \mathrm{hr}}{242\ \mathrm{hr}} \times 53.3\ \mathrm{m^3/hr} = 74.0\ \mathrm{m^3/hr}\ (326\ \mathrm{gpm})$$

With 33 sprinklers spaced 12.1 m (40 ft) apart along the single portable lateral (see Fig. 3.1), the average sprinkler discharge should be:

$$\frac{74.0 \text{ m}^3/\text{hr}}{33} = 2.24 \text{ m}^3/\text{hr (9.9 gpm)}$$

Sample Calculation 3.2. *Design a simple trickle system.*

GIVEN: A level, square orchard 400 m (1320 ft) on a side with trees spaced 4 × 5 m (13.12 × 16.4 ft). The orchard's transpiration rate is 5 mm/day (0.20 in./day).

FIND: How climate, crop, soil, and spatial information are used in design.

CALCULATIONS: The orchard under trickle irrigation (see Fig. 3.2) is planted with trees on a 4 × 5-m spacing. To correspond with this tree spacing, trickle laterals are spaced at 5-m intervals on the supply manifolds. Emitters are spaced

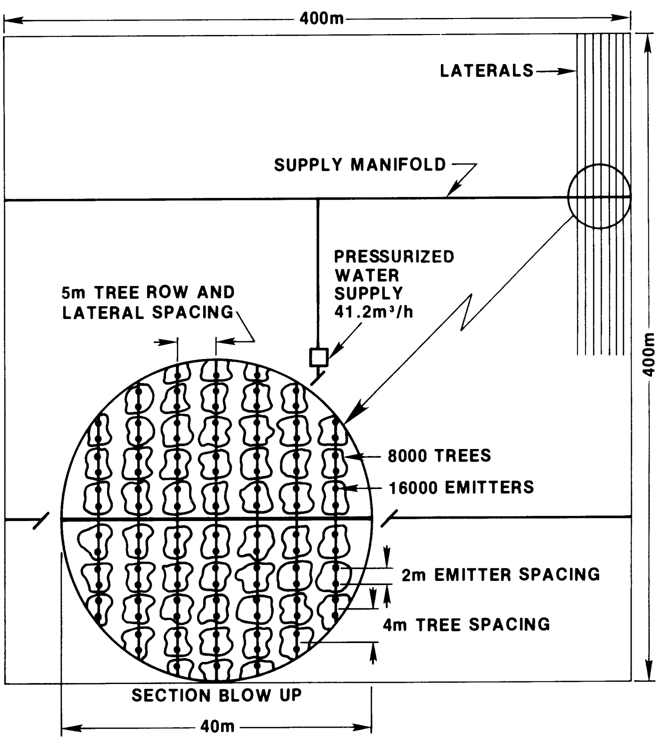


FIG. 3.2. Simple Trickle System Design Layout, with a Blow Up Section to Show the Tree Spacing, and Emitter and Lateral Layout.

every 2 m on the laterals supplying each row of trees, so that each tree is watered by two emitters. The average peak water use rate is 5 mm/day so every tree consumes 4 m $\times$ 5 m $\times$ 5 mm/day = 100 L/day or 4.2 L/hr (1.10 gph). If the system's total water delivery and application efficiency were 90%, then the system would have to supply (4.2 L/hr)/0.90 = 4.61 L/hr (1.22 gph) to each tree, giving a total system capacity of 37.0 m^3/hr (163 gpm) for the 8000 trees in the orchard. Usually trickle systems are run 90% of the time, giving a 10% allowance for downtime and repairs. This contingency boosts the total system flow rate to 41.2 m^3/hr (181 gpm).

Work remains to be done to complete these designs. Operating pressures must be decided upon, pipe sizes chosen, and sprinklers and emitters selected. These aspects of design will be treated later in the book, but already the foundation for these two systems has been laid out.

MANAGEMENT AND SCHEDULING

Generally irrigation systems are designed to meet average peak-water-use requirements. Sometimes, to reduce costs or to stretch limited water supplies, systems are designed to optimize production per unit of water applied. In such cases systems can be designed to apply only about 80% of peak water requirements and still obtain up to 95% of optimum yields. For deep-rooted crops in fine-textured soils, an appreciable amount of water can be stored prior to the critical peak-use periods. By drawing on this stored water, peak system delivery requirements can be reduced without reducing yield potential.

Salinity Control

All irrigation water contains some dissolved salts that are pushed downward by sprinkling and rainfall. By applying more water than the plants consume, most of the salts can be pushed or leached below the root zone. The first step in computing the additional water required for leaching is to determine the leaching requirement by:

$$LR = \frac{EC_w}{5EC_e - EC_w} \tag{3.3}$$

where

LR = leaching requirement ratio for sprinkle or surface irrigation
EC_w = electrical conductivity of the irrigation water, dS/m (mmhos/cm)

EC_e = estimated electrical conductivity of the average saturation extract of the soil root zone profile for an appropriate yield reduction, dS/m (mmhos/cm)

It is recommended that the EC_e value presented in Table 3.5 be used in Eq. 3.3. These are values that will give an approximate 10% yield reduction, as

Table 3.5. Values of EC_e that will give 10% yield reduction for various crops[1]

Crop	EC_e − dS/m	Crop	EC_e − dS/m
Field crops			
Barley	10.0	Rice	3.8
Cotton	9.6	Corn	2.5
Sugar beets	8.7	Flax	2.5
Wheat	7.4	Broadbeans	2.6
Soybean	5.5	Cowpeas	2.2
Sorghum	5.1	Beans	1.5
Groundnut	3.5		
Fruit and nut crops			
Date palm	6.8	Apricot	2.0
Fig, olive	3.8	Grape	2.5
Pomegranate	3.8	Almond	2.0
Grapefruit	2.4	Plum	2.1
Orange	2.3	Blackberry	2.0
Lemon	2.3	Boysenberry	2.0
Apple, pear	2.3	Avocado	1.8
Walnut	2.3	Raspberry	1.4
Peach	2.2	Strawberry	1.3
Vegetable crops			
Beets	5.1	Sweet corn	2.5
Broccoli	3.9	Sweet potato	2.4
Tomato	3.5	Pepper	2.2
Cucumber	3.3	Lettuce	2.1
Cantaloupe	3.6	Radish	2.0
Spinach	3.3	Onion	1.8
Cabbage	2.8	Carrot	1.7
Potato	2.5	Beans	1.5
Forage crops			
Tall wheat grass	9.9	Wild rye grass	4.4
Bermuda grass	8.5	Vetch	3.9
Barley (hay)	7.4	Alfalfa	3.4
Rye grass	6.9	Corn (forage)	3.2
Crested wheat grass	6.0	Berseem clover	3.2
Tall fescue	5.8	Orchard grass	3.1
Sudan grass	5.1	Clover	2.3

[1]Adapted from Ayers and Westcott (1985)

presented by Ayers and Westcott (1985). (For conversion purposes: 1.0 ppm $\simeq 640 \times EC$ in dS/cm.)

Under full irrigation, where $LR < 0.1$, the annual deep percolation losses, even in most of the least watered areas, will normally be sufficient to provide the necessary leaching. However, under deficit irrigation or when $LR < 0.1$, water in addition to the consumptive use should be applied or available at some time during the year to satisfy leaching requirements. The ratio of the total depth of irrigation water required with and without leaching is equal to $1/(1 - LR)$.

Drainage

The assumption above is that the unavoidable excess depth of applied water is at least 10% on all parts of an area that is sufficiently irrigated to meet evapotranspiration demands. With poor irrigation scheduling and application uniformity, the excesses would be much greater. In either case some means for disposing of the excess water is needed.

Drainage of the excess water from the soil profile and conveying it to a sump for reuse or disposal is as important as irrigation. In fact natural or man-made subsurface drainage is essential for sustaining irrigated agricultural production. Without it salts will accumulate until they become toxic to plant growth, and the water table will rise and literally drown the plants. Furthermore, the productive capability of the land itself may be severely damaged and require major reclamation to become productive again.

It is beyond the scope of the text to deal further with drainage. Like irrigation, it is another subdiscipline of agricultural engineering.

Sample Calculation 3.3 *Computing leaching requirement for a sprinkle irrigation system.*

GIVEN: Corn irrigated by a traveling sprinkler with water having an electrical conductivity of $EC_w = 2.1$ dS/m.

FIND: The leaching requirement.

CALCULATIONS: From Table 3.5, the electrical conductivity of the average saturation extract of the soil root zone profile that would give a 10% yield reduction, $EC_e = 2.5$ dS/m, and by Eq. 3.3:

$$LR = \frac{2.1}{5(2.5) - 2.1} = 0.20$$

Application Depth and Frequency

For periodic-move, and low-frequency, continuous-move systems, such as traveling sprinklers, it is desirable to irrigate as infrequently as practical to reduce labor costs. For trickle and for solid-set and center-pivot sprinkle system, the degree of system automation is generally high enough that labor costs will not be a major consideration in determining irrigation frequency. Under these conditions, an irrigation frequency can be selected that will provide the optimal environment for plant growth given the physical limitations of the system.

Systems are usually designed so that their discharge, depths of application, and irrigation frequency meet crop water requirements during the peak consumptive use period. For this reason the systems must be managed to avoid wasting water during other periods of the crops' growth cycle when water requirements are less (see Fig. 3.3); when the crops' roots may not have penetrated to their full depth: and during rainy periods. This requires scheduling the irrigations to fit crop requirements. Water, labor, and energy would be wasted during rainy periods and at the beginning and end of the crop season if the systems were run using the full system design capacity. Furthermore, during the very early crop growth stages, when roots are shallow, more frequent, light irrigations may be required.

Irrigations can be scientifically scheduled from water budgets based on evaporation estimates or soil-moisture observations. For details on scheduling pro-

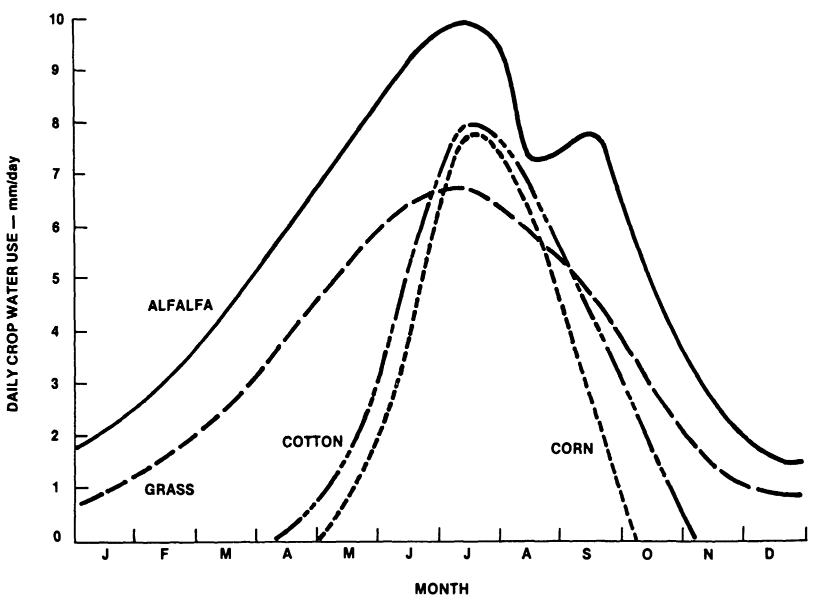


FIG. 3.3. Typical Crop Water Use Curves for Alfalfa, Cotton, Corn, and Grass in Mid-portion of the Central Valley of California.

uling procedures and estimating crop water requirements refer to: Doorenbos and Pruitt (1977); Doorenbos and Kassam (1979); Hargraves and Samani (1985); and Jensen et al. (1990).

REFERENCES

Ayers, R. S., and D. W. Westcott, 1985. Water quality for agricultures. *Food and Agricultural Organization of the United Nations, Irrigation and Drainage Paper 29, Rev. 1.*

Doorenbos, J., and W. O. Pruitt. Revised 1977. Crop water requirements. *Food and Agricultural Organization of the United Nations, Irrigation and Drainage Paper 24.*

Doorenbos, J., and A. H. Kassam. 1979. Yield response to water. *Food and Agricultural Organization of the United Nations, Irrigation and Drainage Paper 33.*

Hargraves, G. H., and Z. A. Samani. 1985. Reference crop evapotranspiration from temperature. *Applied Engineering in Agriculture* 1(2):96–99.

Jensen, M. E., R. D. Burman, and R. G. Allen (eds.). 1990. *Evaporation and Irrigation Water Requirements.* Committee on Irrigation Water Requirements. New York: American Society of Civil Engineers.

RECOMMENDED READING

Hansen, V. E., O. W. Israelsen, and G. E. Stringham. 1979. *Irrigation Principles and Practices.* New York: Wiley.

James, L. G. 1988. *Principles of Farm Irrigation system Design.* New York: Wiley.

Soil Conservation Service (SCS). 1970. Irrigation water requirements. *USDA-SCS Technical Release 21.*

II

SPRINKLE IRRIGATION

4

Types of Sprinkle Systems

There are 10 major types of sprinkle systems and several versions of each type. These types of systems may be divided into two basic groups: set systems that operate with the sprinklers set in a fixed position, and continuous-move systems that operate while the sprinkler is moving through the field. Set systems may be further divided according to whether or not sprinklers must be moved through a series of positions during the course of irrigating a field. Those systems that must be moved are called *periodic-move* systems, and those not requiring any movement are called *fixed* systems.

SET SYSTEMS

The major types of periodic-move systems are: hand-move, end-tow, and side-roll laterals; side-move laterals with or without trail lines; and gun and boom sprinklers. Fixed systems are usually either small or big gun sprinklers mounted at stationary positions.

Hand-Move Lateral System

The hand-move, portable lateral system is composed of either portable or buried main line pipe with valve outlets at intervals for attaching the portable laterals. These laterals are assembled from sections of aluminum tubing connected by quick couplers. Each pipe section has either center-mounted or end-mounted riser pipe supporting a sprinkler head. This system is used to irrigate more area than any other type of set system and is used on almost all crops and on all types of topography. A disadvantage of the system is its high labor requirement. This system is the basis from which all the mechanized systems were developed. Figure 2.2 shows a typical hand-move sprinkler lateral in operation.

A modification can be made to hand-move systems to reduce labor by the addition of a "tee" at each sprinkler riser. A hose or tube one lateral spacing long is then connected to the "tee." With a row of sprinklers along the lateral and another at the ends of the hoses, a two-sprinkler-wide strip is irrigated. This modification reduces the number of hand-move laterals by half; however, the system is more difficult to move than a conventional hand-move lateral.

End-Tow Lateral System

An end-tow lateral system is similar to one with hand-move laterals except that is consists of rigidly coupled lateral pipe connected to a main line. The main line should be buried and positioned in the center of the field for convenience of operation. Laterals are towed lengthwise over the main line from one side to the other in an "S" fashion, as depicted in Fig. 4.1. By draining the pipe through automatic quick-drain valves, a 20- to 30-hp tractor can easily pull a 400 -m-long (quarter-mile-long) 100-mm (4-in.)-diameter lateral.

Two carriage types are available for end-tow systems. One is a skid plate attached to each coupler to slightly raise the pipe off the soil, protect the quick drain valve, and provide a wear surface when towing the pipe. Two or three outriggers are required on a 400-m (quarter-mile) lateral to keep the sprinklers upright. The other type uses small metal wheels at or midway between each coupler to allow easy towing on sandy soils.

End-tow laterals are the least expensive of the mechanically moved systems. However, they are not adapted to small or irregular areas, steep or rough topography, row crops planted on contours, or fields with physical obstructions. They work best in grasses, legumes, and other close-growing crops and fairly well in row crops. However, the laterals can be easily damaged by careless operation. It is important that operators avoid moving laterals before they have drained, making too sharp as "S" turn, or moving too fast. End-tow systems are not, therefore, recommended for projects where the quality of labor is undependable.

When used in row crops, a 75-m (250-ft)-wide turning area is required along the length of the main line, as shown in Fig. 4.1. The turning area can be planted in alfalfa or grass. Crop damage in the turning areas can be minimized

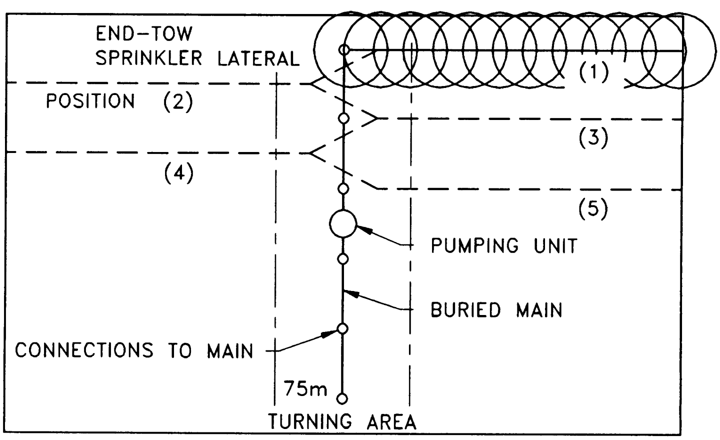


FIG. 4.1. Schematic of Move Sequence for End-tow Sprinkler Lateral.

by making an offset equal to one-half the distance between lateral positions each time the lateral is towed across the main line (see Fig. 4.1), instead of a full offset every other time. Irrigating a tall crop, such as corn, requires a special crop-planting arrangement, such as 16 rows of corn followed by 4 rows of a low-growing crop that the tractor can drive over without causing much damage.

Side-Roll Lateral System

A side-roll lateral system is similar to a system with hand-move laterals. The lateral pipes are rigidly coupled, and each joint of pipe is supported by a large wheel, as shown in Fig. 4.2. The lateral line forms the axle for the wheels, and when it is twisted, the line rolls sideways. This unit is mechanically moved by an engine mounted at the center of the line or by an outside power source at one end of the line.

Side-roll laterals work well in low-growing crops. They are best adapted to rectangular fields with fairly uniform topography and no physical obstructions. The diameter of the wheels should be selected so that the lateral clears the crop and the specified lateral move distance is a whole number of rotations of the line.

Side-roll laterals up to 490 m (1600 ft) long are satisfactory for use on close-planted crops and smooth topography. For rough or steep topography and for row crops with deep furrows, such as potatoes, lateral lengths should not exceed

FIG. 4.2. Side-roll Sprinkler Lateral in Operation (Source: Nelson Irrigation Corp.).

400 m (one-fourth mile). Typically, 100-or 125-mm (4- or 5-in.) diameter aluminum tubing is used. For a standard, 400-m-(quarter-mile)-long lateral on a close-spaced crop, at least three lengths of pipe to either side of a central power unit should be heavy-walled, 1.83-mm (0.072-in.)-thick aluminum tubing. For longer lines and in deep-furrowed row crops or on steep topography, more heavy-walled tubing should be used to enable the laterals to roll smoothly and uniformly with little chance of breaking.

A well-designed side-roll lateral should have quick drains at each coupler. All sprinklers should be provided with a self-leveler so that, regardless of the position at which the lateral pipe is stopped, each sprinkler will be upright. In addition, the lateral should be provided with at least two wind braces, one on either side of the power mover, as well as a flexible or telescoping section to connect the lateral to the main line hydrant valves.

Trail tubes one lateral spacing long are sometimes added to heavy-walled, 5-in. side-roll lines. With sprinklers mounted along the lateral and at the ends of the trail tubes, these lines have the capacity to irrigate as much land as two conventional side-roll laterals. Special couplers with a rotating section are needed, so that the lateral can be rolled forward. Quick couplers are also required at each trail tube, so they can be detached when the lateral reaches its last operating position. Laterals must then be rolled back to the starting location, where the trail tubes are reattached for the beginning of a new irrigation cycle.

Side-Move Laterals

Side-move laterals are periodically moved across the field in a manner similar to side-roll laterals. An important difference is that the pipeline is carried above the wheels on small "A" frames instead of serving as the axle. Typically, the pipe is carried about 1.5 m (5 ft) above the ground, and the wheel carriages are spaced 15.2 m (50 ft) apart. Trail tubes with up to 11 sprinklers mounted at 9.2-m (30-ft) intervals are pulled behind each wheel carriage. Thus, the system wets a strip up to 100.6 m (330 ft) wide, allowing a 400-m (quarter-mile)-long lateral to irrigate approximately 4.0 ha (10 A)) at a setting. Such a system produces high uniformity and low application rates.

Side-move lateral systems are suitable for most field and vegetable crops. However, for field corn, the trail tubes cannot be used, and the "A" frames must be extended to provide a minimum ground clearance of 2.1 mm (7 ft). To irrigate a wide strip of land, small gun sprinklers discharging 4 to 6 L/s (60 to 90 gpm) are mounted at every other carriage (see Fig. 4.3). The system produces a wetted strip 46 m (150 ft) wide, and a 400-m-(quarter-mile)-long lateral will irrigate 1.8 ha (4.5 A) per setting. However, application rates are relatively high, approximately 13 mm/hr (0.5 in./hr).

Positioning a hand-move system is hard work, requiring more than twice the

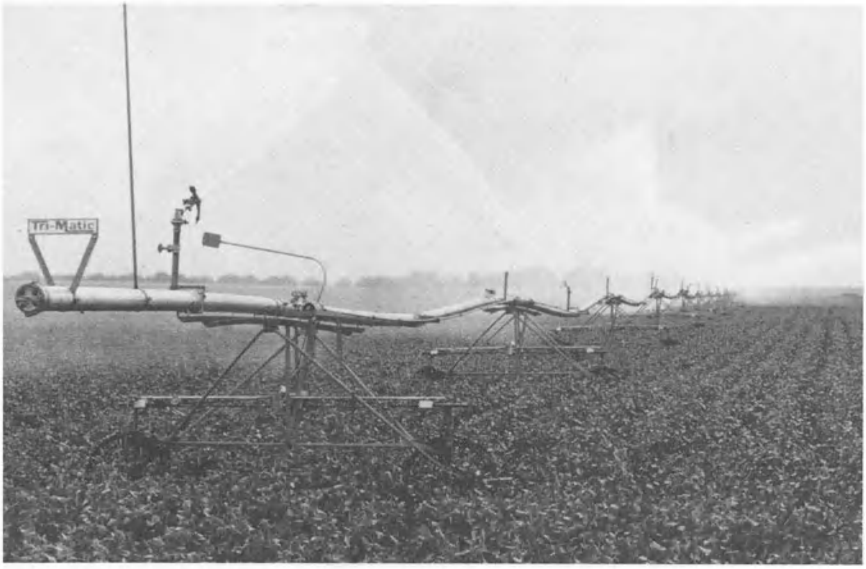


FIG. 4.3. Periodic-move Lateral with High Wheel Carriages to Support Gun Sprinklers (Source: Nelson Irrigation Corp.).

amount of time per unit of irrigated area as moving an end-tow, side-roll, or side-move system. However, a major inconvenience of these mechanical-move systems occurs when the laterals reach the end of an irrigation cycle. When this happens with a hand-move system, the laterals at the field boundaries can be disassembled, loaded on a trailer, and hauled to the starting position at the opposite boundary. Unfortunately, the mechanical-move laterals cannot be readily disassembled. Therefore, each one must be reversed and returned to its starting position. This operation is quite time-consuming, especially where trail tubes are involved.

Gun and Boom Sprinklers

Gun, or giant, sprinklers have 16-mm (5/8-in.) or larger range nozzles attached to long discharge tubes. Most gun sprinklers are rotated by means of a rocker arm drive (see Fig. 4.4), and many can be set to irrigate a part circle.

Boom sprinklers (see Fig. 4.5) have rotating arms 18 to 36 m (60 to 120 ft) in length. Water is discharged through nozzles strategically positioned along the arms. The arms, or booms, are supported by a cable suspension system and mounted on a four-wheel trailer. They are rotated by the thrust of the jets.

Gun or boom sprinkler systems are both well-adapted to supplemental irrigation and to use on irregularly shaped fields or fields with obstructions. Each has its comparative advantages and disadvantages. Boom sprinklers provide a

FIG. 4.4. Part-circle Gun Sprinkler with Rocker Arm Drive.

more uniform water application and somewhat smaller droplets and application rates than gun sprinklers. Gun sprinklers, however, are considerably less expensive and simpler to operate than booms; consequently, there are more gun than boom sprinklers in use. Gun and boom sprinklers usually discharge at least 6.3 L/s (100 gpm). Most typically these sprinklers discharge approximately

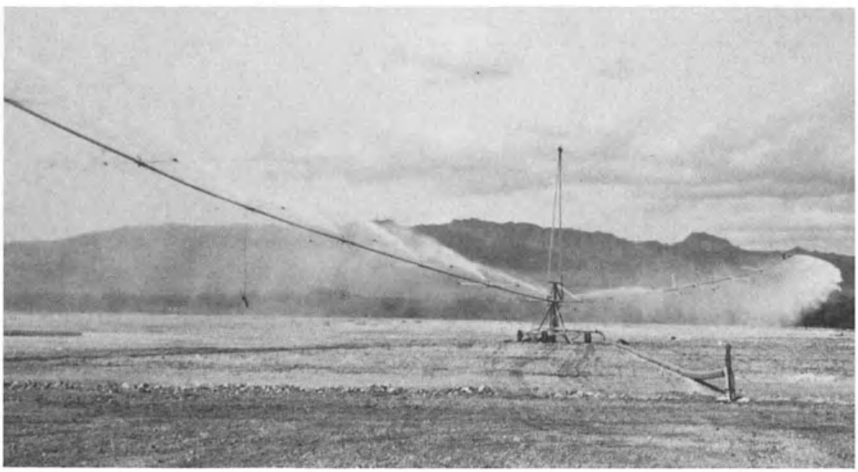


FIG. 4.5. Boom Sprinkler in Operation.

31.5 L/s (500 gpm) at operating pressures in the neighborhood of 620 kPa (90 psi), and gun sprinklers are usually operated individually rather than along laterals.

Gun and boom sprinklers can be used on most crops, but they produce relatively high application rates and large water drops that tend to compact the soil surface and to create runoff problems. Therefore, these sprinklers are most suitable for coarse-textured soils with high infiltration rates and for relatively mature crops that need only supplemental irrigation. Gun and boom sprinklers are not recommended for use in extremely windy areas, because their water distribution patterns become too distorted. Even under calm conditions, their application uniformity is relatively low.

Large gun sprinklers are usually trailer-or skid-mounted and, like boom sprinklers, are towed from position to position by a tractor. Boom sprinklers are unstable and can tip over when being towed over rolling or steep topography.

Fixed Sprinkler Systems

A fixed sprinkler system has enough lateral pipe and sprinkler heads so that none of the laterals need to be moved for irrigation purposes after being placed in the field. Thus, to irrigate the field, the sprinklers (or laterals) need only to be cycled on and off. The three main types of fixed systems are those with solid-set portable, hand-move laterals (see Fig. 4.6); buried, or permanent, laterals; and sequencing-valve laterals. Most fixed sprinkler systems have small sprin-

FIG. 4.6. Solid-set Sprinkler Laterals Connected to Portable Aluminum Mainline.

klers spaced 9.1 to 24.4 m (30 to 80 ft) apart, but some systems use small gun sprinklers spaced 30.5 to 48.8 m (100 to 160 ft) apart.

Portable solid-set systems are used for potatoes and other high-value crops, where the system can be moved from field to field as the crop rotation or irrigation plan for the farm is changed. These systems are also moved from field to field to germinate such crops as lettuce, which is then furrow-irrigated after germination. Moving the laterals into and out of the field requires much labor, although this requirement can be reduced by the use of special trailers on which the portable lateral pipe can be stacked by hand. After a trailer has been partly loaded, the pipe is banded in several places to form a bundle that can be lifted off the trailer at the farm storage yard with a mechanical lifter. The procedure is reversed when returning the laterals to the field for the next season.

Permanent buried laterals are placed underground 45 to 75 cm (18 to 30 in.) deep, with only the riser pipe and sprinkler head above the surface. Many systems of this type are used in citrus groves, orchards, vineyards, berries, and specialty crops.

The sequencing-valve lateral may be buried, placed on the soil surface, or suspended on cables above the crop. The heart of the system is a valve on each sprinkler riser that turns the sprinkler on or off when a control signal is received. Most sequencing-valve systems with small sprinklers use a pressure change in the water supply to activate the valves.

The portable lateral, buried or permanent lateral, and sequencing-valve lateral systems with small gun sprinklers can be automated by the use of electric or air-operated valves activated by controllers. These automatic controllers can be programmed for irrigation, crop cooling, and frost protection and can be activated by soil moisture and temperature-sensing devices.

Other Set Sprinkle Systems

Because of recent concerns about availability and cost of energy, interest in perforated pipe, hose-fed sprinklers, and orchard systems has revived. These systems afford a means of very low pressure 35- to 140-kPa (5- to 20-psi) sprinkle irrigation. Often gravity pressure produced by the difference in elevation between the water supply and irrigated area is sufficient to operate the systems without pumps. Furthermore, inexpensive low-pressure pipe, such as unreinforced concrete and thin-wall plastic or asbestos cement, can be used to distribute the water to the sprinklers. These systems do have the disadvantage of a high labor requirement when being periodically moved, because the spacing between lateral positions is usually only 6 to 9 m (20 to 30 ft).

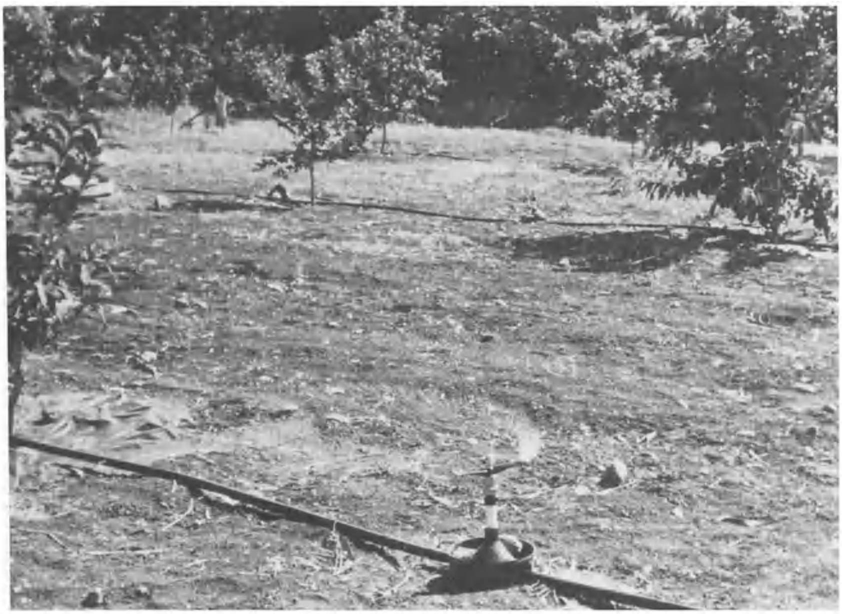
Perforated-pipe sprinkle irrigation almost became obsolete for agricultural irrigation, but continued to be widely used for home lawn systems. Perforated-pipe systems spray water from 1.6-mm (1/16-in.)-diameter or smaller holes drilled at uniform distances along the top and sides of a lateral pipe. The holes are sized and spaced so as to apply water reasonably uniformly between adjacent lines of perforated pipe. The water issues from the holes and produces a rainlike application over a rectangular strip (see Fig. 4.7). Each hole emits a jet of water, which, in rising and falling, breaks up into small drops that are spread by air turbulence over the irrigated area. The spread, which ranges from 7.5 to 15. m (25 to 50 ft), increases as pressure increases. Such systems can operate effectively at pressures between 35 and 210 kPa (5 and 30 psi). However, they can be used only on coarse-textured soils, such as loamy sands, having high capacities for infiltration, because the minimum practical application from perforated pipe is 13 mm/hr (0.5 in./hr).

Hose-fed sprinkler grid systems employ hoses to supply individual small sprinklers, which are operated at pressures as low as 35 to 70 kPa (5 to 10 psi). These systems can also produce relatively uniform wetting provided that the sprinklers are moved in a systematic grid pattern with sufficient overlap. However, these systems are not in common use except in home gardens and turf irrigation, although they do hold promise for rather broad use on small farms (see Fig. 4.8A) especially in developing countries, where capital and power resources are limited and labor is relatively abundant.

The *orchard sprinkler* is a small spinner or impact sprinkler designed to cover the space between adjacent trees; there is little or no overlap between the areas wetted by neighboring sprinklers. Orchard sprinklers are designed to be operated at pressures between 70 and 210 kPa (10 and 30 psi). Typically the di-

FIG. 4.7. Perforated Pipe Lateral in Operation.

A. Impact Sprinkler on Portable Stand (Source: Harward Irrigation Systems)



B. Orchard Sprinkler on Hose-pulled Skid

FIG. 4.8. Hose-fed Impact Sprinkle Irrigation Systems.

ameter of coverage is between 4.5 and 9 m (15 and 30 ft). They are located under the tree canopies to provide relatively uniform volumes of water for each individual tree. Water should be applied fairly evenly to areas wetted, although some soil around each tree may receive little to no irrigation. The individual sprinklers can be supplied by hoses (see Fig. 4.8B) and periodically moved to cover several positions, or a sprinkler can be provided for each position.

CONTINUOUS-MOVE

The major types of continuous-move systems are traveling (gun or boom) sprinklers, center-pivot systems, and linear-moving laterals.

Traveling Sprinkler

The traveling sprinkler, or traveler, is a high-capacity sprinkler fed with water through a flexible hose; it is mounted on a self-powered chassis and travels along a straight line while watering (see Fig. 2.3). The most common type of traveler used in the United States for agriculture utilizes a gun-type sprinkler that discharges approximately 32 L/s (500 gpm). The sprinkler is mounted on a moving vehicle and wets a diameter of more than 120 m (400 ft). The vehicle is equipped with a water piston or turbine-powered winch that reels in a cable. The cable guides the unit along a path as it tows a flexible, high-pressure, lay-flat hose. The hose is connected to a buried pipeline that supplies water under pressure. The typical hose is 100 mm (4 in.) in diameter and is 200 m (660 ft) long, allowing the unit to travel 400 m (1320 ft), unattended.

Figure 4.9 shows a typical layout for a cable-drawn traveling sprinkler. The entire strip between towpaths is irrigated without stopping. The unattended travel distance can be as long as the cable and twice the length of the hose. After use, the hose can be drained, flattened, and wound onto a reel. European travelers are typically supplied from more rigid hose, which does not lie flat, and the sprinkler is pulled by reeling up the hose.

Some traveling sprinklers have a self-contained pumping plant mounted on the vehicle that pumps water directly from an open ditch while the unit moves.

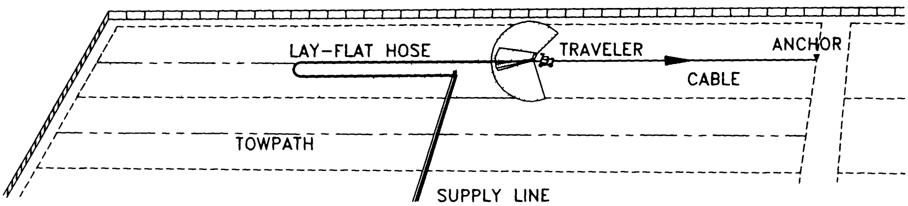


FIG. 4.9. Aerial View of Layout with Cable-drawn Traveling Sprinkler in Operation.

The supply ditches replace the hose. Other travelers are equipped with boom sprinklers instead of guns.

As the traveler moves along its path, the sprinkler wets a strip of land some 120 m (400 ft) wide, rather than the circular area wetted by a stationary sprinkler. After the unit reaches the end of a towpath, it is moved and set up to water an adjacent strip of land. The wetted overlap between adjacent strips depends on the distance between towpaths and on the diameter of the area wetted by the sprinkler. Frequently, a part-circle sprinkler is used, and the dry part of the pattern is positioned over the towpath so the unit travels on dry ground, as depicted in Fig. 4.9.

Traveling sprinklers require the highest pressure of any system. In addition to the 450- to 690-kPa (65- to 100-psi) or higher pressure required at the sprinkler nozzles, hose losses add another 140 to 280 kPa (20 to 40 psi) to the required system pressure. Therefore, travelers are best suited for supplemental irrigation where seasonal irrigation requirements are small, thus mitigating the high power costs associated with high operating pressures.

Traveling sprinklers can be used in tall field crops, such as corn and sugar cane, and have even been used in orchards. They have many of the same advantages and disadvantages discussed under gun and boom sprinklers; however, because they are moving, traveling sprinklers have a higher uniformity and lower application rate than guns and booms. Nevertheless, the uniformity of irrigation from travelers is only fair in the central portion of the field, leaving 30- to 60-m (100- to 200-ft)-wide strips along the sides and ends of the field poorly irrigated.

Center-Pivot

The center-pivot system sprinkles water from a continuously moving lateral pipeline. The self-propelled lateral is fixed at one end and rotates to irrigate a large circular area. The fixed end of the lateral, called the *pivot point*, is connected to the water supply. The lateral consists of a series of spans ranging in length from 27 to 76 m (90 to 250 ft) long, carried about 3 m (10 ft) above the ground by "drive units" that consist of an "A-frame" supported on motor-driven wheels (see Fig. 4.10).

Devices are installed at each drive unit to keep the lateral in line between the pivot and end drive unit. The end drive unit is set to control the speed of rotation. The most common center-pivot lateral uses 168-mm (6 5/8-in.) pipe, is 400 m (1320 ft) long, and covers a circle of 50 ha (126 A) inscribed within a square "quarter-section" of land with an area of 65 ha (160 A). An additional 1 to 4 ha (2 to 10 A) of the quarter-section may be irrigated by the pivot's end gun. Laterals as short as 70 m (230 ft) and as long as a "half-mile," 800 m (2640 ft) are available with pipe sizes up to 255 mm (10 in.).

The moving lateral pipeline is fitted with impact, spinner, or spray nozzle

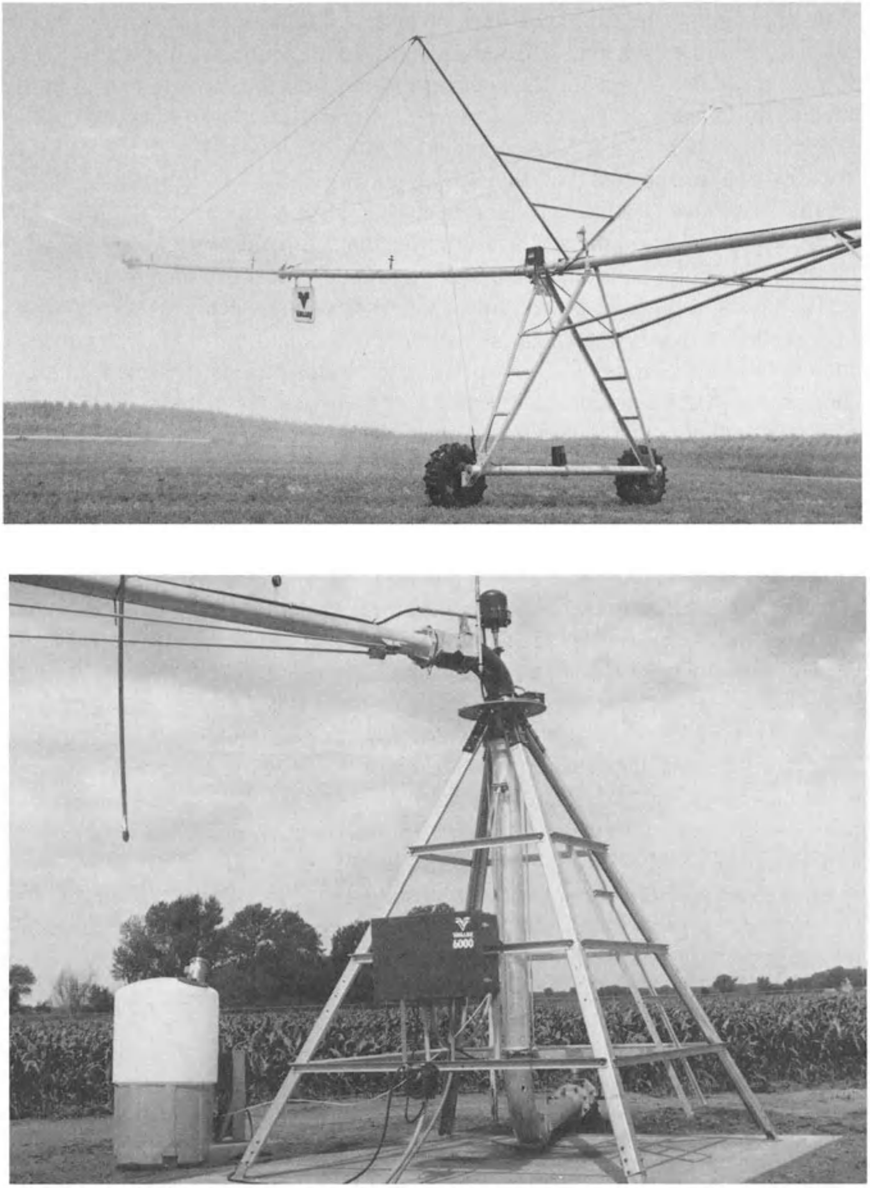


FIG. 4.10. Outer (Top) and Pivot (Bottom) Ends of Center-pivot Lateral (Source: Valmont Industries, Inc.).

sprinklers to spread the water uniformly over the circular field. The area irrigated by each sprinkler, if set at a uniform sprinkler spacing along the lateral, grows progressively larger toward the moving end. Therefore, to achieve uniform application, the sprinklers must be designed to have progressively greater discharges, closer spacings, or both, toward the moving end. Typically, when impact sprinklers are used, the application rate near the moving end is in the vicinity of 25 mm/hr (1.0 in./hr). With spray nozzles it may be as high as 250 mm/hr (10 in./hr). These application rates exceed the intake rate of many soils except for the first few minutes at the beginning of each irrigation. To minimize surface ponding and runoff, the laterals are usually rotated every 10 to 72 hr depending on the soil's infiltration characteristics, the system capacity and nozzling configuration, and the maximum desired soil moisture deficit.

The different types of power units used to drive the wheels on center-pivots are: electric motors, water pistons, water spinners and turbines, hydraulic oil motors, and air pistons. The first pivots used water pistons; however, electric motors are most common today because of their speed, reliability, and ability to rotate the lateral clockwise or counterclockwise.

Center-pivot sprinkler systems are suitable for almost all field crops, including corn, but require fields free of any above-ground obstructions, such as telephone lines, electric power poles, buildings, and trees. They are best adapted for use on soils having relatively high intake rates and uniform topography. When they are used on soils with low intake rates and irregular topography, the resulting runoff causes erosion and puddles that may interfere with the uniform circular movement of the lateral around the pivot point.

Where center-pivot systems are used on square fields, some means of irrigating the four corners must be provided, or other uses must be made of the areas not irrigated. In a 64-ha (160-A), quarter-section, squared field, from 9 to 12 ha (22 to 30 A) are not irrigated by the center-pivot system unless the pivot has a special corner-irrigating apparatus (see Fig. 4.11). With some corner systems, only about 3 ha (8 A) are left unirrigated.

Most pivot systems are permanently installed in a given field. However, for supplemental irrigation or for double-cropping, it is practical to move a center-pivot lateral back and forth between fields.

Linear-Moving Laterals

Self-propelled, linear-moving laterals combine the structure and guidance system of a center-pivot lateral with a traveling water-feed system similar to a traveling sprinkler.

For efficient operation, linear-moving laterals require rectangular fields free from obstructions. Measured water distribution from these systems has given the highest uniformity coefficients of any sprinkle system for single irrigations under windy conditions.

Systems that pump water from open channels must be installed on nearly level fields (see Fig. 4.12). Even where the system is supplied by a flexible hose, the field must have fairly uniform topography for the guidance system to work effectively.

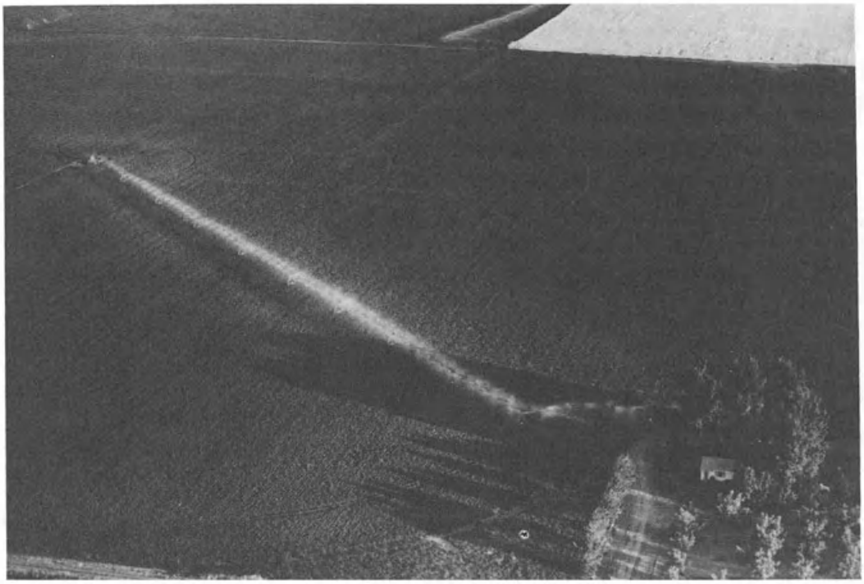


FIG. 4.11. Corner Systems for Use on End of Center-pivot Lateral (Source: Valmont Industries, Inc.).

A major disadvantage of linear-moving lateral systems, compared with center-pivot systems, is the problem of bringing the lateral back to the starting position or across both sides of the water-supply channel or pipeline. Because

FIG. 4.12. Linear-moving System with Open Channel Water Supply (Source: Valmont Industries, Inc.).

a center-pivot lateral operates in a circle, it automatically ends each irrigation cycle at the beginning of the next cycle. A linear-moving lateral travels from one end of the field to the other and must be driven or towed back to its starting position. However, due to this difference in lateral line movement, a linear-moving system can irrigate all of a rectangular field, unlike a center-pivot system, which can irrigate only a circular portion of the field (unless it is provided with a corner system).

APPLICATION EFFICIENCIES AND DEPTHS

Table 4.1 gives typical application efficiencies for well-managed sprinkle irrigation systems. These efficiency values can be used for preliminary design purposes or as final values where more refined data are unavailable.

The efficiencies in Table 4.1 are based on average expected coefficient of uniformity, CU, values (see Chapter 6 for detailed discussion) as a measure of application uniformity. Set system application rates between 2.5 and 5.0 mm/hr (0.1 and 0.2 in./hr) are considered low; between 5.0 and 10.0 mm/hr (0.2 and 0.4 in./hr) are considered medium; and over 10.0 mm/hr (0.4 in./hr) are considered high. In Table 4.1 wind speeds are referred to as low when

Table 4.1. Typical application efficiencies for well-managed sprinkle systems

Systems[1] and environmental conditions[2]	Efficiency E_h%
Moving and *set* systems with excellent uniformity in cool or humid climates and low winds	85
Typical efficiency for *moving* systems in most climates and winds; and *set* systems with medium to high applications rates and good uniformity in most climates and low winds	80
Typical efficiency used for average *set* systems in most climates and winds; and for *moving* systems in desert climates and high winds	75
Set systems with high application rate in desert climates with high winds or low application rates in other climates with high winds; and *travelers*	70
Set systems with moderately low application rates in desert climates and high winds or low application rates in high desert climates and high winds	65
Set systems with low application rates with small drops operating in low desert climates and medium to high winds; and *gun* or *boom* sprinklers	60

[1]*Moving* and *set* refer to systems with laterals fitted with small sprnklers.
[2]See text for application rate and wind speed ranges.

between 0 and 8 km/hr (0 and 5 mph); medium when between 8 and 16 km/hr (5 and 10 mph); and high when over 16 km/hr (10 mph).

RECOMMENDED READING

Addink, J. W. Keller, C. H. Pair, R. E. Sneed, and J. W. Wolfe. 1980. Design and operation of sprinkle systems. In *Design and Operation of Farm Irrigation Systems*, ASAE Monograph 3, ed. M. E. Jensen, pp. 621-660. St. Joseph, Michigan: American Society of Agricultural Engineers.

Pair, C. H., W. H. Hing, K. R. Frost, R. E. Sneed, and T. J. Schilty, (eds.). 1983. *Irrigation*, 5th ed. Arlington, Virginia: The Irrigation Association.

Rolland, Lionel. 1982. Mechanized sprinkler irrigation. *Food and Agriculture Organization of the United Nations Irrigation and Drainage Paper 35.*

5

Sprinkle Irrigation Planning Factors

A complete farm sprinkle system can be defined as a system planned exclusively for a given design area or farm unit on which sprinkling will be the primary method of water application. Planning for complete systems includes considering specified crops and crop rotations, water quality, and the soils found in the specified design area.

A farm sprinkle irrigation system includes sprinklers and related hardware; lateral, submain, and main pipelines; pumping plant and boosters; operation-control equipment; and other accessories required for the efficient application of water. The field system shown in Fig. 2.4 is of the periodic-move type, having a buried main line and a portable sprinkler lateral operating in rotation up one side and down the other side of the main.

Larger farm systems, such as the one shown in Fig. 5.1, are made up of several field systems. A field system is designed either for use on several fields of a farm unit or for movement between fields on several farm units. Field systems are planned for stated conditions, generally for preirrigation, for bringing up seedlings, or for use on specialty crops in a crop rotation. Considerations of distribution efficiency, labor utilization, and power economy may be entirely different for field systems than for complete farm systems. Field systems can be fully portable or semiportable.

Failure to recognize the fundamental difference between field and farm systems, either by the planner or the owner, has led to poorly planned systems of both kinds. In between these two are systems initially used as a field system, but designed to become part of a complete farm system.

Failure to anticipate the capacity required of the completed system has led to many piecemeal systems with poor distribution efficiencies, excessive initial costs, and high annual water-application charges. This situation is not always the fault of the system planner, for he may not always be informed whether future expansion is intended. However, the planner has a responsibility to inform the owner of possible alternatives for future development when preparing a field system plan.

DESIGN PROCEDURE

The first activity in the design process should be to collect basic farm-resource data. This information includes a topographic map showing obstacles and farm

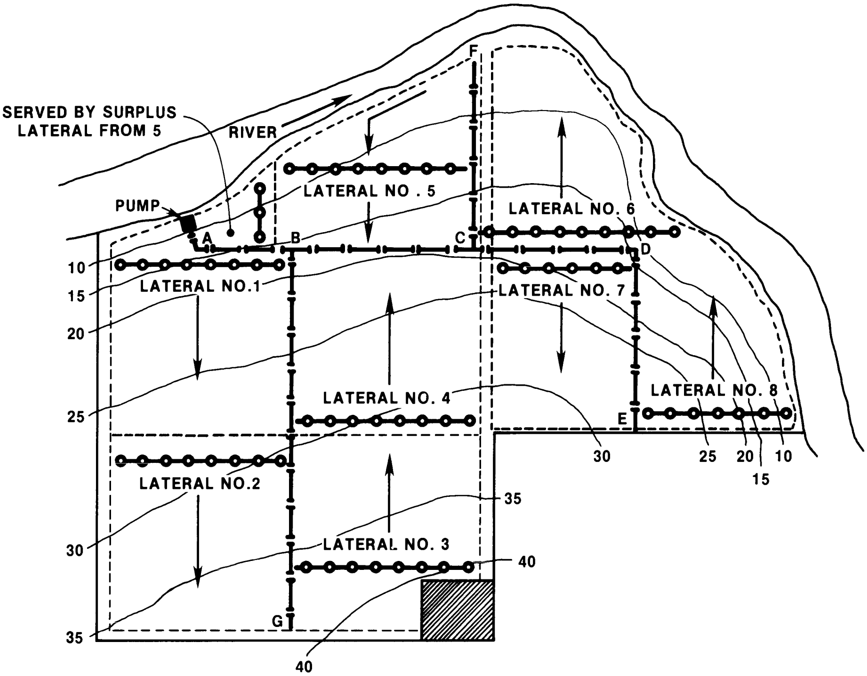


FIG. 5.1. Layout of a Complete Hand-move Sprinkle System.

and field boundaries, as well as data on water quality and quantity, weather, crops, and soils. The preliminary design factors (see Fig. 5.2) lead to determining peak-use rate, infiltration capacity, maximum depth of application per irrigation, application rate, and system capacity. The concept for the form shown in Fig. 5.2 was developed by McCulloch et al. (1957).

The designer should inquire about the farmer's financial, labor, and management capabilities. Once the data on the farm's resources have been assembled, the system selection, layout, and hydraulic design process can proceed according to the farm's physical, human, and financial constraints.

To facilitate this evaluation, a step-by-step checklist of the procedure normally used in planning a sprinkle irrigation system follows:

1. Make an inventory of available resources and operating conditions. Include information on soils, topography, water supply, source of power, crops, and farm operation schedules.
2. Using actual field data, local irrigation guides, or data from Tables 3.1, 3.2, and 3.4, estimate the depth or quantity of water to be applied at each irrigation.
3. Determine the average peak-period, daily consumptive use rates and the annual irrigation requirements for the crops under consideration. For general planning purposes the needed information is available from Ta-

I. CROP (TYPE)						
	(a) Root depth – mm (ft)	z				
	(b) Growing season – days					
	(c) Water use rate – mm/day (in./day)	U_d				
	(d) Seasonal water use – mm (in.)	U				
II. SOILS [Area – ha (A)]						
	(a) Surface texture depth – cm (ft) Moisture capacity – mm/m (in./ft)	W_a				
	(b) Subsurface texture depth – cm (ft) Moisture capacity – mm/m (in./ft)	W_a				
	(c) Moisture capacity – mm (in.)					
	(d) Allowable depletion – mm (in.)	d_x				
	(e) Intake rate – mm/hr (in./hr)					
III. IRRIGATION						
	(a) Interval – days	f'				
	(b) Net depth – mm (in.)	d_n				
	(c) Efficiency – %	E_a				
	(d) Gross depth – mm (in.)	d				
IV. WATER REQUIREMENT						
	(a) Net seasonal – mm (in.)	U				
	(b) Effective rain – mm (in.)	R_n				
	(c) Stored moisture – mm (in.)	M_s				
	(d) Net irrigation – mm (in.)	D_n				
	(e) Gross irrigation – mm (in.)	D_g				
	(f) Number of irrigations					
V. SYSTEM CAPACITY						
	(a) Application rate – mm/hr (in./hr)	I				
	(b) Time per set – hr	T_a				
	(c) Settings per day					
	(d) Days of operation per interval	f				
	(e) Preliminary system capacity – L/s (gpm)					

FIG. 5.2. Factors for Preliminary Sprinkle Irrigation System Design.

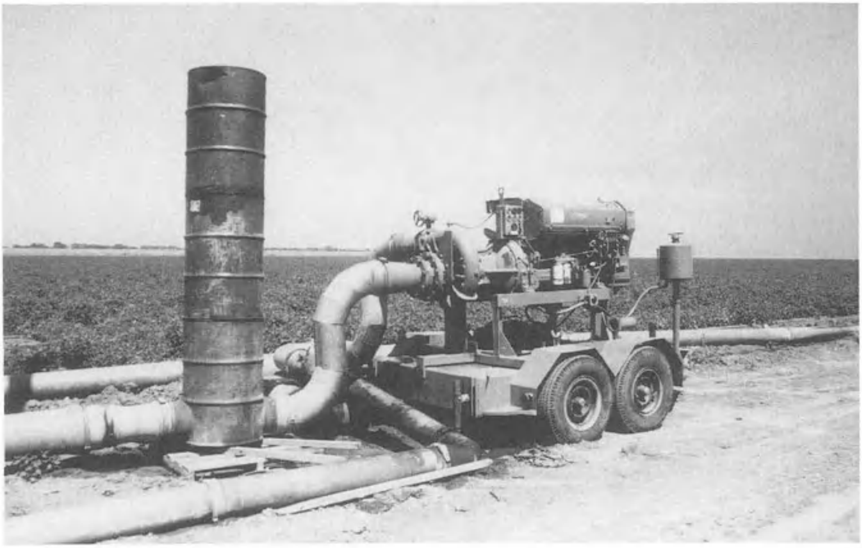
ble 3.3. For final planning consult a local irrigation guide or use more precise computational procedures.

4. Determine design-use frequency of irrigation or the length of the shortest irrigation period. This step is unnecessary for fully automated fixed systems or for center-pivot systems. The needed information may be available from local irrigation guides.
5. Determine capacity requirements of the system.
6. Determine the optimum water-application rate. Maximum (not necessarily optimum) rates are often obtainable form local irrigation guides.
7. Consider several alternative types of sprinkler systems. The landowner should be given alternatives from which to make a selection (see Fig. 1.2).
8. For periodic-move and fixed sprinkle systems determine:
   a. The sprinkler spacing, discharge, nozzle sizes, and operating pressure for the optimum water-application rate;
   b. The number of sprinklers that must be operated simultaneously to meet system capacity requirements;
   c. The best layout of main and lateral pipelines for simultaneous operation of the approximate number of sprinklers required;
   d. Final adjustments to meet layout conditions;
   e. Required sizes of lateral line pipe; and
   f. Maximum total pressure required for individual lateral lines.
9. Determine required sizes of main line pipe.
10. Check main line pipe sizes for power economy.
11. Determine maximum and minimum operating (pressure and discharge) conditions.
12. Select the pump and power unit for maximum operating efficiency within the expected range of operating conditions.
13. Prepare plans, schedules, and instructions for proper layout and operation.

To make a rational system selection, it may be necessary to design and analyze two or more systems. Finally, the owner should be encouraged to make a careful feasibility study of the system he ultimately selects. (The procedures for designing continuous-move systems will be covered later.)

CHARACTERISTICS OF SPRINKLER HEADS

The major components of a set sprinkle system are shown in Figs. 2.4 and 5.3. The hardware design process should begin with the sprinkler selection, continue with the system layout, and end with the design of the lateral, main line, and pumping plant (see Fig. 1.3).



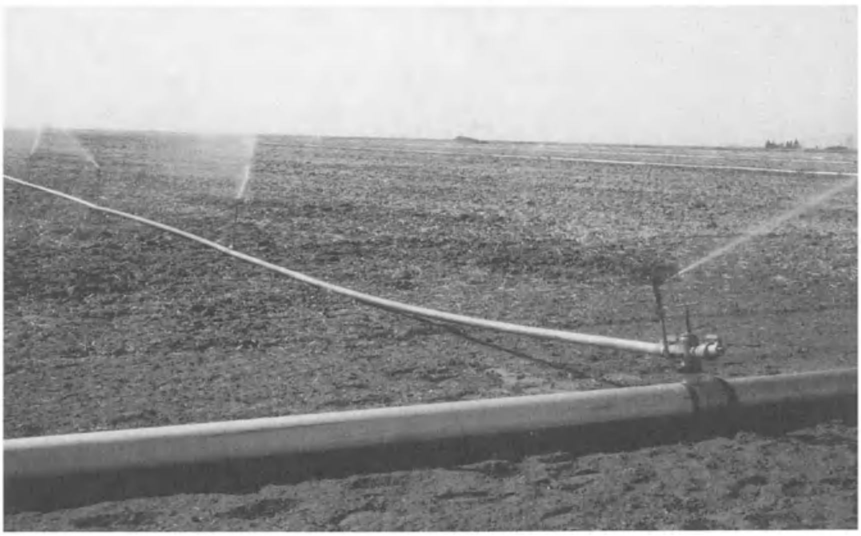


FIG. 5.3. Components of a Set Sprinkle Irrigation System Showing Booster Pump Supplying a Portable Main Line (Top) and Portable Main Line, Hydrant, and Sprinkler Lateral (Bottom).

Sprinklers are classified according to their operating pressure range and their position in relation to irrigated crops. Table 5.1 describes the different classifications, with the characteristics and adaptability of each.

Table 5.1. Classification of sprinklers and their adaptability

Sprinkler operating characteristics	Type of sprinkler and operating pressure range						
	Impact or spray low pressure 35–140 kPa (5–20 psi)	Impact or gear moderate pressure 105–210 kPa (15–30 psi)	Impact or gear medium pressure 210–410 kPa (30–60 psi)	Impact or gear high pressure 345–690 kPa (50–100 psi)	Rocker arm gun or giant 550–830 kPa (80–120 psi)	Spinner or spray undertree 70–245 kPa (10–50 psi)	Fixed perforated pipe 30–40 kPa (4–20 psi)
General characteristics	Special thrust springs or reaction-type arms	Usually single nozzle oscillating or long-arm dual-nozzle design	Either single or dual-nozzle design	Either single or dual-nozzle design	One large nozzle with smaller supplemental nozzles to fill	Stream trajectories below fruit and foliage by lowering the nozzle angle	Portable irrigation pipe with small perforations
Range of wetted diameters	6 to 15 m (20 to 50 ft)	18 to 24 m (60 to 80 ft)	23 to 37 m (75 to 100 ft)	34 to 90 m (110 to 230 ft)	61 to 122 m (200 to 400 ft)	12 to 27 m (40 to 90 ft)	Rectangular 3 to 15 m (10 to 50 ft) wide strips
Recommended minimum application rate	10 mm/hr (0.40 in./hr)	3 mm/hr (0.12 in./hr)	2 5 mm/hr (0 10 in./hr)	10 mm/hr (0.40 in./hr)	15 mm/hr (0.60 in /hr)	5 mm/hr (0 20 in./hr)	13 mm/hr (0.50 in /hr)
Jet characteristics (with proper pressure and nozzle size)	Water drops are large due to low pressure	Water drops are fairly well broken	Water drops are well broken over entire wetted diameter	Water drops are well broken over entire wetted diameter	Water drops are extremely well broken	Water drops are fairly well broken	Water drops are large due to low pressure
Water distribution with proper spacing pressure and nozzle size	Fair	Fair to good at upper limits of pressure range	Very good	Good except where wind velocities exceed 6.4 km/hr (4 mph)	Acceptable in calm air. Severely distorted by wind	Fairly good. Use diamond pattern where laterals are spaced wider then trees	Good. Pattern is rectangular
Adaptations and limitations for periodic-move or fixed systems	For small acreages and soils with intake rates exceeding 13 mm/hr (0.50 in./hr)	For fields crops, vegetables, and undertree sprinkling in orchards	For all field crops and most irrigable soils. Well adapted to overtree sprinkling	Same as for intermediate pressure sprinklers except where wind is excessive	For odd-shaped areas. Limited to soils with high intake rates or good cover	For all orchards. Ideal in windy areas and where pressure is too low for over-tree sprinkling	For low-growing crops and soils with relatively high intake rates Best adapted to small fields
Adaptations and limitations for traveling systems	Spray nozzles adaptable on center-pivot and linear-move	Center-pivot and linear-move on high-infiltrate-rate soils	Center-pivot and linear-move on most soils	Center-pivot end guns and travelers	Center-pivot end guns and travelers	Not applicable	Not applicable

Precipitation Profiles and Recommended Spacings

In choosing a sprinkler, the aim is to find the combination of sprinkler spacing, operating pressure, and nozzle size that will most nearly provide the optimum water-application rate with the greatest degree of uniformity of distribution.

The degree of uniformity obtainable with a set sprinkle system depends largely on the water-distribution pattern and spacing of the sprinklers. Figure 5.4, adapted from Christiansen (1942), shows the distribution pattern and precipitation profiles obtained from a typical double-nozzle sprinkler operating at proper pressure in low wind.

Each type of sprinkler has certain precipitation profile characteristics that vary with nozzle size and operating pressure and result in an optimal range of operating pressures for each nozzle size. In selecting nozzle sizes and operating pressures for a required sprinkler discharge, the designer should know that different pressures affect the profile as follows:

1. At the lower side of the specified pressure range for any nozzle, the water remains in large drops. When pressure falls too low, the water from the nozzle concentrates in a ring a distance away from the sprinkler, giving a poor precipitation profile (see Fig. 5.5A);

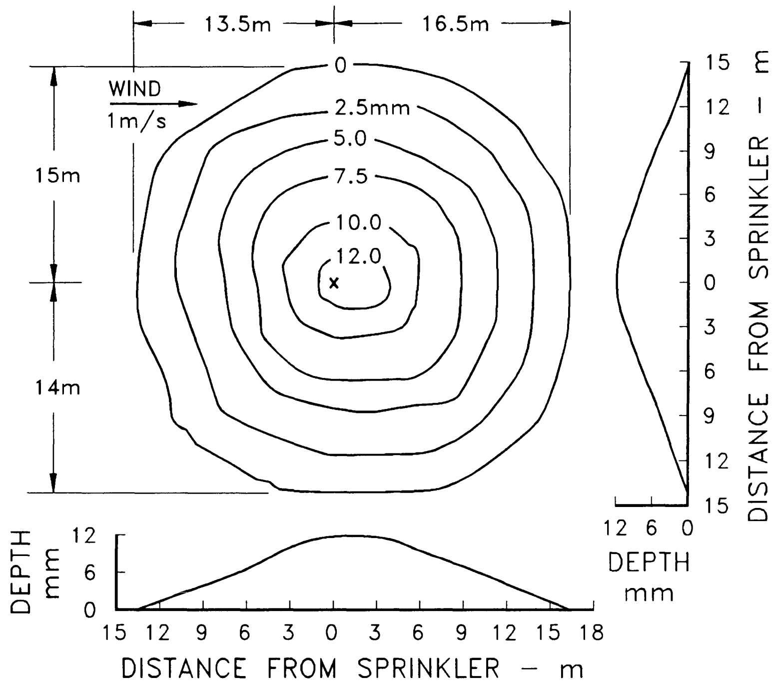


FIG. 5.4. Distribution Pattern and Precipitation Profiles from a Typical Double Nozzle Sprinkler Operating under Favorable Conditions (1 m = 3.28 ft; 1 mm = 0.04 in.).

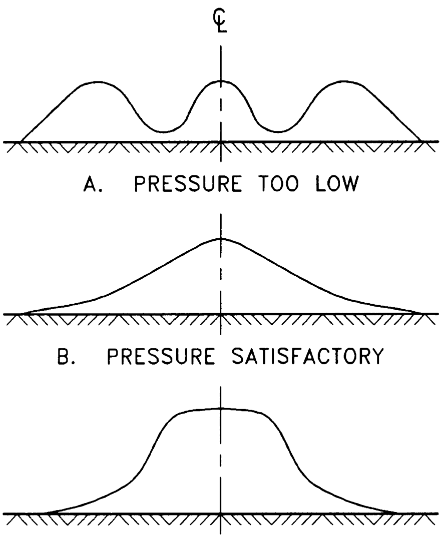


FIG. 5.5. Relative Effects of Different Pressures on Precipitation Profiles for a Typical Double Nozzle Sprinkler.

2. On the high side of the pressure range, the water from the nozzle breaks up into fine drops and settles around the sprinkler (see Fig. 5.5C). Under such conditions, the profile is easily distorted by wind movement; and
3. Within the desirable range, the sprinkler should produce a precipitation profile similar to Fig. 5.5B.

At a given pressure, large drops are obtained from large nozzles and fine sprays from small ones. All manufacturers of revolving sprinklers recommend operating pressures or ranges of pressures that will result in the most desirable application pattern for each combination of sprinkler and nozzle size.

Wind distorts the application pattern, and the higher the wind velocity, the greater the distortion. Figure 5.6 shows test results of an intermediate double-nozzle sprinkler operating under a wind velocity of 5 m/s (11.2 mph). This distortion must be considered when selecting the sprinkler spacing.

The depth of water applied to an area surrounding a revolving sprinkler varies as the distance from the sprinkler increases. Thus, to obtain a reasonably high degree of uniformity of application, water from adjacent sprinklers must be added. Figure 5.7, adapted form Christiansen (1942), illustrates the depth distribution obtained by overlapping the distribution patterns of adjacent sprinklers.

Manufacturers of sprinklers specify a wetted diameter for all nozzle-size and operating-pressure combinations for each type of sprinkler in their lines. As it is common for sprinkler-spacing recommendations to be made on the basis of

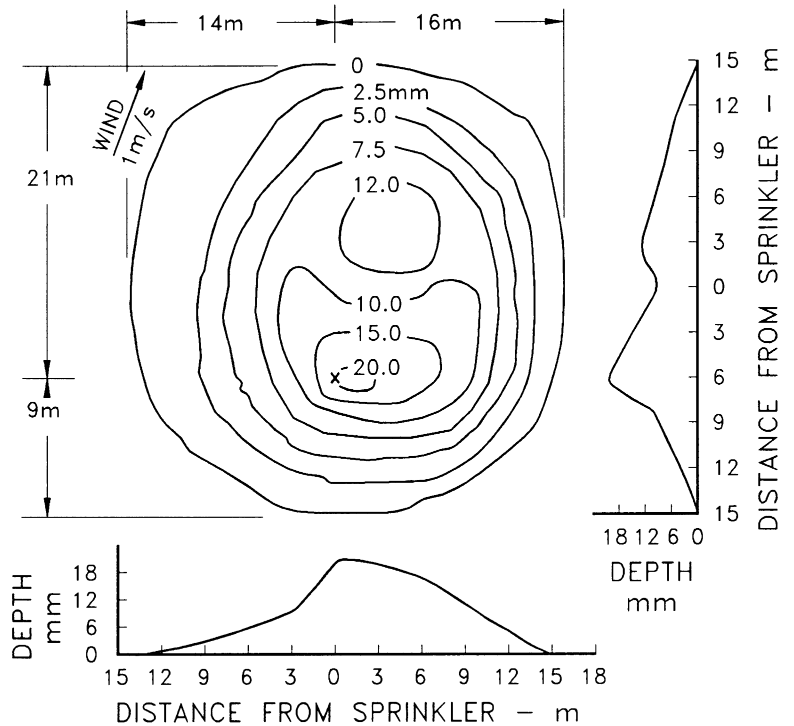


FIG. 5.6. Effect of Wind on Distribution Pattern and Precipitation Profiles from a Typical Intermediate Double Nozzle Sprinkler (1 m = 3.28 ft; 1 mm = 0.04 in.)

these diameters, they must be carefully considered by the planner. The precipitation profile is also important when making sprinkler-spacing recommendations.

Sprinklers operating in low winds produce characteristic precipitation profiles. Stylized profiles (Christiansen 1942) are shown in Fig. 5.8, along with spacing recommendations based on the diameter of effective coverage under the particular field conditions of operation. Conditions that affect both the diameter and profile of a sprinkler's precipitation pattern are: direction and velocity of the wind measured from the ground level to the top of the jet trajectory; height and angle of risers; turbulence in the stream of water entering and leaving the nozzle; pressure of the nozzle; and size of the nozzle. Characteristics of the sprinkler itself that affect its performance are the angle of stream trajectory and the design of the driving mechanism that determines the speed and uniformity of rotation. With such a complex set of conditions, the practical way to determine a sprinkler's profile type and diameter is to place catch gauges in the precipitation area and record and evaluate the results.

Profile types A and B are characteristic of sprinklers having two or more nozzles. Profile types C and D are characteristic of single-nozzle sprinklers at

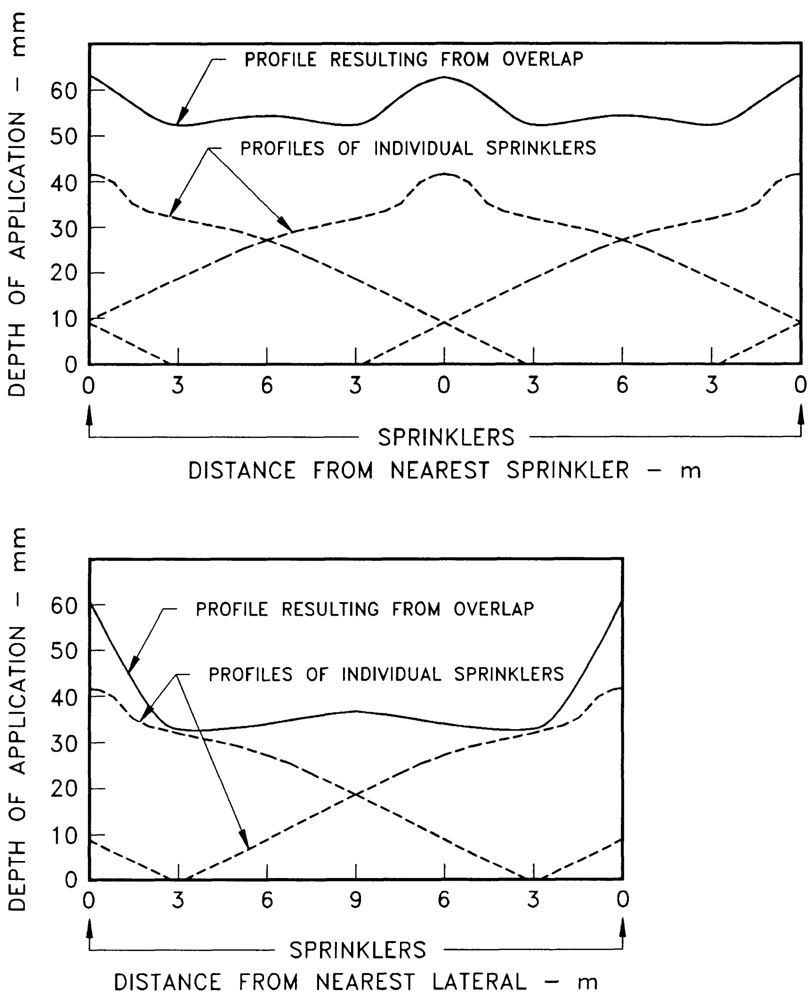


FIG. 5.7. Example of the Distribution Patterns Between Sprinklers Spaced 12 m Apart Along the Lateral with 18 m Spacing Between Laterals (1 m = 3.28 ft; 1 mm = 0.04 in.).

the recommended pressure. Profile type E is generally produced with gun sprinklers or sprinklers operating at pressures lower than those recommended for the nozzle size. Sprinklers with straightening vanes just upstream from the range nozzle also tend to produce a type E profile. The vanes increase the diameter of throw, but pressures must be increased by 70 to 105 kPa (10 to 15 psi) to keep the dip in the center of the profile from becoming too low.

The spacing recommendations in Fig. 5.8 should give acceptable application uniformities when a realistic effective diameter is used. Operating conditions in the field affect both the diameter and the precipitation profile. *Wind is the chief modifier reducing the diameter of throw and changing profiles* to a mixed type

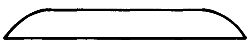
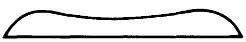
SPRINKLER PROFILE		RECOMMENDED SPACING AS A PERCENTAGE OF DIAMETER		
TYPE	SHAPE	SQUARE	TRIANGULAR EQUILATERAL	RECTANGULAR SHORT x LONG
A		50	50	40 x 60 to 65
B		55	66	40 x 60
C		60	65	40 x 60 to 65
D		40 70 (FAIR)	70 to 75	40 x 70 to 75
E		40 80 (FAIR)	80	40 x 80

FIG. 5.8. Christiansen's Geometrical Sprinkler Application Rate Profiles and Optimum Set Sprinkler Spacings as a Percentage of the Effective Wetted Diameter for Square, Triangular and Rectangular Layouts.

such as a short A or B type on the upwind side of the sprinkler, a D or E type downwind, and C type crosswind. (See Fig. 5.6.) The wetted diameters of sprinklers listed in manufacturers' brochures are usually based on tests in essentially no wind and are measured to where the application rate falls below 0.25 mm/hr (0.01 in./hr). Under field conditions *with 0 to 5 km/hr (0 to 3 mph) wind*, such *diameters should be shortened by 10% from the listed figure to obtain the effective diameter*. Effective diameters should be further reduced for winds over 5 km/hr (3 mph). *A reduction of 2.5% for each 1.6 km/hr (1 mph) over 5 km/hr (3 mph) is a fair estimate for the usual range of wind conditions under which sprinklers are operated.*

In general, highest uniformities are obtained at spacings of 40% or less of the wetted diameter, but such close spacings raise both precipitation rates and system cost. Overly narrow or wide spacings between lines can result in poor uniformities of coverage. Certain profile types, notably D and E, have a narrow range of lateral line spacings that give high uniformities. Thus, the uniformity can change drastically with changes in wind speed. In fact, when sprinklers with D and E profiles are closely spaced for windy conditions, the uniformity can actually decrease as wind velocity decreases because of too much overlap.

As a general recommendation, moderate- and intermediate-pressured sprinklers should be spaced as follows:

1. For a rectangular spacing use 40 by 67% of the effective diameter based on the average wind speed during the setting;
2. For a square spacing use 50% of the effective diameter based on average wind speed during the setting; and

3. For an equilateral triangular spacing use 62% of the effective diameter based on average wind speed during the setting.

Nozzle Size and Pressure

Table 5.2 lists the expected discharge and wetted diameters in conditions of no wind from typical 13- and 19-mm (1/2- and 3/4-in.) bearing impact sprinklers with angles of trajectory between 22 and 28° and having standard nozzles without vanes. The various values in the table are for different nozzle sizes between 2.4 and 5.6 mm (3/32 and 7/32 in.) and base of sprinkler pressures between 140 and 480 kPa (20 and 70 psi).

In general, the relationship between pressure or pressure head and discharge from a sprinkler can be expressed by the orifice equation:

$$q = K_d \sqrt{P} \tag{5.1a}$$

or

$$q = K_d \sqrt{H} \tag{5.1b}$$

where

q = sprinkler discharge, L/min (gpm)
K_d = appropriate discharge coefficient for the sprinkler and nozzle combined and the specific units used
P = sprinkler operating pressure, kPa (psi)
H = sprinkler operating pressure head, m (ft)

The relationship between the pressure, P, and the pressure head, H, are fully discussed at the beginning of Chapter 11.

The K_d can be determined for any combination of sprinkler and nozzle if any value of P and the corresponding q are known. Because of internal sprinkler friction losses, K_d decreases slightly as P and consequently q increase. However, over the normal operating range of most sprinklers, it can be assumed constant. The average values of K_d over the recommended range of operating pressures for each nozzle size are given in Table 5.2.

Equation 5.1a can be manipulated to give:

$$P = P'(q/q')^2 \tag{5.2}$$

where P' and q' are corresponding values that are known (from Table 5.2 or a manufacturer's table), and either q or P is not known. Equation 5.1b can be manipulated in a similar manner.

Table 5.2. Nozzle discharges and wetted diameters for typical 1/2- and 3/4-in. bearing impact sprinklers with trajectory angles between 22 and 28° and standard nozzles without vanes[1]

Sprinkler Pressure	Nozzle diameter—in. (mm)																	
	3/32 (2.4)		7/64 (2.8)		1/8 (3.2)		9/64 (3.6)		5/32 (4.0)		11/64 (4.4)		3/16 (4.8)		13/64 (5.2)		7/32 (5.6)	
	Nozzle discharge and wetted diameter																	
psi	gpm	ft	gpm	ft	gpm	ft	gpm	ft	gpm	ft	gpm	ft	gpm	ft	gpm	ft	gpm	ft
20	1.14	63	1.55	73														
25	1.27	64	1.73	76	2.25	76	2.88	79	3.52	82		[2]						
30	1.40	65	1.89	77	2.47	77	3.16	80	3.85	85	4.64	88	5.50	91	6.50	94	7.58	96
35	1.51	66	2.05	77	2.68	78	3.40	81	4.16	87	5.02	90	5.97	94	7.06	97	8.25	100
40	1.62	67	2.20	78	2.87	79	3.64	82	4.45	88	5.37	92	6.40	96	7.55	99	8.82	102
45	1.72	68	2.32	79	3.05	80	3.85	83	4.72	89	5.70	94	6.80	98	8.00	101	9.35	104
50	1.80	69	2.45	80	3.22	81	4.01	84	4.98	90	6.01	95	7.17	100	8.45	103	9.88	106
55	1.88	70	2.58	80	3.39	82	4.25	85	5.22	91	6.30	96	7.52	101	8.85	104	10.34	107
60	1.98	71	2.70	81	3.54	83	4.42	86	5.45	92	6.57	97	7.84	102	9.24	105	10.75	108
65					3.68	84	4.65	87	5.71	93	6.83	98	8.19	103	9.60	106	11.10	109
70					3.81	84	4.82	88	5.92	94	7.09	99	8.49	104	9.95	107	11.40	110
K_d^3	0.255 (1.151)[4]		0.346 (1.561)		0.453 (2.044)		0.575 (2.595)		0.704 (3.177)		0.849 (3.831)		1.012 (4.566)		1.193 (5.383)		1.394 (6.290)	

[1]The use of straightening vanes or special long discharge tubes increases the wetted diameter by approximately 5%.
[2]Horizontal lines represent upper and lower recommended pressure boundaries.
[3]$q = K_d\sqrt{P}$ with q in gpm and P in psi.
[4]Numbers in ( ) for $q = K_d\sqrt{H}$ with q in L/min and H in m.
NOTE: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 gpm = 3.785 L/min = 0.063083 L/s; 1 psi = 6.895k Pa = 0.703 m.

Now that the basic characteristics of sprinkler performance have been introduced, the preliminary design of set sprinkle systems can be addressed (see Fig. 1.2). The material presented from here through the end of Chapter 7 is needed for preliminary design as well as the final design (see Fig. 1.3).

PRELIMINARY DESIGN

The first six steps of the design procedure outlined earlier in this chapter are often referred to as the preliminary design factors. Sections I through IV of Fig. 5.2 are useful for organizing the information gained by carrying out these steps. Section V of the form is set up specifically for periodic-move and fixed sprinkle systems. The four columns in Fig. 5.2 enable the form to be used for different crops or for different fields on the same farm.

Gross Application Depth

When $LR \leq 0.1$ (as computed by Eq. 3.3) the unavoidable deep percolation losses will normally satisfy the leaching requirement. Therefore, the depth of application per irrigation, d, is computed by dividing the net depth, d_n, per irrigation by the irrigation efficiency. As mentioned earlier, the value selected for d_n should be equal to or less than d_x computed by Eq. 3.1. In addition, d_n will depend on system design and environmental factors and:

$$d = \frac{d_n}{E_a/100} \tag{5.3a}$$

When $LR > 0.1$ the depth of application per irrigation to satisfy both consumptive use and leaching requirements can be computed by:

$$d = \frac{0.9\ d_n}{(1.0 - LR)E_a/100} \tag{5.3b}$$

where

d = gross depth per irrigation application, mm (in.)
E_a = application efficiency, %

The 0.9 in Eq. 5.3b is included to account for the unavoidable deep percolation losses that normally will satisfy approximately 10% of the leaching needed. In Eq. 5.3 d_n can be replaced by d_x (as computed by Eq. 3.1) to give a preliminary maximum gross depth of irrigation. Furthermore, E_a can be taken from Table 4.1 or replaced by the computed application efficiency of the low half, E_h, or low quarter, E_q, which will be discussed in Chapter 6.

System Capacity Requirements

The required capacity of a sprinkle system depends on the size of the area irrigated (design area), the gross depth of water applied at each irrigation (computed by Eq. 5.3), and the net operating time allowed to apply this depth. The capacity of a system can be computed by the formula:

$$Q_s = K\frac{Ad}{fT} \tag{5.4}$$

where

Q_s = system discharge capacity, L/s (gpm)
K = conversion constant, 2.78 for metric units (453 for English units)
A = design area, ha (acres)
d = gross depth of application, mm (in.)
f = operating time allowed for completion of one irrigation, days
T = average actual operating time per day, hr/day

The value of f must be less than or equal to the irrigation interval, f', determined by Eq. 3.2. The value of d should be computed from Eq. 5.3. For fully automatic fixed systems, it is best to let d equal the gross depth required per day and $f = f' = 1.0$ days. To allow for some breakdown or moving of systems, T should be reduced by at least 5 to 10% from the potential value of 24 hr.

In this equation, d, f, and T are of major importance in that they have a direct bearing on the capital investment per acre required for equipment. From Eq. 5.4, it is obvious that the greater the operating time (fT) for applying a given depth, d, the smaller will be the system capacity and, therefore, the cost for a given design area, A. The *capacity* (and cost) of periodic-move systems designed to apply light (small d), frequent (small f) irrigations must be relatively large unless labor is available to move the system at night.

With center-pivot and automated fixed systems, light, frequent irrigations are quite practical, because both system capacities and labor requirements are minimal. With these systems, irrigation frequency should be based on maintaining optimum soil-plant-water conditions, rather than on allowing soil moisture depletion levels that are a compromise between optimizing labor requirements, capital costs, and growing conditions.

Before a sprinkle system is planned, the designer should thoroughly acquaint the owner with these facts, and together they should reach a clear understanding on the number of operating hours that can be allowed for completing one irrigation. Also, the farmer should understand the labor required to run the sprinkle system, so its operation offers minimal interference with other farming operations.

Sample Calculation 5.1 has been prepared as an example of the use of Eqs. 5.3 and 5.4 where a single crop is irrigated in the design area. The design moisture use rate and irrigation frequency can be obtained from irrigation guides where available. Otherwise, they may be computed from actual field data or estimated for preliminary design purposes from the data presented in Tables 3.1 through 3.4. In design areas containing more than one soil type, the design should be based on the dominant soil type. If different soils cover roughly equal areas, the soil with the lowest moisture-holding capacity should be the basis for design.

Sample Calculation 5.1 *Computing system capacity requirements for a single crop in the design area.*

GIVEN: Field of corn, $A = 16$ ha
Design moisture use rate, $U_d = 5$ mm/day
Moisture replaced in soil at each irrigation, $d_n = 60$ mm
Irrigation efficiency, $E_a = 75\%$
Irrigation period, $f = 10$ days in a 12-day interval
System operating time per day, $T = 20$ hr/day
Electrical conductivity of the irrigation water, $EC_w = 2.1$ dS/m

CALCULATIONS: From Sample Calculation 3.3 $LR = 0.20$; thus Eq. 5.3b should be used to compute the gross depth of water application per irrigation as:

$$d = \frac{0.9 \times 60}{(1 - 0.20) \times 75/100} = 90 \text{ mm (3.54 in.)}$$

Using Eq. 5.4 to compute the system capacity:

$$Q_s = \frac{2.78\, Ad}{fT} = \frac{2.78 \times 16 \times 90}{10 \times 20} = 20.0 \text{ L/s (317 gpm)}$$

Where two or more design areas with different crops are being irrigated by the same system, and peak design-use rates for the crops occur at about the same time of year, the capacity for each area is computed as shown in Sample Calculation 5.1. Capacities for each area are added to obtain the required capacity of the system. The days allotted for completing one irrigation over all areas (f) must be no longer than the shortest interval-frequency period, as computed by dividing the design soil water depletions allowed by the peak-water-use rate.

System capacity requirements for a design area in a crop rotation are calculated to satisfy the peak period of water use. The maximum requirement may, but does not always, occur when all crops in the rotation are being irrigated. Allowances must be made for the differences in time when the peak-use requirements for each crop occur (Sample Calculation 5.2).

Sample Calculation 5.2 *Computing capacity requirements for a crop rotation.*

GIVEN: Design area of 90 A with crop acreages and peak-use requirements as follows:

10 A Irish potatoes, last irrigation May 31
2.6-in. gross application lasts 12 days in May (peak period)
30 A corn, last irrigation August 20
2.9-in. gross application lasts 12 days in May
3.4-in. gross application lasts 12 days in July (peak period)
50 A alfalfa, irrigated through frost-free period
3.6-in. gross application lasts 12 days in May
4.3-in. gross application lasts 12 days in July (peak period)
Irrigation period f = 10 days in 12-day irrigation interval
System is to be operated 16 hr per day

CALCULATIONS: Using Eq. 5.4, the capacity requirements for May when all three crops are being irrigated is:

$$Q = \frac{453 \times 10 \times 2.6}{10 \times 16} = 74 \text{ gpm for Irish potatoes}$$

$$Q = \frac{453 \times 30 \times 2.9}{10 \times 16} = 246 \text{ gpm for corn}$$

$$Q = \frac{453 \times 50 \times 3.6}{10 \times 16} = 510 \text{ gpm for alfalfa}$$

$$\text{Total for May, } Q_s = 830 \text{ gpm}$$

Capacity requirements for July when potatoes have been harvested, but corn and alfalfa are using moisture at the peak rate:

$$Q = \frac{453 \times 30 \times 3.4}{10 \times 16} = 289 \text{ gpm for corn}$$

$$Q = \frac{453 \times 50 \times 4.3}{10 \times 16} = 609 \text{ gpm for alfalfa}$$

$$\text{Total for July, } Q = 898 \text{ gpm}$$

Although only two of the three crops are being irrigated, the maximum capacity requirement of the system is in July.

Besides the requirements for system capacity dictated by ordinary irrigation, other contingencies may enter into the calculation of system capacity.

Leaching. Most water is of good enough quality that no extra system capacity is required during the peak-use period for leaching. Leaching requirements can usually be adequately satisfied before and after the peak-use period. Therefore, the system capacity seldom needs to be increased to accommodate leaching. However, where relatively high-salinity irrigation water is to be used on salt-sensitive crops (when the conductivity of the irrigation water is more than half the conductivity values given in Table 3.5), it is advisable to provide a portion of the annual leaching requirement at each irrigation. In such cases the depth of each irrigation should be increased to provide for leaching, which is effectively accomplished by use of Eq. 5.3b, as demonstrated in Sample Calculation 5.1.

Wind. Under extremely windy conditions, the efficiency of sprinkle irrigation may be very low due to poor uniformity and excessive drift and evaporation losses. This is especially true with periodic-move systems on low-infiltration soils, which require low application rates. Therefore, during high-wind conditions, it may not be wise to irrigate. Because this reduces the effective number of sprinkling hours per day, system capacities must be increased proportionately.

Underirrigation. In water-short areas, it is sometimes practical to purposely underirrigate to conserve water at the expense of some reduction in potential yields. Optimum yields per unit of water applied often occur with system capacities about 20% lower than are specified for conventional periodic-move systems in the same area. Underirrigation is best achieved by using a larger irrigation interval than normally recommended for optimum yields.

Fixed systems. Fixed systems can be used for ordinary irrigation, high-frequency irrigation, crop cooling, and frost protection. Special consideration is required when estimating the system capacity needed by each of these uses. All fixed systems are ideal for applying water-soluble fertilizers and other chemicals.

Some fixed systems may be installed in permanent and other deep-rooted crops where relatively long irrigation intervals are employed. The capacity of such systems can be 5 to 10% less than conventional periodic-move systems covering the same area, because downtime is not needed for moving laterals. The capacity should be sufficient to apply the peak "net" crop water requirements for low-frequency (1- or 2-week interval) irrigations when the system is operated 24 hr per day, 7 days per week. These systems can be controlled by hand valves.

For fixed systems designed to apply irrigations once or twice a day to control soil temperatures and to hold the soil moisture content within a narrow band, a greater system capacity will be required. The net system capacity should be increased by 10 to 20% over conventional periodic-move systems. This additional capacity is necessary, because the crop will always be consuming water at the peak potential evapotranspiration rate. By contrast, under lower frequency irrigation, as the soil moisture decreases the consumptive use rate falls below the peak potential rate. Typically, the major justification for the high cost of a fixed system is to keep the crop performing at a peak rate to increase crop quality and yield. Clearly, crops that do not respond favorably to uniform high soil moisture conditions are not particularly good candidates for fixed systems. High-frequency systems can be operated with hand valves; however, automatic valving is attractive.

Intake and Optimum Application Rates

The rate at which water should be applied depends on the following:

- The infiltration characteristics of the soil, the field slope, and the crop cover;
- The minimum application rate that will produce a uniform sprinkler distribution pattern and satisfactory efficiency under prevalent wind and evaporative demand conditions; and
- The coordination of the lateral moves for periodic-move systems with other operations on the farm.

Drop impact tends to cause surface sealing and to reduce infiltration, especially on bare soils. The kinetic energy of a falling drop is the product of one-half its mass and the square of its velocity. With sprinkle irrigation, drop sizes typically range from 0.5 to 5.0 mm and have terminal falling velocities varying from about 2 to 22 m/s (6 to 72 ft/s), respectively. With a typical fall distance equivalent to about 3 to 6 m (10 to 20 ft), most drops come close to reaching their respective terminal velocities. Table 5.3 presents terminal velocities and kinetic energies associated with different drop sizes.

Drop size is reduced as pressure increases, as shown in Fig. 5.9, or as nozzle size decreases (Schleusener and Kidder, 1960). Drop sizes can also be reduced by using means other than high pressures to cause jet breakup. Some devices used to reduce drop size are the use of pins that penetrate the jet close to where it leaves the orifice; sharp orifices instead of tapered nozzles; triangular, rectangular, or oval orifices; and impinging jets. The interest in obtaining small drops without high pressures has been accelerated due to escalating energy costs.

Surface sealing and reduction in infiltration due to drop impact depend on the soil texture and structure, amount and type of crop cover, and the application

Table 5.3. Terminal velocities and kinetic energies associated with different-size raindrops

Drop diameter (mm)	Drop volume mm^3	Terminal velocity m/s	Kinetic energy values in relation to a 1.0-mm drop	Kinetic energy values per mm of rain Joules/m^2
0.5	0.07	1.8	0.03	1.6
1.0	0.5	3.8	1.0	7.3
1.5	1.8	5.3	6.5	14.1
2.0	4.2	6.5	22.8	20.8
2.5	8.2	7.3	57.0	26.6
3.0	14.2	7.9	115.7	31.1
3.5	22.5	8.4	205.0	34.8
4.0	33.5	8.7	332.0	37.6
4.5	47.8	8.9	499.0	39.8
5.0	65.5	9.1	707.5	41.2

1 m/s = 3.3 ft; 1 Joule/m^2 per mm = 1.74 ft-lb/ft^2 per in.

rate. Figure 5.10 (Levine, 1952) shows the general relation between drop size and reduction in infiltration rate on three different, freshly tilled bare soils for an application rate of approximately 13 mm/hr (0.5 in./hr). The reduction in infiltration rate on the clay loam soil approached the maximum level about 20 min after the beginning of application.

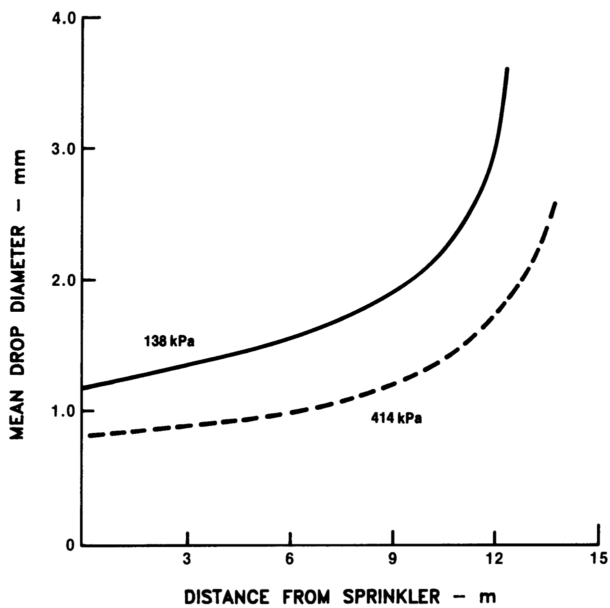


FIG. 5.9. Drop Sizes at Various Distances from a Standard 4 mm Nozzle Operating at 138 and 414 *kPa*.

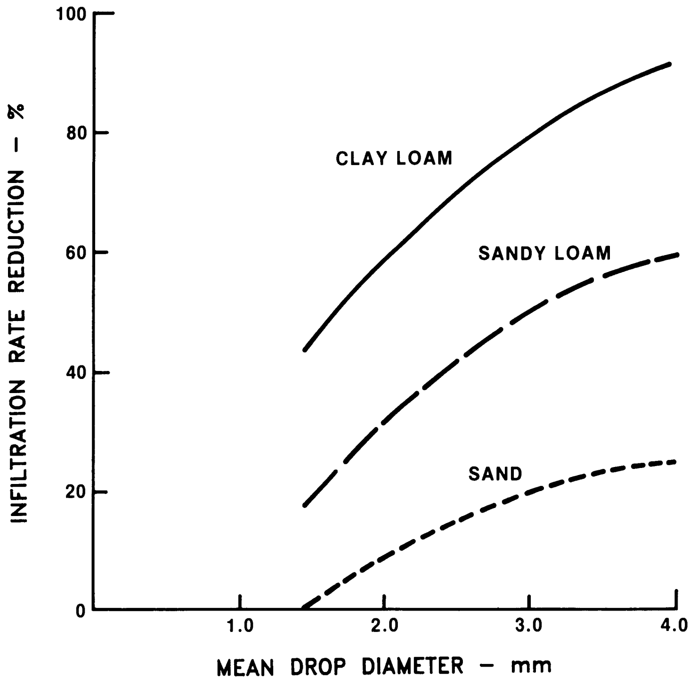


FIG. 5.10. Relation of Infiltration Rate Reduction Due to Sprinkling Three Different Freshly-tilled Soils at an Application Rate of Approximately 13 mm/hr.

Impact sprinklers produce a circular wetted area. At any one moment, all the water in the jet lands in a small segment of the total wetted area. Usually, the application rate on this area exceeds the infiltration capacity of the soil. The excess water momentarily ponds, forming a film on the soil surface that lubricates the surface soil particles. This also eliminates surface tension forces, which might otherwise help hold the surface soil grains in place. Droplets striking the ponded surface tend to dislodge silt and clay particles, which then become suspended. These particles settle out on the soil surface and are also carried into the soil profile by the infiltrating water, causing vertical erosion, surface sealing, and compaction. With coarse-textured soils, such as sands, surface sealing is usually not a problem because of good porosity and stability and the absence of silt and clay particles. However, surface sealing is often a problem on medium- and fine-textured soils with weak structures.

In all cases, the selected water application rate must fall somewhere between the maximum and minimum values set forth at the beginning of this section. Local irrigation guides should be used where available to obtain suggested values for maximum water-application rates for different combinations of soils, slopes, and cover. However, actual field data should be used for final design purposes. Maximum application rates for good ground cover should be used only when such cover can be preestablished and maintained.

Table 5.4. Suggested maximum sprinkler application rates for average soil, slope, and tilth

	Slope			
	0–5%	5–8%	8–12%	12–16%
	Maximum application rate			
Soil texture and profile	mm/hr (in./hr)	mm/hr (in./hr)	mm/hr (in./hr)	mm/hr (in./hr)
Coarse sandy soil to 1.8 m (6 ft)	50 (2.0)	38 (1.5)	25 (1.0)	13 (0.50)
Coarse sandy soils over more compact soils	38 (1.5)	25 (1.0)	19 (0.75)	10 (0.40)
Light sandy loams to 1.8 m (6 ft)	25 (1.0)	20 (0.80)	15 (0.60)	10 (0.40)
Light sandy loams over more compact soils	19 (0.75)	13 (0.50)	10 (0.40)	8 (0.30)
Silt loams to 1.8 m (6 ft)	13 (0.50)	10 (0.40)	8 (0.30)	5 (0.20)
Silt loams over more compact soils	8 (0.30)	6 (0.25)	4 (0.15)	2.5 (0.10)
Heavy textured clays or clay loams	4 (0.40)	2.5 (0.10)	2 (0.08)	1.5 (0.06)

Table 5.4 can be used for suggested maximum application rates for periodic-move systems. The table is based on average soil conditions for the irrigation of all crops except grasses and alfalfa. Application-rate values for slopes ranging from 0 to 16% are included in the table. For bare ground and poor soil conditions, the values should be reduced by about 25%. For grasses and alfalfa, the values may be increased by about 25%. In addition, application rates should be reduced by 25% for gun and some boom sprinklers, because they produce an abundance of large-diameter drops and have high instantaneous application rates.

For most irrigated crops, the minimum practical rate of application to obtain reasonably good distribution and high efficiency under favorable climatic conditions is about 3 mm/hr (0.12 in./hr). Where high temperatures and high wind velocities are common, the minimum application rate will be higher. Establishing minimum application rates for local conditions requires experience and judgment.

Once maximum and minimum rates of application have been determined, the designer of periodic-move systems needs to arrive at a final rate that will give set periods that fit into the farm operation schedule. For periodic-move systems, it is usually desirable to have intervals that give one, two, or, at most, three changes per day and that avoid nighttime changes. Changes just before or after mealtimes are normally preferred, because this leaves most of the day for other work. For fixed systems (especially automated ones) any number of changes per day can be achieved.

Computing Set Sprinkler Application Rates

The average application rate from a sprinkler is computed by:

$$I = \frac{Kq}{S_e \times S_l} \tag{5.5}$$

where

I = average application rate, mm/hr, (in./hr)
K = conversion constant, 60 for metric units (96.3 for English units)
q = sprinkler discharge, L/min (gpm)
S_e = spacing of sprinklers along the laterals, m (ft)
S_l = spacing of laterals along the main line, m (ft)

To compute the average instantaneous application rate, I_i, for a sprinkler having a radius of throw, R_j, and wetting an angular segment, S_a, Eq. 5.5 can be modified to:

$$I_i = \frac{Kq}{\pi (R_j)^2 \times S_a/360°} \tag{5.6}$$

where

K = same as for Eq. 5.5
R_j = radius of wetted area, m (ft)
S_a = angular segment (from a top view) wetted by a stationary sprinkler jet, degrees

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Sample Calculation 5.3 *Computing average and instantaneous application rates for set sprinkle systems.*

GIVEN: A typical impact sprinkler with a 4-mm (5/32-in.) nozzle operating at 345 kPa (50 psi) and discharging, q = 19 L/min (5.0 gpm)
Spacing of sprinklers along laterals, S_e = 9.1 m (30 ft)
Spacing of laterals along main line, S_l = 15.2 m (50 ft)

CALCULATIONS: From Eq. 5.5 the average application rate is:

$$I = \frac{60 \times 19}{9.1 \times 15.2} = 8.2 \text{ mm/hr (0.32 in./hr)}$$

If the above sprinkler produced a wetted radius of R_j = 13.7 m (45 ft), and the jet stream wetted an angular segment of S_a = 6°, then, by Eq. 5.6, the instantaneous application rate is:

$$I_i = \frac{96.3 \times 5.0}{\pi \times (45)^2 \times \dfrac{6}{360}}$$
$$= 4.5 \text{ in./hr (114 mm/hr)}$$

This is considerably higher than the infiltration rate of almost any agricultural soil except during the first moments of an irrigation.

Increasing sprinkler pressures or applying other means to reduce drop size tends to decrease the instantaneous application rate, I_i. The smaller drops and lower I_i work together to reduce surface sealing. The greatest drop impact and highest I_i is toward the periphery of throw and downwind from the sprinkler.

A jet of water rotating quickly over the soil surface will cause less sealing than a slower moving stream. A good rotational speed for the jet at the periphery of the wetted area is 1.5 m/s (5 ft/s), which is a typical walking speed of 5.6 km/hr (3.5 mph). Thus, a typical impact sprinkler that produces a 30-m (100-ft) wetted diameter should rotate about once a minute. However, a gun sprinkler, which wets an area over 120 m (400 ft) in diameter, should turn only once every 4 to 5 min.

REFERENCES

Christiansen, J. E. 1942. Irrigation by sprinkling. *California Agric. Exp. Stn.* Bull. 670. University of California, Berkeley.

Levine, G. 1952. Effect of irrigation droplet size on infiltration and aggregate breakdown. *Agricultural Engineering* 33(9):559–560.

McCulloch, A. W., J. Keller, R. M. Sherman, and R. C. Mueller, 1957. Revised by Keller, 1967. *Irrigation Handbook.* Milpitas, California: W. R. Ames Co.

Schleusener, P. E. and E. H. Kidder. 1960. Energy of falling drops from medium pressure sprinklers. *Agricultural Engineering* 42(2):100–103.

6

Set Sprinkler Uniformity and Efficiency

Irrigation efficiency is a concept used extensively in system design and management. It can be divided into two components, uniformity of application and losses. If either uniformity is poor or losses are large, efficiency will be low. Several factors affect the water-application efficiency of sprinkle irrigation systems:

- Variation of individual sprinkler discharge throughout the lateral lines. This variation can be held to a minimum by proper pipe network design or by employing pressure- or flow-control devices at each sprinkler or sprinkler nozzle.
- Variation in water distribution within the sprinkler-spacing area. This variation is caused primarily by wind. It can be partly overcome for set sprinkler systems by close spacing of the sprinklers. In addition to the variation caused by wind, there is a variability in the distribution pattern of individual sprinklers. The extent of this variability depends on sprinkler design, operating pressure, and sprinkler rotation.
- Loss of water by direct evaporation from the spray. Losses increase as temperature and wind velocities increase, and as drop size and application rate decrease.
- Evaporation from the soil surface before the water is used by the plants. This loss will grow proportionately lower as greater depths of water are applied.

UNIFORMITY

The uniformity of application is of primary concern in the sprinkle irrigation design procedure.

Uniformity Calculations

A useful term for placing a numerical value on the uniformity of application for agricultural irrigation systems is Distribution Uniformity, DU (Merrian and

Keller, 1978). The DU indicates the uniformity of application throughout the field and is computed by:

$$\mathrm{DU} = \frac{\text{Average low-quarter depth of water received}}{\text{Average depth of water received}} \times 100 \qquad (6.1)$$

The average low-quarter depth of water received is the average of the lowest one-quarter of the measured values, where each value represents an equal area.

Another parameter that is widely used to evaluate sprinkle irrigation uniformity is the coefficient of uniformity developed by Christiansen (1942):

$$\mathrm{CU} = 100\left(1.0 - \frac{\Sigma X}{nm}\right)$$

or

$$\mathrm{CU} = 100\left(1.0 - \frac{\Sigma|z - m|}{\Sigma z}\right) \qquad (6.2)$$

where

CU = coefficient of uniformity developed by Christiansen, %
z = individual depth of catch observations from uniformity test, mm (in.)
$X = |z - m|$ = absolute deviation of the individual observations from the mean, mm (in.)
$m = (\Sigma z)/n$ = mean depth of observations, mm (in.)
n = number of observations

The test data for CU > 70% usually forms a bell-shaped normal distribution and is reasonably symmetrical about the mean. Therefore, CU can be approximated by:

$$\mathrm{CU} \simeq \frac{\text{Average low-half depth of water received}}{m} \times 100 \qquad (6.2a)$$

and the relationship between DU and CU can be approximated by:

$$\mathrm{CU} \simeq 100 - 0.63(100 - \mathrm{DU}) \qquad (6.3a)$$

or

$$\mathrm{DU} \simeq 100 - 1.59(100 - \mathrm{CU}) \qquad (6.3b)$$

and the relationship between CU and the standard deviation, *sd*, of the individual depth of catch observations can be approximated by:

$$CU \simeq 100\left(1.0 - \frac{sd}{m}(2/\pi)^{0.5}\right) \tag{6.3c}$$

which can be rearranged to give:

$$sd \simeq \frac{m}{(2/\pi)^{0.5}}\left(1.0 - \frac{CU}{100}\right) \tag{6.3d}$$

Uniformity Problems

Even with nearly identical sprinklers operating simultaneously, the uniformity test values may vary by a significant percentage. Usually the accuracy of the catch data results in a deviation of ± 1 to 2%. In addition, the normal variation of CU, v_c (Solomon, 1978), and variation of DU, v_d, can be approximated by:

$$v_c = \pm[0.2(100 - CU)]\% \tag{6.4a}$$

$$v_d = \pm[0.2(100 - DU)]\% \tag{6.4b}$$

Some of the things that affect uniformity tend to average out during a series of irrigation applications. Other aspects of nonuniformity tend to concentrate, that is, the same areas tend to be over- or underirrigated during each irrigation application. The major concern is with those aspects that concentrate in the problem areas.

The components of uniformity in sprinkle irrigation systems that tend to smooth or cancel out, especially with hand-move systems, are:

1. Nonuniformity of operation of the sprinklers in periodic-move systems. This includes: variations in turning speed; variations in discharge between sprinklers caused by differences in nozzle size and wear; and irregularity of trajectory angle caused by riser straightness.
2. Nonuniformity of the lateral line set time for periodic-move systems will generally smooth itself out, especially if care is taken to do such things as to alternate between day and night sets.

One item that tends to smooth out, but also has some tendencies to concentrate, is:

3. Nonuniform aerial distribution of water between sprinklers. This is a function of overlap, sprinkler pattern shape, and wind effects on the over-

lap and pattern shape. Because the wind is usually different during each irrigation, there is some tendency for uniformity to improve over several irrigations. Also, management programs, such as alternating day and night sets and changing the lateral positions for each irrigation, smooth out some nonuniformities. In general, close sprinkle spacings give higher uniformities irrespective of wind conditions.

The following items tend to be additive, and thus they concentrate the under- and overwatering problems, causing poor uniformity:

4. Differences in sprinkler discharges throughout the system caused by elevation and friction loss.
5. Surface movement of water (both micro- and macrorunoff). Normally one thinks of all the water infiltrating into the soil where it falls; however, this is not always the case.
6. Poor water distribution around field boundaries. This is especially true in the case of boom or gun sprinklers that, by necessity, produce a poor watering pattern around all field boundaries. The outer 30 m (100 ft) of a 64-ha (160-A) gun sprinkler-irrigated field contains 15% of the field area. Tipping the risers outward to shorten the wetted radius or using part-circle sprinklers along the field edges can greatly improve the application uniformity around field boundaries.

Evaluating Sprinkler Uniformity

Most of the effort to evaluate sprinkle irrigation system uniformity and efficiency is done with "can" (catch container) tests. Such tests typically measure only the uniformity problems associated with Item 3 above. With close sprinkler spacings on set systems, a high level of uniformity, with DU values above 90%, is possible in the test area. However, the other problems causing lower uniformity reduce the highest practical overall DU to about 85%.

A low DU or CU value indicates that losses due to deep percolation may be large if adequate irrigation is applied to all areas. Although the concept of low values is relative, values of DU $< 60\%$ (CU $< 75\%$) are generally considered relatively low, even for general field and forage crops. For higher value crops a DU $> 75\%$ (CU $> 84\%$) is recommended. However, the optimum uniformity is determined by the economics of crop and applied water values, the crop response to water and deficits, and drainage economics.

Figure 6.1 shows the catch container layout for measuring the uniformity of distribution along a sprinkler lateral line. By overlapping the right- and left-hand catch data, the total catch between adjacent lateral positions can be simulated. In addition to collecting the catch data, the sprinkler discharge and pressure should also be determined during the field test (see Figs. 6.2 and 6.3).

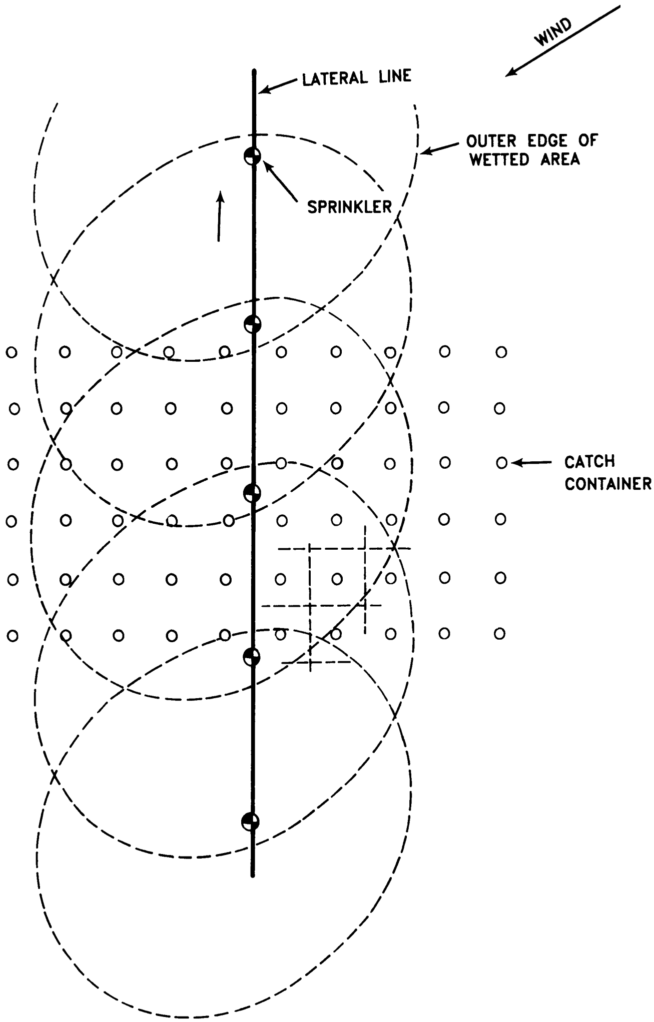


FIG. 6.1. Layout of Catch Containers for Testing the Uniformity of Distribution along a Sprinkler Lateral Line.

Complete details for conducting and evaluating field tests are presented in a handbook by Merriam and Keller (1978).

Test facility data from a single operating sprinkler can be used to simulate various sprinkler spacings along the lateral, S_e, as well as spacings between lateral lines, S_l. For such tests the simulated composite catch data can best be used for S_e and S_l values that are multiples of the can spacing used for the test. However, simple interpolation can be used with reasonable accuracy when the two spacings do not match. Complete details for conducting simple sprinkler tests for research purposes or performance reporting are presented in the Amer-

FIG. 6.2. Measuring Pressure at Sprinkler Nozzle with Gauge Connected to Pivot Tube.

FIG. 6.3. Measuring Sprinkler Discharge Using a Hose to Direct the Water into a Container of Known Volume.

ican Society of Agricultural Engineers Standards ASAE S330.1 or S398.1 (see Appendix C).

Sample Calculation 6.1. *Compare the distribution uniformity and Christiansen's Uniformity Coefficient from field test data.*

GIVEN: The field test data presented in Fig. 6.4. (For conversion to metric: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 gpm = 3.79 L/min; 1 gal = 3.79 L; 1 psi = 6.895 kPa.)

CALCULATIONS: Figure 6.5 shows the data (converted to in./hr) gathered between sprinklers 5 and 6 (see Fig. 6.4) and overlapped to simulate a 50-ft lateral spacing, $S_l = 50$ ft. The sprinklers were spaced 30 ft apart on the lateral. $S_e = 30$ ft; thus, the sprinkler spacing is referred to as a 30- by 50-ft spacing. The right-side catch is added to the left-side catch; the totals at each point represent a complete 1.0-hr irrigation for a 30- by 50-ft spacing. For the simulated 50-ft lateral spacing, the total of the catch rates at all 15 grid points is 3.97 in./hr, which gives:

$$\text{Average catch rate} = \frac{3.97}{15} = 0.265 \text{ in./hr}$$

The average of the lowest one-quarter of the catch rates (use 4 out of 15) is:

$$\text{Average low quarter rate} = \frac{0.20 + 0.22 + 0.22 + 0.23}{4} = 0.218 \text{ in./hr}$$

and from Eq. 6.1:

$$\text{DU} = \frac{0.218}{0.265} \times 100 = 82\%$$

To estimate the CU, the absolute deviations, $X = |z - m|$, of the individual observations from the mean must be determined, as shown by the numbers in parentheses on Fig. 6.5. The sum of these deviations is 0.51, and from Eq. 6.2:

$$\text{CU} = 100\left(1.0 - \frac{0.51}{3.97}\right) = 87\%$$

As mentioned earlier, the CU can be approximated from the average low-half and mean values of the observations by Eq. 6.2a:

$$\text{CU} \simeq \frac{1.86/8}{0.265} \times 100 = 88\%$$

Furthermore, the CU can also be approximated from the DU = 82% by Eq. 6.3a:

$$\text{CU} \simeq 100 - 0.63(100 - 82) = 89\%$$

1. Location *Field C-22*, Observer *JLM*, Date *9-30-75*
2. Crop *Tomatoes*, Root zone depth *4.0* ft, MAD *50* %, MAD *4.4* in
3. Soil: texture *clay loam*, available moisture *2.2* in/ft, SMD *4.4* in
4. Sprinkler: make *Rain Bird*, model *29B*, nozzles *5/32* by ____ in
5. Sprinkler spacing *30* by *50* ft, Irrigation duration *23.5* hrs
6. Rated sprinkler discharge *4.4* gpm at *40* psi giving *0.28* in/hr
7. Lateral: diameter *2* in, slope *1½* %, Riser Height *18* in
8. Actual sprinkler pressure and discharge rate:

	Sprinkler location number on test lateral					
	1	*4*	*5*	*6*	*10*	*15 end*
Initial pressure (psi)	*45*	*40*	*40*	*40*	*39*	*40*
Final pressure (psi)	*45*		*40*		*39*	*40*
Catch volume (gal)	*1.0*	*1.0*	*1.0*	*1.0*		*1.0*
Catch time (min or sec)	*0.21*	*0.22*	*0.22*	*0.22*		*0.22*
Discharge (gpm)	*4.8*	*4.6*	*4.6*	*4.6*		*4.6*

9. Wind: Direction relative to
   Part 10: initial ____, during ____, final ____
   Speed (mph): initial *2±*, during *5±*, final *5±*
10. Container grid test data in units of *ml*, Volume/depth *200* ml/in
    Container grid spacing *10* by *10* ft
    Test: start *2:55 pm*, stop *4:30 pm*, duration *1* hr *35* min = *1.58* hr

			32	*68*	*77*	*90*	6	*73*	*66*	*9*		*ml*
			.10	*.21*	*.24*	*.28*		*.23*	*.21*	*.03*		in./h
			35	*66*	*84*	*100*		*100*	*52*	*3*		
			.11	*.21*	*.26*	*.31*		*.31*	*.16*	*.01*		
			32	*50*	*70*	*104*		*99*	*48*	*12*		
			.10	*.16*	*.22*	*.32*	5	*.31*	*.15*	*.04*		
			31	*74*	*88*	*104*		*86*	*56*	*11*		
			.10	*.23*	*.27*	*.32*		*.27*	*.17*	*.03*		
			27	*64*	*80*	*96*		*112*	*62*	*9*		
			.08	*.20*	*.25*	*.30*		*.35*	*.19*	*.03*		
			20	*49*	*59*	*107*		*87*	*36*	*13*		
			.06	*.16*	*.19*	*.33*	4	*.27*	*.11*	*.04*		

11. Evaporation container: initial *2.15* final *2.10* loss *0.05* in
12. Sprinkler pressures: max *45* psi; min *39* psi, ave *40* psi
13. Comments *Test duration was too short. Depths caught measured in 1000 ml graduated cylinder. Wind velocities are less than normal.*

FIG. 6.4. Sprinkler-lateral Irrigation Evaluation Form with Field Data.

The deviation of the approximated values of CU from the value computed by Eq. 6.2 results from the small size of the sample and consequent deviation from a typical normal distribution.

Although the system was designed for a 50-ft lateral move, the effect on uniformity of choosing other move distances can also be evaluated from the field test data. Table 6.1 summarizes computations for DU and CU for four

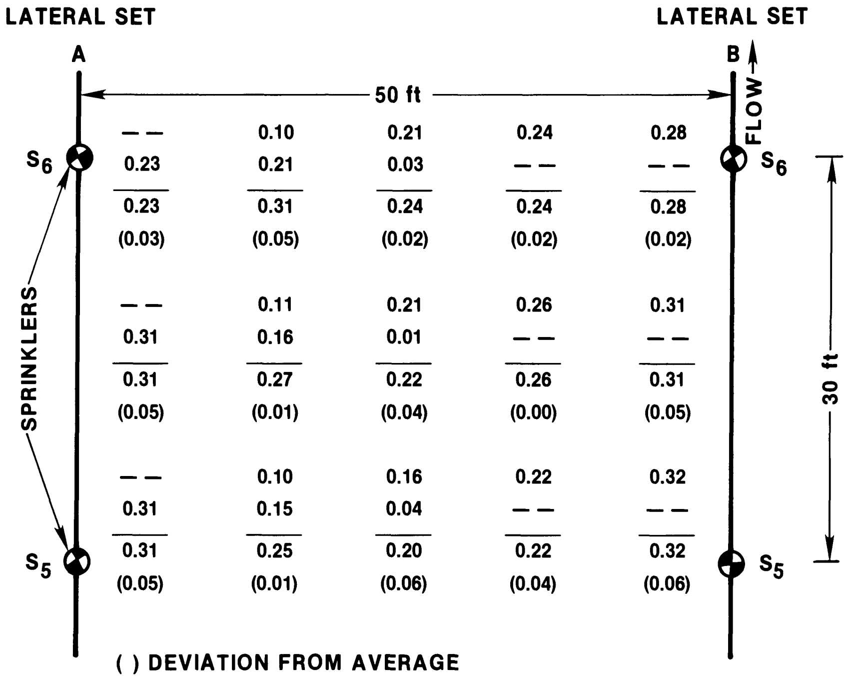


FIG. 6.5. Combined Catch Pattern Data (in in./hr) Between Sprinklers 5 and 6 for a 50-Foot Lateral Spacing.

typical lateral spacings, for the area between sprinklers 5 and 6 and the area between sprinklers 4 and 5. All these values have been computed as above from the data in Fig. 6.4, parts 8 and 10.

Comparing uniformity measurements illustrates the importance of choosing a representative site for evaluation. The application on some sites in a field is undoubtedly less uniform than on others. Therefore, it is important that the site selected for testing be useful for evaluating the entire system. As indicated by Eq. 6.4, even with nearly identical sprinklers operating simultaneously, the uniformity test values may vary by a significant percentage. Furthermore, the accuracy of the catch data themselves results in a deviation of $\pm$ 1 to 2%.

Evaluating System Uniformity

Nozzle discharge varies as the square root of the nozzle pressure, unless special, flexible-orifice nozzles are used to control flow. Figure 6.6B shows the relationship between discharge and pressure for a standard 4-mm (5/32-in.) nozzle that gives 19 L/min (5.0 gpm) at 330 kPa (48 psi) and for a flexible-orifice

Table 6.1. DU and CU values for four standard sprinkler spacings for areas between sprinklers 5 and 6 and sprinklers 4 and 5 in Fig. 6.4

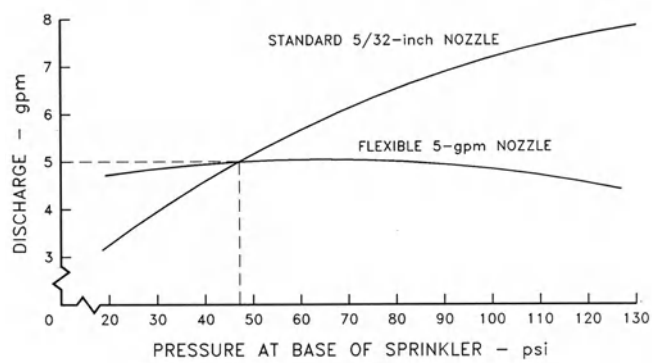
Test area criteria	Sprinkler spacing (ft) 30 × 40	30 × 50	30 × 60	30 × 60 alt[1]
	Area between sprinklers 5 and 6			
DU	81	84	64	91
CU	87	87	75	93
	Area between sprinklers 4 and 5			
DU	79	76	60	82
CU	86	89	70	91

[1]The alternate set values were computed using the test data for a 30- × 30-ft spacing.

nozzle designed to give approximately 19 L/min (5 gpm), over a range of pressures.

Unfortunately, it is difficult to manufacture the flexible-orifice nozzles precisely, and they may have up to ±10% variation in flow, even with uniform pressures. Thus, flexible-orifice nozzles should be used only when the difference in pressure throughout the system is expected to exceed about 25% of the desired average operating pressure, P_a. The flow variation when pressure reg-

A. Nozzle Operation (Source: Nelson Irrigation Corp.).



B. Comparison of Pressure Versus Discharge Relationship for a Standard Fixed Nozzle and a Flexible Orifice Nozzle.

FIG. 6.6. Flexible Orifice Nozzle Characteristics.

ulators are used at the base of each sprinkler with fixed nozzles is less than half as much. Therefore, pressure regulators will improve system performance when expected pressure variations exceed about 0.10 P_a.

The flow cross section of flexible-orifice nozzles reduces as the pressure increases (see Fig. 6.6B). Thus, they maintain constant flow without causing the 7- to 14-kPa (1- to 2-psi) pressure drop typical of even better pressure regulators used at the base of sprinklers. However, when pressures are above 550 kPa (80 psi), the jet breakup and wind drift from flexible-orifice nozzles may be excessive, and the sprinklers may turn erratically. Therefore, for such high-pressure operation, either pressure regulators or flexible flow-control orifices should be used at the base of the sprinklers.

When flexible-orifice nozzles are used, the DU and CU test values should be multiplied by approximately 0.90 to obtain the system uniformity. Where pressure regulators are used under each sprinkler, multiply by approximately 0.95 to obtain the system uniformity. (A detailed discussion on the use of *discharge-control devices* with each sprinkler and their effect on sprinkler discharge uniformity is presented in Chapter 14.)

When pressure-control devices are not used, pressure variations throughout the system may cause the overall uniformity to be lower than the uniformity in the test area. Sprinkle discharge varies as the square root of pressure (see Eq. 5.1), and CU varies as the average discharge in the low-half (see Eq. 6.2a). By assuming a linear distribution of pressure variations between the average and minimum sprinkler pressures, we can compute the system CU by:

$$\text{System CU} = \text{CU} \times \frac{1 + (P_n/P_a)^{0.5}}{2} \tag{6.5a}$$

And noting that DU varies as the average discharge of the low quarter (see Eq. 6.1), we can compute the system DU by:

$$\text{System DU} = \text{DU} \times \frac{1 + 3(P_n/P_a)^{0.5}}{4} \tag{6.5b}$$

where

P_n = minimum sprinkler pressure, kPa (psi)
P_a = average sprinkler pressure, kPa (psi)

Although the pressure distribution is not exactly linear between P_n and P_a, Eqs 6.5a and 6.5b give very practical and reasonable results. The P_a can be estimated by taking the average of a large representative sample of pressure readings throughout the system. (If the sample is large enough, the computed CU and DU of the estimated sprinkler discharges can be used in place of the entire second or fractional terms in Eqs. 6.5a and 6.5b, respectively.) With a limited number of pressure readings, P_a can be estimated as $(2P_n + P_x)/3$, where P_x is the maximum sprinkler pressure. For design purposes, P_a is always known, and P_n can be computed from friction loss and elevation data.

Sample Calculation 6.2. *Determine the System DU and CU.*

GIVEN: The data from Fig. 6.4, Part 12 and Sample Calculation 6.1.

CALCULATIONS: Using Eq. 6.5b with the test DU = 82%:

$$\text{System DU} = 82 \times \frac{1 + 3(39/40)^{0.5}}{4} = 81\%$$

and using Eq. 6.5a with the test CU = 87%:

$$\text{System CU} = 87 \times \frac{1 + (39/40)^{0.5}}{2} = 86\%$$

DESIGN CRITERIA

The leading manufacturers and independent testing agencies, such as CIT,[1] conduct sprinkler tests to obtain data on sprinklers operating under various simulated and actual field conditions. Such data should be used for planning purposes as a basis for selecting the combination of spacing, discharge, nozzle size, and operating pressure that will result in the highest practical DU or CU for the anticipated operating conditions. The data should be obtained from an independent source like CIT if they are available. Data from manufacturers should be used for backup.

Sprinkler Head Selection

Once manufacturer preference has been determined, actual sprinkler head selection is based on the discharge rate, height of trajectory, and sprinkler distribution characteristics desired. There is little difference between sprinkler selection for periodic-move and fixed sprinkler systems. The main exception is for permanent installations using buried pipe, where the sprinkler-spacing selection can be independent of the standard pipe lengths. Therefore, more economical

[1] Center for Irrigation Technology, California State Univ., Fresno, CA 93740-0018, phone (209) 394-2066.

systems can be designed with low-discharge sprinklers set at the widest practical spacings.

By keeping sprinkler discharge rates as low as possible while still using wide sprinkler spacings, the size and amount of pipe, as well as the requirement for labor, are kept to a minimum. Where soil surface sealing and infiltration are not limiting, such as over dense crop covers or on stable, permeable soils, the sprinkler giving the most economical overall system should be selected. However, when bare soil surfaces that tend to seal will be watered, sprinklers having small-diameter nozzles between 2.0 and 3.6 mm (5/64 and 9/64 in.) and operating at pressures over 345 kPa (50 psi) should be utilized.

Under-tree orchard systems require low-trajectory sprinklers to reduce foliar wetting and interference. Under-tree, rather than over-tree, sprinkling is required for such sensitive tree crops as citrus when the irrigation water is of low enough quality to cause leaf burn. In general, sprinklers that produce an E-type pattern (see Fig. 5.8) by throwing a large proportion of water to the outer perimeter of the wetted area produce the best under-tree results. This is because tree and foliar interference tends to deflect water close to the sprinklers where the application would otherwise be lightest.

On over-crop systems in very windy areas, low-angle sprinklers with a trajectory of 18 to 21° produce better results than higher angle sprinklers with 25 to 28° trajectories. Many sprinkler manufacturers have compromised on a trajectory angle of between 22 and 24° to achieve reasonable performance under various wind conditions. Where winds are always very low, high-angle sprinklers give the best results with a minimum of pressure.

Sprinkler Spacing

The basic criterion governing the selection of a spacing for any given sprinkler-nozzle-pressure and wind combination is the uniformity of distribution. In general, a CU of at least 85% is recommended for delicate and shallow-rooted crops, such as potatoes and most other vegetables. A CU between 75% and 83% is generally adequate for deep-rooted field crops, such as alfalfa, corn, cotton, and sugar beets. Tree and vine crops that have deep-spreading root systems can be adequately irrigated if the CU is above 70%. However, when applying chemicals through the system, a CU above 80% is recommended. Where systems have low CUs due to wind, chemicals should be applied only during calm periods.

Alternate Sets. Uniformity can be improved by positioning the laterals midway between the previous settings for alternate irrigations. This practice is called *alternate sets*, and the composite application uniformity is roughly equivalent to having a lateral spacing only half as wide for each pair of irrigations. The uniformity of a pair of irrigations using alternate sets can be approximated by:

$$CU_a = 10(CU)^{0.5} \tag{6.6a}$$

or

$$DU_a = 10(DU)^{0.5} \tag{6.6b}$$

For gun or boom sprinklers, CU values of 60 to 75% are typical for low and moderate wind conditions. These sprinklers are not recommended for use in high winds (or in arid areas). By using alternate sets along the lateral or between laterals when practical, CU_a values in the neighborhood of 80% can be obtained in the central portion of a field. However, Eq. 6.6 should be used with caution for low DU and CU values, especially where the duration between irrigations is relatively long. This is because the soil moisture depletion in the least watered areas may become excessive between the irrigations.

Interpretation of CU

Table 6.2 is presented to give a more useful meaning to the concept of CU. The water distribution efficiencies, DE, given in the body of the table represent

Table 6.2. Design water distribution efficiency values,[1] DE_{pa}, expressed as percentages for various CUs and percentages of land area adequately irrigated, *pa*[2]

	Land area adequately irrigated − *pa*, %								
CU, %	95	90	85	80	75	70	65	60	50
	Water distribution efficiencies − DE_{pa}, %								
94	88	90	92	9;	95	96	97	98	100
92	83	87	90	92	93	95	96	97	100
90	79	84	87	89	92	93	95	97	100
88	75	81	84	87	90	92	94	96	100
86	71	77	82	85	88	91	93	96	100
84	67	74	79	83	86	89	92	95	100
82	63	71	77	81	85	88	91	94	100
80	59	68	74	79	83	87	90	94	100
78	55	65	71	77	81	86	89	93	100
76	50	61	69	75	80	84	88	92	100
74	46	58	66	73	78	83	87	92	100
72	42	55	64	70	76	82	86	91	100
70	38	52	61	68	75	80	85	90	100
68	34	49	58	66	73	79	85	90	100
66	30	45	56	64	71	78	84	89	100
56	9	29	43	54	63	71	79	86	100

[1] Adapted from Hart and Reynolds (1965).
[2] Example: DE_{80} = 85% is the distribution efficiency when the water requirement at the time of irrigation is met or exceeded in 80% of the area irrigated for a CU = 86%.

values for different CUs assuming the water requirement at the time of irrigation is met on 95, 90, 85, . . . , or 50% of the irrigated area (Hart and Reynolds, 1965; and Hart et al., 1979). Thus, Table 6.2 combines the measurement of application uniformity with the concept of the area adequately irrigated to obtain a measure of distribution efficiency. For example, if a sprinkle system has a CU of 86%, then, from Table 6.2, $DE_{80} = 85\%$. This implies that for each unit, i.e., millimeter (inch) of the average application of water received by the crop or soil, 80% of the area would receive 85% of the average application or more, and 20% of the area would receive less than 85%. Expressed as a formula:

$$DE_{80} = \frac{\text{Minimum net depth received by wettest 80\% of area}}{\text{Average net depth received over entire area}}$$

To apply a net application depth of 1.0 unit of water to at least 80% of the area with a system having a CU of 86%, the average net application (after allowing for wind drift and evaporation losses) must be:

$$\frac{1.0}{85/100} = 1.18 \text{ units of water}$$

With a CU of only 70%, $DE_{80} = 68\%$, and an average net application of 1.47 would be required to apply a net depth of 1.0 or more units of water to 80% of the irrigated area.

Figure 6.7 illustrates the relation between surface area and depth of water applied at the CU values discussed above. Both 70 and 86% CU values leave 20% of the area underirrigated and adequately (or over-) irrigate 80% of the area. However, to do this requires a gross application of approximately 25% more water with the 70% CU than with the 86% CU. Data for constructing the curves in Fig. 6.7 were taken from Table 6.2. This was done by multiplying the average net applications by the appropriate DE values and assuming the curves to be symmetrical around the centerline.

It is interesting to note in Table 6.2 that when CU values are used as distribution efficiencies, the adequacy of irrigation will be approximately 80%, i.e., the values under the DE_{80} column correspond almost perfectly with the values under the CU column. In other words, $CU \simeq DE_{80}$. It can also be demonstrated that $DU \simeq DE_{90}$. For example, from Eq. 6.3b, when $CU = 86\%$, $DU \simeq 78\%$, and from Table 6.2, $DE_{90} = 77\%$ for $CU = 86\%$. Thus, when DU values are used as distribution efficiencies, the adequacy of irrigation will be approximately 90%.

Table 6.3 is presented to extend the understanding of CU to give a feel of relative productivity, especially when dealing with forage or other vegetative

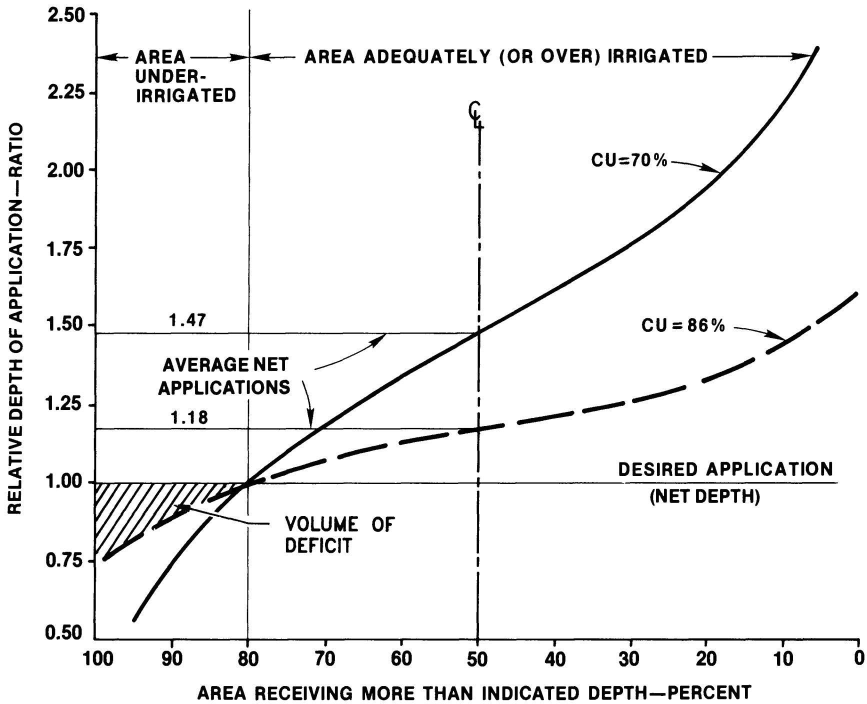


FIG. 6.7. Relation Between Surface Area and Depth of Water Applied for *CU* Values of 70 and 86% When 20% of the Area is Under-irrigated and the Remaining 80% of the Area Is Adequately (or Over-) Irrigated.

Table 6.3. Relative percentages of optimum productivity (where overwatering does not reduce yields) for various values of CU and percentages of land area adequately irrigated

	Land area adequately irrigated – %							
CU, %	95	90	85	80	75	70	65	60
	Relative production – $100(Y_a/Y_p)$, %							
90	100	99	99	98	98	97	97	96
86	100	99	98	98	97	96	96	95
82	99	99	98	97	96	95	94	93
78	99	98	97	96	95	94	93	91
74	98	97	96	95	94	93	91	90
70	98	97	95	94	92	91	90	89
64	97	97	95	94	91	90	88	87

crops. For such crops, yield is almost a linear function of available water, and overwatering does not reduce yield. (The relationship between water and yield for various crops is covered in a later section.)

The data in Table 6.3 demonstrate that nearly optimal yields may be obtained with a system having a low CU. For example, with a CU of 86 and 80% of the area adequately irrigated 98% of optimum yield might be obtained with an average net application 1.18 times the net depth required after allowing for wind drift and evaporation losses (refer to Fig. 6.7). With a CU of only 70%, 94% of the optimum yields might be obtained if 80% of the area were adequately irrigated. However, the average net application would need to be 1.47 times the adjusted net requirement. If CU = 70% and only 1.18 times the required net depth were applied, to give DE = 85%, the adequacy of the irrigation would be pa = 65% (see Table 6.2), and at best only 90% of optimum yields could be expected (see Table 6.3).

Design Uniformity

Sprinkler performance is a function of the sprinkler's physical characteristics as well as nozzle size and pressure. Therefore, the DU or CU values used for final design computations should be based on field or test facility data. However, for preliminary design or when test data are not available, Tables 6.4 to 6.7 can be used for planning purposes. These tables allow designers to obtain estimated values of CU for various wind conditions and application rates for the most common periodic-move sprinkler spacings.

The CU estimates presented in the tables were derived by W. C. Strong in 1961 from an analysis of numerous tests of impact sprinklers from various sources (McCulloch, 1967). They had 1/2- or 3/4-in. bearings, standard 22 to 28° trajectory angles, and nozzles without straightening vanes (to consolidate the jets). The four tables are separated according to wind speeds. Using vanes and/or lower trajectory angles between 8 to 21° may improve application uniformities in higher wind speeds. With vanes and/or low angles, Table 6.5 can be used with caution instead of Table 6.6 for 16- to 24-km/hr (10- to 15-mph) winds, and Table 6.6 can be used instead of Table 6.7 for 24- to 32-km/hr (15- to 20-mph) winds.

Sprinkle Head Discharge and Pressure Requirements

The required average discharge, q, of each sprinkler is a function of the average application rate, I, and the sprinkler spacing. The required application rate depends on time per set, net depth to be applied per irrigation, and application efficiency. It is only practical to change periodic-move laterals once or twice per day unless they are automated. For one change per day, the time per set will be 24 hr minus the length of time required to change the lateral position.

Table 6.4. A guide to recommended nozzle sizes and pressures with expected average CU values for different application rates and sprinkler spacings under *low wind* conditions [0 to 6.4 km/hr (0 to 4 mph)]

Sprinkler		Water application rate, in./hr ± 0.02 in./hr						
Spacing ft × ft	Operation	0.10	0.15	0.20	0.25	0.30	0.35	0.40
30 × 40	Nozzle, -inch	3/32	3/32	7/64	1/8	9/64	5/32	9/64 × 3/32
	Pressure, psi	30	50	45	45	45	40	40
	CU, %	82	83	82	83	83	85	88
30 × 50	Nozzle, -inch	3/32	7/64	1/8	9/64	5/32	11/64	11/64
	Pressure, psi	40	40	45	50	45	40	50
	CU, %	83	88	86	86	84	85	86
30 × 60	Nozzle, -inch		1/8	9/64	5/32	11/64	3/16	3/16
	Pressure, psi		40	45	45	45	45	50
	CU, %		88	88	89	88	85	87
40 × 40	Nozzle, -inch	7/64	1/8	9/64	1/8 × 3/32	5/32 × 3/32	5/32 × 3/32	5/32 × 1/8
	Pressure, psi	30	35	35	40	35	40	35
	CU, %	78	82	86	87	88	89	90
40 × 50	Nozzle, -inch			5/32	5/32 × 3/32	5/32 × 3/32	11/64 × 3/32	3/16 × 3/32
	Pressure, psi			35	35	45	40	40
	CU, %			78	83	84	88	89
40 × 60	Nozzle, -inch			5/32	11/64	3/16	13/64	7/32
	Pressure, psi			50	50	50	50	50
	CU, %			83	85	85	84	86
60 × 60	Nozzle, -inch			3/16	13/64	7/32	1/4	1/4
	Pressure, psi			60	65	65	50	65
	CU, %			88	88	88	88	88

NOTE: 1 in./hr = 25.4 mm/hr; 1 ft = 0.305 m; 1 psi = 6.895 kPa.

Table 6.5. A guide to recommended nozzle sizes and pressures with expected average CU values for different application rates and sprinkler spacings under *moderate wind* conditions [6.4 to 16 km/hr (4 to 10 mph)]

Sprinkler		Water Application rate, in./hr ± 0.02 in./hr						
Spacing ft × ft	Operation	0.10	0.15	0.20	0.25	0.30	0.35	0.40
30 × 40	Nozzle, -inch	3/32	3/32	7/64	1/8	9/64	5/32	9/64 × 3/32
	Pressure, psi	30	50	45	45	45	40	40
	CU, %	82	85	85	82	83	84	85
30 × 50	Nozzle, -inch	3/32	7/64	1/8	9/64	5/32	11/64	11/64
	Pressure, psi	40	40	45	50	45	40	50
	CU, %	70	75	84	84	84	87	85
30 × 60	Nozzle, -inch		1/8	9/64	5/32	11/64	3/16	3/16
	Pressure, psi		40	45	45	45	45	50
	CU, %		80	84	84	84	85	86
40 × 40	Nozzle, -inch	7/64	1/8	9/64	1/8 × 3/32	5/32 × 3/32	5/32 × 3/22	5/32 × 1/8
	Pressure, psi	30	35	35	40	35	40	35
	CU, %	80	83	83	83	84	87	86
40 × 50	Nozzle, -inch			5/32	5/32 × 3/32	5/32 × 3/32	11/64 × 3/32	3/16 × 3/32
	Pressure, psi			35	35	45	40	40
	CU, %			76	76	76	83	84
40 × 60	Nozzle, -inch			5/32	11/64	3/16	13/64	7/32
	Pressure, psi			50	50	50	50	50
	CU, %			77	81	83	84	85
60 × 60	Nozzle, -inch			3/16	13/64	7/32	1/4	1/4
	Pressure, psi			60	65	65	50	65
	CU, %			80	82	83	83	84

NOTE: 1 in./hr = 25.4 mm/hr; 1 ft = 0.305 m; 1 psi = 6.895 kPa.

Table 6.6 A guide to recommended nozzle sizes and pressures with expected average CU values for different application rates and sprinkler spacings under *high wind* conditions [16 to 24 km/hr (10 to 15 mph)]

Sprinkler		Water application rate, in./hr ± 0.02 in./hr						
Spacing ft × ft	Operation	0.10	0.15	0.20	0.25	0.30	0.35	0.40
30 × 40	Nozzle, -inch	3/32	3/32	7/64	1/8	9/64	5/32	5/32
	Pressure, psi	30	50	45	45	45	40	45
	CU, %	75	80	80	84	84	85	86
30 × 50	Nozzle, -inch		7/64	1/8	9/64	5/32	11/64	11/64
	Pressure, psi		40	45	50	45	45	55
	CU, %		70	81	82	87	88	88
30 × 60	Nozzle, -inch			9/64	5/32	11/64	3/16	3/16
	Pressure, psi			45	45	45	45	50
	CU, %			72	75	81	84	86
40 × 40	Nozzle, -inch		1/8	9/64	5/32	11/64	11/64	3/16
	Pressure, psi		35	35	35	35	45	45
	CU, %		80	82	81	80	86	85
40 × 50	Nozzle, -inch			5/32	5/32	11/64	3/16	13/64
	Pressure, psi			35	50	50	50	50
	CU, %			77	78	80	80	82
40 × 60	Nozzle, -inch			5/32	11/64	3/16	13/64	7/32
	Pressure, psi			50	50	50	50	50
	CU, %			68	74	78	81	82
60 × 60	Nozzle, -inch			3/16	13/64	7/32	1/4	1/4
	Pressure, psi			60	65	65	50	65
	CU, %			64	66	68	75	82

NOTE: 1 in./hr = 25.4 mm/hr; 1 ft = 0.305 m; 1 psi = 6.895 kPa.

Table 6.7. A guide to recommended nozzle sizes and pressures with expected average CU values for different application rates and sprinkler spacings under *extreme wind* conditions [24 to 32 km/hr (15 to 20 mph)]

Sprinkler		Water application rate, in./hr ± 0.02 in./hr						
Spacing ft × ft	Operation	0.10	0.15	0.20	0.25	0.30	0.35	0.40
30 × 40	Nozzle, -inch	3/32	3/32	7/64	1/8	9/64	5/32	5/32
	Pressure, psi	30	50	45	45	45	40	45
	CU, %	69	72	73	75	76	82	85
30 × 50	Nozzle, -inch			1/8	9/64	5/32	11/64	11/64
	Pressure, psi			45	50	45	45	55
	CU, %			74	77	80	81	84
30 × 60	Nozzle, -inch			9/64	5/32	11/64	3/16	3/16
	Pressure, psi			45	45	45	45	50
	CU, %			60	65	75	80	83
40 × 40	Nozzle, -inch			9/64	5/32	11/64	11/64	3/16
	Pressure, psi			35	35	35	45	45
	CU, %			70	72	76	81	84
40 × 50	Nozzle, -inch			5/32	5/32	61/64	3/16	13/64
	Pressure, psi			35	50	50	50	50
	CU, %			55	60	70	75	77
40 × 60	Nozzle, -inch			5/32	11/64	3/16	13/64	7/32
	Pressure, psi			50	50	50	50	50
	CU, %			64	70	73	74	75
60 × 60	Nozzle, -inch						1/4	1/4
	Pressure, psi						50	65
	CU, %						66	75

NOTE: 1 in./hr = 25.4 mm/hr; 1 ft = 0.305 m; 1 psi = 6.895 kPa.

This leaves a total set time of 23.0 to 23.5 operating hours. For two changes per day, set times will range between 11.0 and 11.5 hr.

The nozzle sizes and pressures in Tables 6.4 to 6.7 for each spacing will give application rates, I, that fall within 0.5 mm/hr (0.02 in./hr) of the rates indicated by the column headings. Equation 5.5 should be used to compute the precise flow rate needed for a given I. Then manufacturers' sprinkler tables should be consulted to determine the required operating pressure that will give the desired flow rate. Pressures for standard nozzles should be selected to fall within the following ranges:

Nozzle Sizes mm (in.)	Pressure Range* kPa (psi)
2.0 to 2.4 (5/64 to 3/32)	140 to 310 (20 to 45)
2.8 to 3.6 (7/64 to 9/64)	170 to 345 (25 to 50)
4.0 to 4.4 (5/32 to 11/64)	205 to 380 (30 to 55)
4.8 to 5.5 (3/16 to 7/32)	240 to 415 (35 to 60)

*When straightening vanes are used, add 35 kPa (5 psi).

The low side of the pressure ranges given above should be increased by 35 to 70 kPa (5 to 10 psi) when sprinkling bare soils that tend to seal. High pressures should be avoided to save energy and eliminate excessive drift and evaporation losses.

Riser Height

Riser pipes elevate and support the sprinklers above the crop and provide the connecting link to the lateral (see Fig. 6.3). They also help remove the turbulence set up when part of the flow in the lateral pipeline is diverted to an individual sprinkler. If not removed, this turbulence may carry through the nozzle and cause premature stream breakup and reduced diameter of coverage and hence produce a poorer distribution pattern (Wiersma, 1955). The length of pipe needed to remove turbulence varies with sprinkler discharge. Following are recommended *minimum* riser lengths (heights) for different discharges:

Sprinkler Discharge		Minimum Riser Height	
L/s	(gpm)	cm	(in.)
<0.6	(<10)	15	(6)
0.6 to 1.6	(10 to 25)	23	(9)
1.6 to 3.2	(25 to 50)	30	(12)
3.2 to 7.6	(50 to 120)	45	(18)
>7.6	(>120)	90	(36)

Most crops exceed 30 cm (12 in.) in height, so, except for clean, cultivated orchards where low riser pipes are desirable for under-tree sprinkling, the choice will be the minimum height to clear the crop. Some research studies indicate that 30 to 60 cm (12 to 24 in.) additional height improves the sprinkler distribution efficiency. However, there are obvious disadvantages to this, such as additional wind drift and problems with handling lateral pipes with long risers attached. Farmers usually prefer 45- to 60-cm (18- to 24-in.) risers except when irrigating higher growing crops, such as cotton and corn, or for fixed systems with buried laterals.

LOSSES

Although efforts are often concentrated on evaluating systems by dealing with uniformity problems, losses of water also reduce system efficiency. Frequently, designers assume that systems will be perfectly managed and losses will be minimal, but this is seldom so. Overwatering is perhaps the greatest cause of loss in any irrigation system. In addition to overwatering due to poor irrigation scheduling, the major sources of losses associated with sprinkle irrigation are: evaporation from droplets and wet soil surfaces, transpiration from unwanted vegetation, wind drift, field border losses, leaks, and system drainage.

Wind Drift and Evaporation

Wind drift and evaporation losses may be as little as a few percent when irrigating a crop with a full vegetative canopy in low winds. Under more common conditions, wind drift and evaporation losses range between 5 and 10%. However, under very severe conditions, they can be considerably greater.

Figure 6.8 has been developed as a guide for estimating the effective portion of the applied water that reaches the soil-plant surface. The values given for the effectiveness portion of the applied water for different potential evapotranspiration rates are based in part on the work by Frost and Schwalen (1955). A full plant canopy, high field-application efficiency, operating laterals spaced far apart, and the average of day and night application were assumed. The fine-spray curves are based on 4.8-mm (3/16-in.) nozzles operating at 415 kPa (60 psi) with a 12- by 18-m (40- by 60-ft) spacing. The coarse spray is for 4.8-mm (3/16-in.) nozzles operating at 210 kPa (30 psi) with a 9- by 18-m (30- by 60-ft) spacing.

To enter Fig. 6.8, it is necessary to know whether the spray from a sprinkler is coarse, fine, or somewhere in between. To make this determination, a Coarseness Index, CI, is used. This index can be calculated by the following method:

$$\mathrm{CI} = K \frac{P^{1.3}}{B} \qquad (6.7)$$

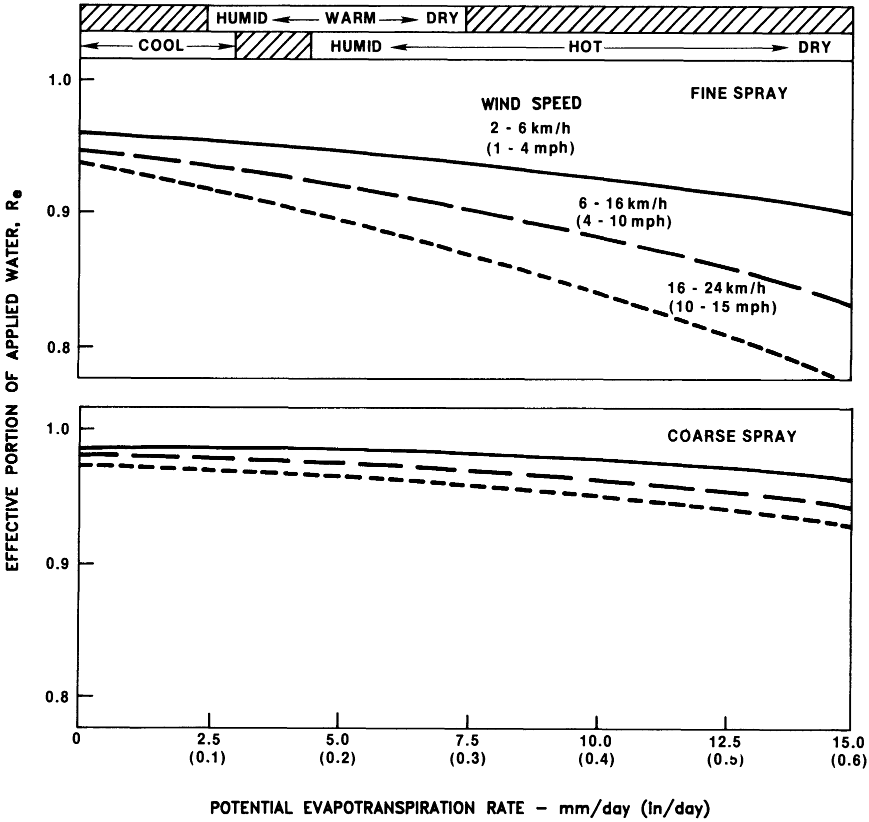


FIG. 6.8. Effective Portion of Water Applied by Sprinkling, R_e, Which Reaches Soil Surface Under Different Environmental and Spray Conditions.

where

P = nozzle operating pressure, kPa (psi)
B = nozzle diameter, mm (in.)
K = conversion constant, 0.032 for metric units (1/64 for English units)

If the value of CI $\leq$ 7, the spray is coarse, and the lower portion of Fig. 6.8 should be used to find R_e. If CI $\geq$ 17, then the spray is fine, and the upper portion of the figure should be used. When the value of CI falls between 7 and 17, the R_e value may be interpolated by:

$$R_e = \frac{(\mathrm{CI} - 7)}{10} (R_e)_f + \frac{(17 - \mathrm{CI})}{10} (R_e)_c \tag{6.8a}$$

Alternatively, the following regression equation describes and interpolates among all curves in Fig. 6.8:[2]

$$R_e = 0.976 + 0.005\ \mathrm{ET} - 0.00017\ \mathrm{ET}^2 + 0.0012\ \mathrm{WS}$$
$$-\mathrm{CI}(0.00043\ \mathrm{ET} + 0.00018\ \mathrm{WS} + 0.000016\ \mathrm{ET\ WS});$$
$$\text{for } 7 \le \mathrm{CI} \le 17$$
$$\text{If CI} < 7, \text{ let CI} = 7, \text{ and if CI} > 17 \text{ let CI} = 17 \qquad (6.8\text{b})$$

where

R_e = effective portions of water emitted from sprinklers, most of which reaches the irrigated soil surface, decimal
$(R_e)_c$ = R_e value from Fig. 6.8 from coarse spray curves
$(R_e)_f$ = R_e value from fine spray curves
ET = potential or reference evapotranspiration or water consumptive use rate, mm/day (for English units 1.0 in./day = 25.4 mm/day)
WS = wind speed, km/hr, (for English units 1.0 mph = 1.6 km/hr)
CI = coarseness index from Eq. 6.7

Leaks and Drainage Losses

For well-maintained systems, leaks and drainage losses can be held to less than 1% of system capacity. Thus, the ratio of the water effectively discharged through the sprinklers to the total system discharge, $0.99 < O_e < 1.0$. In buried (permanent) systems these losses can be eliminated by using antidrain valves at the sprinklers, so $O_e = 1.0$. However, poorly maintained systems have been known to have leakage and drainage losses of up to 10%, giving $O_e = 0.9$. Major areas where leaks occur are at sprinkler bearings and couplers. Excess applications due to eroded (enlarged) nozzles might also be thought of as leakage.

EFFICIENCY

Useful application efficiency terms for agroirrigation have been summarized by the On-farm Irrigation Committee of ASAE (1978). Perhaps the most often used irrigation efficiency term is the "Classical Field Application Efficiency," E'_a.

[2] Developed in 1988 by K. M. Fisher and R. G. Allen, Agric. and Irrig. Eng. Dept., Utah State Univ., Logan, UT.

This is the ratio of the average depth of irrigation water available for ET to the gross depth of irrigation water delivered to the field.

By itself E'_a only indicates the losses, because it merely shows the fraction of delivered water stored within the root zone that is potentially accessible for evaporation and transpiration. Thus, E'_a gives no indication of the adequacy of the irrigation and, with exaggerated underirrigation, it can equal 100%.

Typically a slightly different application efficiency term E_a is used in sprinkle irrigation design. This is the E_a term used in Eq. 5.3, to determine gross depth of water required, d, from the net depth of irrigation desired, d_n. Values of the design E_a are selected with a "general feeling," for some level of adequacy, rather than a specific one. To be more useful, an irrigation efficiency concept should combine some measure of uniformity and adequacy of irrigation as well as losses.

General Sprinkle Application Efficiency

A general equation for computing d, given d_n, should have an application efficiency term that includes the effects of losses due to nonuniformity of application, deep percolation, spray drift and evaporation, and pipe leakage. The term that will be designated the designer E_{pa} should also be specific as to adequacy—the percentage area, pa, receiving the desired d_n.

The designer E_{pa} for any percentage of the area adequately irrigated can be computed by:

$$E_{pa} = DE_{pa} R_e O_e \tag{6.9}$$

where

E_{pa} = application efficiency based on adequately irrigating a percentage, pa, of the field, %
DE_{pa} = distribution efficiency for the desired percentage adequacy, pa, (computed by Eq. 6.10 or taken from Table 6.2), %
R_e = effective portion of applied water from Eq. 6.8, decimal
O_e = ratio of water effectively discharged through sprinkler orifices or nozzles to total system discharge, decimal

For computing DE_{pa} use the equation:[3]

$$DE_{pa} = 100 + [606 - 24.9\,pa + 0.349\,pa^2 - 0.00186\,pa^3] \cdot (1 - \mathrm{CU}/100) \tag{6.10a}$$

[3] Developed in 1987 by R. G. Allen, Agric. and Irrig. Eng. Dept., Utah State Univ., Logan, UT.

or

$$DE_{pa} = 100 - 1581(1 - \mathrm{CU}/100) + 1581 \exp\left[-\left(\frac{pa - 40}{150}\right)^2\right] \cdot (1 - \mathrm{CU}/100) \qquad (6.10b)$$

The E_{pa} is not a "Classical Application Efficiency," E'_a, for if it were, d_n would be the minimum depth applied over the entire field. This is clearly not so, as implied by pa, which sets the limit on the portion that will be allowed to remain underirrigated. But underwatering some portion of the field is practical, because for the entire field to receive a minimum depth of water equal to d_n would be very difficult (except for very high CU values). For example, even with the relatively high CU of 86% in Fig. 6.7 the gross application would need to be increased from 1.18 to 1.57 units of water. This will be necessary for all of the field to receive the desired 1.0 units of water.

A "classic" version of E'_{pa} is also useful. Values for the classic version can be completed by replacing DE_{pa} in Eq. 6.9 with the classic version of DE'_{pa}. DE'_{pa} values can be taken from Table 6.8 or computed using:[3]

$$DE'_{pa} = 100 + [432 - 21.3\,pa + 0.323\,pa^2 - 0.001785\,pa^3] \cdot (1 - \mathrm{CU}/100) \qquad (6.11)$$

The difference between DE_{pa} and DE'_{pa} values for the same CU and pa values represents the difference between the desired d_n and the average net depth available to satisfy crop water-use requirements, d'_n. The value of d'_n will always be less than d_n when some portions of the field are underirrigated. When Tables 6.2 and 6.8 are compared, the differences between DE_{pa} and DE'_{pa} are small for high uniformities and/or adequacies of irrigation, but are quite large for the lower uniformities and adequacies.

Effect on Yield

Typically, E_{pa} is used for design purposes to determine d given d_n as in equations like Eq. 5.3. In such cases the ratio d'_n/d_n represents the resulting absolute adequacy of each irrigation, as well as the total irrigation for the season in terms of satisfying crop water-use requirements. In desert areas with crops for which yield is directly proportional to ET and on soils where overwatering is not a problem, the ratio of d'_n/d_n gives the actual yield, Y_a, compared to potential yield, Y_p. Obviously, $d'_n/d_n = DE'_{pa}/DE_{pa} = E'_{pa}/E_{pa}$ for the same values of uniformity and pa. The relative production percentage values given in Table 6.3 are equal to this ratio.

Table 6.8. Classic water distribution efficiency values, DE'_{pa}, expressed as percentages for various CUs and percentages of land area adequately irrigated, *pa*

CU, %	Land area adequately irrigated − *pa*, %							
	95	90	85	80	75	70	65	60
	Water distribution efficiencies − DE'_{pa}, %							
94	87	90	92	93	94	95	95	96
92	83	87	89	90	92	93	94	95
90	79	83	86	88	90	91	92	93
88	75	80	83	86	88	89	91	92
86	71	77	81	83	86	88	89	91
84	67	73	78	81	84	86	88	89
82	62	70	75	79	82	84	86	88
80	58	67	72	76	79	82	85	87
78	54	63	69	74	77	80	83	85
76	50	60	67	71	75	79	82	84
74	46	57	64	69	73	77	80	83
72	42	53	61	67	71	75	78	81
70	37	50	58	64	69	73	77	80
68	33	47	55	62	67	72	75	79
66	29	44	53	60	65	70	74	77
56	8	27	39	48	55	61	66	71

Actually, there are few crops for which the relative production is directly proportional to d'_n/d_n; furthermore, in many areas a significant part of the crop water requirements is from precipitation. Thus, a more comprehensive relationship for estimating actual yield is warranted. The following relationship, which is adapted from Doorenbos and Kassam (1979), is quite simple and accounts for both crop response and effective rain:

$$Y_a = Y_p \left(1 - k_y + k_y \frac{D'_n + R_n}{D_n + R_n}\right) \tag{6.12}$$

where

Y_a = estimated actual crop yield, units/ha (units/A)
Y_p = expected or potential yield with no water deficit, units/ha (units/A)
k_y = specific yield response factor for each crop taken from Table 6.9, decimal
D'_n = average actual (or classic) net seasonal depth of irrigation water applied and available for crop use, mm (in.)
D_n = average (design) net seasonal depth of irrigation water applied and required, mm (in.)
R_n = effective precipitation available for crop use, mm (in.)

Table 6.9. Specific yield response factors,[1] k_y, for average water deficits during the total growing season

Crop	Response Factor − k_y	Crop	Response Factor − k_y
Alfalfa	0.7–1.1	Potato	1.1
Banana	1.2–1.35	Safflower	0.8
Bean	1.15	Sorghum	0.9
Cabbage	0.95	Soybean	0.85
Citrus	0.8–1.1	Sugarbeet	0.7–1.1
Cotton	0.85	Sugarcane	1.2
Grape	0.85	Sunflower	0.95
Groundnut	0.7	Tobacco	0.9
Maize	1.25	Tomato	1.05
Onion	1.1	Watermelon	1.1
Pea	1.15	Wheat, spring	1.15
Pepper	1.1	Wheat, winter	1.0

[1] Adapted from Doorenbos and Kassam (1979).

When using Eq. 6.9 with k_y values taken from Table 6.9 it is assumed that irrigation is precise. It is also assumed that the R_n used is what will actually be available to the crop (and not include deep percolation losses). Where the expected yield, Y_p, is uncertain, Eq. 6.9 can be used to obtain Y_a/Y_p in order to obtain a relative evaluation of different design criteria.

Special Sprinkle Application Efficiencies

The values in Table 6.2 are based on the statistical (mathematical) relationships between CU and corresponding DE_{pa} values, assuming the overlapped patterns are represented by a ''normal'' distribution function. As mentioned earlier, it is interesting to note the close relationship between DE_{80} and CU for all values of CU. Thus, when CU is used in place of DE_{pa} in Eq. 6.9, the computed application efficiency is approximately equal to E_{80}. A similar relationship holds true for values of DU and DE_{90}.

Using DU in place of DE_{pa} in Eq. 6.9 gives what will be called the *design* application efficiency of the low quarter, E_q. The E_q is a useful term for placing a numerical value on irrigation efficiency for *medium- to high-value crops.*

When the soil moisture deficit, SMD, is divided by E_q to determine the gross depth of irrigation, d, only about 10% of the area will remain below field capacity, FC. Conversely, about 90% of the area will be adequately irrigated and will receive varying amounts of overirrigation, as discussed previously. Though this is practical for medium- to high-value crops, it may be unjustified for lower value field and forage crops. For such crops, an application efficiency based on the average low-half depth is usually more appropriate. For design purposes, the application efficiency of the low half, E_h, can be estimated by using CU in place of DE_{pa} in Eq. 6.9. Thus, when E_h is used to estimate d to replenish a given SMD, only about 20% of the area will remain below FC (underirrigated).

The range of probable E_q and E_h values for the various types of set sprinkler systems are:

Type	E_q	E_h
Periodic-move lateral	60 to 75%	70 to 85%
Gun or boom sprinklers	50 to 60%	60 to 75%
Fixed lateral	60 to 85%	70 to 88%

The above efficiency values are based on crops with full canopies and systems that are well-designed and carefully maintained. The values are merely estimates and should be considered accordingly. Obviously, considerably lower values would be obtained with poor management or where systems are poorly designed or ill-suited to the prevailing conditions.

PRELIMINARY DESIGN COMPUTATIONS

Figure 6.9 shows a copy of Fig. 5.2 filled in for a sample field of alfalfa and potatoes. Sample Calculations 6.3 and 6.4 illustrate the procedure for determining the desired application rate, I, and related average sprinkle discharge, q, for the alfalfa field and the potato field, respectively.

Sample Calculation 6.3. *Preliminary design computations for alfalfa.*

GIVEN: The information in Parts I and II of Fig. 6.9 for alfalfa where the average wind speed is 4 to 10 mph and:

The soil moisture depletion, MAD = 50%;
There will be one set change per day;
The sprinkler spacing is 40 × 60 ft;
The leaching requirement, LR = 0.05;
System leakage is insignificant; therefore, $O_e \simeq 1.0$.

FIND: The net depth per irrigation, irrigation interval, irrigation efficiency, application rate, and sprinkler discharge requirement.

CALCULATION: For an MAD = 50%, the allowable soil water depletion, d_x, is 50% of the total available water-holding capacity of the root zone. From Eq. 3.1 it is:

$$d_x = 6 \text{ ft} \times 2.0 \text{ in./ft} \times \frac{50}{100} = 6.0 \text{ in.}$$

		Alfalfa	Potato		
I. CROP (TYPE)		*Alfalfa*	*Potato*		
(a) Root depth – mm (ft)	z	*6*	*2.5*	*Table 3.2*	
(b) Growing season – days		*165*	*135*	*From an*	
(c) Water use rate – mm/day (in./day)	U_d	*0.30*	*0.25*	*Irrigation*	
(d) Seasonal water use – mm (in.)	U	*30.0*	*19.0*	*Guide*	
II. SOILS [Area – ha (A)]			*80*		
(a) Surface texture depth – cm (ft) Moisture capacity – mm/m (in./ft)	W_a	*Loam* *8* *2.0*	*Sandy loam 4* *1.6*	*Table 3.1*	
(b) Subsurface texture depth – cm (ft) Moisture capacity – mm/m (in./ft)	W_a	*same*	*same*		
(c) Moisture capacity – mm (in.)		*12.0*	*4.0*		
(d) Allowable depletion – mm (in.)	d_x	*6.0*	*2 0*	*50% of Total*	
(e) Intake rate – mm/hr (in./hr)		*0.6*	*0 4*	*Table 5 4*	
III. IRRIGATION					
(a) Interval – days	f′	*20*	*8*		
(b) Net depth – mm (in.)	d_n	*6.0*	*2.0*		
(c) Efficiency – %	E_a	*79*	*75*		
(d) Gross depth – mm (in.)	d	*7.6*	*2.7*		
IV. WATER REQUIREMENT					
(a) Net seasonal – mm (in.)	U	*30*	*19*		
(b) Effective rain – mm (in.)	R_n	*3*	*2*		
(c) Stored moisture – mm (in.)	M_s	*5*	*2*		
(d) Net irrigation – mm (in.)	D_n	*22*	*15*		
(e) Gross irrigation – mm (in.)	D_g	*28*	*20*		
(f) Number of irrigations		*3 to 4*	*7 to 8*		
V. SYSTEM CAPACITY					
(a) Application rate – mm/hr (in./hr)	I	*0.33*	*0.23*		
(b) Time per set – hr	T_a	*23*	*11.5*		
(c) Settings per day		*1*	*2*		
(d) Days of operation per interval	f	*18*	*8*		
(e) Preliminary system capacity – L/s (gpm)			*532*		

NOTE: 1 ft = 0.305 m; 1 in. = 25.4 mm; 1 gpm = 0.0632 L/s

FIG. 6.9. Preliminary Set Sprinkler Irrigation System Design Factors with Data in (English Units).

The maximum allowable irrigation interval during the peak use period from Eq. 3.2 is:

$$f_x = \frac{\text{allowable depletion (in.)}}{\text{water-use rate (in./day)}} = \frac{6.0}{0.30} = 20 \text{ days}$$

This equation uses the maximum allowable depletion to give the corresponding maximum interval during the peak-use period that will give the desired level of productivity. To fit the final system design, lesser net applications and correspondingly smaller intervals may be used.

The application efficiency can be estimated from the effective portion of the applied water R_e, and the uniformity of application. Assuming the spray will be midway between coarse and fine, from Fig. 6.8, for a potential evapotranspiration rate of 0.3 in./day, the effective portion is:

$$R_e = \frac{0.97 + 0.91}{2} = 0.94$$

Because alfalfa is a relatively low-value crop, an application efficiency, E_h, based on the average low-half depth is appropriate, i.e., use CU as the measure of distribution uniformity. Assuming an E_h of 75% from Table 4.1, the preliminary gross application depth, (d), by Eq. 5.3a (for LR < 0.1) is:

$$(d) = \frac{6.0}{75/100} = 8.0 \text{ in.}$$

Assuming it will take 1 hr to change the position of a hand-move lateral, the time per set with one change per day will be 23 hr. Thus, the preliminary application rate, (I), is:

$$(I) = \frac{8.0 \text{ in.}}{23 \text{ hr}} = 0.35 \text{ in./hr}$$

From Table 6.5 for 4- to 10-mph winds, the anticipated CU = 84% on a 40- × 60-ft spacing with water applied at 0.35 in./hr. A more specific estimate of CU can often be obtained directly from a supplier. The expected application efficiency can be estimated in Eq. 6.9 by substituting CU for DE_{pa} to give:

$$E_h = \text{CU} \times R_e \times O_e = 84 \times 0.94 \times 1.0 = 79\%$$

The required gross application can now be more accurately computed as:

$$d = \frac{6.0}{79/100} = 7.6 \text{ in.}$$

and the required application rate is:

$$I = \frac{7.6 \text{ in.}}{23 \text{ hr}} = 0.33 \text{ in./hr}$$

The required sprinkler discharge can now be calculated by Eq. 5.5:

$$q = \frac{I(S_e \times S_l)}{96.3}$$

$$= \frac{0.33 \times 40 \times 60}{96.3} = 8.22 \text{ gpm}$$

Sample Calculation 6.4. *Preliminary design computations for potatoes.*

GIVEN: The information in Parts I and II of Fig. 6.9 for potatoes where the average wind is 10 to 15 mph and:

The soil moisture depletion, MAD = 50%;
Side-roll laterals with two changes per day;
The sprinkler spacing is 40 × 50 ft; and
System leakage is insignificant, $O_e = 1.0$.

FIND: The irrigation efficiency, application rate, and sprinkler discharge required.

CALCULATION: Determine $R_e = 0.92$ from Fig. 6.8 for ET = 0.25 in./day for 10- to 15-mph wind and average spray midway between fine and coarse. Because potatoes are a relatively high-value, shallow-rooted crop, an application efficiency, E_q, based on the average low-quarter depth, is appropriate, so use DU as the measure of uniformity. This will leave approximately 10% of the area underwatered. Assuming an E_q of 67%, the required gross application would be:

$$d = \frac{2.0}{67/100} = 3.0 \text{ in.}$$

Assuming it will take 30 min to change the position of a side-role lateral, the time per set with two changes per day will be 11.5 hr. Thus, the preliminary application rate, (I), is:

$$(I) = \frac{3.0 \text{ in.}}{11.5 \text{ hr}} = 0.26 \text{ in./hr}$$

From Table 6.6, for 10- to 15-mph winds, the anticipated CU = 78%. If alternate sets are used, the improved CU_a can be estimated by Eq. 6.6a as:

$$CU_a = 10(CU)^{1/2} = 10(78)^{1/2} = 88\%$$

Two processes can be used to find the expected E_q. An estimated DU_a can be determined by Eq. 6.3a as:

$$\mathrm{DU}_a \simeq 100 - 1.59\,(100 - \mathrm{CU}_a)$$

$$\simeq 100 - 1.59\,(100 - 88) = 81\%$$

and letting $\mathrm{DU}_a = DE_{pa}$ in Eq. 6.9:

$$E_q = 81 \times 0.92 \times 1.0 = 75\%$$

The other method is to enter Table 6.2 with $\mathrm{CU}_a = 88\%$ and find that, for 90% of the area adequately irrigated, $DE_{90} = 81\%$.

From Eq. 6.9:

$$E_{(90\%\ \text{adequate})} = 81 \times 0.92 \times 1.0 = 75\%$$

The required gross application, assuming $E_q = 75\%$, can now be determined by Eq. 5.3a since LR < 0.1 as:

$$d = \frac{2.0}{75/100} = 2.7 \text{ in.}$$

and the required application rate is:

$$I = \frac{2.7 \text{ in.}}{11.5 \text{ hr}} = 0.23 \text{ in./hr}$$

The required sprinkler discharge can now be computed by Eq. 5.5 as:

$$q = \frac{0.23 \times 40 \times 50}{96.3} = 4.78 \text{ gpm}$$

Sample Calculation 6.5. *Benefit of alternate sets.*

GIVEN: The application efficiency and uniformity data developed in Sample Calculation 6.4: CU = 78%, $\mathrm{CU}_a = 88\%$, $R_e = 0.92$, $O_e = 1.0$

FIND: An estimate of the productivity increase from alternate sets.

CALCULATIONS: In Sample Calculation 6.4, if alternate sets had not been used, the application efficiency for 90% adequacy would have been much lower. From Table 6.2 for CU = 78%, $DE_{90} = 65\%$ and by Eq. 6.9:

$$E_{90} = 65 \times 0.92 \times 1.0 = 60\%$$

instead of $E_{90} = 75\%$ using alternate sets.

On the other hand, if the same application efficiency of 75% is assumed and alternate sets are not used, then the adequacy of irrigation would be much lower. From Eq. 6.9, and Table 6.2, the percentage of adequacy, subscript *pa*, can be determined by noting that:

$$E_{pa} = DE_{pa} \times .92 \times 1.0 = 75\%$$

therefore

$$DE_{pa} = 81\%$$

From Table 6.2 for CU = 78%, find $DE_{pa} = 81\% = DE_{75}$. Therefore, the percentage of adequacy is only 75% instead of 90% using alternate sets if the same application efficiency is assumed with and without alternate sets.

The productive value of having 90% adequacy by using alternate sets versus using regular sets that would give only 75% adequacy with the same gross application can be demonstrated. Table 6.3 gives relative percentages of optimum productivity for different CU and adequacy values, assuming overwatering does not reduce yields. With a CU = 78 and 75% adequacy, the relative production is 95%, and, for a CU = 88 and 90% adequacy, it is 99%. Thus, the use of alternate sets can be expected to improve yields by at least (99 − 95) = 4%. If uneven watering causes losses of production or quality difference (due to leaching of fertilizer or waterlogging), the gross yield or income differences may be considerably larger than 4%.

Sample Calculation 6.6. *Comparison of graphical with numerical determination of design and classic net application depths.*

GIVEN: The graphical relationship between surface area and depth of water applied for CU values of 70 and 86%, when $pa = 20\%$ of the area is underirrigated and the remaining 80% of the area is adequately (or over-) irrigated as shown in Fig. 6.7.

CALCULATIONS: The shaded area in Fig. 6.7 represents the volume of the irrigation deficit with CU = DE_{80} = 86%, when the relative depth of application is ≥ 1.0 over $pa = 80\%$ of the irrigated area. Since the relative depth values are the ratio d'_n/d_n, the average relative depth stored, which we will call $(d_n)_R$ is:

$$(d_n)_R = \frac{1.0 \times 100\% - (\text{average volume of deficit})}{1.0 \times 100\%}$$

and by inspection the average volume of deficit is about 0.1 × 20% so:

$$(d_n)_R = \frac{100 - 0.1 \times 20}{100} = 0.98$$

thus, $(d'_n/d_n) = 0.98$.

This is the same as the relative production value of 98% presented in Table 6.3 for CU = 86% and $pa = 80\%$. It can also be computed by the ratio of the classic/design water distribution efficiency values presented in Tables 6.8 and 6.2 for CU = 86% as:

$$d_n'/d_n = DE_{80}'/DE_{80} = 83/85 = 0.98$$

In a similar manner for CU = 70%, from the graph:

$$d_n'/d_n \approx (100 - 0.25 \times 20)/100 = 0.95$$

and from Table 6.3 the relative production is 94% or from Tables 6.8 and 6.2 for CU = 70%:

$$d_n'/d_n = DE_{80}'/DE_{80} = 64/68 = 0.94$$

Sample Calculation 6.7. *Determination of nozzle size and average operating pressure.*

GIVEN: The sprinkler spacing of 40 by 60 ft and the average sprinkler discharge of q_a = 8.22 gpm, giving an application rate of I = 0.33 in./hr for the alfalfa field considered in Sample Calculation 6.3.

CALCULATION: From Table 6.5 a sprinkler with a 13/64-in. nozzle should be appropriate (see column for 0.35 ± 0.02 in./hr). Furthermore, from Table 5.2 or from manufacturers' charts, a 13/64-in. nozzle will discharge 8.00 gpm at 45 psi and 8.45 gpm at 50 psi. Thus the average sprinkler pressure, P_a, that will give the required discharge can be interpolated as P_a = 47 psi.

Another way to estimate P_a is by Eq. 5.2:

$$P_a = 45 \left(\frac{8.22}{8.00}\right)^2 = 47 \text{ psi}$$

or by Eq. 5.1 with K_d = 1.193 from Table 5.2:

$$P_a = \left(\frac{q_a}{K_d}\right)^2 = \left(\frac{8.22}{1.193}\right)^2 = 47 \text{ psi}$$

REFERENCES

ASCE On-Farm Irrigation Committee. 1978. Describing irrigation efficiency. *Journal of the Irrigation and Drainage Division*, ASCE 104(IR1):35–41.

Christiansen, J. E. 1942. Irrigation by sprinkling. *California Agric. Exp. Stn. Bull.* 670. University of California, Berkeley.

Doorenbos, J., and A. H. Kassam. 1979. Yield response to water. *Food and Agriculture Organization of the United Nations, Irrigation and Drainage Paper* 33.

Frost, K. R., and H. C. Schivalen. 1955. Sprinkler evaporation losses. *Agricultural Engineering* 36(8):526–528.

Hart, W. E., and W. N. Reynolds. 1965. Analytical design of sprinkle systems. *ASAE Transactions* 8(1):83–85, 89.

Hart, W. E., G. Peri, and G. V. Skogerboe. 1979. *Journal of the Irrigation and Drainage Division, ASCE* 105(IR3):275–288.

McCulloch, A. W., J. Keller, R. M. Sherman, and R. C. Mueller. 1957. Revised by Keller, 1967. *Irrigation Handbook.* Milpitas, California: W. R. Ames Co.

Merrian, J. L., and J. Keller. 1978. *Farm Irrigation System Evaluation*, 3rd ed., Logan, Utah: Agricultural and Irrigation Engineering Department, Utah State University.

Solomon, K. H. 1978. Variability of sprinkler coefficient of uniformity test results. *ASAE Technical Paper* no. 78-2010.

Wiersma, J. L. 1955. Effect of wind variation on water distribution from rotating sprinklers. *South Dakota Agric. Exp. Stn. Bull.* 16. South Dakota State College, Brookings.

7

Layout of Set Sprinkler Systems

Often the layout of a system will be simple, as in the case of small, regularly shaped areas (McCulloch et al., 1967). On the other hand, large, odd-shaped tracts with broken topography may present a complex engineering problem requiring alternate layouts and careful pipe-size analysis (Benami and Ofen, 1983). The most important points to consider when planning a portable or fixed lateral system layout (see Fig. 2.4) and the general rules to follow are presented below. These rules provide only general guidance to the planner. In the more complex layouts, considerably more imagination and judgment are necessary.

LATERAL LAYOUT

The ideal lateral layout depends on the number of sprinklers and lateral settings required, the topography, and wind conditions.

Number of Sprinklers

The system layout must provide for simultaneous operation of the average number of sprinklers that will satisfy the required system capacity, Q_s, determined by Eq. 5.4. The required average number is:

$$N_n = \frac{Q_s}{q_a} \tag{7.1a}$$

where

N_n = minimum average number of sprinklers operating
Q_s = total system discharge capacity, L/s (gpm)
q_a = average sprinkler discharge, L/s (gpm)

The variation in the number of sprinklers operated from time to time during an irrigation should be kept to a minimum. This should be done to facilitate lateral routing and to maintain a nearly constant load on the pumping plant. There need be no variation in a rectangular area (except for the last day of an irrigation cycle where the number of lateral positions is a fractional multiple of the number of laterals.) For this reason, farmers should be encouraged to relocate fences, drainage ditches, roads, and other field boundaries, where practicable, to obtain a rectangular area.

Pipe lengths are generally standardized, and small sprinklers on portable systems are normally spaced at 9-, 12-, and 18-m (30-, 40-, and 60-ft) intervals on the laterals. Furthermore, the spacing between laterals is usually at intervals of 12, 15, 18, and 24 m (40, 50, 60, and 80 ft) along the main line. Because whole laterals must be operated simultaneously, the preliminary system capacity determined by Eq. 5.4 may be lower than the required capacity, even on rectangular fields. However, the depth per irrigation, d or the length of actual operating time per irrigation, (fT), can usually be adjusted to optimize the fit.

On odd-shaped fields, it is sometimes necessary to operate less than the average required number of sprinklers for one or more lateral settings. In these cases, engine-driven pumps can be throttled down to reduce the discharge. Where two or more laterals are operated simultaneously, either with different numbers of sprinklers or at different positions in the system, valves should be used to control their inlet pressures. For most odd-shaped fields, the number of sprinklers needed will exceed the computed theoretical minimum number, and extra equipment will be necessary to serve irregular parts of the field.

Where the design area is subdivided, the number of sprinklers required for each subdivision must be computed separately.

Number of Lateral Positions

The maximum number of positions that each periodic-move lateral can handle depends on the number of moves per day and the number of operating days allowed for completing one irrigation cycle during the peak-use period, f. The required number of positions per lateral must not exceed the product of these two factors.

If the system layout provides for the theoretical minimum number of operating sprinklers, N_n, then the number of settings required per lateral will not exceed: (moves/day) f. Long, narrow, or irregularly shaped parts of a field, however, may require additional lateral settings. Thus, more equipment is necessary if such areas are to be served within the allowable period.

Topographic Effects

To create a successful design, the crop, soil, and sprinkler characteristic considerations introduced in the preceding chapters must be integrated into a system that conforms to the shape and relief of the field to be irrigated.

The sketches in Fig. 7.1 show how the lateral layout can be designed to fit the topography. To obtain near-uniform application of water along the length

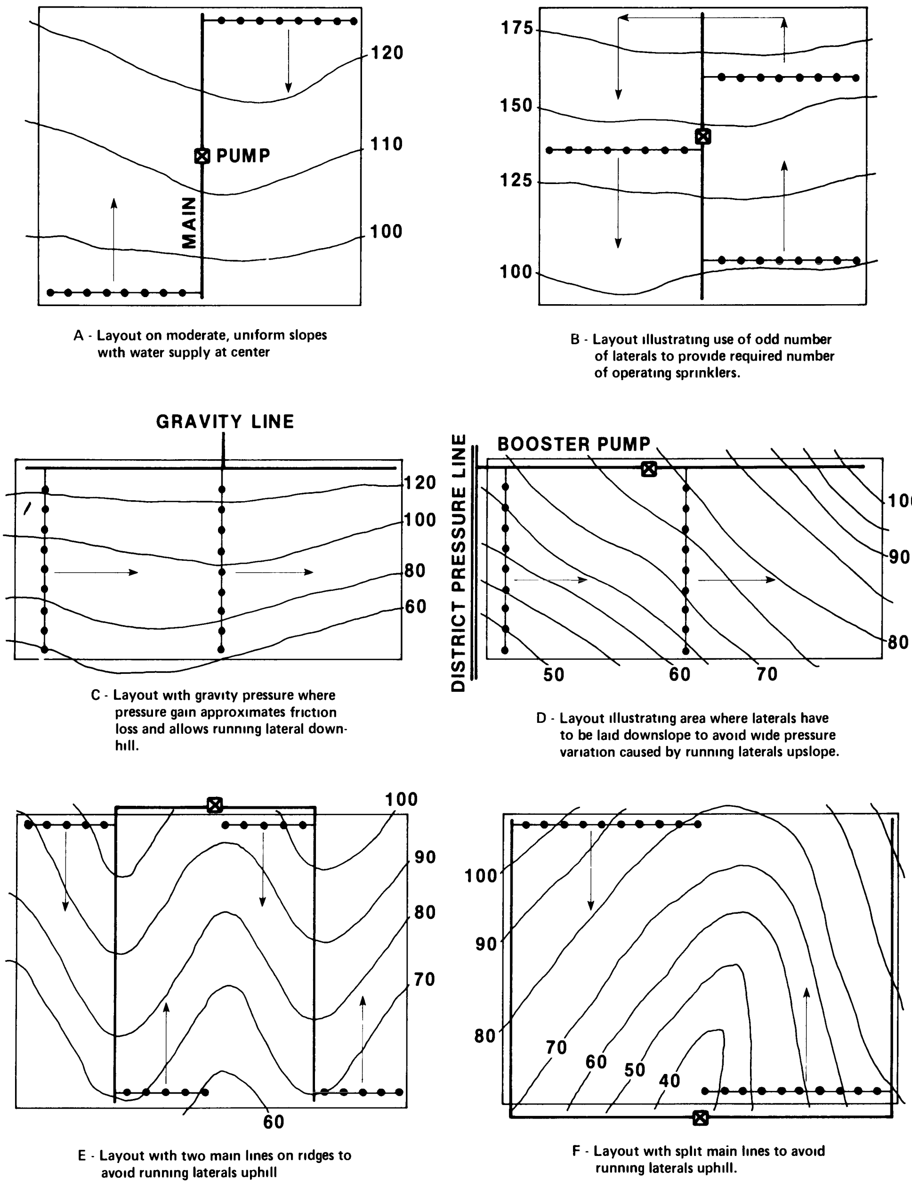


FIG. 7.1. Lateral Layouts for Periodic-move Sprinkle Systems with Different Topographic Situations.

of a lateral, the pipe diameter, length, and alignment must be selected so as to result in a minimum variation in discharge between individual sprinklers. Normally, the variation in discharge should not exceed 10% unless economically justified. Therefore, either pressure (or flow) regulation must be provided for each sprinkler or laterals must be located and pipe sizes selected so that the pressure head variations in the lateral, due to both friction loss and elevations differences, *will not exceed 20% of the average design operating pressure for the sprinklers, P_a.*

To meet this pressure variation criterion, it is usually preferable to lay laterals on the contour or across prominent land slopes (see Fig. 7.1A, B, and C). Thus, for a given average sprinkle discharge, q_a, and set of pipe sizes, the lateral length is limited only to that length in which the friction loss equals 20% of P_a.

Running laterals uphill should be avoided wherever possible. An uphill lateral of a given pipe size and fixed q_a is limited to that length in which the pressure loss due to friction is equal to 20% of P_a, minus the static pressure difference due to elevation. For example, if the static head caused by the difference in elevation between ends of the lateral amounts to 12% of P_a, then the line is limited to that length in which only 8% of P_a is lost due to friction. For this reason, where uphill laterals must be used, they need to be shorter than level laterals run on the same system, unless pressure or flow regulators are installed to compensate for the differences in slope.

Running laterals downslope is often a distinct advantage, provided the slope is fairly constant and not too steep (see Fig. 7.1D, E, and F). Under downslope conditions the difference in elevation between the two ends of the line results in a gain in pressure head rather than a loss; therefore, downslope laterals may be longer than similar laterals laid on level ground.

Where the ground slope along the lateral is about equal to the friction loss gradient, the pressure along the lateral will be nearly constant. When the ground slope along the lateral increases for successive setting, intermediate control valves may be required. Such valves are needed to avoid building up excessive pressures and exceeding the 20% of P_a variation limit.

Contours and Terraces

Farming operations and row directions often influence the layout of laterals. *Sprinkling of contoured row crops* can be done only with hand-move or solid-set systems. Nonparallel contours present special problems, such as difficulty in placing and moving the laterals, which make it difficult to obtain uniform coverage unless hose-fed sprinklers are used (see Fig. 4.8A).

Where sloping land is terraced and the slopes are not uniform, lateral lines laid between crop rows will not be parallel. Thus, the lateral spacing will vary between two adjacent lines. This variation adversely affects uniformity of application and efficiency of water use.

Where the land is terraced and the topography broken, curves in the alignment of the rows may be sharper than can be negotiated with the limited deflection angle of the coupling devices used on portable irrigation pipe. This difficulty may be overcome by land grading to improve terrace and row alignment. This in turn may allow laterals to be run parallel and downhill, even though both the rows and terraces must be crossed by the pipelines.

Other Considerations

Hand-move lateral lines need to be limited to one or two pipe sizes for simplicity of operation. The trend in recent years has been toward the use of a single pipe size.

Lateral lines should be located at right angles to the prevailing wind direction where possible. They should be moved in the direction of the wind if the water contains more than 1000 ppm of salts. By moving in the direction of the wind, salt deposits from the drifting spray will be washed off the crop during subsequent lateral settings.

Many times laterals are kept in a single design area and are not moved from field to field. In such cases, they should be oriented with respect to the main line so they can be rotated around it, thereby minimizing the hauling of pipe back to the starting point for subsequent irrigations (see Fig. 7.1A, B, and E).

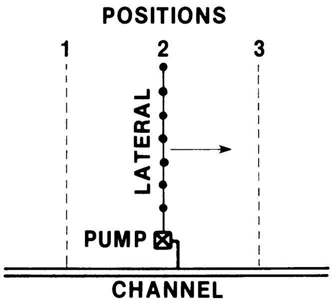
MAIN LINE AND PUMPING PLANT LAYOUT

Figures 7.1 and 7.2 show various main line configurations and pumping plant locations.

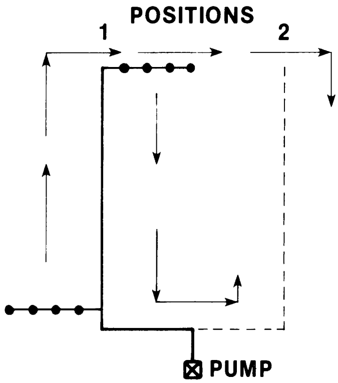
Main Line Layout

Main lines or submains should usually run up and down predominant land slopes. Where laterals are downslope, the main line will often be located along a ridge, with laterals sloping downward on each side (see Fig. 7.1E) or in a split main line fashion as in Fig. 7.1F.

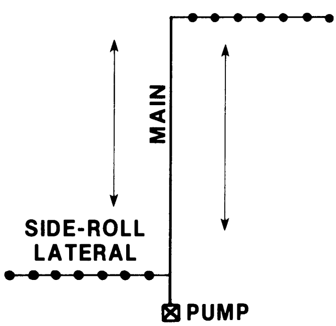
Where possible, main lines should be located so that laterals can be rotated around them in a "split-line" operation, as discussed above and further illustrated in Fig. 7.2B, C, and D. This not only reduces labor but also minimizes main line pipe friction losses. It should be pointed out, however, that the farmers' planting, cultural, and harvesting operations do not always permit a split-line operation. An example would be harvesting flue-cured tobacco over a period of several weeks while irrigation is still in progress. Water is usually applied to part of a field immediately after a picking of ripened leaves. However, most growers object to picking in several parts of the field simultaneously, as would be necessary to stay ahead of the lateral moves in a split-line operation. A similar situation holds true for harvesting forage crops.



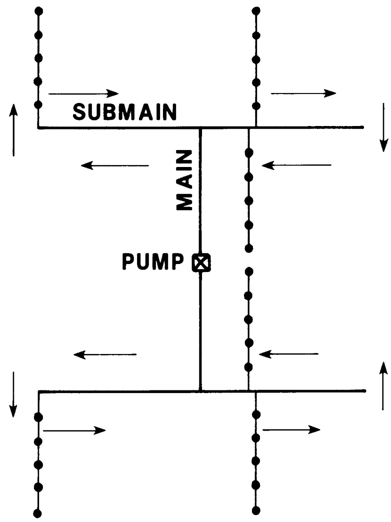
A - Fully portable sprinkler system with portable lateral and movable pumping plant.



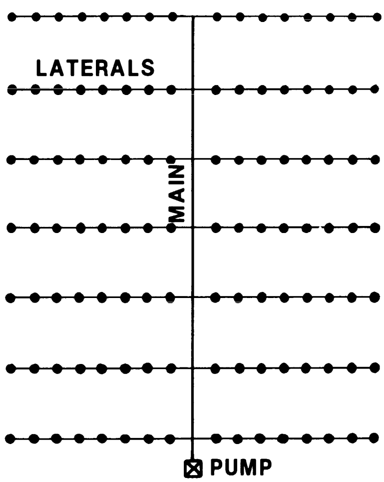
B - Portable laterals and main lines with stationary pumping plant.



C - Portable laterals, buried main line, and stationary pumping plant.



D - Buried mains and submains with portable laterals.



E - Buried main lines and fixed laterals.

FIG. 7.2. Mainline and Pumping Plant Layouts for General Types of Periodic-move Sprinkle Systems.

Water Source and Pumping Plant

Where possible, the *water supply* should be located near the center of the design area. This gives the least cost for main line pipe and for pumping. However, choosing the location of the water supply is usually possible only when a well is the source. On sloping fields the well should be located uphill from center (see Fig. 7.1B) to better balance up- and downhill pressures and minimize pipe sizes.

Where surface water is utilized, flexibility for locating the *pumping plant* is generally limited. However, the pumping plant should be located as central as practical to all parts of the design area for the reasons already mentioned (see Figs. 7.1A and 7.2D). Figure 5.1 illustrates this condition where the choice of pump locations is between points A and F, but the most central pump location is at A, which gives the least cost of main line pipe.

On flat or gently sloping land where water is to be pumped from ditches, main line costs will be reduced if water is run in a ditch to the center of the design area. However, the ditch will present an obstacle to farming operations. On steep land, the water supply may be high enough above the field so that sufficient pressure can be obtained by gravity. In such cases, cost will usually be lowest if the gravity-pressured supply line enters the design area at the center of the top boundary, as in Fig. 7.1C.

A booster pump should be considered where a small part of the design area requires higher pressures than the main body of the system. In such cases, a booster pump can eliminate the need for supplying higher pressures at the main pumping plant to meet the pressure required for only a small fraction of the total discharge. A booster pump may also be recommended where the static head is so great that two pumps prove more economical than a single unit. A careful analysis of pipe as well as pumping costs is required in both cases. By reducing pressures near the main pumping plant, lighter, lower cost pipe may be satisfactory.

ADJUSTMENTS TO MEET LAYOUT CONDITIONS

Figures 1.2 and 1.3 show the relationships between the design process activities. After completing the preliminary layout of laterals and main lines, it is often necessary to adjust one or more of the following variables:

Number of sprinklers operating, N_n;
Water-application rate, I;
Gross depth of each irrigation, d;
Average sprinkler discharge, q_a;
Spacing of sprinklers, S_e and/or S_l;
Actual operating time per day, T;

Days to complete one irrigation, f;
Total operating time per irrigation, (fT); and
Total system capacity, Q_s. (Note lower-case q is used to denote outlet discharge and capital Q to denote system or pipeline flow rates.)

Experienced designers can foresee these adjustments during the layout process. On rectangular fields, the layout can usually be determined early in the design procedure, and the subsequent steps developed on the basis of fixed layout requirements.

Application Rate

The application rate, I, can be adjusted according to the flexibility in time allowed for applying the required gross depth of water, d. This flexibility is limited by the maximum water-application rate (see Table 5.4), which is fixed by the water-intake rate of the soil and the minimum water-application rates practical for the design.

Because q_a is a function of I and sprinkler spacing, q_a can be modified only to the extent that I, the spacing, or both can be modified for a constant d. However, d and the frequency of irrigation can also be adjusted if further modification is needed.

The sprinkler spacing can be adjusted within limits in order to maintain a fixed I. Changes in spacing, S_e or S_l, can be made in 3-m (10-ft) increments to alter the number of operating sprinklers on a fixed length of lateral or the number of lateral positions across the field.

Major adjustments in I to fit requirements of a good layout must be compensated for by modifying the total operating time per irrigation, (fT), to fit d.

System Capacity

Before the layout is made, T and f are assumed in computing Q_s by Eq. 5.4. If the total time of operation, (fT), is increased, Q may be proportionately reduced; however, f obviously cannot exceed the irrigation interval, f', computed for the design, d. The actual system capacity is the product of the maximum number of operating sprinklers, N_x, and q_a. Rewriting Eq. 7.1a and replacing the minimum number of sprinklers, N_n, with N_x:

$$Q_s = N_x \times q_a \tag{7.1b}$$

Thus, the final adjustment is to compute the total system capacity needed to satisfy maximum demands. Sample Calculation 7.1 illustrates the problem of adjusting system capacity to meet layout requirements.

Sample Calculation 7.1 *Determine system capacity and adjust operating conditions to meet layout requirements.*

GIVEN: An 80-A potato field with dimensions of 1320 × 2640 ft (see Fig. 7.3)

The information from Fig. 6.9 for potatoes and from Sample Calculation 6.4 gives $d = 2.7$ in.; $q_a = 4.78$ gpm; 8-day irrigation interval during peak-use period; two 11.5-hr sets per day, and 40- by 5-ft sprinkler spacing

LAYOUT CALCULATIONS: Calculate the preliminary system capacity by Eq. 5.4:

$$Q_s = \frac{453\ Ad}{fT} = \frac{453 \times 80 \times 2.7}{8 \times 2 \times 11.5} = 532 \text{ gpm}$$

Determine the minimum number of sprinklers by Eq. 7.1a:

$$N_n = \frac{Q_s}{q_a} = \frac{532}{4.78} = 111 \text{ sprinklers}$$

Fix the layout with one main line 1320 ft long, through the center of the field with laterals 1320 ft long on either side.

With $S_e = 40$ ft, the number of sprinklers per lateral is:

$$\frac{1320}{40} = 33 \text{ sprinklers per lateral}$$

The minimum whole number of laterals required is:

$$\frac{111}{33} = 3.4 = 4 \text{ laterals}$$

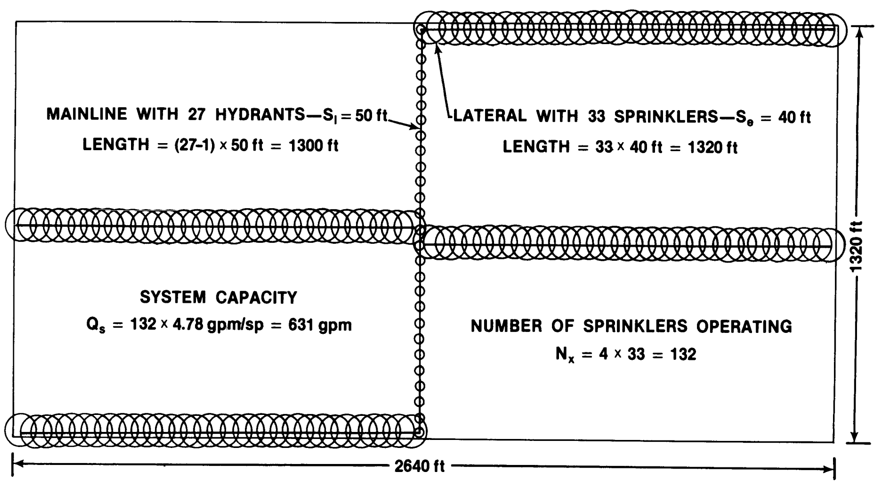


FIG. 7.3. Sprinkler System Layout in 80-Acre Field with 40- by 50-Foot Sprinkler Spacing.

The number of lateral positions on each side of the main line with $S_l = 50$ ft is:

$$\frac{1320}{50} = 26.4 = 27 \text{ positions}$$

which gives a total of 54 positions (on both sides of the main line (see Fig. 7.3).

The average number of settings (positions) for each of the four laterals is:

$$\frac{54}{4} \text{ laterals} = 13.5 \text{ settings per lateral}$$

Thus two of the laterals will make 14 settings and the other two will make 13 settings to complete one irrigation cycle.

The time required to complete one irrigation with two lateral settings per day is:

$$f = \frac{14}{2} = 7 \text{ days}$$

ADJUSTMENT CALCULATIONS: With all four laterals operating, the maximum number of sprinklers running is:

$$N_x = 4 \times 33 = 132 \text{ sprinklers}$$

The actual system capacity computed by Eq. 7.1b is:

$$Q_s = N_x \times q_a = 132 \times 4.78 = 631 \text{ gpm}$$

This is higher than the preliminary capacity, which was based on an 8-day irrigation interval. The final system capacity could be reduced to be more nearly equal to the preliminary $Q = 532$ gpm by letting $d = 2.4$ in. and reducing the irrigation interval to 7 days. This would require changing q_a to about 4.25 gpm (depending on the effect on the application efficiency, E_q). However, it was decided to leave the 8-day interval, to provide a margin of safety since the water supply was sufficient. Furthermore, the savings in system cost afforded by a lower application rate would be more than offset by the added labor cost of more frequent irrigations.

REFERENCES

Benami, A., and A. Ofen. 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).

McCulloch, A. W., J. Keller, R. M. Sherman, and R. C. Mueller. 1957. Revised by Keller, 1967. *Irrigation Handbook*. Milpitas, California: W. R. Ames Co.

8

Pipeline Hydraulics and Economics

This chapter contains information about the hydraulics of pipe systems used for both sprinkle and trickle irrigation. The economic relation between pipe friction and the costs of capitalizing and operating an irrigation piping system is presented as a basis for selecting pipe diameters.

PRESSURE AND HEAD RELATIONSHIPS

For water at rest in a container, the *pressure* at any point is equal to the product of the unit weight of water 1000 kg/m^3 at 20°C (62.4 lb/ft^3 at 60°F) and the height of the water above that point. The height of the water in the column is called the *head*. Head and pressure are simply different ways of expressing the same thing. Pressure usually is expressed in kilopascals, kPa (pounds per square inch, psi) and head is expressed in m (ft) of water: 1 kPa (1 psi) of pressure is equivalent to 0.102 m (2.31 ft) of water head; or 1 m (1 ft) of water pressure head is equivalent to 9.8 kPa (0.433 psi) of pressure.

In an irrigation system the head consists of several components. At any point in a system, the following hold true:

- *Static head, H_e, at a given point* is simply equal to the difference in elevation, ΔEl, between the highest discharge point in the system and that point;
- Pressure head, H, is equal to the pressure, P, at that point divided by the unit weight of water;
- Velocity head is the head required to accelerate the water from rest to the velocity at that point. It is numerically equal to $V^2/2_g$, where V is the velocity, in m/s (ft/s), and g is the acceleration due to gravity 9.81 m/s^2 (32.2 ft/s^2).
- *Friction head*, h_f, is the energy required for water to flow between two points at the same elevation expressed in meters (ft) of water; and
- Elevation above datum, El, is the distance of a given point in the system above some arbitrary datum. Elevation is assumed to be positive above the datum and negative below the datum.

Static Pressure or Head

A system's total static head is the vertical distance, El, the water must be raised or lowered between the water source and the highest point of discharge. Since the static head is the same as the difference in elevation it is positive (+) if the water must be raised and negative (−) if it must be lowered.

The static pressure (or pressure head) in laterals is considered in the design procedure for determining the lateral inlet pressure, P_l, or inlet pressure head, H_l, required for proper operation. (Lateral line design is fully discussed in Chapter 9.) Therefore, the elevation differences between the pump and highest and lowest hydrants serving the laterals along the main line or submains give the maximum and minimum main line static head values. These must be included in computing the total dynamic head for maximum and minimum operating conditions, as discussed in Chapter 11.

Suction lift, or the difference between the elevation of the water source and the elevation of the pump, is a form of static head that must be included in total head computations. For wells, the drawdown while pumping at the maximum required discharge should also be included.

Velocity Head

The velocity of flow in a sprinkler system will seldom exceed 2.5 m (8 ft) per second. Therefore, the velocity head will seldom exceed 0.3 m (1.0 ft) and may be disregarded except in computing suction requirements for centrifugal pumps.

CALCULATION OF PIPE FRICTION

The formula most commonly used for estimating the friction loss in sprinkle and trickle system laterals and main lines of various pipe materials is the Hazen-Williams equation:

$$J = \frac{h_f}{L/100} = K\left(\frac{Q}{C}\right)^{1.852} D^{-4.87} \tag{8.1}$$

where

J = head loss gradient, m/100 m (ft/100 ft)
K = conversion constant, 1.212×10^{12} for metric units (1050 for English units)
h_f = head loss due to pipe friction, m (ft)
L = length of pipe, m (ft)
Q = flow rate in the pipe, L/s (gpm)

C = friction coefficient, which is a function of pipe material characteristics
D = inside diameter of the pipe, mm (in.)

Typical values of C for use in the Hazen-Williams equation are:

Pipe Material	C
Plastic	150
Epoxy-coated steel	145
Cement asbestos	140
Galvanized steel	135
Aluminum (with couplers every 30 ft)	130
Steel (new)	130
Steel (15 years old) or concrete	100

The Hazen-Williams equation was developed from study of water-distribution systems that used 75-mm (3-in.) or larger diameter pipes and discharges greater than 3.2 L/s (50 gpm). Under these flow conditions, the Reynolds number is greater than 5×10^4, and the formula predicts friction loss satisfactorily. However, for the small-diameter, smooth-walled pipe used in trickle irrigation systems, the Hazen-Williams equation with a C value of 150 underestimates the friction losses. This phenomenon is demonstrated by Fig. 8.1, which shows laboratory test results for a plain 13-m (1/2-in.) trickle hose superimposed on a Moody diagram. The Reynolds number, R_y, for 21°C (70°F) water flowing through a pipe is:

$$R_y = K\frac{Q}{D} \tag{8.2}$$

where

K = the conversion constant, 1.30×10^6 for metric units (3214 for English units)

The Moody diagram shows the relationship between the relative roughness of pipes, e/D, and the friction factor, F_f, for different values of R_y. The F_f is related to the head loss in the pipe, h_f, by the Darcy-Weisbach equation:

$$h_f = F_f\frac{L}{D}\frac{V^2}{2g} \tag{8.3}$$

where

F_f = Darcy-Weisbach pipe friction factor
V = velocity of flow in the pipe, m/s (ft/s)

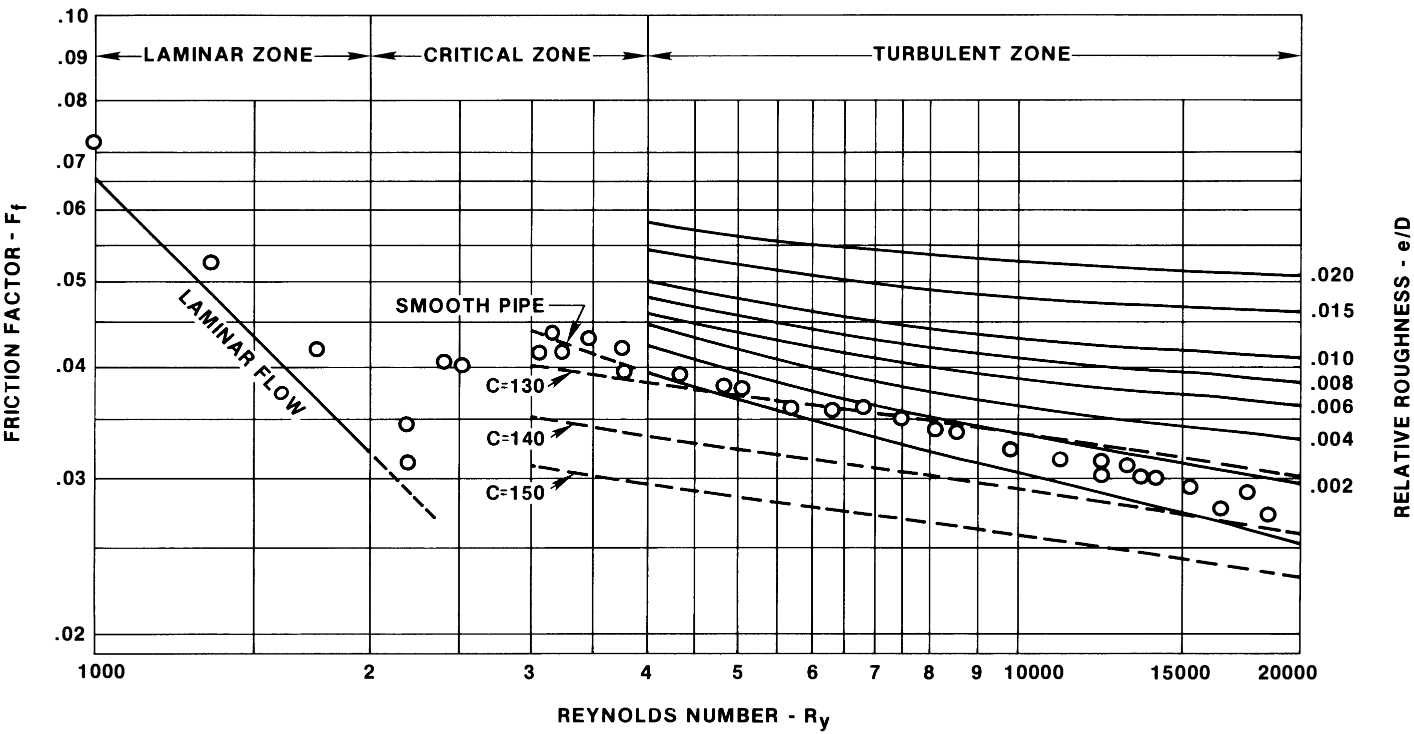


FIG. 8.1. Friction Factors for 13-Millimeter (1/2-Inch) Plastic Irrigation Tubing (Small Circles) and Hazen-Williams (Dashed Curves) versus Darcy-Weisbach (Solid Curves) Friction Coefficients in the Low Reynolds Number Range.

g = acceleration due to gravity, 9.81 m/s (32.2 ft/s)
D = inside pipe diameter, m (ft)

The "smooth pipe" line on the Moody diagram is generally considered the ultimate in pipe smoothness. For comparison, the "equivalent" F_f values for Hazen-Williams C values of 130, 140, 150 are also plotted on Fig. 8.1. The position of the C value lines clearly shows a discrepancy in the "smooth pipe" concept in this low range of Reynolds numbers. The $C = 150$ line, which represents Hazen-Williams smooth pipes, is well below the friction factor of Darcy-Weisbach smooth pipes.

The range of Reynolds numbers shown on Fig. 8.1 represents hose discharge rates between 0.01 and 0.20 L/s (0.2 and 3.0 gpm) for 13 mm (1/2-in.) hose. The hose test data plot somewhat above the Moody "smooth pipe" line and appear nearer to an average C value of about 130. Note that the data points seem to follow the curvature of the lines on the Moody diagram described by Eq. 8.3, rather than the constant C value lines. This observation strongly supports the conclusion that the Darcy-Weisbach equation represents the friction losses in small-diameter pipe and hoses better than does the Hazen-Williams formula.

The friction factor, F_f, for flow in smooth pipes is given by the following classic equations:

for laminar flow where $R_y < 2000$,

$$F_f = \frac{64}{Ry} \tag{8.4a}$$

and for turbulent flow where $R_y \geq 2000$,

$$\frac{1}{(F_f)^{0.5}} = -0.80 + 2.0 \log R_y(F_f)^{0.5} \tag{8.4b}$$

An equation developed by Churchill (1977) nicely handles the entire range of R_y values for determining F_f in all types of pipes. It lends itself to use in spreadsheets as:

$$F_f = 8\left[(8/R_y)^{12} + 1/(k_1 + k_2)^{1.5}\right]^{1/12} \tag{8.5}$$

where

$$k_1 = \left[2.457 \ln\left(\frac{1}{(7/R_y)^{0.9} + 0.27(e/D)}\right)\right]^{16}$$

$$k_2 = \left(\frac{37530}{R_y}\right)^{16}$$

The Churchill equation is uncanny in that it gives F_f values almost identical to those obtained from Eqs. 8.4a and 8.4b and even handles the discontinuities through the "critical zone" (see Fig. 8.1). Typical values of e for use in Eq. 8.5 are:

Pipe Material	Roughness values − e mm	(in.)	Size (-in.)
1. Plastic	0.013	0.0005	-
2. Epoxy-coated steel	0.028	0.0011	6 to 8
3. Cement asbestos or tar enamel steel	0.076	0.003	12
4. Galvanized steel	0.102	0.004	6 to 8
5. Aluminum (with couplers)	0.127	0.005	3 to 6
6. Plain steel (new)	0.203	0.008	12
7. Plain steel (old) or concrete	1.52	0.06	12

These e values are equivalent to the C values presented for use in Eq. 8.1 for the ranges of pipe sizes of each pipe material ordinarily used in field irrigation systems.

Obviously, Eq. 8.4b is quite tedious, and Eq. 8.5 is somewhat cumbersome to use; so for calculator computations, the following equation is recommended for smooth pipes for R_y between 2000 and 100,000:

$$F_f = 0.32\, R_y^{-0.25} \tag{8.6}$$

Equation 8.6 is called the *Blasius equation* and is useful for small-diameter plastic pipes and hoses used in the low Reynolds number range found in sprinkle and trickle irrigation systems.

Simple Formulas and Tables

To simplify desk computation further, Eqs. 8.2, 8.3, and 8.6 can be combined and the constant adjusted for average conditions. This gives a simple equation developed by Watters and Keller (1978) for use with smooth plastic pipes and hoses *less than* 125 mm (5 in.) in diameter:

$$J = \frac{100\, h_f}{L} = K \frac{Q^{1.75}}{D^{4.75}} \text{ (for small pipe)} \tag{8.7a}$$

where

K = conversion constant, 7.89×10^7 for metric units (0.133 for English units)

For larger plastic pipe, where the diameter is *greater than* 125 mm (5 in.), the friction head loss gradient can be approximated by:

$$J = \frac{100\, h_f}{L} = K \frac{Q^{1.83}}{D^{4.83}} \text{ (for large pipe)} \tag{8.7b}$$

where

K = conversion constant, 9.58×10^7 for metric units (0.100 for English units)

These formulas are as easy to use as the Hazen-Williams equation and more accurately predict friction loss for 21 °C (70°F) water flowing in smooth plastic pipe.

Tables 8.1 through 8.6 give friction gradient or J values in units of head loss per 100 units of pipe length for aluminum and plastic pipe materials used in sprinkle and trickle irrigation systems. Other types of pipe material, such as asbestos-cement, are also available and practical for sprinkle system main lines. As a general rule, pipe manufacturers will provide friction-loss tables for the particular types and classes of pipe they offer. It is impractical to include all such tables in this text.

Most friction-loss tables are for new pipe unless otherwise stated. An *allowance for aging of the* pipe can be made by multiplying the friction-loss gradient, J, by a constant appropriate for the type of pipe material and the average life of the pipe. Normally, no allowance should be made for an increase in J for plastic, aluminum, asbestos cement, or galvanized (or plastic-coated) steel pipes unless mineral deposits are expected. However, due to slow corrosion, the friction gradient for unprotected steel pipe is typically assumed to increase linearly by a factor of about 1.7 over a 15-year period.

FRICTION LOSSES IN PIPE WITH OUTLETS

Flow of water through the length of a closed pipeline of a given diameter causes more friction loss than does flow through a line with a number of equally spaced outlets. The reason for this reduction in friction loss is that the volume of flow decreases each time an outlet is passed.

The method developed by Christiansen (1942) for computing head or pressure losses in multiple-outlet pipelines has been widely accepted and is used here. It involves first computing the friction loss in the line without multiple outlets and then multiplying by a factor, F, based on the number of outlets in the line, N. Thus, the head loss, h_f, in a pipe with uniformly spaced outlets is:

$$h_f = J\, F \frac{L}{100} \tag{8.8a}$$

Table 8.1. Friction-loss gradients, *J*, in m per 100 m (ft per 100 ft) for different flow rates in *portable aluminum pipe* used for *sprinkle irrigation laterals* with 1.27-mm (0.050-in.) wall thickness and couplings every 9.1 m (30 ft)[1]

Flow rate		Aluminum pipe size			
L/s	(gpm)	2-in.[2]	3-in.	4-in.	5-in.
0.63	10	0.40	0.05		
1.26	20	1.44	0.18		
1.89	30	3.05	0.39		
2.52	40	5.20	0.66		
3.15	50	7.85	1.00		
3.79	60	11.01	1.40	0.33	
4.42	70	14.65	1.87	0.44	
5.05	80	18.76	2.39	0.57	0.19
5.68	90	23.33	2.98	0.70	0.23
6.31	100	28.36	3.62	0.85	0.28
7.57	120		5.07	1.20	0.39
8.83	140		6.74	1.59	0.52
10.1	160		8.64	2.04	0.67
11.4	180		10.74	2.54	0.83
12.6	200		13.06	3.08	1.01
13.9	220		15.58	3.68	1.21
15.1	240		18.30	4.32	1.42
16.4	260		21.22	5.01	1.65
17.1	280		24.35	5.75	1.89
18.9	300			6.54	2.15
20.2	320			7.37	2.42
21.5	340			8.24	2.71
22.7	360			9.16	3.01
24.0	380			10.13	3.33
25.2	400			11.14	3.66
26.5	420			12.19	4.01
27.8	440			13.28	4.37
29.0	460			14.42	4.75
30.3	480			15.61	5.14
31.2	500			16.83	5.54
32.8	520				5.96
34.1	540				6.39
35.3	560				6.83
36.6	580				7.29
37.9	600				7.76

[1]Based on Hazen-Williams Eq. 8.1 with C = 130.
[2]Outside and nominal diameter; 1 in. = 25.4 mm

Table 8.2. Friction-loss gradients, *J*, in m per 100 m (ft per 100 ft) for different flow rates in *polyethylene hose* used for *trickle irrigation laterals*[1]

Flow rate		Inside diameter		Flow rate		Inside diameter	
L/min	(gpm)	14.7 mm (0.58 in.)	17.8 mm (0.70 in.)	L/min	(gpm)	14.7 mm (0.58 in.)	17.8 mm (0.70 in.)
0.4	0.1	0.05	0.03	13.6	3.6	15.99	6.54
0.8	0.2	0.11	0.05	14.0	3.7	16.77	6.86
1.1	0.3	0.17	0.08	14.4	3.8	17.53	7.19
1.5	0.4	0.37	0.11	14.8	3.9	18.40	7.53
1.9	0.5	0.53	0.22	15.1	4.0	19.23	7.87
2.3	0.6	0.72	0.30	15.5	4.1	20.09	8.21
2.6	0.7	0.94	0.39	15.9	4.2	20.96	8.57
3.0	0.8	1.18	0.49	16.3	4.3	21.84	8.93
3.4	0.9	1.45	0.60	16.7	4.4	22.74	9.30
3.8	1.0	1.73	0.71	17.0	4.5	23.66	9.57
4.2	1.1	2.04	0.84	17.4	4.6	24.59	10.05
4.5	1.2	2.37	0.98	17.8	4.7	25.54	10.44
4.9	1.3	2.72	1.12	18.2	4.7	26.51	10.83
5.3	1.4	3.09	1.27	18.5	4.9	27.43	11.23
5.7	1.5	3.43	1.43	18.9	5.0	28.49	11.64
6.1	1.6	3.89	1.60	19.3	5.1	29.50	12.05
6.4	1.7	4.32	1.78	19.7	5.2	30.52	12.47
6.8	1.7	4.77	1.96	20.1	5.3	31.57	12.89
7.2	1.9	5.24	2.15	20.4	5.4	32.63	13.32
7.6	2.0	5.73	2.35	20.8	5.5	33.70	13.76
7.9	2.1	6.24	2.56	21.2	5.6	34.79	14.20
8.3	2.2	6.76	2.77	21.6	5.7	35.83	14.65
8.7	2.3	7.31	3.00	22.0	5.8	37.01	15.11
9.1	2.4	7.87	3.23	22.3	5.9	38.15	15.57
9.5	2.5	8.45	3.46	22.7	6.0	39.30	16.04
9.8	2.6	9.05	3.71	23.1	0.1	40.46	16.51
10.2	2.7	9.66	3.96	23.5	6.2	41.64	17.00
10.6	2.8	10.30	4.22	23.8	6.3	42.84	17.43
11.0	2.9	10.95	4.48	24.2	6.4	44.05	17.97
11.4	3.0	11.62	4.76	24.6	6.5	45.27	18.47
11.7	3.1	12.30	5.04	25.0	6.6	46.51	18.98
12.1	3.2	13.01	5.33	25.4	6.7	47.76	19.49
12.5	3.3	13.73	5.62	25.7	6.8	49.03	20.00
12.9	3.4	14.46	5.92	26.1	6.9	50.32	20.53
13.2	3.5	15.22	6.23	26.5	7.0	51.61	21.05

[1]Based on Eqs. 8.3 and 8.5.

Table 8.3. Friction loss gradients, *J*, in m per 100 m (ft per 100 ft), for different flow rates in *IPS-PVC thermoplastic pipe*, used for *sprinkle irrigation laterals* and *trickle manifolds*[1]

Flow rate		Nominal pipe size and inside diameter, mm (in.)					
		$1\frac{1}{4}$-in. 38.9 (1.532)	$1\frac{1}{2}$-in. 44.6 (1.754)	2-in. 55.4 (2.193)	$2\frac{1}{2}$-in. 67.4 (2.655)	3-in. 83.4 (3.284)	4-in. 108.7 (4.280)
L/s	(gpm)						
0.25	4	0.19	0.10	0.01			
0.38	6	0.39	0.20	0.07	0.03		
0.63	10	0.95	0.50	0.17	0.07	0.03	
0.88	14	1.71	0.90	0.31	0.13	0.05	
1.14	18	2.67	1.40	0.48	0.19	0.07	
1.39	22	3.81	2.00	0.69	0.28	0.10	0.03
1.64	26	5.13	2.69	0.93	0.37	0.14	0.04
1.89	30	6.62[2]	3.46	1.19	0.48	0.17	0.05
2.15	34	8.27	4.33	1.49	0.60	0.22	0.06
2.40	38	10.09	5.28	1.81	0.73	0.26	0.08
2.65	42	12.06[3]	6.31	2.17	0.87	0.32	0.09
2.90	46	14.19	7.42	2.55	1.02	0.37	0.10
3.15	50	16.48	8.62	2.96	2.29	0.43	0.12
3.41	54	18.92	9.89	3.39	1.36	0.49	0.14
3.66	58	21.50	11.24	3.86	1.54	0.56	0.16
4.16	66		14.17	4.86	1.95	0.70	0.20
4.67	74		17.41	5.96	2.39	0.86	0.25
5.17	82			7.17	2.87	1.04	0.30
5.68	90			8.47	3.39	1.22	0.34
6.31	100			10.24	4.09	1.48	0.42
6.94	110			12.16	4.86	1.75	0.49
7.57	120			14.22	5.68	2.05	0.58
8.20	130				6.56	2.37	0.65
8.83	140				7.50	2.70	0.76
9.46	150				8.49	3.06	0.86
10.09	160				9.53	3.44	0.96
10.73	170				10.64	3.83	1.07
11.36	180				11.79	4.25	1.19
11.99	190					4.68	1.31
12.62	200					5.13	1.44
13.88	220					6.10	1.71
15.14	240					7.14	2.00
17.67	280					9.43	2.64
20.19	300						3.36
22.71	360						4.16
25.24	400						5.03

[1]Based on Eqs. 8.3 and 8.5; $1\frac{1}{4}$- to $2\frac{1}{2}$-in. pipe is SDR 26 (Class 10.9 atm or 160 psi); 3-in. is SDR 32.5 (Class 8.5 atm or 125 psi) 4-in. is SDR 41 (Class 6.8 atm or 100 psi).
[2]For flow rates below solid lines, the velocity exceeds 1.5 m/s (5 ft/s).
[3]For flow rates below dashed lines, the velocity exceeds 2.1 m/s (7 ft/s).

Table 8.4. Friction loss gradient, *J*, in m per 100 m (ft per 100 ft), for different flow rates in *sprinkle irrigation main line* of *portable aluminum pipe* with couplers connecting 9.1-m (30 ft) lengths[1]

Flow rate		Aluminum pipe size, (thickness and inside diameter, in.)[2]				
L/s	(gpm)	5-in.[2] (0.050) (4.900)	6-in. (0.058) (5.884)	8-in. (0.072) (7.856)	10-in. (0.091) (9.818)	12-in. (0.091) (11.818)
6.3	100	0.28	0.12			
9.5	150	0.60	0.24			
12.6	200	1.01	0.42	0.10		
15.8	250	1.53	0.63	0.15		
18.9	300	2.15	0.88	0.22		
22.1	350	2.86	1.17	0.29		
25.2	400	3.66	1.50	0.37	0.12	
28.4	450	4.56	1.87	0.46	0.15	
31.5	500	5.54	2.27	0.56	0.19	
34.7	550	6.61	2.71	0.66	0.22	
37.9	600	7.76	3.18	0.78	0.26	
41.0	650	9.00	3.69	0.90	0.31	
44.2	700		4.24	1.04	0.35	0.14
47.3	750		4.81	1.18	0.40	0.16
50.5	800		5.42	1.33	0.45	0.18
53.6	850		6.07	1.49	0.50	0.20
56.8	900			1.65	0.56	0.23
59.9	950			1.83	0.62	0.25
63.1	1000			2.01	0.68	0.27
69.4	1100			2.39	0.81	0.33
75.7	1200			3.81	0.95	0.39
82.0	1300			3.26	1.10	0.45
88.3	1400			3.74	1.26	0.51
94.6	1500			4.25	1.44	0.58
100.9	1600			4.79	1.62	0.66
113.6	1800			5.96	2.01	0.82
126.2	2000			7.25	2.45	0.99
138.8	2200			8.64	2.92	1.18
151.4	2400				3.43	1.39
164.0	2600				3.98	1.61
176.7	2800				4.56	1.85
189.3	3000				5.18	2.10
220.8	3500					2.80
252.4	4000					3.58

[1]Based on Hazen-Williams Eq. 8.1 with C = 130; for 6.1-m (20 ft) pipe increase by 7% and for 12.2-m (40-ft) pipe decrease by 3%.
[2]1.0 in. = 25.4 mm.

Table 8.5. Friction loss gradient, *J*, in m per 100 m (ft per 100 ft) for different flow rates in SDR 41—*IPS-PVC* (class 6.8 atm or 100 psi) *thermoplastic pipe* used for *sprinkle and trickle irrigation main lines*[1]

Flow rate		Nominal pipe size and inside diameter, mm (in.)				
L/s	(gpm)	4-in. 108.7 (4.280)	6-in. 160.0 (6.301)	8-in. 208.4 (8.205)	10-in. 259.7 (10.226)	12-in. 308.1 (12.128)
6.3	100	0.42				
9.5	150	0.86				
12.6	200	1.42				
15.8	250	2.09[2]				
16.9	300	2.88	0.47			
22.1	350	3.77[3]	0.62			
25.2	400	4.77	0.80			
28.4	450	5.86	0.99			
31.5	500		1.20	0.33		
34.7	550		1.42	0.40		
37.9	600		1.67	0.47		
41.0	650		1.93	0.54		
44.2	700		2.22	0.62	0.21	
47.3	750		2.51	0.70	0.24	
50.5	800		2.83	0.79	0.27	
53.6	850		3.16	0.88	0.30	
56.8	900		3.51	0.98	0.34	
63.1	1000		4.25	1.19	0.41	0.18
69.4	1100		5.07	1.41	0.49	0.21
75.7	1200		5.94	1.66	0.57	0.25
82.0	1300			1.92	0.66	0.29
88.3	1400			2.20	0.76	0.33
94.6	1500			2.50	0.86	0.38
100.9	1600			2.81	0.97	0.43
107.3	1700			3.14	1.08	0.48
113.6	1800			3.48	1.20	0.53
126.2	2000			4.23	1.46	0.64
138.8	2200			5.03	1.74	0.76
151.4	2400			5.90	2.04	0.89
164.0	2600				2.36	1.03
176.7	2800				2.70	1.18
189.3	3000				3.05	1.34
201.9	3200				3.45	1.51
227.1	3600				4.28	1.88
252.4	4000				5.19	2.28

[1]Based on Eq. 8.7.
[2]For flow rates below the solid lines, the velocity exceeds 1.5 m/s (5 ft/s).
[3]For flow rates below the dashed lines, the velocity exceeds 2.1 m/s (7 ft/s).

Table 8.6. Friction loss gradient, *J*, in m per 100 m (ft per 100 ft) for different flow rates in SDR 41 *PIP-PVC* (class 6.8 atm 100 psi) *thermoplastic pipe* used for *sprinkle and trickle irrigation main lines*[1]

Flow rate		Nominal pipe size and inside diameter, mm (in.)				
L/s	(gpm)	6-in. 148.3 (5.840)	8-in. 197.1 (7.762)	10-in. 246.4 (9.702)	12-in. 295.7 (11.642)	15-in. 369.7 (14.554)
18.9	300	0.68				
22.1	350	0.90				
25.2	400	1.15				
28.4	450	1.42[2]				
31.5	500	1.73	0.44			
34.7	550	2.06	0.52			
37.9	600	2.41[3]	0.61			
41.0	650	2.79	0.71			
44.2	700	3.20	0.81	0.28		
50.5	800	4.08	1.03	0.35		
56.8	900	5.06	1.28	0.44		
63.1	1000	6.14	1.55	0.53		
69.4	1100		1.85	0.63	0.26	
75.7	1200		2.17	0.74	0.31	
82.0	1300		2.51	0.86	0.35	
88.3	1400		2.88	0.98	0.41	
100.9	1600		3.67	1.25	0.52	0.18
113.6	1800		4.56	1.55	0.64	0.22
126.2	2000		5.52	1.88	0.78	0.27
138.8	2200		6.58	2.24	0.93	0.32
151.4	2400			2.63	1.09	0.37
164.0	2600			3.04	1.26	0.43
176.7	2800			3.48	1.44	0.49
189.3	3000			3.95	1.64	0.56
220.8	3500				2.17	0.74
252.4	4000				2.77	0.94
283.9	4500				3.44	1.17
315.5	5000				4.17	1.42
347.0	5500					1.69
378.5	6000					1.98
410.1	6500					2.29
441.6	7000					2.63

[1]Based on Eq. 8.7.
[2]For flow rates below the solid lines, the velocity exceeds 1.5 m/s (5 ft/s).
[3]For flow rates below the dashed lines, the velocity exceeds 2.1 m/s (7 ft/s).

or to determine the pressure loss, P_f:

$$P_f = KJF \frac{L}{100} \tag{8.8b}$$

where P_f = pressure loss due to friction, kPa (psi), and K = conversion constant, 9.8 for metric units (1/2.31 or 0.433 for English units).

Christiansen's equation for computing the reduction coefficient, F, for multiple-outlet pipelines where the first outlet is S_e form the main line is:

$$F = \frac{1}{b+1} + \frac{1}{2N} + \frac{(b-1)^{0.5}}{6N^2} \tag{8.9a}$$

where b = velocity or flow exponent of the head loss equation used, and N = number of outlets in the line, where the first outlet is αS_e form the main line ($0 \le \alpha \le 1$):

$$F(\alpha) = \frac{NF - (1-\alpha)}{N - (1-\alpha)} \tag{8.9b}$$

and in the special case where the first outlet is $S_e/2$ from the main line ($\alpha = 1/2$):

$$F(1/2) = \frac{2N}{2N-1}\left(\left(\frac{1}{b+1}\right) + \frac{(b-1)^{0.5}}{6N^2}\right) \tag{8.9c}$$

Table 8.7 shows values of F for different numbers of outlets. These values were computed by dividing the actual computed head loss in multiple-outlet pipelines (with equal discharge per outlet) by the computed head loss in pipe-

Table 8.7. Reduction coefficient, *F*, for multiple-outlet pipelines

Number of outlets	*F* (end)[1]	*F* (mid)[2]	Number of outlets	*F* (end)	*F* (mid)
1	1.00	1.00	8	0.42	0.38
2	0.64	0.52	9	0.41	0.37
3	0.54	0.44	10–11	0.40	0.37
4	0.49	0.41	12–15	0.39	0.37
5	0.46	0.40	16–20	0.38	0.36
6	0.44	0.39	21–30	0.37	0.36
7	0.43	0.38	≥31	0.36	0.36

[1]Where the first outlet is a full space from the pipe inlet, i.e., at the end of the first pipe.
[2]When the first outlet is one-half space from the pipe inlet, i.e., at the middle of the first pipe.

lines of equal diameter and length, but with only one outlet (they were not computed by Eq. 8.9, but are essentially the same as values calculated by the equation with $b = 1.83$).

Sample Calculation 8.1. *Computation of lateral friction loss.*

GIVEN: A 100-mm (4-in.) aluminum lateral pipeline 396 m long with 12 m between outlets and sprinklers discharging 30 L/min. (The inside diameter of the pipe is 99 mm).

CALCULATIONS: The number of sprinklers: $N = \dfrac{400}{12.2} = 33$

The lateral discharge is:

$$Q_l = 33 \times 30 = 990 \text{ L/min} = 16.5 \text{ L/s}$$

And by Eq. 8.1:

$$J = 1.21 \times 10^{12} \left(\frac{16.5}{130}\right)^{1.852} (99)^{-4.87} = 5.06$$

and by Eq. 8.8 with F = 0.36 from Table 8.7:

$$h_f = 5.06 \times 0.36 \times \frac{396}{100} = 7.21 \text{ m} = 7.07 \text{ kPa}$$

DIMENSIONLESS PIPE FRICTION CURVE

The head loss along any multiple-outlet, single-diameter pipeline that has uniform outlet spacing and discharge can be represented by a single curve on a dimensionless plot. Figure 8.2 shows such a plot where the horizontal scale is the dimensionless ratio of any length, x, measured from the closed end of the line to the total length, L. The vertical axis is the dimensionless ratio of the friction loss in length x, h_{fx}, to the friction loss in the total length, h_f. The dimensionless ratios used in plotting the curve are presented in Table 8.8. This general friction curve can be adapted to a specific problem by multiplying the dimensionless length ratios by L and the friction-loss ratios by h_f for a specific lateral or manifold pipe diameter, flow rate, number of outlets, and length.

The data in Table 8.8 used in plotting the general friction curve can be obtained from an outlet-by-outlet analysis of a typical multiple-outlet line. It can also be determined mathematically by noting that the flow rate at any distance x from the closed end is $(x/L)Q$. Combining Eqs. 8.1, 8.7a, or 8.7b with 8.8a and replacing Q with $(x/L)Q$ and L with $(x/L)L$ gives:

$$h_{fx} = J\,F\,\frac{L}{100}\left(\frac{x}{L}\right)^{(1+b)} \tag{8.10a}$$

which can be reduced further to give:

$$h_{fx} = h_f\left(\frac{x}{L}\right)^{(1+b)} \tag{8.10b}$$

where

x = distance from the closed end, m (ft)
h_{fx} = friction head loss from x to the closed end, m (ft)
L = length of the multiple-outlet (lateral) pipeline, m (ft)
J = head loss gradient in the lateral pipe between the inlet and the first outlet, m/100 m (ft/100 ft)
F = reduction coefficient to compensate for the discharge along the lateral pipe of length L
b = velocity of flow exponent of the head loss equation used

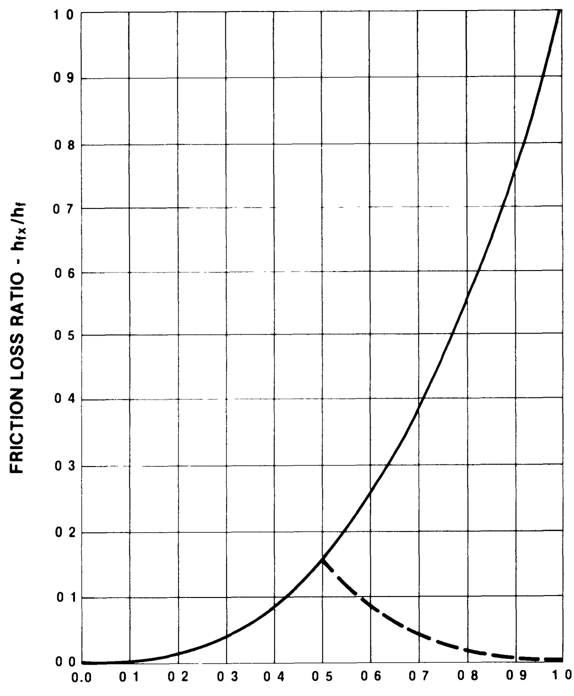


FIG. 8.2. General Friction Curve for a Multi-outlet Pipeline of Uniform Diameter that Has Uniform Spacing Between Outlets, Uniform Flow Per Outlet, and a 1.75 Flow Exponent.

Table 8.8. Dimensionless values of relative distance and relative pipe fraction to the closed end of multiple outlet pipelines for a 1.75 flow exponent

x/L	h_{fx}/h_f	x/L	h_{fx}/h_f
0.10	0.002	0.60	0.245
0.20	0.013	0.65	0.305
0.25	0.023	0.70	0.374
0.30	0.037	0.75	0.452
0.35	0.057	0.80	0.540
0.40	0.081	0.85	0.638
0.45	0.112	0.90	0.747
0.50	0.149	0.95	0.868
0.55	0.193	1.00	1.000

LIFE-CYCLE COSTING

The most economical size (or combination of sizes) of pipe in a main line or submain is the one that will result in a reasonable balance between the annual fixed cost of owning the pipe and the annual operating cost of pumping water through it. This balance depends on: the annual hours of operation; the unit cost and expected rate of inflation of fuel used; pipe prices, anticipated life and friction characteristics; and the annual interest rate.

To optimize the life-cycle cost of a system, we must find the set of pipe sizes that gives the minimum sum of fixed plus operating costs. To visualize this, think of selecting the diameter of an irrigation water-supply line in an arid area where it will be in operation for 3000 hr each year. If a very small pipe is used, the fixed cost will be low, but the operating (power) cost of overcoming friction losses in the pipe will be relatively high. As the pipe diameter is increased, the fixed cost will also increase, but the power cost will decrease. The optimum pipe size is the one that minimizes the sum of the fixed plus power cost (see Fig. 8.3). If the supply line were delivering water in a humid area and operated only 500 hr per year, the power cost curve would be much lower. This would shift the "minimum sum" in Fig. 8.3 to the left, resulting in a smaller optimum pipe size.

Costing Factors

Life-cycle cost analysis can be made on a present-worth or an annualized basis. In either case the interest rate, i, the expected life of the investment, n, and an estimate of the expected annual rate of escalation in energy costs, e, must be considered. The present worth of the escalating energy factor, $PW(e)$, and the equivalent annualized cost of escalating energy factor, $EAE(e)$, can be computed by the following equations, taken from Pearson (1974), for $e \neq i$:

$$PW(e) = \left[\frac{(1 + e)^n - (1 + i)^n}{(1 + e) - (1 + i)}\right] \cdot \left[\frac{1}{(1 + i)^n}\right] \tag{8.11}$$

and

$$EAE(e) = \left[\frac{(1 + e)^n - (1 + i)^n}{(1 + e) - (1 + i)}\right] \cdot \left[\frac{i}{(1 + i)^n - 1}\right] \tag{8.12}$$

The standard capital recovery factor is computed by:

$$CRF = \frac{i(1 + i)^n}{(1 + i)^n - 1} \tag{8.13}$$

where:

e = decimal equivalent annual rate of energy escalation
i = time value of unsecured money to the developer or the decimal equivalent annual interest rate
n = number of years in the life cycle

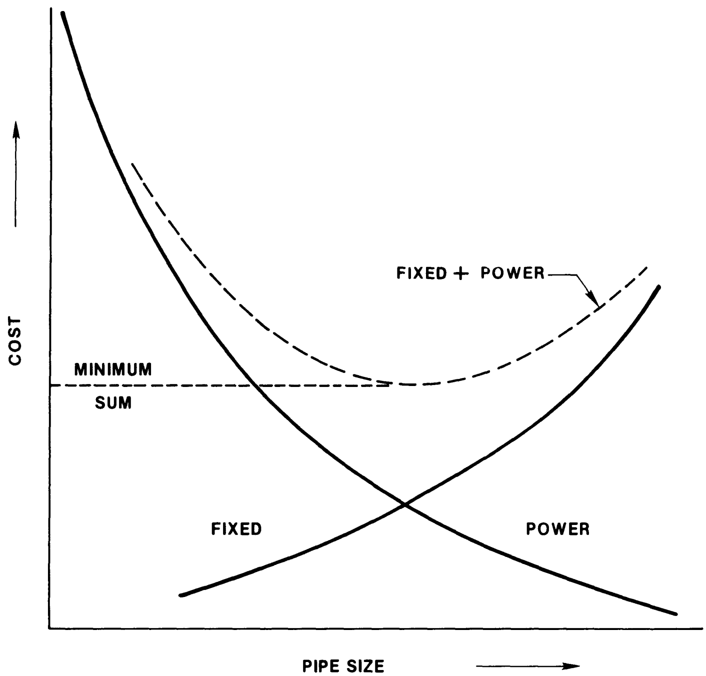


FIG. 8.3. Influence of Pipe Size on Fixed, Power, and Total Costs for a Given Flow Rate.

$PW(e)$ = present worth factor of escalating energy costs taking into account the time value of money over the life cycle

$EAE(e)$ = equivalent annualized cost factor of escalating energy taking into account the time value of money over the life cycle

CRF = uniform series annual payment (capital recovery factor), which takes into account the time value of money and depreciation over the life cycle

When considering life-cycle costing, the time value of unsecured money to the developer should be used as the appropriate interest rate, i. Using larger pipe to save energy should be profitable, not just a break-even situation for the developers. Therefore, due to the uncertainties involved, interest rates in the neighborhood of 5 to 10% higher than the interest rates on high-grade securities are typically used.

Table 8.9 gives the necessary factors for either a present-worth or annualized

Table 8.9. Present worth and annualized economic factors for assumed annual escalation in energy costs of 9 and 13.5% and various interest rates and life cycles

Factor	Interest i, %	Life cycle − n, years					
		7	10	15	20	30	40
$PW(13.5\%)$	10	7.004	10.509	17.135	24.884	44.547	71.442
$EAE(13.5\%)$		1.439	1.710	2.253	2.923	4.726	7.306
$PW(9\%)$		6.193	8.728	12.802	16.694	23.965	30.601
$EAE(9\%)$		1.272	1.420	1.683	1.961	2.542	3.129
CRF		0.205	0.163	0.132	0.118	0.106	0.102
$PW(0\%)$		4.868	6.145	7.606	8.514	9.427	9.779
$PW(13.5\%)$	15	5.854	8.203	11.917	15.396	21.704	27.236
$EAE(13.5\%)$		1.407	1.634	2.038	2.460	3.306	4.101
$PW(9\%)$		5.213	6.914	9.206	10.960	13.327	14.712
$EAE(9\%)$		1.253	1.378	1.574	1.751	2.030	2.215
CRF		0.240	0.199	0.171	0.160	0.152	0.151
$PW(0\%)$		4.160	5.019	5.848	6.259	6.566	6.642
$PW(13.5\%)$	20	4.967	6.569	8.712	10.334	12.490	13.726
$EAE(13.5\%)$		1.378	1.567	1.863	2.122	2.509	2.747
$PW(9\%)$		4.453	5.615	6.942	7.762	8.583	8.897
$EAE(9\%)$		1.235	1.339	1.485	1.594	1.724	1.781
CRF		0.277	0.239	0.214	0.205	0.201	0.200
$PW(0\%)$		3.605	4.193	4.676	4.870	4.979	4.997
$PW(13.5\%)$	25	4.271	5.383	6.651	7.434	8.215	8.513
$EAE(13.5\%)$		1.351	1.508	1.723	1.880	2.056	2.128
$PW(9\%)$		3.854	4.661	5.449	5.846	6.147	6.224
$EAE(9\%)$		1.219	1.306	1.412	1.479	1.539	1.556
CRF		0.316	0.280	0.259	0.253	0.250	0.250
$PW(0\%)$		3.161	3.571	3.859	3.954	3.995	4.000

life-cycle cost analysis. The table gives factors for 9 and 13.5% annual escalation in energy costs, for 10 to 25% interest rates and for life cycles of 7 to 40 years. The value $PW(0\%)$ is the present worth factor of nonescalating energy, taking into account the time value of money over the life cycle; clearly $PW(0\%) = 1/CRF$.

Fixed and Energy Cost Analysis

The expected life of different main line pipe materials is:

Portable aluminum	10–20 years
Coated welded steel	10–20 years
PVC plastic	20–40 years
Asbestos-cement	20–40 years

However, because of obsolescence, life cycles of $n = 20$ or less are frequently used for all pipes.

Some interesting observations can be made from Table 8.9 concerning the long-term effects of escalating energy costs.

1. Low time values of money deemphasize high first costs, as indicated by low *CRF*s.
2. Low time values of money emphasize escalating energy costs, as indicated by high $PW(e)$s, and $EAE(e)$s, but have considerably less effect on nonescalating energy costs as indicated by $PW(0\%)$.
3. High time values of money emphasize high first costs, but deemphasize energy costs.
4. Long useful life deemphasizes high first costs, but emphasizes energy costs.
5. Escalating energy costs have a maximum effect when the time value of money is low and the life cycle is long.
6. The relative effect of escalating versus nonescalating energy costs can be observed by comparing $PW(9\%)$ to $PW(0\%)$ or $EAE(9\%)$ to $EAE(0\%) = 1.0$ for any life cycle and time value of money.

The number of brake horsepower hours* per unit of fuel that can be expected from efficient power units is:

Diesel	4.0 hp-hr/L (15.0 hp-hr/U.S. gal)
Gasoline (water-cooled)	2.8 hp-hr/L (10.5 hp-hr/U.S. gal)
Tractor fuel	2.2 hp-hr/L (8.5 hp-hr/U.S. gal)
Butane-propane	2.5 hp-hr/L (9.5 hp-hr/U.S. gal)
Natural gas	3.0 hp-hr/m^3 (8.5 hp-hr/100 ft^3)
Electric	1.20 hp-hr/kWh at meter

*To convert to kW-hr from equivalent brake power outputs, multiply hp-hr by 0.746 for English hp (550 ft-lb$_f$/s) and 0.735 for metric hp (75 m-kg$_f$/s).

The factors presented in Table 8.9 can be used with the present value of the annual power costs, E, and the cost of the irrigation system, M, to estimate the following:

1. The present worth of energy escalating at 9% per year is equal to $E \times PW(9\%)$;
2. The equivalent annual cost of energy, E', escalating at 9% per year is $E' = E \times EAE(9\%)$;
3. The annual fixed cost of the irrigation system is $M \times CRF$;
4. The present worth of nonescalating energy is $E \times PW(0\%)$;
5. In addition, it is obvious that the annual cost of nonescalating energy is equal to E; and
6. The present worth of the irrigation system is equal to M.

Economic Pipe Sizing

The main line between the pump and the critical outlet must be identified and sized first (as discussed in Chapter 9 on main line design). The sections between outlets will have uniform flow rates. These can be analyzed, one section at a time, to find the pipe diameter that gives the minimum sum of fixed plus power costs (see Fig. 8.3).

Rather than work with the total length of each section, it is as accurate and easier (Keller, 1965) to work with a unit length of pipe, for example 100 m (100 ft). This is because, when a given diameter of pipe is the most economical for any part of the section, it is also the most economical for the rest of the section with that flow rate.

Only a limited set of discrete pipe diameters are available that would be physically or logically suitable for a given flow rate. Therefore, only these diameters need to be checked to determine which one would give the lowest total annual fixed plus annual energy cost. This is a fairly tedious process, as will be demonstrated later in conjunction with Sample Calculations 8.3 and 8.5 and verification of the following Economic Pipe-Selection Chart design method (see Table 8.13).

For systems with downhill or branching main lines, the pipe-size-selection process becomes even more complex. However, as a beginning point, pipes should still be sized by the economic method. Then the pressure at each lateral inlet point should be computed to find the inlet point that requires the highest pump discharge head. Pipe sizes can then be reduced for the rest of the system, so that all lateral inlet pressures are the same. These adjustments are demonstrated in Sample Calculation 10.3, Chapter 10.

Although the selection of economical pipe sizes is an important engineering decision, it is often given insufficient attention, especially in relatively simple irrigation systems. Many designers use an arbitrary flow velocity or a unit friction loss to size pipe because they consider the economic pipe-size-selection methods to be too time-consuming, limited, or complex (see Sample Calculation 8.5).

Economic Pipe-Selection Chart

The Economic Pipe-Selection chart was developed (Keller, 1975) to simplify the design process. The chart can be constructed for a given set of economic parameters and entered to directly select the most economical pipe diameters for nonlooping systems having a single pump station. The chart approach to economic design is particularly useful when technicians are employed to design a number of simple systems having the same economic parameters.

The Economic Pipe-Selection Chart (see Fig. 8.4) gives the locus (or region) of pipe section flow rates where each discrete diameter of pipe is most economical for various system flow rates. The total system flow rate, Q_s, affects the

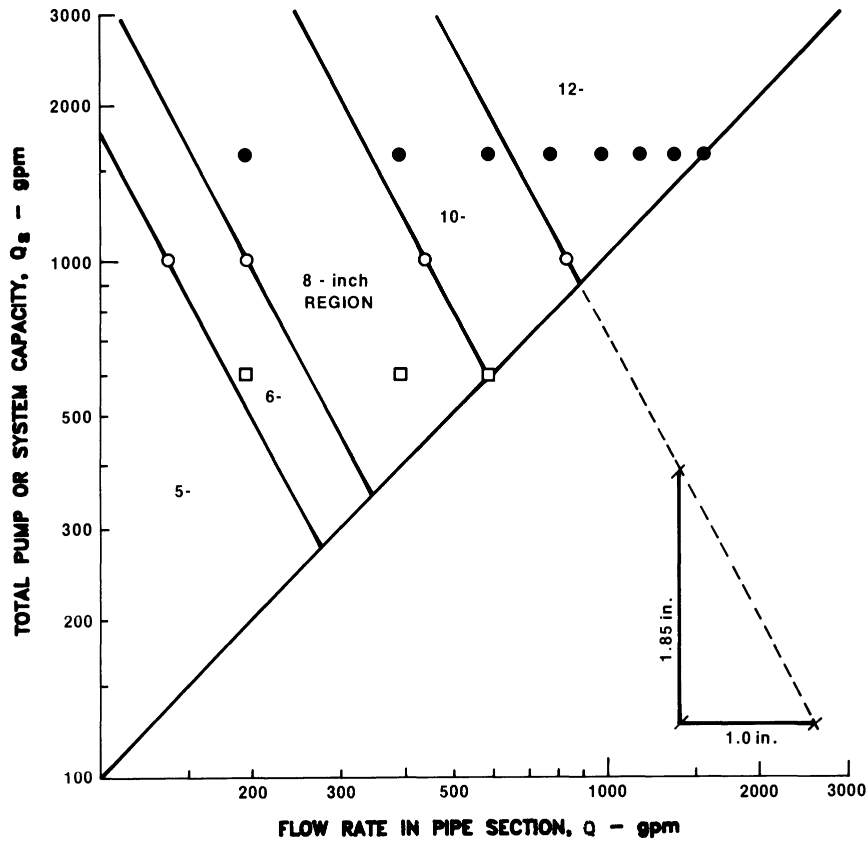


FIG. 8.4. "Economic Pipe Selection Chart" for Portable Aluminum Pipe with C = 130, *CRF* = 0.214 and *E'* = $138.60/whp-year.

selection, because the pressure loss due to pipe friction must be included in the total dynamic head at the pump's discharge. This is demonstrated in Sample Calculation 8.5.

The sloping boundary lines between the pipe-size regions in Fig. 8.4 are the locations where the total annual cost is the same for adjacent sizes of pipe. In other words, it would make no difference, in terms of total annual fixed plus annual power costs, which diameter of pipe is used.

Chart Construction Procedure. The following example demonstrates how the chart is constructed. (This section can be skipped for the time being and returned to later.) The example is worked in English dimensional units using a unit length of 100 ft of pipe. When working in metric units the unit length should be 100 m:

Step 1. The necessary economic data must be obtained:

a. For a time value of money $i = 20\%$ and expected life cycle of aluminum main line pipe of 15 years from Table 8.9, $CRF = 0.214$ and $EAE(9\%) = 1.485$;

b.

Nominal diameter	Price / 100 ft	Annualized fixed cost / 100 ft (0.214 × price / 100 ft)
5-in.	\$150	\$32.10
6-in.	\$200	\$42.80
8-in.	\$250	\$53.50
10-in	\$300	\$64.20
12-in.	\$350	\$74.90

c. Diesel fuel at \$1.05 / U.S. gal. gives a full cost per unit of brake power output, $C_f = \$0.07/\text{hp-hr}$;

d. Estimated hours of operation per year is $O_t = 1000$ hr; and

e. Hazen-Williams resistance coefficient for portable aluminum mainline pipe is $C = 130$.

Step 2. Determine the yearly fixed cost difference between adjacent pipe sizes and enter this in Table 8.10.

Step 3. The equivalent annual cost per water horsepower (hp) of energy escalating at 9% per year assuming a pump efficiency, $E_p = 75\%$ is:

a. The present annual cost of energy per unit of water power output, E, is:

$$E = \frac{O_t C_f}{(E_p/100)} \tag{8.14}$$

$$= \frac{1000 \times 0.07}{0.75} = \$93.33/\text{hp-year}$$

b. The equivalent annual cost of escalating energy per water hp, E', with $EAE\,(9\%) = 1.485$ (form Eq. 8.12 or Table 8.9) is:

$$E' = 1.485 \times \$93.33/\text{hp-year}$$
$$= \$138.60/\text{hp-year}$$

Step 4. The water hp savings needed to offset the annual fixed cost difference between adjacent pipe diameters is equal to the fixed cost difference divided by E'. The required values are presented in Table 8.10, and an example calculation for 8- and 10-in. pipe is:

$$\Delta\text{Power}_{(8-10)} = \frac{\$10.70/100\ \text{ft-year}}{\$138.60/\text{hp-year}}$$
$$= 0.077\ \text{hp}/100\ \text{ft}$$

Step 5. The differences in head loss gradient between adjacent (small-big) pipe diameters, $\Delta J_{(s-b)}$, needed to obtain the above $\Delta\text{Power}_{(s-b)}$ values, are presented in Table 8.10. The general equation and an example calculation are:

$$\Delta J_{(s-b)} = \frac{K\ \Delta\text{Power}_{(s-b)}}{Q_s} \tag{8.15}$$

where

$\Delta J_{(s-b)}$ = difference in head loss gradient between adjacent pipe diameters, m/100 m (ft/100 ft)
K = conversion constant, which is 102 for metric units (3960 for English units)
$\Delta\text{Power}_{(s-b)}$ = water power savings needed to offset the annual fixed cost difference between adjacent pipe diameters, kW/100 m (hp/100 ft)
Q_s = total system capacity, L/s (gpm)

Table 8.10. Sample data and procedure for locating economic pipe-size regions on selection chart for: $C = 130$; CRF $= 0.214$; $E' = \$138.60$/hp-year; and $Q_s = 1000$ gpm

		Adjacent pipe size pairs Nominal diameters-in.			
Step	Item	5–6	6–8	8–10	10–12
2	Yearly fixed cost difference – \$/100 ft	10.70	10.70	10.70	10.70
4	ΔPower – hp/100 ft	0.077	0.077	0.077	0.077
5	ΔJ – ft/100 ft	0.31	0.31	0.31	0.31
6	Q – gpm	140	200	450	850

For an assumed system flow rate, $Q_s = 1000$ gpm:

$$\Delta J_{(8-10)} = \frac{3960 \times 0.077 \text{ hp}/100 \text{ ft}}{1000 \text{ gpm}}$$
$$= 0.31 \text{ ft}/100 \text{ ft}$$

Step 6. The flow rates, Q, that would produce the required $\Delta J_{(s-b)}$ between adjacent pipe sizes are shown in Table 8.10. These flow rates can be determined by trial and error, using J values from pipe friction loss calculations or tables. For example, to get $J_{(8-10)} = 0.31$ ft/100 by trial and error using Table 8.4, find that at $Q = 450$ gpm:

$$\begin{aligned} J_{(8)} &= 0.46 \text{ ft}/100 \text{ ft} \\ \underline{-J_{(10)}} &\underline{= 0.15 \text{ ft}/100 \text{ ft}} \\ \Delta J_{(8-10)} &= 0.31 \text{ ft}/100 \text{ ft} \end{aligned}$$

(To obtain $\Delta J_{(s-b)}$ values graphically, construct a log-log plot of flow versus head loss differences between adjacent pipe sizes.)

Step 7. Plot the points representing the system flow used in Step 5, $Q_s = 1000$ gpm, at the pipe flow rates determined in Step 6, on log-graph paper in Fig. 8.4 (see the open circles).

Step 8. Draw lines of negative slope through each of the points plotted in Step 7. The slope should be equal to the exponent of the flow or velocity term used in the pipe friction equation used, which for this example is a slope of -1.85 (since the Hazen-Williams equation was used). These lines represent the set of pipe flow rates, Q, that give the same fixed plus operating cost as adjacent sizes of pipe for different system flow rates, Q_s. Each pair of lines defines the region in which the size common to both lines is the most economical pipe to use.

Figure 8.4 shows the complete Economic Pipe-Size-Selection Chart. The circles on the 2×2 cycle log-log graph paper at a system capacity or flow rate $Q_s = 1000$ gpm represent the pipeline flow rates, Q, found in Step 6 and presented in the last line of Table 8.10.

Changing any of the economic factors will shift the lines in the chart shown in Fig. 8.4. Developing a new chart for a new set of economic factors is simple when the spacing between lines remains constant, such as for a different E' or *CRF*, or when the pipe prices are all changed proportionally. Construction Steps 1 through 6 need be repeated for only one pair of adjacent pipe sizes at a single Q_s. This Q_s versus Q point locates the new position for the line in question, and all other lines can be shifted an equal distance from and drawn parallel to their original positions.

Design of Economical Main Line

The negative sloping lines on Fig. 8.4 represent all the possible Q_s versus Q values for each of the adjacent pairs of pipe sizes that will give the same sum of fixed plus operating costs. The zone between adjacent lines defines the region of Q_s versus Q values where the pipe size common to both lines is the most economical choice. The chart is universally applicable for pipe-size selections in any series nonlooping system for the economic boundary conditions assumed.

Sample Calculation 8.2. *Use of Economic Pipe-Selection Chart.*

GIVEN: Pipe system flow rates and layouts shown in Fig. 8.5A and B Economic Pipe-Selection Chart, Fig. 8.4.

FIND: The most economical pipe sizes for Systems (A) and (B), using portable aluminum pipe with the same economic parameters considered in developing Fig. 8.4.

CALCULATION:

System (A)

This pipe system is to deliver 200 gpm to each of eight different hydrants in series, as shown in Fig. 8.5A. The pump discharge is $Q_s = 8 \times 200$ gpm = 1600 gpm, which is also the flow rate in the first section of pipe. The flow rate in the pipe will decrease by 200 gpm at each outlet, with the final section carrying only 200 gpm. The solid dots plotted on Fig. 8.4 are the Q_s versus Q points representing this system. The Pipe Size Region where each point falls is the pipe size to use for that section. The pipe sizes and flow rates for each reach are shown on Fig. 8.5A. Because 12-in. pipe is the largest pipe size considered in setting up the chart, the 12-in. region is exaggerated. If 14-in. pipe had been considered, some flows would have fallen in its region.

System (B)

This system has three 200-gpm hydrants in series, so that Q_s = 600 gpm, as shown in Fig. 8.5B. The Square symbols plotted on Fig 8.4 are the Q_s versus Q points representing the system. The flow rates and recommended pipe sizes for each reach are shown on Fig. 8.5B. It is interesting to note that where Q = 200 gpm in the smaller system, 6-in. pipe should be installed, and in the larger system 8-in. pipe should be recommended. Similarly, where Q = 600, the larger system should have 10-in. pipe, though the smaller requires only 8-in. pipe. This is because the added power cost to offset friction for a given q increases with Q_s.

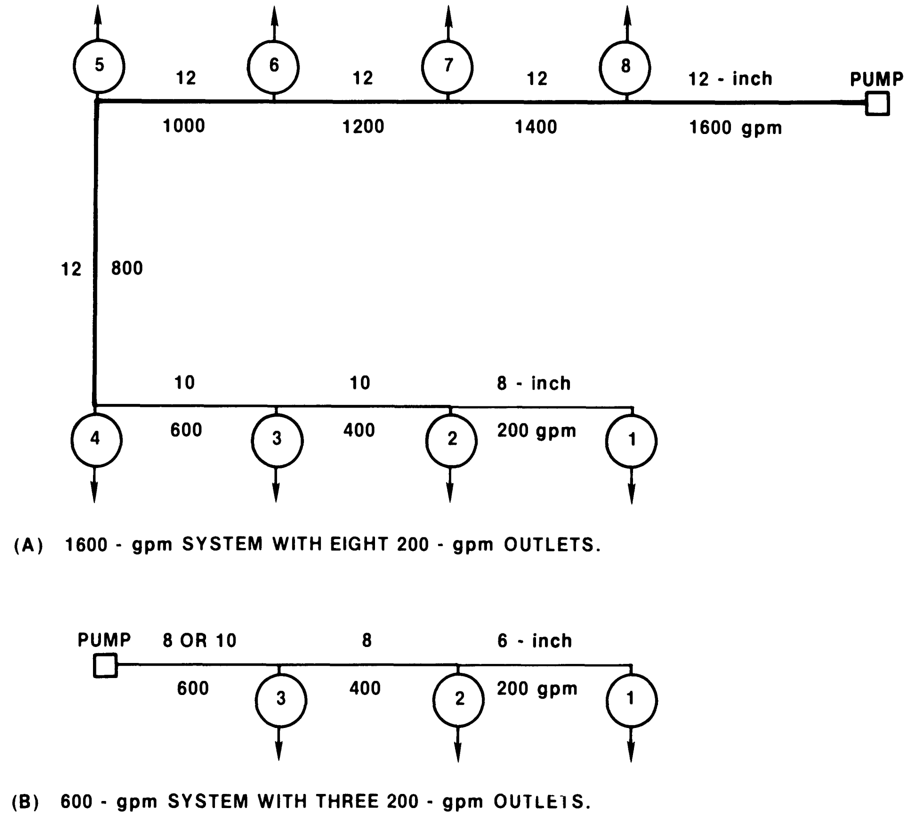


FIG. 8.5. Water Distribution Systems with Pipe Diameters Selected from the "Economic Pipe Size Selection Chart," Shown in Fig. 8.4.

The preceding example with solutions shown in Fig. 8.5 are applicable for the main branch of any nonlooping pipeline system when that branch is uphill, level, or moderately downhill from the pump. Many practical system layouts involve boundary conditions that do not meet these criteria. For these situations the trial-and-error solutions for determining the most economical pipe sizes become even more time-consuming, and the Economic Chart method requires some adjustment. Some instances requiring adjustment of the chart method are the following: (a) subbranch, parallel, or branched series pipelines; and (b) pipelines running down steep slopes where pipe sizes selected by the Economic Chart method would result in pressure gains due to elevation differences greater than their pressure losses due to friction. Although in these cases the pipe sizes selected using the Economic Chart method as presented in Fig. 8.4 must be adjusted downward, the adjustments are direct and yield the most economical pipe sizes for such conditions. Sample Calculation 10.1 in Chapter 10 demonstrates the use of these adjustments.

PIPE-DIAMETER-SELECTION METHODS

Various designers use different methods to size sprinkle and trickle system main lines.

The recommended technique is the:

1. *Economic method*—selecting the least sum of fixed plus power costs, as described in the above section on life-cycle costing;

Other methods in use are the:

2. *Unit head loss method*—setting a limit on the head loss per unit length, for example, 2.0 m/100 m (2.0 ft/100 ft);
3. *Velocity method*—setting a limit on the velocity, usually between 1.5 and 3 m/s (5 and 10 ft/s); and
4. *Percent head loss method*—setting a limit on the friction head loss in the main line network, for example, by allowing main line pressure to vary by 10 to 20% of the desired average sprinkler operating pressure.

The economic method can be done by constructing and utilizing an Economic Pipe-Selection Chart, such as Fig. 8.4, or by merely comparing the fixed power costs of the most reasonable combinations of pipe sizes. In the following example all the selection methods are compared, to demonstrate the value of the Economic Chart method.

Sample Calculation 8.3. *Comparison of pipe sizes selected using different methods.*

GIVEN: System layout shown in Fig. 8.6;
Aluminum pipe and cost data used in developing the example; and
Economic Pipe-Selection Chart, Fig. 8.4, and the economic parameters used for constructing it.

FIND: Pipe diameters for each section based on:
1. Head loss gradient of 2.0 ft/100 ft or less;
2. Maximum flow velocity of 7.0 ft/s or less;
3. Maximum total main line friction head loss of 15% of $P_a = 50$ psi, which is 7.5 psi or 17.3 ft; and
4. Economic method.

CALCULATIONS: *Selection by Head Loss Gradient:* Select pipe sizes from Table 8.4 such that the head loss gradient, J, will be less than but as close to 2.0 ft/100 ft as possible for each reach of pipe. The results of this procedure, which gives a total head loss of 21.4 ft due to pipe friction, are shown in Table 8.11.

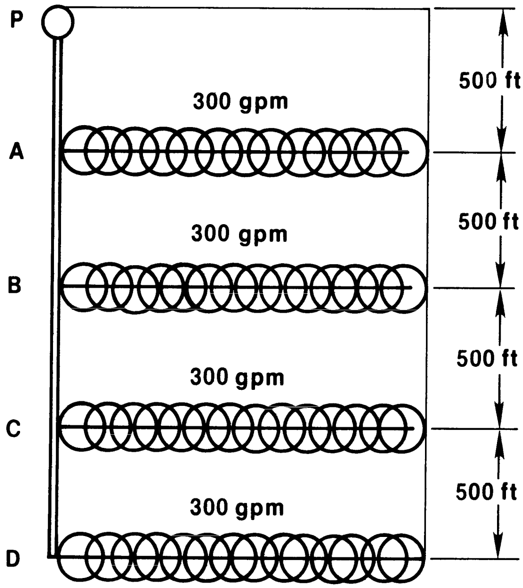


FIG. 8.6. Sprinkle System Layout for Sample Calculations 8.3 and 8.4.

Selection by Velocity Method: Select pipe sizes such that the flow velocity will be less than but as close to 7.0 ft/s as possible for each reach of pipe. This results in a total head loss of 39.8 ft due to pipe friction, as shown in Table 8.11. Flow rate limitations for each size of pipe were computed by:

$$Q = \frac{VD^2}{K} \tag{8.16}$$

where

Q = flow rate, L/s (gpm)
K = conversion constant, 1273 for metric units (0.4085 for English units)
V = velocity of flow in pipe, m/s (ft/s)
D = inside diameter of pipe, mm (in.)

Selection by Percent Head Loss Method: Select pipe sizes such that the total head loss does not exceed 17.3 ft., i.e., 15% of 50 psi (115.5 ft), for example. For a beginning point let the maximum unit head loss be 2.0 psi/100 ft. This will be the same as for the head loss gradient method, in which the total head loss is 21.4 ft. Therefore, some pipe diameters must be increased to reduce the total head loss. First, the pipe size in the section having the greatest unit head loss should be increased; in this case the diameter in Section A-B is increased from 8- to 10-in. pipe. If this had not decreased the total head loss sufficiently, the pipe diameter in the section with the next highest unit head loss should have

Table 8.11. Data for Sample Calculation 8.3 showing the total pipe friction head loss obtained by different pipe-size selection methods

Pipe section	Flow gpm	Length ft	Diameter in.	J, ft/100 ft	Loss, ft
		Selection by head loss gradient			
P-A	1200	500	10	0.95	4.8
A-B	900	500	8	1.65	8.3
B-C	600	500	8	0.78	3.9
C-D	300	500	6	0.88	4.4
				Total h_f =	21.4
		Selection by velocity method			
P-A	1200	500	10	0.95	4.8
A-B	900	500	8	1.65	8.3
B-C	600	500	6	3.18	15.9
C-D	300	500	5	2.15	10.8
				Total h_f =	39.8
		Selection by percent head loss method			
P-A	1200	500	10	0.95	4.8
A-B	900	500	10	0.56	2.8
B-C	600	500	8	0.78	3.9
C-D	300	500	6	0.88	4.4
				Total h_f =	15.9
		Selection by economic method			
P-A	1200	500	12	0.39	2.0
A-B	900	500	12	0.23	1.2
B-C	600	500	10	0.26	1.3
C-D	300	500	8	0.22	1.1
				Total h_f =	5.6

been increased, and so on. The results of this procedure, which gives a total head loss of 15.9 ft, are shown in Table 8.11.

Selection by Economic Method: Select pipe sizes that will give the least sum of pumping (fuel) plus annual fixed (investment) costs, as discussed earlier under Life-Cycle Costing. In this simple example the set of practical pipe diameter combinations that should be considered are:

Section	Flow, gpm	Diameters, in.
P-A	1200	12, 10, or 8
A-B	900	12, 10, or 8
B-C	600	10, 8, or 6
C-D	300	8, 6, or 5

This results in 28 iterations if all combinations are considered in which an upstream pipe diameter is never smaller than a downstream section.

The Economic Pipe-Selection Chart presented as Fig. 8.4 was used to simplify the selection process. (If the economic parameters had been different for this problem, a new chart would have been required.) The resulting total head loss is 5.6 ft due to pipe friction, as shown in Table 8.11.

Comparative Analysis of Methods

At first glance it may be surprising that such large pipe diameters are called for by the economic method in Sample Calculation 8.3. The validity of the economic method can be tested by comparing the *total annual fixed plus operating costs* of the different sets of pipes in Table 8.11. To accomplish this the total pipe cost should be multiplied by the capital recovery factor, *CRF*, to obtain the annual fixed cost. The annual energy operating cost, CE', is equal to the total head loss, h_f, times the annual energy cost per unit of head loss. The CE' can be computed by:

$$CE' = \frac{EAE(e)\ E\ Q_s}{K} h_f \tag{8.17}$$

where

CE' = equivalent annual energy cost of head loss, \$
K = conversion constant, 102 for metric units (3960 for English units)
$EAE(e)$ = equivalent annualized cost factor of escalating energy
E = present annual cost of energy per unit of water power output from Eq. 8.14, \$/kW-year (\$/hp-year)
Q_s = total system capacity, L/s (gpm)
h_f = total head loss due to pipe friction, m (ft)

Table 8.12 shows a comparison of the total annual costs for the different pipe-size combinations presented in Table 8.11. From Table 8.12 it is apparent that the economic selection method gives the lowest total annual cost.

Even though the economic method usually gives the lowest total annual cost, developers often hesitate to make the higher initial investment required. This is because they are worried about initial capital costs in view of various economic risks involved. If this is true for the n, i, and e values used, then they can be changed, because they are actually projections or estimations. Reducing n or e or increasing i will cause the pipe-size regions on the Economic Pipe-Size Chart to shift to the right, thus giving smaller diameter pipes. The easiest thing to do is simply reduce n to the number of years the developers are willing to wait for energy savings to offset initial investment costs. If n is made small enough the ''economic'' method will give a lower capital cost than the ''percent'' method (see Table 8.12). However, it would still always give the optimum relative pipe sizing, which is not so for the other pipe-sizing methods.

Table 8.12. Comparison of total annual costs for different pipe-size combinations determined in Sample Calculation 8.3

Method (or size for C-D)	Initial capital cost, $	Annual fixed cost, $	Total h_f ft	Annual energy cost, $[2]	Total annual cost, $[3]
Gradient	5000	1070	21.4	899	1969
Velocity	4500	963	39.7	1667	2630
Percent	5250	1124	15.9	668	1792
Economic	6250	1338	5.6	235	1573

[1]CRF = 0.214 from Table 9.9 for n = 15 years and i = 20%.
[2]CE' = $42 for each foot of head loss as computed by Eq. 8.17 in which: Q_s = 1200 gpm; h_f = 1.0 ft; E = $93.33/whp-year from Eq. 8.14 for 1000 hr/yr, $0.07/bhp-hr and 75% pump efficiency; and EAE = 1.485 from Table 8.9 for n = 15 years, i = 20%, and e = 9%.
[3](Annual fixed cost) + (annual energy cost) = (total annual cost).

Verifying Economic Pipe-Selection Chart Method

The Economic Pipe-Selection Chart can be verified by an analysis of a unit length of each reach or section. This is demonstrated in Table 8.13 for section C-D of Fig. 8.6, where the flow rate is only 300 gpm. However, the total system capacity must be used in Eq. 8.17 to determine the annual cost of the head loss in Section C-D. This is necessary because the extra pressure head needed to compensate for the friction loss in any section of pipe must be provided (at the pumping plant) to the total system flow of Q_s = 1200 gpm.

In Table 8.13 the 8-in. pipe has the least total annual cost; thus, 8-in. pipe would be the most economical for section C-D. This is in agreement with the selection based on the Economic Pipe-Selection Chart (see Fig. 8.4).

UNIVERSAL ECONOMIC PIPE-SELECTION CHART

Figure 8.7 is a "Universal Economic Pipe-Selection Chart" developed for IPS-PVC thermoplastic pipe with SDR ratings of 26 for up to 2-1/2-in., 32.5 for 3-in., and 41 for 4-in. or larger pipe diameters. (These are the same pipe diameters and wall thicknesses used in computing friction loss in Tables 8.3 and 8.5.) The chart can be adjusted for a given set of economic parameters and

Table 8.13. Comparison of total annual cost for different pipe-size combinations for main line section C-D in fig. 8.6

Nominal pipe size in.	Initial capital cost, $/100 ft	Annual fixed cost,[1] $/100 ft	Head loss C-D, h_f/100 ft	Annual energy cost,[2] $/100 ft	Annual total cost,[3] $/100 ft
10	300	64	0.07	3	67
8	250	54	0.22	9	63
6	200	43	0.88	37	80
5	150	32	2.15	90	122

[1], [2], and [3], see footnotes to Table 8.12.

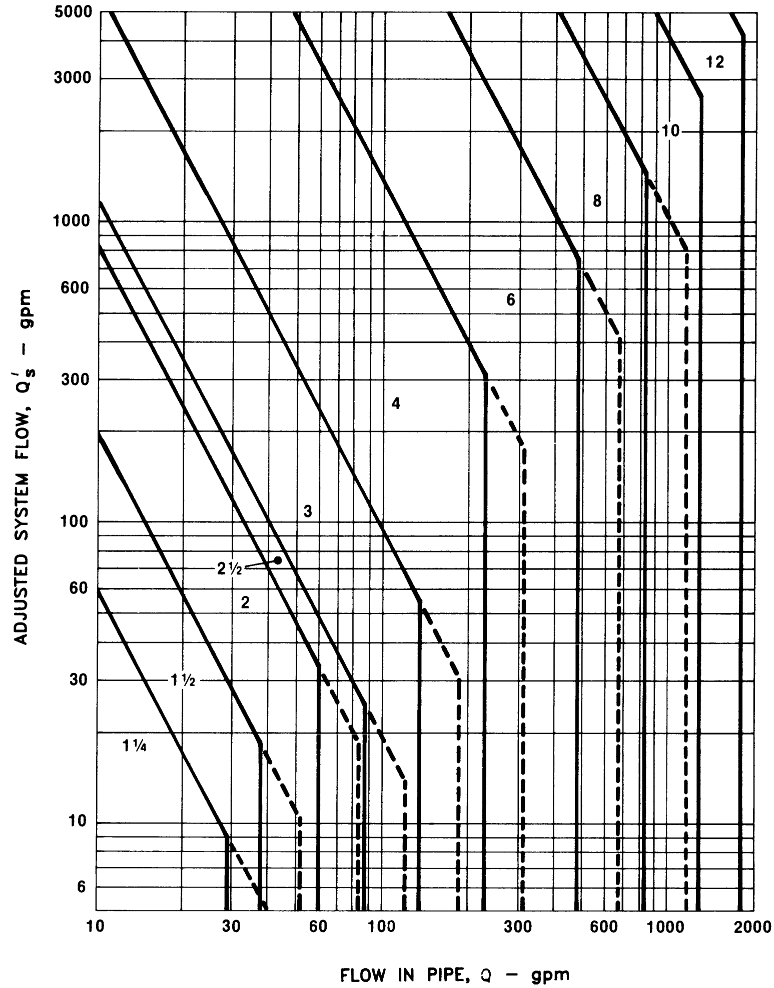


FIG. 8.7. "Universal Economic Pipe Size Selection Chart" for PVC Thermoplastic IPS Pipe with SDR Ratings of 26 and up Through 2 1/2-Inch, 32.5 for 3-Inch and 41 for 4-Inch and Larger Diameters.

entered to directly select the most economical pipe sizes for nonlooping systems that have a single pump station. The chart is "universal" only for the above set of pipe diameters and SDR ratings (pipe wall thicknesses). The relative positions of the lines would be slightly different for different SDR (pressure) rating.

Chart Construction

A Universal Economic Chart for PVC pipe is constructed assuming some convenient values for: the capital recovery factor, *CRF*; the equivalent cost per water horsepower per year, *E'*; and the cost of PVC per unit weight. The steps in developing the chart are the same as those used in developing Fig. 8.4. Pipe costs are estimated by multiplying the cost of PVC per unit weight by the weight of PVC per unit length of pipe.

A convenient system flow rate, Q_s, can be used in Eq. 8.15 (see Step 5) to compute each $\Delta J_{(s-b)}$. Use the following equation to obtain the flow rate Q that will produce the desired $\Delta J_{(s-b)}$ directly (rather than by trial and error as in Step 6 under the Economic Pipe-Selection Chart construction procedures presented earlier):

$$Q = \left[\frac{\Delta J_{(s-b)}}{K(D_s)^c - K(D_b)^c}\right]^{1/b} \tag{8.18}$$

where

Q = flow rate that will produce $\Delta J_{(s-b)}$, L/s (gpm)
$\Delta J_{(s-b)}$ = difference in head loss gradient between adjacent pipe diameters, m/100 m (ft/100 ft)
D_s = inside diameter of smaller pipe, mm (in.)
D_b = inside diameter of next size larger (or bigger) pipe, mm (in.)

For use with the Hazen-Williams equation, Eq. 8.1:
K = conversion constant $(1.212 \times 10^{12})/C^b$ for metric units $(1050/C^b$ for English units); $b = 1.852$ and $c = -4.87$;

For small-diameter plastic pipes with D $\leq$ *125 m (5 in.), Eq. 8.7a*:
K = conversion constant 7.89×10^7 in metric units (0.133 for English units); $b = 1.75$ and $c = -4.75$

For large-diameter plastic pipe with D > *125 mm (5 in.), Eq. 8.7b*:
K = conversion constant 9.58×10^7 for metric units (0.100 for English units); $b = 1.83$ and $c = -4.83$;

The points should be plotted as in Step 7 under the Economic Pipe-Selection Chart procedure, and lines with a slope of −1.80 should be drawn through each point to define the regions in which the pipe size common to the lines on both sides is the most economical size to use. (The slope −1.80 corresponds to the average discharge exponent in Eqs 8.7a and 8.7b.)

At the lower end of each sloped line in Fig. 8.7 two vertical lines are drawn. The solid lines represent the flow rate that would give a velocity in excess of 1.5 m/s (5 ft/s). Since velocity restrictions override economic considerations, the vertical lines define the boundary between adjacent pipe regions at the higher flow rates. The dashed extensions are for velocities of 2.1 m/s (7 ft/s).

Use of Universal Chart

The Universal Economic Pipe-Selection Chart, Fig. 8.7, is based on: pipe costing \$1.00 per English pound weight for PVC thermoplastic IPS pipe with minimum acceptable SDR ratings, C_p = \$1.00/lb; E' = \$100/hp-year; and CRF = 0.100.

To use Fig. 8.7 for systems having different economic parameters, the design flow rate for the system, Q_s, must be adjusted to compensate for different pipe cost per unit weight, E', and CRF values. To do this, first determine E' by:

$$E' = \frac{O_t C_f \, EAE(e)}{(E_p/100)} \tag{8.19}$$

where

E' = equivalent annual cost of escalating energy per unit of water power output, \$/kW-year, (\$/hp-year)
O_t = total hours of operation per year, hr/year
C_f = fuel cost per unit of brake power output, (unit cost of fuel)/(output per unit of fuel), \$/kW-hr (\$/hp-hr)
$EAE(e)$ = equivalent annualized cost factor of escalating energy taken from Table 8.11 or computed by Eq. 8.12
E_p = pump efficiency, %

Next, determine the CRF from Table 8.9 or by Eq. 8.13 and compute the system flow-rate-adjustment factor:

$$A_f = \frac{K_{uc} E'}{CRF \; C_p} \tag{8.20}$$

where

A_f = system flow-rate-adjustment factor
K_{uc} = Universal Pipe-Selection Chart coefficient
C_p = pipe cost per unit weight, \$/kg (\$/lb)

The value of K_{uc} is dependent on the economic parameters used in developing a given universal chart. It is computed by rearranging Eq. 8.20 to solve for K_{uc}, and setting A_f = 1.0 and the economic parameters to the values used in developing the chart. For example, for Fig. 8.7:

$$K_{uc} = 1.0 \left[\frac{CRF \; C_p}{E'} \right]_{\text{chart}}$$

which for the parameters used in Fig. 8.7 is:

$$K_{uc} = \frac{1.0 \times 0.100 \times 1.00}{100} = 0.001$$

The system flow rate for entering a universal chart, such as Fig. 8.7, is equal to:

$$Q_s' = A_f Q_s \tag{8.21}$$

where Q_s' = adjusted system flow rate for entering the Universal Economic Pipe-Selection chart, L/s (gpm), and Q_s = system flow rate under consideration, L/s (gpm).

The procedure for using the chart is demonstrated in Sample Calculation 8.4, which follows.

Sample Calculation 8.4. *Use of a Universal Economic Pipe-Selection Chart.*

GIVEN: An electric pumping plant for an irrigation system that will be operated for an average of O_t = 2300 hr/yr;

The unit cost of electricity is \$0.07/kWh;
The pump efficiency is E_p = 82%;
Electricity is expected to have an annual rate of cost escalation, e = 9% = 0.09;
The life cycle desired is n = 20 years;
The desired rate of return is i = 20% = 0.20; and
The cost of PVC plastic pipe is C_p = \$0.76/lb.

FIND: Select the most economical PVC pipe sizes for the sprinkler system layout shown in Fig. 8.6 using the universal chart, Fig. 8.7.

CALCULATIONS: *First* using Table 8.9 (or Eqs. 8.12 and 8.13) find *EAE* (9%) = 1.594 and *CRF* = 0.205 for e = 9%, i = 20%, and n = 20 years.

Then determine the equivalent cost factor of escalating energy, E'. Assuming a meter to motor efficiency of 90%, the conversion factor for electricity is 1.20 hp-hr/kWh. Thus, C_f = 0.07/1.20 and by Eq. 8.19:

$$E' = \frac{2300 \times 0.07 \times 1.594}{(82/100) \times 1.20} = \$260.81/\text{hp-year}$$

Next determine the system flow-rate-adjustment factor by Eq. 8.20:

$$A_f = \frac{0.001 \times 260.81}{0.205(0.76)} = 1.67$$

From Fig. 8.6 the system flow rate is $Q_s = 1200$ gpm; therefore, by Eq. 8.21 the adjusted system flow rate for entering the universal chart is:

$$Q_s' = 1.67 \times 1200 = 2009 \text{ gpm}$$

From Fig. 8.7 the most economical pipe sizes for the sprinkler system depicted in Fig. 8.6 are:

P-A, $Q = 1200$ gpm; use 10-in. pipe;
A-B, $Q = 900$ gpm; use 10-in. pipe;
B-C, $Q = 600$ gpm; use 8-in. pipe; and
C-D, $Q = 300$ gpm; use 8-in. pipe.

It is interesting to note that $Q = 1200$ gpm is on the 12-in. edge of the 10-in. pipe size region due to the velocity restraint of $V = 5$ ft/s. This can be demonstrated by computing the velocity of flow in the 10-in. pipe (which has an inside diameter of 10.226 in.; see Table 8.5) using Eq. 8.16:

$$V = 0.4085 \frac{1200}{(10.226)^2} = 4.7 \text{ ft/s}$$

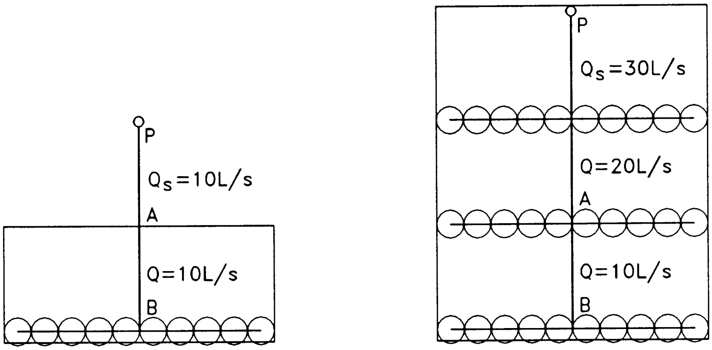
The vertical portion of the line separating the 10- and 12-in. regions is drawn at the pipe flow rate, $Q = 1280$ gpm, which produces $V = 5.0$ ft/s in the 10-in. pipe.

Sample Calculation 8.5. *Demonstration of numerical economic pipe size selection process and how the total system flow rate Q_s affects it.*

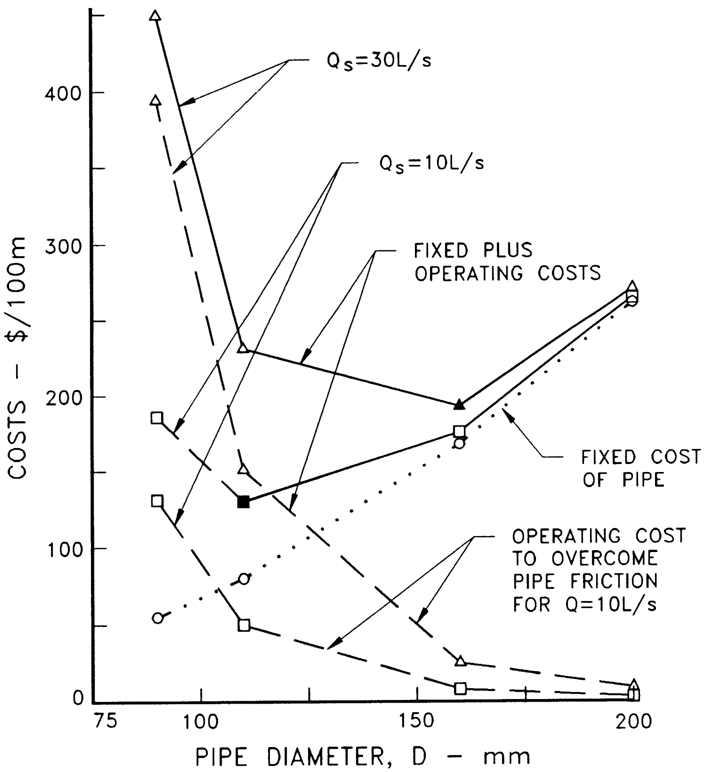
GIVEN: Two sprinkle irrigation systems as shown in Fig. 8.8A with the following general operating and economic parameters:

Interest rate of 10% so $i = 0.10$;	Diesel fuel cost \$0.20/L;
Economic life of $n = 10$ years;	Fuel output is 3.0 kW-hr/L;
Inflation rate of 9% so $e = 0.09$;	Pump efficiency is $E_p = 75\%$; and
Average annual operating time is $O_t = 3400$ hr.	

A set of 6-atmosphere pressure rated PVC plastic pipe with the following dimensions and costs based on PVC resin costing \$3.00/kg:



A. SYSTEM LAYOUTS



B. COSTS VERSUS DIAMETER FOR PIPE SECTIONS A–B

FIG. 8.8. Demonstration of Effect of System Flow Rate, Q_s, on the Sum of Annual Fired Plus Operating Costs.

Nominal Diameter mm	Wall Thickness mm	Inside Diameter mm	Weight kg/m	Cost $/100 m
90	2.7	84.6	1.130	339
110	3.2	103.6	1.640	492
160	4.7	150.6	3.440	1032
200	5.9	188.2	5.370	1611

FIND: The most economical pipe size for section A-B in the two systems shown in Fig. 8.8A and graphically demonstrate that the optimum pipe size is a function of the system discharge Q_s.

CALCULATIONS: Obtain the capital recovery factor, CRF = 0.163 from Table 8.9 (or by Eq. 8.13) for $n = 10$ years and $i = 10\%$. Then multiply the pipe costs by 0.163 to obtain the annual fixed costs of 100 m of each size of pipe.

Determine the equivalent annulized cost factor for escalating energy at $e = 0.09$, interest at $i = 0.10$, and a life cycle of $n = 10$ years from Table 8.9 or by Eq. 8.12 to obtain:

$$\text{EAE}(9) = \left[\frac{(1 + 0.09)^{10} - (1 + 0.10)^{10}}{(1 + 0.09) - (1 + 0.10)}\right] \cdot \left[\frac{0.10}{(1 + 0.10)^{10} - 1}\right]$$

$$= 1.420$$

Then compute the equivalent annual escalating cost of energy by:

$$E' = \frac{O_t\,(\text{unit fuel cost})\ \text{EAE}(e)}{(E_p/100)(\text{hp-hr per unit of fuel})}$$

$$= \frac{3400 \times 0.20 \times 1.420}{(75/100) \times 3.0} = \$429/\text{kW-year} \tag{8.19}$$

Compute the head loss per 100 m, J, for 10.0 L/s in the various pipe sizes by:

$$J = 7.89 \times 10^7 \frac{Q^{1.75}}{D^{4.75}} \quad D \leq 125\ \text{mm} \tag{8.7a}$$

or

$$J = 9.58 \times 10^7 \frac{Q^{1.83}}{D^{4.83}} \quad D > 125\ \text{mm} \tag{8.7b}$$

For example, for 90 mm pipe ID = 84.6 mm and:

$$J = 7.89 \times 10^7 \frac{(10)^{1.75}}{(84.6)^{4.75}} = 3.11\ \text{m}/100\ \text{m}$$

Then compute the cost of overcoming the friction head loss per 100 m, J, for system (pump) discharges of $Q_s = 10.0$ L/s and $Q_s = 30.0$ L/s for the different potential pipe sizes in sections A-B by Eq. 8.17 which can be reformulated as:

Table 8.14 Summary of calculated data giving the fixed, operating, and total annual cost for different pipe diameters in sections A-B of Fig. 8.8A.

Nominal Diameter mm	Fixed Cost \$/100 m	J at Q = 10 L/s m/100 m	Annual Operating Costs Q_s = 10 L/s \$/100 m	Annual Operating Costs Q_s = 30 L/s \$/100 m	Total Cost \$/100 m
90	55	3.11	131	—	186
110	80	1.19	50	—	130
160	168	0.196	8	—	176
200	262	0.067	3	—	265
90	55	3.11	—	392	447
110	80	1.19	—	150	230
160	168	0.196	—	25	193
200	262	0.067	—	8	270

$$\text{Annual Operating Cost} = CE' = \frac{E'Q_s}{K} J \tag{8.22}$$

where K = conversion constant which is 102 for metric units (3960 for English units)

For example, for the system with Q_s = 10.0 L/s:

$$(\text{Annual Operating Cost})_{90} = \frac{429 \times 10.0}{102} \times 3.11 = \$131/100 \text{ m}$$

And for the system with Q_s = 30.0 L/s:

$$(\text{Annual Operating Cost})_{90} = \frac{429 \times 30.0}{102} \times 3.11 = \$392/100 \text{ m}$$

Add the annual fixed and operating costs for each set of pipe diameters to determine the pipe size that gives the lowest total cost for each system as shown in Table 8.14.

Figure 8.8B, which is a plot of the data shows that 110 mm pipe is the optimum size for the system with Q_s = 10.0 L/s and 160 mm is optimum for the system with 30.0 L/s.

REFERENCES

Christiansen, J. E., 1942. Irrigation by sprinkling. *California Agric. Exp. Stn. Bull.* 670. University of California, Berkeley.

Churchill, S. W. 1977. Friction-factor equation spans all fluid-flow regimes. *Journal of Chemical Engineering*, 7 Nov., 1977:91–92.

Keller, Jack. 1965. Selection of economical pipe sizes in sprinkler irrigation systems. *ASAE Transactions* 8(2):186–191.

Keller, Jack. 1975. Economic pipe size selection chart. Proceedings of ASCE Irrigation and Drainage Division Specialty Conference, 13–15 August 1968, at Utah State University, Logan, Utah.

Pearson, G. F. 1974. Life-cycle costing in an energy crisis era. *Professional Engineer*, July, 1974:26–29.

Watters, G. Z., and J. Keller. 1978. Trickle irrigation tubing hydraulics. *ASCE Technical Paper* no. 78-2015.

9

Set Sprinkler Lateral Design

It is not practical to have the same pressure at each lateral outlet. Therefore, lateral line pipe sizes should be chosen so the total pressure variation in the line, due to both friction head and elevation (see Fig. 2.2), falls within the limits outlined in Chapter 7. This is typically taken as 20% of the average design operating pressure for the sprinklers. If this criterion is adhered to, the system CU will be approximately 0.97 × CU, and the system DU will be approximately 0.96 × DU (see Eq. 6.5) when the lateral inlet pressure, P_l, is regulated.

LATERAL DIAMETER AND INLET PRESSURE

Laterals must be specifically designed for fields that are level or slope uphill or downhill from the lateral inlet. Sample Calculation 9.1 is presented to demonstrate the general hydraulic characteristics of multiple-outlet pipelines. Included in the calculations are: the use of the reduction coefficient for computing the friction loss; a plot of the pipe friction-loss curve; and the relationship between the average, inlet, and minimum pressure heads, H_a, H_l, and H_n, along a lateral pipeline.

Sample Calculation 9.1. *Verifying the use of a reduction coefficient to compute head loss in a multioutlet line.*

GIVEN: A 2-in. aluminum pipe line that is 1000 ft long and has 10 outlets spaced 100 ft apart, with each outlet discharging 10 gpm.

FIND: Determine the head loss in the multiple-outlet pipeline by two methods: using data from Table 8.1 directly; and by Eq. 8.8a. Plot the pipe friction-loss curve and determine the average pressure head along the line and the location where the average occurs.

CALCULATIONS: *Using Table 8.1 directly* the total friction loss is equal to the sum of the individual losses in each 100-ft section of pipe. For example, the first section carries 100 gpm and has a loss of 28.36 ft; the second section carries 90 gpm with a loss of 23.33 ft; and the end section carries 10 gpm with a loss of 0.40 ft. The sum of the losses in the 10 sections is:

$$h_f = \text{sum of losses in each section} = 114.05 \text{ ft}$$

By Eq. 8.8a with $J = 28.36$ ft/100 ft for 100 gpm from Table 8.1 and $F = 0.40$ for 10 outlets (end) from Table 8.7:

$$h_f = JF\frac{L}{100} = 28.36 \times 0.40 \times \frac{1000}{100} = 113.4 \text{ ft}$$

This is essentially the same value as was obtained using Table 8.1 directly.

To plot the loss curve first compute the running sum of the losses beginning at the closed end of the pipeline and using the data from Table 8.1 for 2-inch aluminum pipe:

Section	Flow-gpm	h_f/100 ft − ft	Sum of h_f − ft
2-1	10	0.40	0.40
3-2	20	1.44	1.84
4-3	30	3.05	4.89
5-4	40	5.20	10.09
6-5	50	7.85	17.94
7-6	60	11.01	28.95
8-7	70	14.65	43.60
9-8	80	18.76	62.36
10-9	90	23.33	85.69
M-10	100	28.36	$h_f = 114.05$

The pressure head decreases from the inlet at M to a minimum at the number 1 outlet at the distal (closed) end of the lateral or manifold. The pressure head above minimum at each outlet is:

Outlet	Pressure head above H_n − ft	Outlet	Pressure head above H_n − ft
1	0.0	7	29.0
2	0.4	8	43.6
3	1.8	9	63.4
4	4.9	10	85.7
5	10.1	M	—
6	17.9		$\Sigma = 256$ ft

The solid-line curve between outlet 1 and the inlet at M in Fig. 9.1 is a plot of these data. (The solid-line curve between outlets 1 and 6 plus the dashed curve between outlet 6 and M is for a lateral with half 3-in. and half 2-in. aluminum pipe, which will be discussed in Sample Calculation 9.3.) In dimensionless form the data for the plot would be essentially the same as in Table 8.8, which is plotted to obtain Fig. 8.2.

The average outlet pressure head along the lateral, H_a is the sum of the outlet pressure heads above H_n divided by the number of outlets plus the minimum pressure head, H_n:

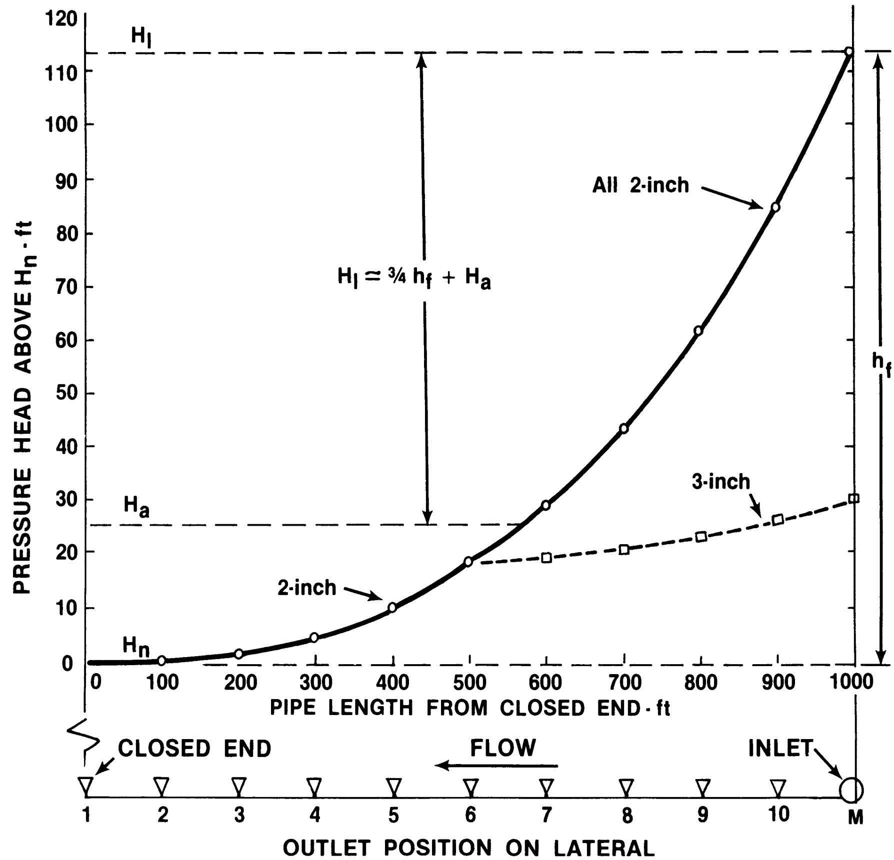


FIG. 9.1. Pipe Friction Loss Curves for an All 2-Inch (and a Half 2-Inch and Half 3-Inch) Aluminum Pipe with 10 Outlets Each Spaced 100 ft Apart and Discharging 10 gpm.

$$H_a = H_n + \frac{256}{10} = H_n + 25.6 \text{ ft}$$

The H_a occurs just downstream from the number 7 outlet or 60% of the distance from the closed to the inlet end of the pipeline. Furthermore, approximately $\frac{3}{4}$ of the total $h_f = 114$ ft occurs between the inlet at M and outlet number 7, which operates nearest H_a, i.e.:

$$\left[114 - \tfrac{3}{4}(114)\right] = 28.5 \text{ ft}$$

Average Sprinkler Discharge

The average sprinkler discharge, q_a, along the lateral will be approximately the same as the discharge from a sprinkler operating at the average outlet pressure head along the lateral, H_a. This is demonstrated in Sample Calculation 9.2.

Sample Calculation 9.2. *Verifying that a sprinkler operating at H_a has a discharge of approximately q_a.*

GIVEN: The data generated in Sample Calculation 9.1 for a 2-in. aluminum lateral with 10 sprinklers.

FIND: The average discharge of the 10 sprinklers along the lateral assuming $H_a = 400$ ft and a sprinkler operating at $H = 400$ ft discharges $q = 10.0$ gpm.

CALCULATIONS: Substituting in Eq. 5.1b to solve for the orifice discharge coefficient K_d yields:

$$K_d = \frac{10.0}{(400)^{1/2}} = 0.50$$

From the friction loss data found in Sample Calculation 9.1:

$$H_1 = H_n = 400 - 25.6 = 374.4 \text{ ft}$$

and using Eq. 5.1 with $K_d = 0.50$ to find the discharge of the end sprinkler:

$$q_1 = 0.50(374.4)^{1/2} = 9.67 \text{ gpm}$$

The pressure head at each successive upstream sprinkler can be computed by adding the friction head loss between the outlets, for example:

$$H_2 = H_1 + (h_f)_{1\text{-}2} = 374.4 + 0.4 = 375 \text{ ft}$$

and each sprinkler's discharge can be computed as above to give:

Outlet	h_f – ft*	H – ft	q – gpm
1	–	374.4	9.67
2	0.4	375	9.68
3	1.4	376	9.70
4	3.1	379	9.73
5	5.2	384	9.80
6	7.9	392	9.90
7	11.0	403	10.04
8	14.7	418	10.22
9	18.8	437	10.45
10	23.3	460	10.72
		TOTAL =	99.91

*The friction loss to the next downstream outlet.

Because there are 10 sprinklers, the average sprinkler discharge, $q_a = 9.99$ gpm. Thus, the above computations demonstrate that a sprinkler operating at H_a has a discharge of approximately q_a.

Lateral Head-Loss Gradient

The general equation for the allowable head-loss gradient, J_a, along a lateral line is:

$$J_a = K \frac{(0.20\, P_a - \Delta P_e)}{F(L/100)} \tag{9.1a}$$

$$= K \frac{(P_f)_a}{F(L/100)} \tag{9.1b}$$

$$= \frac{(h_f)_a}{F(L/100)} \tag{9.1c}$$

where

J_a = allowable head-loss gradient, m/100 m (ft/100 ft)
K = conversion constant, 0.102 for metric units (2.31 for English units)
P_a = average sprinkler (or emitter) operating pressure, kPa (psi)
ΔP_e = static pressure difference between the inlet and closed ends due to elevation difference along the lateral, which is positive (+) for *uphill* and negative (−) for *downhill* laterals, kPa (psi)
$(P_f)_a$ = allowable pressure loss due to pipe friction, kPa (psi)
$(h_f)_a$ = allowable pressure head loss due to pipe friction, m (ft)
F = multiple outlet adjustment coefficient from Table 8.7
L = lateral line length, m (ft)

To determine the necessary lateral pipe diameter, first compute J_a by Eq. 9.1a. Then enter Table 8.1 for aluminum or Table 8.3 for PVC thermoplastic pipe at the flow rate corresponding to the total lateral discharge. Now, moving along that line to the right, find the pipe-size column that contains a J value just equal to or less than J_a. This is the pipe size required. Reverse the procedure using the selected J value in Eq. 8.8b to determine the actual pressure loss due to friction, P_f.

Lateral Inlet Pressure

The general equation for the lateral inlet pressure (the pressure required at the main line end) for single pipe size laterals is:

$$P_l = P_a + \tfrac{3}{4}P_f + \tfrac{1}{2}\,\Delta P_e + P_r \tag{9.2a}$$

and,

$$H_l = H_a + \tfrac{3}{4}h_f + \tfrac{1}{2}\,\Delta H_e + H_r \tag{9.2b}$$

And for dual pipe size laterals, which have a more linear friction-loss curve, replace the $\frac{3}{4}$ with $\frac{5}{8}$ to obtain:

$$P_l = P_a + \tfrac{5}{8}P_f + \tfrac{1}{2}\Delta P_e + P_r \tag{9.3}$$

where

P_l = lateral inlet pressure (pressure required at the supply end), kPa (psi)
P_f = pressure loss due to pipe friction, kPa (psi)
P_r = pressure required to lift water up the riser, which is 9.8 kPa/m (0.43 psi/ft) of riser, kPa (psi)
H_l = lateral inlet pressure head, m (ft)
H_a = average sprinkler (or emitter) operating pressure head, m (ft)
h_f = head loss due to pipe friction, m (ft)
ΔH_e = static pressure head difference between the inlet and closed ends due to elevation difference, ΔEl, along the lateral, which is positive (+) for *uphill* and negative (−) for *downhill*, m (ft)
H_r = height of riser, m (ft)

(A discussion of the relationship between P_e and H_e is given in the first part of Chapter 8.)

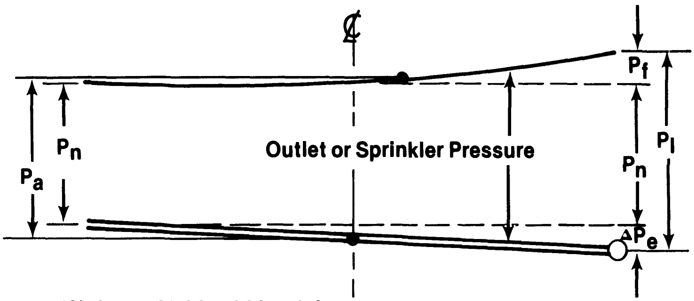
Equations 9.2a and 9.3 give the P_l required to obtain the design P_a for the systems. The design P_a is the pressure that will give the design q_a for the sprinkle configuration selected (see Sample Calculations 6.3, 6.4, and 6.6).

Lateral Design

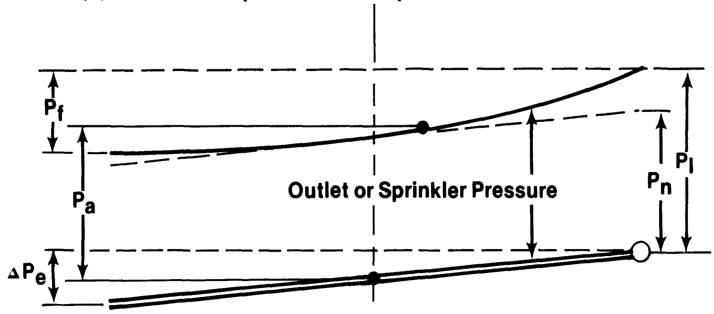
For laterals on level ground, $\Delta P_e = 0.0$ ($\Delta H_e = 0.0$), and the allowable pressure loss due to friction in the lateral line will be equal to 20% of P_a.

For *uphill laterals* (see Fig. 9.2a), P_f may be equal to 20% of P_a *minus* the static pressure difference due to elevation, ΔP_e, which is the difference in elevation between the inlet and closed ends of the lateral in meters multiplied by 9.8 to get kPa (or in feet multiplied by 0.43 to get psi). For uphill laterals ΔP_e *is positive* and increases as the elevation increases. Thus, it reduces the allowable pressure loss available for pipe friction, $(P_f)_a$, in Eq. 9.1b and increases the required lateral inlet pressure, P_l, determined by Eq. 9.2a or 9.3. Sample Calculation 9.4 illustrates the procedure for selecting the pipe diameter and determining P_l for uphill laterals.

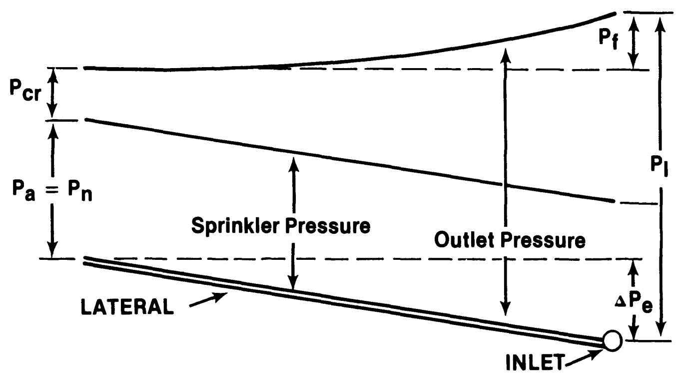
For uphill laterals the minimum pressure, P_n, occurs at the closed end of the pipeline, as is evident in Fig. 9.2A. Also, the pressure at any outlet is repre-



(A) Lateral laid uphill · minimum pressure occurs at closed end.



(B) Lateral laid down hill · minimum pressure occurs when pipe friction gradient equals slope.



(C) Lateral laid uphill with flow-or pressure- control valves- minimum equals average pressure.

FIG. 9.2. Pressure Relationship for Laterals Laid Uphill, Downhill, and Uphill with Flow- (or Pressure-) Control Valves (Assuming P_r = 0).

sented by the distance between the lateral and the pipe-friction curve. Furthermore, the average sprinkler pressure, P_a, is represented by the distance between the average P_e and average P_f (as indicated by the black dots on Fig. 9.2A).

For *downhill laterals* (see Fig. 9.2B) the allowable P_f is 20% of P_a plus the static pressure gain due to the decrease in elevation between the inlet and closed ends of the lateral, ΔP_e. For *downhill laterals* ΔP_e *is negative* as it decreases as elevation decreases along the pipeline. Thus, it increases the allowable pipe friction $(P_f)_a$, in Eq. 9.1b and decreases the required lateral inlet pressure, P_l, determined by Eq. 9.2a or 9.3.

For relatively *mild downhill slopes* the allowable head-loss gradient can be computed by Eq. 9.1. However, on *steep downhill slopes* where $(-\Delta P_e) >$ (0.3 to 0.4 P_a), it is usually desirable to minimize the pressure variation along the line. This can be done by selecting pipe sizes so that the friction loss approximately equals the static pressure gain along the lateral due to elevation, $P_f \approx (-\Delta P_e)$. Sample Calculation 9.5 illustrates the procedure for selecting the pipe diameter and determining P_l for downhill laterals.

For downhill laterals P_n occurs at the point along the lateral where the pipe-friction gradient equals the slope of the lateral (ground), as shown in Fig. 9.2B. As with uphill (or "level") laterals, the pressure at any outlet or the average sprinkler pressure is represented by the distance between the lateral and pipe friction curve, or average P_e and average P_f, respectively.

Laterals with Two Pipe Sizes

Most farmers prefer *portable aluminum lateral* lines of a single pipe size for convenience. A few want to use two pipe sizes where their use will result in a reduction in initial costs. Portable laterals containing more than two pipe sizes should never be considered; however, permanently buried laterals of multiple pipe sizes are practical.

The design of *buried plastic laterals* for permanent systems is essentially the same as for portable aluminum laterals. The main differences in design come from the difference in pipe friction and the fact that up to four different pipe sizes are often used.

The design of set sprinkler laterals and manifolds serving hose-fed sprinkler or trickle laterals are very similar, particularly in the case of buried fixed systems. For this reason further insight into the design of set sprinkler laterals may be gained by referring to Chapter 23, "Trickle Manifold Design."

Friction loss Tables 8.1 or 8.3 and F factor Table 8.7 can be used to find the nearest uniform pipe size for a lateral line that will result in a friction loss equal to or less than $(P_f)_a$. The tables may also be used to obtain the lengths of each of two pipe sizes to obtain a given P_f along a lateral line, by using the following procedure:

Step 1. Compute the allowable pressure loss due to pipe friction, $(P_f)_a$, for the total length of the line, as described in the previous sections.

Step 2. Compute the allowable head-loss gradient, J_a, by Eq. 9.1

Step 3. Enter Table 8.1 or 8.3 with the total lateral line capacity, Q_l, to find the two adjacent pipe sizes that have head-loss gradients, J values, greater and less than J_a.

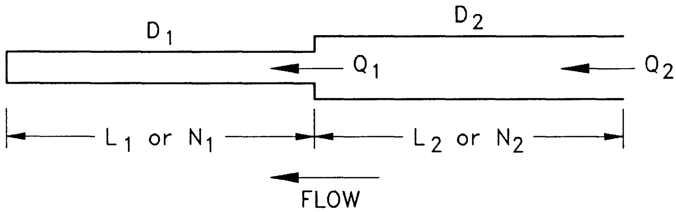


FIG. 9.3. Descriptive Labeling for Lateral with Two Pipe Diameters.

Step 4. Determining the specific lengths of each of the two pipe sizes required to obtain $(P_f)_a$ is an iterative procedure. First estimate lengths L_1 and L_2, where L_1 is the length of the smaller diameter pipe, and L_2 is the length of the bigger (or larger) pipe (see Fig. 9.3). Then compute the total pressure loss, P_f, due to friction for these lengths by the following procedures. (Note that all friction-loss calculations using Eq. 8.8 must consider the closed end of the multioutlet line; and all computations can be made in terms of head loss, h_f, in place of pressure loss, P_f.)

Step 5. Assume that the larger pipe diameter D_2 extends for the full length of the lateral line, and find the loss for length $L_1 + L_2 = L$ supplying $N_1 + N_2 = N$ sprinklers and discharging $Q_2 = Q_l$.

Step 6. Find the loss in length L_1 for pipe diameter D_2 supplying N_1 sprinklers and discharging Q_1.

Step 7. Then find the loss in length L_1 of the smaller pipe diameter, D_1, supplying N_1 sprinklers and discharging Q_1.

Step 8. Combine the losses to determine the pressure loss, P_f, as follows:

$$\begin{aligned} P_f &= \text{Step 5} - \text{Step 6} + \text{Step 7} \qquad (9.4a) \\ &= P_{f(1+2)}(\text{for all } D_2) - P_{f(1)}(\text{for } D_2) + P_{f(1)}(\text{for } D_1) \end{aligned}$$

Should this loss fall above or below $(P_f)_a$, choose different values of L_1 and L_2, and repeat Steps 5 through 8 until $P_f \approx (P_f)_a$.

There is a *direct solution* for determining the approximate length of (or number of sprinklers on) each pipe diameter that will give $(P_f)_a$. This solution is based on the logic that leads to Eq. 8.10b, combined with that used in the above iterative procedure, as follows:

Steps 1–3. Complete as before.

Step 4. This step is omitted because the solution is direct.

Step 5. Complete as before to find the pressure loss due to pipe friction, assuming the larger diameter, D_2, extends for the full length of the lateral, $L_1 + L_2 = L$, to give:

$$P_{f(1+2)}(\text{for all } D_2) = (P_f)_2$$

Also find the pressure loss due to pipe friction, assuming the smaller diameter, D_1, extends for the full length of the lateral to give:

$$P_{f(1+2)}(\text{for all } D_1) = (P_f)_1$$

Step 6. According to the logic used in developing Eq. 8.10b, the pressure loss due to pipe friction in L_1 for diameter D_2 supplying N_1 sprinklers is:

$$(P_f)_{(1)}(\text{for } D_2) = \left(\frac{L_1}{L}\right)^{2.8} \cdot (P_f)_2$$

Step 7. In a similar manner, the loss in L_1 for D_1 is:

$$(P_f)_{(1)}(\text{for } D_1) = \left(\frac{L_1}{L}\right)^{2.8} \cdot (P_f)_1$$

Step 8. Replacing P_f with $(P_f)_a$, substituting the results of Steps 5, 6, and 7 into Eq. 9.4a and rearranging gives:

$$(P_f)_a = (P_f)_2 + \left(\frac{L_1}{L}\right)^{2.8} \cdot [(P_f)_1 - (P_f)_2] \tag{9.4b}$$

and solving for L_1 gives:

$$L_1 = L\left[\frac{(P_f)_a - (P_f)_2}{(P_f)_1 - (P_f)_2}\right]^{1/2.8} \tag{9.4c}$$

In working with Eq. 9.4b or c, the numbers of sprinklers can be substituted for the L_1, i.e., replace L with N and L_1 with N_1. As before, the head-loss, h_f, values can be used in place of P_f values.

Equation 9.4c can be simplified further by replacing the P_f values with the corresponding J values used in computing them. This is possible because the F and L values used in computing the P_f values are the same and cancel out in Eq. 9.4c to give:

$$L_1 = L\left[\frac{J_a - J_2}{J_1 - J_2}\right]^{1/2.8} \tag{9.4d}$$

Sample Calculation 9.3. *Verifying the procedure for computing the pipe-friction loss in laterals with two or more pipe sizes.*

GIVEN: An aluminum pipe line, as in Sample Calculation 9.1, that is 1000 ft long and has 10 outlets spaced 100 ft apart, with each outlet discharging 10 gpm. The last 5 outlets on the line are supplied by 2-in. pipe and the first 5 by 3-in. pipe.

FIND: Determine the head loss, h_f, in the multiple-outlet pipeline using data from Table 8.1 directly. Compare this result with the h_f computed by Eq. 8.8a using the two pipe-size computation methods described above. Plot the pipe friction curve, and determine the average pressure head and its location.

CALCULATIONS: According to the procedures used in Sample Calculation 9.1, the running sum of the losses is the same for the section with 2-in. pipe between outlets 1 and 6. With 3-in. aluminum pipe (see Table 8.1) from outlet 6 to M the running sum is:

Section	Flow-gpm	h_f/100 ft − ft	Sum of h_f − ft
From 6 to 1 (which is 5 sections of 2-in.)			17.94
7-6	60	1.40	19.34
8-7	70	1.87	21.21
9-8	80	2.39	23.60
10-9	90	2.98	26.58
M-10	100	3.62	h_f = 30.20

Thus, the total h_f = 30.20 ft by the summation of the individual losses in each 100-ft section taken from Table 8.1.

The pressure head above minimum, H_n, at each outlet is:

Pipe section	Pipe size-in.	Outlet	Pressure head above H_n − ft
1-2	2	1	0
2-3	2	2	0.4
3-4	2	3	1.8
4-5	2	4	4.9
5-6	2	5	10.1
6-7	3	6	17.9
7-8	3	7	19.3
8-9	3	8	21.3
9-10	3	9	23.6
10-M	3	10	26.6
			Σ = 126 ft

These data are plotted on Fig. 9.1 and represented by the solid-line curve from outlet 1 to 6 and the dashed curve from outlet 6 to M, the lateral inlet.

The average outlet pressure head, H_a, computed as before is:

$$H_a = H_n + \frac{126}{10} = H_n + 12.6 \text{ ft}$$

In this case H_a occurs at about outlet 5 or about 40% of the distance from the closed to the inlet end of the pipeline. Furthermore, approximately $\frac{5}{8}$ of the total h_f = 30.2 ft occurs between the inlet at M and outlet number 5, which operates nearest H_a.

By Eq. 8.8a and Tables 8.1 and 8.7, using the procedure described in steps 5 through 8 discussed above, the h_f for all 3-in. pipe (Step 5) is:

$$h_{f(1+2)}\text{ (for all } D_2) = JF\frac{L}{100} = 3.62 \times 0.40 \times 10 = 14.5 \text{ ft}$$

The $h_{f(1)}$ for the last 5 sections ($F = 0.46$) of 3-in. pipe (Step 6) is:

$$h_{f(1)}(\text{for } D_2) = 1.00 \times 0.46 \times 5 = 2.3 \text{ ft}$$

The $h_{f(1)}$ for the last 5 sections with 2-in. pipe (Step 7) is:

$$h_{f(1)}(\text{for } D_1) = 7.85 \times 0.46 \times 5 = 18.1 \text{ ft}$$

And in accordance with Step 8, using Eq. 9.4:

$$h_f = 14.5 - 2.3 + 18.1 = 30.3 \text{ ft}$$

This is essentially the same as for the summation method, which gave $h_f =$ 30.2 ft.

Sample Calculation 9.4. *Design of lateral laid uphill with two pipe diameters.*

GIVEN: A lateral consisting of $L = 960$ ft of portable aluminum irrigation pipe with 24 sprinklers spaced 40 ft apart, discharging $q_a = 12.5$ gpm and operating at $P_a = 44$ psi;

Lateral capacity is $Q_l = 24 \times 12.5 = 300$ gpm;

Elevation difference is $\Delta El = 9.0$ ft (uphill) so $\Delta P_e = 0.43 \times 9.0 = 3.9$ psi; and

Height of risers for corn is $H_r = 8.0$ ft so $P_r = 0.43 \times 8.0 = 3.4$ psi.

FIND: Determine the smallest pipe sizes that will limit pressure loss due to both h_f and ΔEl to 20% of P_a, and compute the lateral inlet pressure, P_l.

CALCULATIONS: The procedures described in Steps 2 through 8 will be followed. Referring to Fig. 9.3, determine J_a by Eq. 9.1a using $F = 0.37$ from Table 8.7 for 24 outlets (Step 2):

$$J_a = 2.31\frac{(0.20 \times 44) - 3.9}{0.37 \times 960/100} = 3.19 \text{ ft}/100 \text{ ft}$$

Entering Table 8.1 with the lateral capacity of $Q_l = 300$ gpm, $J_a = 3.19$, falls between 5- and 4-in. aluminum pipe, i.e., $J_5 = 2.15$ and $J_4 = 6.54$ (Step 3). Assuming 480 ft of 5-in. pipe and 480 ft of 4-in. pipe (Step 4):

$D_2 = 5$-in.	$D_1 = 4$-in.
$L_2 = 480$ ft	$L_1 = 480$ ft
$N_2 = 12$	$N_1 = 12$
$Q_l = 300$ gpm	$Q_1 = 150$ gpm

Using Eq. 8.8b and assuming D_2 = 5-in. for the entire length of the lateral, find the loss in $(L_1 + L_2) = 960$ ft containing $(N_1 + N_2) = 24$ sprinklers and discharging $Q_l = 300$ gpm (Step 5):

$$P_{f(1+2)}\,(\text{for all } D_2) = \frac{2.15 \times 0.37 \times 960}{2.31 \times 100} = 3.13 \text{ psi}$$

Next find the loss in $L_1 = 480$ ft of D_2 = 5-in. pipe containing $N_2 = 12$ sprinklers and discharging $Q_1 = 150$ gpm (Step 6):

$$P_{f(1)}\,(\text{for } D_2) = \frac{0.59 \times 0.39 \times 480}{2.31 \times 100} = 0.48 \text{ psi}$$

And in a similar manner find the loss in the 4-in. pipe (Step 7):

$$P_{f(1)}\,(\text{for } D_1) = \frac{1.81 \times 0.39 \times 480}{2.31 \times 100} = 1.47 \text{ psi}$$

The friction loss for the dual-pipe-size line can now be determined in accordance with Step 8 by Eq. 9.4a:

$$P_f = 3.31 - 0.48 + 1.47 = 4.3 \text{ psi}$$

This value is slightly lower than the allowable P_f:

$$(P_f)_a = 0.20\,P_a - \Delta P_e = 0.20 \times 44 - 3.9 = 4.9 \text{ psi}$$

Therefore, less 5-in. pipe and more 4-in. pipe can be used.

Time could have been saved by solving for N_1 directly. Replacing L_1 and L with N_1 and $N = 24$ in Eq. 9.4c, computing $P_{f(1+2)}$ (for all D_1) $= (P_f)_1 = 10.06$ psi and solving for N_1 directly gives:

$$N_1 = 24 \left[\frac{4.9 - 3.31}{10.06 - 3.31} \right]^{1/2.8} = 14^*$$

We assume 400 ft of 5-in. pipe containing 10 sprinklers and 560 ft of 4-in. pipe containing 14 sprinklers and repeat the above procedure, which results in:

$$P_f = 3.31 - 0.75 + 2.28 = 4.8 \text{ psi}$$

The lateral inlet pressure, P_l, which is the pressure requirement at the main line, can now be determined by Eq. 9.3 for the dual-pipe-size lateral:

$$P_l = P_a + \tfrac{5}{8} P_f + \tfrac{1}{2} \Delta P_e + P_r$$

$$P_l = 44.0 + \tfrac{5}{8}(4.8) + \tfrac{1}{2}(3.9) + 3.4 = 52.6 \text{ psi}$$

*Round down to whole numbers.

Sample Calculation 9.5. *Design of laterals laid downslope with one pipe diameter.*

GIVEN: A lateral consisting of $L = 960$ ft of portable, aluminum irrigation pipe with 24 sprinklers spaced 40 ft apart, discharging $q_a = 12.5$ gpm, and operating at $P_a = 44.0$ psi;

Lateral capacity is $Q_l = 300$ gpm;

Average downhill slope is $s = -3.5\%$ so $\Delta El = (-33.6)$ ft in the total length of the line and $\Delta P_e = 0.43 \times (-33.6) = -14.5$ psi; and

Height of risers for corn is $H_r = 8.0$ ft so $P_r = 0.43 \times 8.0 = 3.4$ psi.

FIND: Determine the smallest pipe size that will result in an approximate balance between pressure loss due to friction and static pressure gain along the lateral due to the elevation decrease, i.e., $(P_f)_a = -\Delta P_e$, (because this is a steep downslope where $(-\Delta P_e) > 0.3\ P_a$, i.e., 14.5 psi > 13.2 psi). Also, find the lateral inlet pressure, P_l, (required at the inlet end), the minimum pressure, P_n, and the pressure at the closed end of the lateral; and make a sketch to illustrate the lateral line hydraulic characteristics.

CALCULATIONS: Letting $(P_f)_a$ be equal to the pressure gain due to elevation, $(-\Delta P_e) = 14.5$ psi in Eq. 9.1b:

$$J_a = 2.31\, \frac{14.5}{0.37 \times 960/100} = 9.43 \text{ ft}/100 \text{ ft}$$

Entering Table 8.1 with the lateral capacity of $Q_l = 300$ gpm indicated some 3-in. and some 4-in. pipe will be required. Because the owner wished to have laterals with only one pipe size, use all 4-in. pipe. The pressure loss due to friction by Eq. 8.8b is:

$$P_f = 0.433 \times \frac{0.37 \times 6.54 \times 960}{100} = 10.1 \text{ psi}$$

With all 4-in. pipe, the percentage of pressure variation between the inlet and closed end of the line is:

$$100 \times \frac{(-\Delta P_e) - P_f}{P_a} = 100 \times \frac{14.5 - 10.1}{44.0} = 10.0\%$$

If all 3-in. pipe were used, $P_f = 42.5$ psi, and the resulting pressure variation would be:

$$= 100 \times \frac{14.5 - 42.5}{44.0} = -64\%$$

This is obviously outside the 20% limit. Thus a line consisting of all 3-in. pipe should not be used.

Using Eq. 9.2a to compute the pressure required at the main line for a 4-in. lateral:

$$P_l = 44.0 + \tfrac{3}{4}(10.1) + \tfrac{1}{2}(-14.5) + 3.4 = 47.7 \text{ psi}$$

The sprinkler pressure at the closed end of the lateral, P_1, can be computed by:

$$\begin{aligned} P_1 &= P_l - P_f - \Delta P_e - P_r \\ &= 47.7 - 10.1 - (-14.5) - 3.4 = 48.7 \text{ psi} \end{aligned}$$

The minimum pressure occurs where the flow rate in the lateral is such that the pipe-friction gradient equals the ground (lateral) slope gradient, $J = s = -3.5$ ft/100 ft. In Table 8.1 for the 4-in. aluminum pipe, note that $J = 3.5$ when the flow rate is approximately 212.5 gpm. This is the flow rate in the section between the 17th and 18th sprinklers from the closed end. Thus, starting from the closed end, P_n can be computed by:

$$\begin{aligned} P_n &= P_1 + (P_f)_{17} + (\Delta P_e)_{17} \\ &= 48.7 + 10.1\left(\frac{17}{24}\right)^{2.8} + (-14.5)\left(\frac{17}{24}\right) \\ &= 48.7 + 3.8 - 10.3 = 42.3 \text{ psi} \end{aligned}$$

The rationale for computing $(P_f)_{17}$ is based on Eq. 8.10b. Computing the pipe-friction-pressure loss by Eq. 8.8b gives the same approximate value:

$$(P_f)_{17} = 0.433 \times 3.5 \times 0.38 \times \frac{17 \times 40}{100} = 3.9 \text{ psi}$$

Figure 9.4 is a sketch of the hydraulic characteristics of the lateral. Several $(P_f)_x$ values were computed by Eq. 8.10b (after substituting P_f for h_f and letting $N_x = x$ and $N_l = L$) to plot the P_f curve. The line representing $(P_f)_{av}$ bisects the P_f curve, so that Area A (above) equals Area B (below). The$(P_e)_{av}$ line is at $\frac{1}{2}\,\Delta P_e$ for a lateral on a uniform slope. The remaining dimensions and portions of Fig. 9.4 were developed by deductive reasoning using the data developed earlier.

LATERALS WITH FLOW-CONTROL DEVICES

Flow- or pressure-control devices are used with lateral lines where the topography is too broken or too steep to permit the pressure variation in the line to be controlled within the 20% of P_a limit by using practical sizes of pipe. These devices are either valves placed at the base of each sprinkler outlet or special

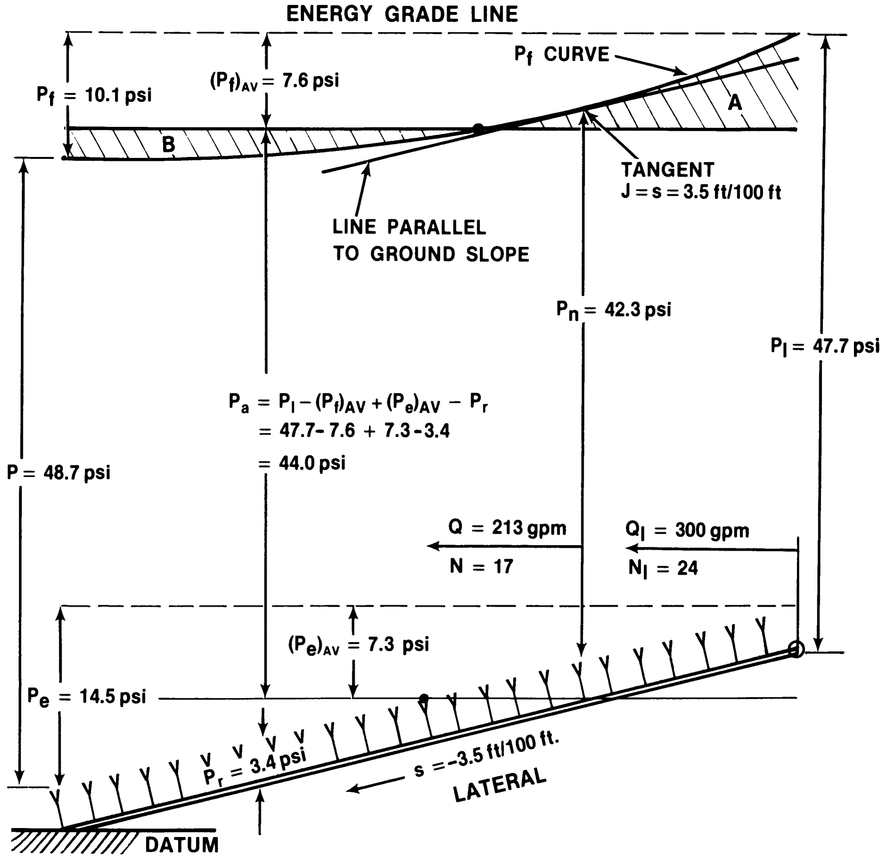


FIG. 9.4. Hydraulic Characteristics of Lateral with 24 Sprinklers Spaced at 40 ft and Discharging 12.5 gpm Each.

flow-control nozzles, as described earlier (see Fig. 6.6). They are designed to provide equal discharge at all sprinklers. When flow- or pressure-control devices are used at the base of each sprinkler, extra pressure is required.

The pressure that must be provided at the distal (closed) end of the lateral will be P_a plus P_r plus the pressure required to overcome friction loss in the control valves, P_{cv}, (see Fig. 9.2C). However, when flexible-orifice nozzles are used to maintain constant flow, P_{cv} is effectively zero, but the discharge may vary from the nominal value (see Fig. 6.6).

Since the valves control the discharge of the sprinklers, the selection of lateral pipe sizes is not a problem of maintaining a specified pressure variation between sprinklers, but one of economics. The allowable head loss due to pipe friction, $(P_f)_a$, should be the one that will result in the lowest annual pumping cost. For many conditions, $(P_f)_a$ may be assumed to be about 0.20 P_a or about 70 kPa (10 psi).

The lateral inlet pressure, P_l, for laterals with a flow-control device at each sprinkler is:

$$P_l = P_a + P_f + \Delta P_e + P_r + P_{cv} \tag{9.5a}$$

and

$$H_l = H_a + h_f + \Delta H_e + H_r + h_{cv} \tag{9.5b}$$

where P_{cv} = pressure loss due to the control device, kPa (psi); and h_{cv} = head loss due to the control device, m (ft).

Valve manufacturers should furnish data on the pressure losses for different discharges through their valves (see Fig. 14.14). Sample Calculation 9.6 illustrates the procedure involved in the design of a lateral line containing flow-control nozzles.

Sample Calculation 9.6. *Design of lateral with flow-control nozzles.*

GIVEN: A lateral 400 m long running up and down slopes on broken topography;

The highest point is 10 m above the inlet end of the lateral (at the main line);

It contains 44 sprinklers spaced at S_e = 9.1 m with q_a = 19 L/min;

The first sprinkler is located $\frac{1}{2}$ S_e from the main line;

Sprinklers with flexible-orifice nozzles designed to discharge approximately 10 L/min (5 gpm) between 275 and 550 kPa (40 and 80 psi), as shown in Fig. 6.6, will be used;

The system will have 1-m risers so $P_r = 9.8 \times 1.0 = 9.8$ kPa; and

The owner desires single-pipe-size laterals.

FIND: Determine the pipe size and P_l required.

CALCULATIONS: The static pressure difference due to the elevation difference of 10 m between the inlet and the high point on the lateral is:

$$\Delta P_e = 9.8 \times 10 = 98 \text{ kPa}$$

Let $(P_f)_a$ = 70 kPa, which is approximately 20% of the pressure required by a standard 4-mm nozzle discharging 19 L/min. By Eq. 9.1b, the allowable head loss gradient is:

$$J_a = 0.102 \frac{70}{0.36 \times 400/100} = 5.0 \text{ m}/100 \text{ m}$$

Entering Table 8.1 with $Q_l = 44 \times 19/60 = 13.9$ L/s, find $J = 3.68$ for 4-in. pipe, which satisfies the criteria for J_a. Using Eq. 8.8b with $J = 3.68$ and $F = 0.36$ from Table 8.7 gives:

$$P_f = 9.8 \frac{0.36 \times 3.68 \times 400}{100} = 53 \text{ kPa}$$

Typically, regulating valves used at the base of a sprinkler have a P_{cv} of between 20 and 35 kPa (3 and 5 psi). However, as mentioned earlier for flexible-orifice nozzles, $P_{cv} = 0$. The flexible-nozzle sprinklers will discharge approximately 19 L/min when operating at any pressure between 275 and 550 kPa, as indicated in Fig. 6.6a. Substituting the lowest permissible operating pressure, 275 kPa, for P_a in Eq. 9.5a gives:

$$P_l = P_a + P_f + \Delta P_e + P_r + P_{cv}$$

$$= 275 + 53 + 98 + 9.8 + 0 = 436 \text{ kPa}$$

PERFORATED PIPE LATERALS

Because perforated pipe laterals (see Figs. 4.7 and 9.5) have equally spaced sequences of outlets, the general principles applicable to the design of laterals with impact sprinklers also apply to perforated pipe. Nevertheless, there are more restrictions on the design of perforated pipe laterals because of their low operating pressure of 35 to 210 kPa (5 to 30 psi). Laterals should be laid very

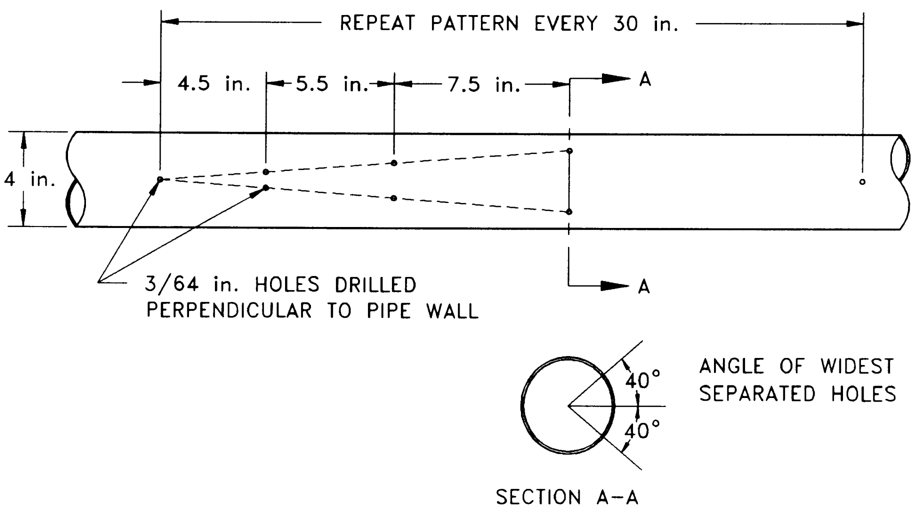


FIG. 9.5. Top View of Typical Perforated Pipe Having Seven-hole Pattern Sequence Every 30 in. (1 in. = 25.4 mm).

nearly on the level so that pressure variation along the lines is kept within acceptable limits. (For example, with P_a = 105 kPa (15 psi) a 1-m (3.3-ft) elevation change will produce a 20% pressure variation.) Pressure-control valves cannot be used for this purpose, and only one pipe size should be used for a given lateral.

Application Rate

Perforated pipe is available for only a few rates of application, the most typical rates being 19 and 25 mm/hr (0.75 and 1.0 in./hr). This limitation results from the fact that strategically placed (see Fig. 9.5) and relatively large perforations (between 1.2 and 1.6 mm, $\frac{3}{64}$ and $\frac{1}{16}$ in.) must be used to obtain uniform coverage and to avoid excessive clogging. This limit in application rates reduces flexibility in design. Furthermore, even with the relatively large perforations, the water must be carefully screened through a 60-mesh final screen to remove particles larger than fine sand (250 microns, or μ).

Lateral Spacing

Figure 9.6 shows a typical wetted pattern profile taken at right angles to a perforated pipeline. The wetted pattern uniformity is surprisingly resilient to light winds, but wind does shift its position relative to the lateral. Perforated pipe should not be operated in open fields where winds exceed 16 km/hr (10 mph) unless wind breaks are installed.

A lateral spacing, S_l, of 1.2 m (4 ft) less than the water spread is customarily used to provide sufficient overlap between wetted patterns to prevent dry areas.

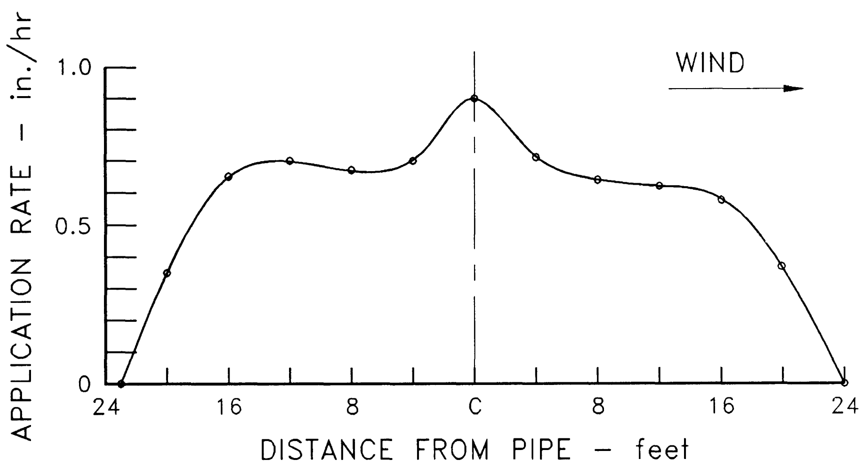


FIG. 9.6. Average Profile of Water Distribution from Five Test Runs for a Typical Perforated Pipe at 152 kPa (22 psi) in 0- to 5.3-km/hr (0- to 3.3-mph) Winds.

Where winds of over 8 km/hr (5 mph) are routinely expected, a 1.5- to 2.0-m (5- to 7-ft) overlap is recommended.

The spread (width) of the wetted pattern is quite sensitive to pressure, as can be observed from the data in Table 9.1. Thus, the pressure can be adjusted to obtain a wide range of lateral spacings. For example, the spread ranges from 9.1 to 15.2 m (30 to 50 ft) as the pressure increases from 40 to 140 kPa (6 to 20 psi).

Some manufacturers of perforated pipe have simplified the design of laterals by furnishing performance tables (McCulloch et al. 1967) for each combination of pipe size and application rate (see Table 9.1). For a given length of line and desired spread (or wetted width), it is easy to read the operating pressure and lateral discharge. Manufacturers of perforated pipe should be able to provide similar performance tables for their products.

Sample Calculation 9.7. *Design of perforated lateral pipeline.*

GIVEN: Table 9.1 for perforated aluminum pipelines of various diameters designed for applying water at the rate of 25 mm/hr (1 in./hr).

FIND: Design a perforated pipe lateral with L = 183 m (600 ft) and S_l = 12.2 m (40 ft) for low winds.

CALCULATIONS: To allow for the sufficient overlap of 1.2 m (4 ft) suggested for low winds, the spread (or wetted width) of the sprinkler pattern should be $S_l + 1.2 = 12.2 + 1.2 = 113.4$ m (44 ft). Entering Table 9.1 with the desired length 183 m (600 ft) and appropriate spread of 13.4 m (44 ft), find:

Lateral pipe diameter: $D =$ 100 mm (4 in.).
Lateral inlet pressure: $P_l =$ 124 kPa (18 psi)
Lateral discharge: $Q_l =$ 19.8 L/s (314 gpm)

HOSE-FED SPRINKLER DESIGN

Hose-fed sprinkle systems for overlapped sprinkler grids and for orchard sprinklers (see Fig. 4.8) involve special design considerations. However, most of the design strategies discussed earlier in this chapter can be used. Each hose may be fitted with 1 to 10 or more sprinklers and either periodically pulled to a new set position or left stationary. (Systems with miniature sprinklers on stationary hoses cannot be distinguished from trickle systems with spray emitters, which are fully considered in the trickle irrigation design chapters of this text.)

Table 9.1. Performance table for perforated pipelines designed to apply 1.0 in./hr

Spread	Pressure	Diameter of lateral																		
		3-in.								4-in.					5-in.					
	P_l flow	Length of lateral – ft																		
feet	Q_l	50	100	150	200	250	300	350	400	450	500	550	600	650	700	750	800	850	900	950
	psi	22	22	23	23	24	26	28	30	24	24	25	26	27						
52	gpm	31	62	90	122	154	188	223	268	279	307	344	375	423						
	psi	20	20	20	20	21	23	25	27	22	22	23	24	25						
50	gpm	29	58	86	114	144	178	213	254	266	293	330	361	404						
	psi	18	18	18	18	19	20	22	24	20	20	21	22	23	20	20	20	21	21	22
48	gpm	27	54	81	107	138	168	202	240	253	279	316	346	384	394	420	450	480	512	544
	psi	16	16	16	16	17	18	19	21	18	18	19	20	21	18	18	19	20	20	21
46	gpm	26	51	76	101	131	158	189	224	240	265	298	330	365	372	397	435	472	497	528
	psi	14	14	14	15	15	16	17	19	16	16	17	18	19	16	17	17	18	18	19
44	gpm	24	48	72	99	123	150	179	213	227	251	280	314	347	353	381	409	446	470	508
	psi	12	12	13	13	14	14	16	17	14	15	15	16	17	14	15	15	16	16	17
42	gpm	22	44	69	92	118	140	173	200	213	243	265	297	326	330	364	386	423	445	479
	psi	11	11	11	12	12	13	14	15	12	13	13	14	15	13	13	13	14	14	15
40	gpm	21	42	63	87	109	136	162	189	196	225	246	278	308	311	339	360	395	416	451
	psi	10	10	10	10	11	11	12	13	11	11	12	12	13	11	12	12	12	13	13
38	gpm	20	40	60	80	104	124	150	177	187	207	237	256	286	292	317	340	364	392	421
	psi	9	9	9	9	9	10	11	12	10	10	10	11	12	10	10	11	11	11	12
36	gpm	19	38	57	76	94	118	142	169	178	197	215	245	276	277	298	321	344	368	394
	psi	8	8	8	8	8	9	10	11	8	9	9	10	11	9	9	9	10	10	11
34	gpm	18	36	54	72	89	112	135	161	160	187	204	233	263	256	277	298	323	350	377
	psi	7	7	7	7	7	8	8	9	7.5	8	8	9	9	8	8	8	9	9	9
32	gpm	17	34	51	68	84	106	122	145	155	177	193	221	237	242	263	282	306	332	348
	psi	6	6	6	6	6.5	7	7	8	7	7	7	8	8	7	7	7	8	8	8
30	gpm	16	32	48	63	81	100	115	137	150	166	182	209	224	234	250	272	291	314	330
	psi	5	5	5	5.5	6	6	6	7	6	6	6	7	7	6	6	6	7	7	8
28	gpm	15	30	45	61	78	93	111	129	140	155	170	196	210	217	238	252	271	295	320
	psi	4.5	4.5	4.5	5	5	5	6	7	5	5	5	6	6	6	6	6	6	6	7
26	gpm	14.5	29	43	59	72	86	107	125	135	149	164	182	195	209	225	241	257	275	292
	psi	4	4	4	4	4	5	5	6											
24	gpm	14	28	42	57	69	83	101	120											

NOTE: 1-in. = 25 mm; 1.0 psi = 6.895 kPa; 1.0 gpm = 0.063 L/s; 1.0 ft = 0.305 m.

Wetting Pattern

When the sprinkler spacing is wider than the plant spacing, complete uniformity of wetting is necessary. In such cases the efficiency and uniformity considerations presented in Chapter 6 hold.

When the sprinkler spacing is the same as the plant spacing, the uniformity of water supplied to each plant is the same as the uniformity of discharges from the sprinklers. In such cases, trickle irrigation design procedures related to wetting pattern considerations (see Chapter 19) are appropriate.

Where the sprinkler spacing is such that each sprinkler position serves two, three, or four plants, the uniformity of coverage within each wetted circular pattern should be symmetrical. The coverage should also be fairly uniform along each radius and extend well beyond each plant. However, it is not necessary that all the spaces between the plants be uniformly wetted and evaluated by DU or CU, provided the plants are *symmetrically* spaced around each sprinkler. In such cases the uniformity of water supplied to each plant is the same as the uniformity of discharge. It is essential, however, that the distance between each sprinkler position and the surrounding two, three, or four plants be equal, when dry areas are allowed by the design. This is necessary for each plant to have access to an equal share of the water discharged at each sprinkler setting.

Design Strategy

Where a manifold feeds hoselines in every other tree row, operating with one or two sprinklers near their ends, the manifold should be treated as an ordinary sprinkler lateral. However, the average pressure along the manifold should be the average sprinkler pressure desired, P_a, plus the friction-head loss in the hose and hydrant. For manifolds with one size of pipe feeding hoses with one or two sprinklers:

$$P_m = P_a + \tfrac{3}{4} P_f + \tfrac{1}{2} \Delta P_e + P_h \tag{9.6a}$$

where P_m = manifold inlet pressure, kPa (psi); and P_h = pressure loss due to hoseline and/or hydrant friction, kpa (psi).

Where each submain serves only one or two hoselines, each with several uniformly spaced sprinklers, the hoseline should be treated as an ordinary sprinkler lateral. Thus, Eq. 8.8b should be used to solve for P_f and Eq. 9.2a used to solve for P_l.

Where several hoselines, each with several sprinklers, are fed from a common manifold, the sprinkle and trickle irrigation design strategies merge. The hydraulics of this type of compound subunit made up of multioutlet laterals supplied from a multioutlet manifold are discussed at length in Chapters 22 and 23. For a single-pipe-size manifold with P_f computed by Eq. 8.8b when feeding hoses discharging Q_l:

Table 9.2. Appropriate friction-loss gradients for plastic hoses and hydrants[1]

Flow rate		Friction-loss gradient, psi/100 ft.			Hydrant loss[2] psi	
L/min	gpm	$\frac{5}{8}$-in.	$\frac{3}{4}$-in.	1-in.	$\frac{3}{4}$-in.	1-in.
7.6	2	1.81	0.76	0.19	0.1	-
15.1	4	6.07	2.55	0.65	0.2	0.1
22.7	6	12.35	5.19	1.32	0.4	0.2
30.3	8	20.43	8.59	2.19	0.8	0.3
37.9	10	30.19	12.70	3.24	1.2	0.5
45.4	12	41.53	17.47	4.45	1.7	0.7
53.0	14		22.88	5.83	2.4	0.9
60.6	16		28.90	7.37	3.1	1.2
68.1	18		35.52	9.06	3.9	1.5
75.7	20			10.89	4.8	1.9
83.3	22			12.87	5.8	2.3
90.8	24			14.98	6.9	2.7
98.4	26			17.24	8.0	3.2
106.0	28			19.62	9.2	3.7
113.6	30			22.14	10.6	4.3

NOTE: 1 psi/100 ft = 22.6 kPa/100 m; 1 psi = 6.895 kPa.
[1]Nominal hose sizes and also inside diameters.
[2]Friction losses in valves vary widely with different makes of equipment. These values should be used only as a guide in determining the size required.

$$P_m = P_l + \tfrac{3}{4} P_f + \tfrac{1}{2} \Delta P_e + P_h \tag{9.6b}$$

In Eq. 9.6b, P_l is the hose inlet pressure found by Eq. 9.2a or 9.3, and P_h is the hose hydrant friction loss (see Table 9.2).

Friction losses in small-diameter hoses can be estimated by Eq. 8.7a. Table 9.2 gives pressure-loss gradients for various sizes of hoses based on Eq. 8.7a and for $\frac{3}{4}$-in. and 1-in. hose hydrants.

Sample Calculation 9.8. *Design of a hose-fed, periodic move sprinkler irrigation system.*

GIVEN: The level 144- by 144-m field with a 5% slope shown in Fig. 9.7. The trees are spaced at 6 by 6 m and the hoselines are laid down every other tree row. Each hose is 72 m long, has two equally spaced sprinklers, and is pulled (while sprinkling) to move the sprinklers 6 m every 24 hr. Thus, it will take 6 days to irrigate on each side of the manifold or a total of 12 days to complete an irrigation.

All $\frac{5}{8}$-in. hose is used, and the average sprinkler discharge, q_a = 3.8 L/min, when operating at an average pressure, H_a = 200 kPa. The hoses are connected

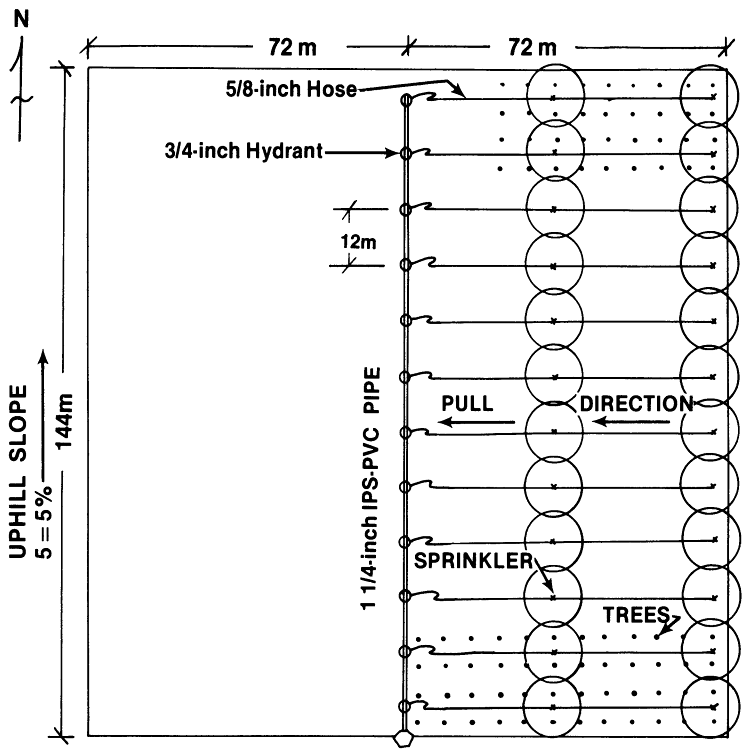


FIG. 9.7. Hose-fed, Periodic-move Sprinkler System Design for an Orchard with 67- by 6-m Tree Spacing and Hoseline with Sprinklers Between Every Other Row of Trees.

to $\frac{3}{4}$-in. hydrants spaced at 12-m intervals along the 1 $\frac{1}{4}$-in. IPS-PVC plastic pipe manifold.

There is a pressure-control valve at the inlet to the manifold, but no other pressure regulation within the irrigated area.

FIND: Determine the required manifold inlet pressure and the pressure variation within the system. Then determine the gross depth per irrigation and estimate the maximum net depth of irrigation per day that the system is capable of applying.

CALCULATIONS: To determine the *manifold inlet pressure*, begin by determining $Q_l = 2 \times 3.8 = 7.6$ L/min and estimating P_l. From Tables 9.2 and 8.7, $J = 1.81$ psi/100 ft, and $F = 0.64$ for two-end outlets, and by Eq. 8.8b the P_f for the 72-m (236-ft) long hoseline is:

$$P_f = 1.81 \times 0.64 \times \frac{236}{100} = 2.73 \text{ psi}$$

$$= 2.73 \text{ psi} \times 6.895 \text{ kPa/psi} = 18.8 \text{ kPa}$$

And P_l, by Eq. 9.2a, is:

$$P_l = 200 + \tfrac{3}{4}(18.8) + \tfrac{1}{2}(0) + 0 = 214.1 \text{ kPa}$$

The manifold flow rate is:

$$Q_m = (12 \times 7.6)/60 = 1.52 \text{ L/s}$$

From Table 8.3, $J = 3.81$ m/100 m for $Q = 1.39$ L/s, therefore, for $Q =$ 1.52 L/s.

$$J = 3.81 \times \left[\frac{1.52}{1.39}\right]^{1.75} = 4.46 \text{ m/100 m}$$

(The exponent 1.75 is the "Q exponent" taken from Eq. 8.7a for the small diameter plastic pipe.) And by Eq. 8.8b with $F = 0.37$ for 12 outlets, with the first outlet $\frac{1}{2}$ spacing from the manifold inlet (see Fig. 9.7), P_f for the 138-m-long manifold is:

$$P_f = 9.8 \times 4.46 \times 0.37 \times \frac{138}{100} = 22.3 \text{ kPa}$$

The $P_h = 0.1$ psi $= 0.7$ kPa from Table 9.2, and the static pressure difference due to the elevation difference along the $s = 5\%$ uphill slope of the manifold is:

$$\Delta P_e = 9.8 \times \frac{5}{100} \times 138 = 67.6 \text{ kPa}$$

The manifold inlet pressure can now be computed by Eq. 9.6b.

$$P_m = 214.1 + \tfrac{3}{4}(22.3) + \tfrac{1}{2}(67.6) + 0.7 = 265 \text{ kPa}$$

To determine the maximum *gross and net depths* of water the system is capable of applying, start by determining the minimum sprinkler pressure head, P_n, which is:

$$\begin{aligned} P_n &= P_m - \Sigma P_f - \Delta P_e - P_h \\ &= 265 - (18.8 + 22.3) - 67.6 - 0.7 = 156 \text{ kPa} \end{aligned}$$

Next, find the system DU by letting DU = 100% in Eq. 6.5b to obtain:

$$\text{System DU} = \text{DU} \times \frac{1 + 3\,(P_n/P_a)^{1/2}}{4}$$

$$100 \times \frac{1 + 3\,(156/200)^{1/2}}{4} = 91\%$$

The rationale for letting DU = 100% is that each of the two (or 4) plants that share water from a common sprinkler set position receive the same amount of water as discussed earlier.

By Eq. 6.9 the system application efficiency with $R_e = 0.94$ for fine spray in low winds (see Fig. 6.8) and almost no leakage, $O_e = 1.0$, is:

$$E_q = \text{system DU} \times R_e \times O_e$$

$$= 91 \times 0.94 \times 1.0 = 86\%$$

The product of the 24-hr set time and the application rate, I, given by Eq. 5.5 is the gross depth of water applied per irrigation cycle:

$$d = 24I = 24\,\frac{60 \times 3.8}{6 \times 12} = 76 \text{ mm}$$

Therefore, with an irrigation cycle or interval of $f' = 12$ days, the net application per day by Eq. 5.3a is:

$$d_n/\text{day} = \frac{86}{100} \times \frac{76}{12} = 5.4 \text{ mm/day}$$

Management

Where sprinklers are pulled down every other row, they should be pulled between alternate rows of trees every other irrigation to help equalize the localized distribution of water between trees. This will also distribute water more uniformly to the entire land area for better root distribution and fertility uptake. Unfortunately, alternating hoseline positions makes it more difficult to obtain the uniform irrigation along the outer edges of the field that is achieved by the layout shown in Fig. 9.7.

Moving all the hoselines down one row spacing will leave the top tow of trees unirrigated and irrigate a strip outside the lower field boundary (see Fig. 9.7). To minimize this problem the following hoseline-shifting sequence can be employed for each set of four irrigation cycles:

Cycle 1

- Layout as shown in Fig. 9.7.

Cycle 2

- Move the southernmost hoseline to the south edge of its between-row spacing.
- Move all the other hoselines one row spacing to the south.

Cycle 3

- Move the hoselines back as in Fig. 9.7.

Cycle 4

- This cycle is similar to Cycle 2 except that the northernmost hoseline should be moved to the north edge of its between-row spacings, and the other hoselines should be moved one row spacing to the north.

Filtration

All sprinkler systems require sufficiently clean water so that nozzles do not clog. To assure a minimum of clogging, the screen openings should be considerably smaller than the nozzle diameter (one-fourth to one-tenth as large). Since the sprinklers used in hose-fed systems are often as small as 1.6 mm ($\frac{1}{16}$ in.) 30- to 60-mesh screen is typically required. A 60-mesh screen will exclude all particles larger than fine sand 0.25 mm (0.01 in.).

For the very small rotating sprinklers often used in hose-fed systems, additional screening or filtration may be required. In addition to protecting against nozzle clogging, the delicate turning mechanisms and bearings may need special consideration. Thus, for the reliable performance (rotation) of many small sprinklers, even the very fine sand particles should be removed. This requires using 200-mesh screens to remove particles larger than 0.075 mm (0.003 in.) and/or sand separators, as discussed in Chapter 18 for trickle irrigation.

REFERENCES

McCulloch, A. W., J. Keller, R. M. Sherman, and R. C. Mueller. 1957. Revised by Keller, 1967. *Irrigation Handbook*, Milpitas, California: W. R. Ames Co.

RECOMMENDED READING

For additional details on the hydraulics of multioutlet pipelines with uniform discharge per outlet, go to Chapter 22, "Trickle Lateral Design," and Chapter 23, "Trickle Manifold Design," herein, and:

Benami, A., and A. Ofen. 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).

10

Main Delivery System Design

Main lines for sprinkle systems vary from short portable feeder lines to intricate networks of buried mains and submains serving large systems (see Figs. 5.1, 7.1, and 7.2). The principal function of main lines and submains is to convey the quantities of water required to all parts of the design area at the pressure required to operate all laterals under maximum flow conditions. The principal design problem is the selection of pipe sizes that will accomplish this function economically. For the purposes here, the line running from the water source to the design area, usually called the *supply line*, will be treated as part of the main line.

The design of main lines or submains requires an analysis of the entire system to determine maximum requirements for capacity and pressure. The classic procedure is to assume, within a reasonable range, several values of allowable head loss due to friction in main lines and submains and to compute the pipe size or sizes for each assumed value. The pipe sizes thus obtained are then checked for energy economy, and the most economical sizes are selected. In Chapter 8 the more efficient procedure of utilizing the Economic Pipe-Selection charts or the algorithms used for their development is presented.

Where gravity pressure (pressure gained by elevation differences) is used, one of two problems may arise. Where elevation differences are scarcely enough to provide adequate pressure for operation of the system, the problem becomes one of conservation of energy, demanding larger than normal pipe sizes. This is necessary to reduce friction losses in order to avoid booster pumping where possible. Sometimes elevation differences are considerably in excess of those required to provide normal operating pressure. In these cases the problem becomes one of reducing pressure gains, requiring small pipe sizes to increase friction losses. On excessively steep slopes, pressure-reducing valves or pressure-breaking structures and small pipes are required for the protection of the main line itself and for that of other equipment in the system.

In addition to pressure-loss considerations, the velocity of flow in main lines should be restricted to eliminate excessive water hammer. This is particularly important in PVC and cement-asbestos pipelines. In PVC pipe, main line velocities should be limited to 2 m/s (7 ft/s). With SDR-41 PVC pipe, the surge pressure is approximately equal to 85 kPa (12.4 psi) for each 0.3 m/s (1.0 ft/s) velocity change.

DESIGN STRATEGIES FOR DIFFERENT LATERAL LAYOUTS

The complexity of the design of main lines depends on the layout.

Design with Single Lateral

When only one lateral is moved along one or both sides of a main line, selecting the main line pipe size is relatively simple. The pipe size may be selected directly from an Economic Design Chart (see Fig. 8.7), tables, or appropriate formulas that will result in a friction loss within allowable limits when the lateral is operating from the distal end of the main line.

Where two laterals are being moved along a main line, but are not rotated in split-line operation, the problem is the same as if a single lateral were being used. The size of pipe selected will be the one that will result in a friction loss within allowable limits when both laterals are operating at the distal end of the main.

Design with Split-Line Layout

The split-line layout consists of two or more laterals rotated around the main line or submains (see Figs. 7.1A, B, and E). Split-line layouts are employed to: equalize the load at the pump regardless of lateral position, minimize the haulback of lateral pipe to the beginning point, and economize by requiring smaller pipe. However, split-line or split-flow layouts complicate farming operations by increasing the number of wet areas in the field, as discussed in Chapter 7.

Figure 10.1 illustrates the problem of main line design, using a split-line layout. In this layout, one lateral is moved up one side of the main line, while the other is moved down the other side (also see Fig. 7.2C).

In Fig. 10.1 it is apparent that at times the full quantity of water, Q_s, will flow from A to B. At such times there will be no flow beyond B. From B to C, the flow will never exceed $Q_s/2$, and when one lateral is operating at C, requiring a flow of $Q_s/2$, at that point, the other lateral will be at A; thus the flow for the entire length of main will be $Q_s/2$.

For any given total head at the pump, the optimum (smallest) pipe sizes will be the ones that give $h_{f1} = (h_{f2} + \Delta H_{e2})$. Note that the static pressure head difference, due to the elevation difference, ΔEl, between B and C (which is ΔH_{e2} in Fig. 10.1) is positive for uphill and negative for downhill lines. This

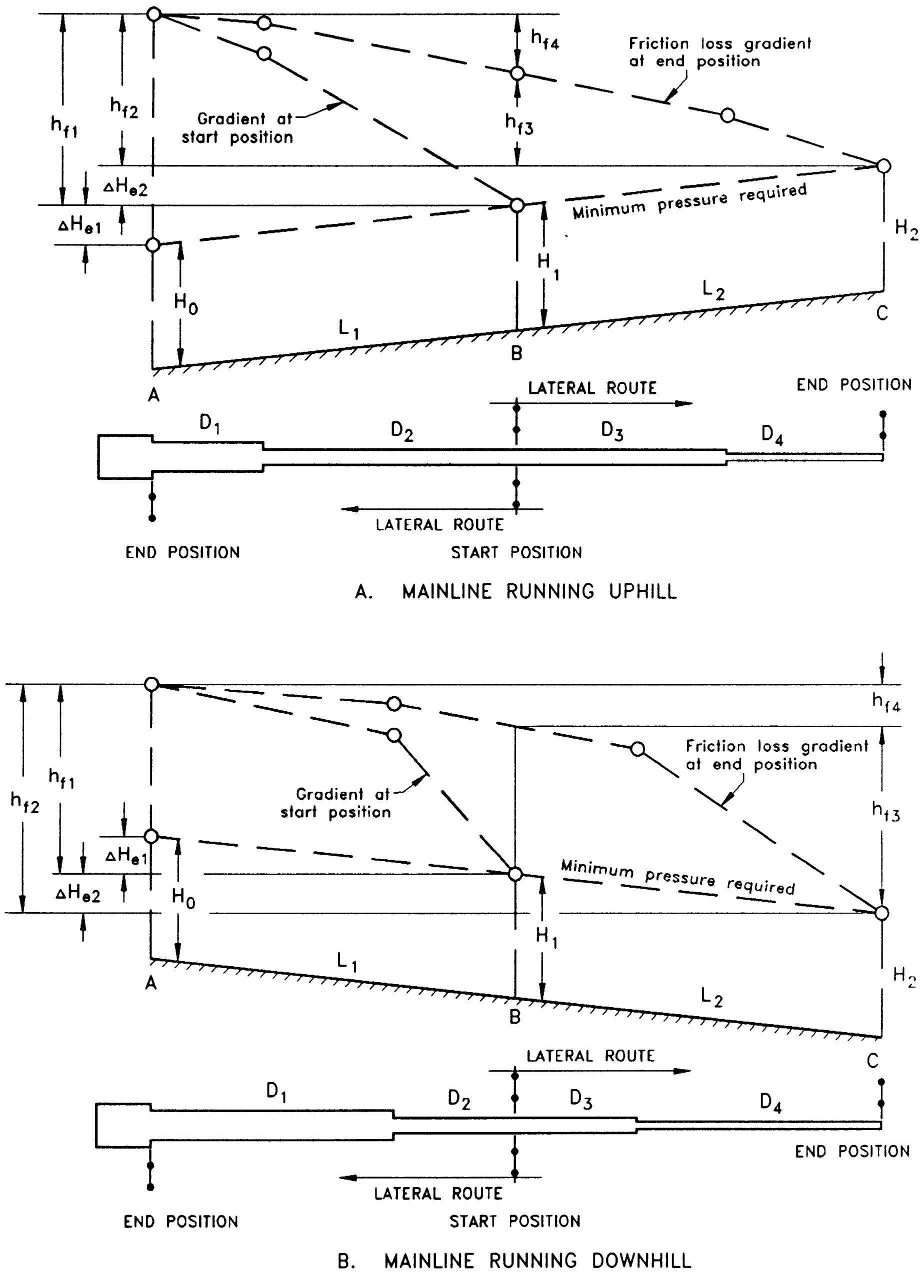


FIG. 10.1. Schematic of Main Line Pipe Sizes and Head Relationships with Twin Laterals, Split-Line Operation.

is because ΔH_e is the H_e at the inlet of the section minus the H_e at the outlet; thus, ΔH_e = (ΔEl between the outlet and inlet).

After pipe sizes have been computed for any reasonable value for head loss, adjustments can be made to balance annual pumping costs and capitalized pipe costs. For mains fed from pressure systems, the available head is fixed, and the smallest pipe sizes that will deliver the required pressure and flow to the laterals (without exceeding velocity restrictions as mentioned earlier) should be used.

A simple procedure to follow in determining minimum pipe sizes for a given allowable head loss, $(h_f)_a$, follows:

Step 1. Find the pipe size (see Fig. 10.1) that will carry the full flow, Q_s, in section A-B of main, L_1, with a friction loss equal to or as close as possible to $(h_f)_a$.

Step 2. If the friction loss for length of pipe L_1 using the selected pipe size exceeds (or is less than) the h_{f1} limit, find the friction loss in the next larger size pipe (or the next smaller pipe size).

Step 3. Determine the proportionate lengths of the two different sizes of pipe for L_1 that will give h_{f1} when the full quantity of water, Q_s, flows from A to B. This can be done by letting $h_{f1} = (h_f)_a$ and $L_1 = L$ in the following general equation for proportioning pipe lengths to give a specific friction head loss:

$$(h_f)_a = \frac{J_b L_b + J_s L_s}{100} = \frac{J_b(L - L_s) + J_s L_s}{100} \qquad (10.1a)$$

Solving for L_s:

$$L_s = \frac{100(h_f)_a - J_b L}{(J_s - J_b)} \qquad (10.1b)$$

where

$(h_f)_a$ = allowable head loss (in section) due to pipe friction, m (ft)
J_b = head-loss gradient in bigger pipe, m/100 m (ft/100 ft)
L_b = length of bigger pipe, m (ft)
J_s = head-loss gradient in smaller pipe, m/100 m m (ft/100 ft)
L_s = length of smaller pipe, m (ft)
L = length of pipe (in section), m (ft)

Step 4. Repeat Steps 1, 2, and 3 for sizing section B-C of main, L_2, with only half the flow $Q_2 = Q_s/2$ through sections A-B and B-C. Since the pipe sizes have already been determined for L_1, the allowable friction head loss for L_2 is:

$$h_{f3} = h_{f2} - h_{f4} = h_{f2} - 0.28\, h_{f1} \qquad (10.2)$$

where

h_{f3} = allowable friction-head loss through L_2, m (ft)
h_{f2} = allowable friction-head loss with Q_2 from A to C, m (ft)
h_{f4} = friction-head loss with Q_2 flowing through L_1 (see Fig. 10.1), m (ft)
h_{f1} = allowable friction-head loss with Q_1 flowing through L_1, m (ft)

(The 0.28 is the reduction in h_f caused by reducing the flow to $Q_s/2$ in L_1, i.e., from Eq. 8.1, $0.28 = (\frac{1}{2})^{1.852}$.) Then let $h_{f3} = (h_f)_a$ and $L_2 = L$ in Eq. 10.1b, and determine the length of smaller pipe in section B-C. Sample Calculation 10.1 illustrates this procedure for main line design where two laterals are operated in a split-line manner.

Sample Calculation 10.1. *Uphill main line with twin lateral split-line operation.*

GIVEN: A sprinkler system with two laterals in rotation as depicted in Fig. 10.1A, assuming a system capacity of Q_s = 500 gpm;

Length of aluminum pipe supply line (with 30-ft sections) from pump at P to A is L_{PA} = 440 ft;

Length of aluminum main line (within design area) is 1200 ft with L_1 = 600 ft and L_2 = 600 ft;

Pressure head required to operate laterals is $H_0 = H_1 = H_2 = 125.0$ ft; and

The static pressure head difference in each section of main line, assuming a uniform uphill slope, is $\Delta H_{e1} = \Delta H_{e2} = 7.0$ ft

FIND: Determine the smallest pipe sizes for both the supply line and main line, assuming the pressure head available at the pump discharge is 172.0 ft.

CALCULATIONS: From Table 8.4, J = 2.27 ft/100 ft for an assumed 6-in. diameter aluminum supply line with $Q_1 = Q_s$ = 500 gpm. Thus the friction loss in the 440-ft supply line is $4.4 \times 2.27 = 10.0$ ft

Referring to Fig. 10.1A and noting that $H_2 = 125$: $h_{f2} = 172.0 - 125.0 - 7.0 - 7.0 - 10.0 = 23.0$ ft

Furthermore: $h_{f1} = h_{f2} + \Delta H_{e2} = 23.0 + 7.0 = 30.0$ ft

The reason h_{f1} is greater than h_{f2} by ΔH_{e2} is because, when both laterals are operating at position B, the pump is not operating against the additional static head ΔH_{e2}. To take advantage of this, the allowable friction loss in section A-B can be increased by ΔH_{e2}.

When both laterals are operating at position B, the average head-loss gradient through length L_1 may be:

$$J_1 = \frac{h_{f1}}{L_1/100} = \frac{30.0}{6} = 5.0 \text{ ft}/100 \text{ ft}$$

From Table 8.4 with $Q_1 = 500$ gpm, the friction gradients for 5-in. and 6-in. pipe fall to either side of 5.0 ft/100 ft. Thus, for D_2 in Fig. 10.1A, use the smaller 5-in. pipe with $J_s = 5.54$ ft/100 ft, and for D_1 use the bigger 6-inch pipe with $J_b = 2.27$ ft/100 ft.

Letting $(h_f)_a = h_{f1} = 30.0$ ft and $L = L_1 = 600$ ft in Eq. 10.1b, the length of the smaller diameter (5-in.) pipe, L_s, is:

$$(L_s)_1 = \frac{100(30.0) - 2.27(600)}{(5.54 - 2.27)} = 500 \text{ ft}$$

Because the pipe is in 30-ft lengths, let $(L_s)_1 = 510$ ft, and determine the length of the bigger diameter (6-in.) pipe:

$$(L_b)_1 = L_1 - L_s = 600 - 510 = 90 \text{ ft}$$

When one lateral is operating at position A and the other is operating at position C, the flow rate in both L_1 and L_2 is $Q_s/2 = 250$ gpm. Thus by Eq. 10.2 the allowable friction lead loss through L_2 is:

$$h_{f3} = 23.0 - 0.277(30) = 14.7 \text{ ft}$$

And the allowable head-loss gradient in L_2 is:

$$J_2 = \frac{h_{f3}}{L_2} = \frac{14.7}{600} = 2.45 \text{ ft}/100 \text{ ft}$$

From Table 8.1 the friction gradients for 4-in. and 5-in. pipe fall to either side of 2.45 ft/100 ft when $Q_2 = 250$ gpm. Thus, for D_4 use 4-in. pipe, $J_s = 4.66$ ft/100 ft, and for D_3 use 5-in. pipe, $J_b = 1.53$ ft/100 ft.

Letting $(h_f)_a = 14.7$ ft and $L = L_2 = 600$ ft in Eq. 10.1b, the length of the smaller diameter (4-in.) pipe is:

$$(L_s)_2 = \frac{100(14.7) - 1.53(600)}{(4.66 - 1.53)} = 176 \text{ ft}$$

Because the pipe is in 30-ft lengths, let $(L_s)_2 = 180$ ft. Therefore, the length of 5-in. pipe is:

$$(L_b)_2 = 600 - 180 = 420 \text{ ft}$$

For this trial design the lengths of the various diameters of supply and main line in Fig. 10.1A are:

Supply line: 440 ft of 6-in.
Main line: 90 ft of 6-in.
510 ft + 420 ft = 930 ft of 5-in.
180 ft of 4-in.

Economic Split-Line Design

In many cases the system or subsystem inlet head is not fixed and pipe friction increases pumping costs. Rather than assuming several $(h_f)_a$ values and determining which gives the lowest sum of fixed plus pumping costs, a more efficient economic selection method can be used. The more efficient method for determining $(h_f)_a$ is:

Step 1. Construct an economic pipe-selection chart for the system operating conditions (see Fig. 8.4), or use Fig. 8.7 if appropriate.
Step 2. Select the most economic pipe sizes assuming Q_s from the supply to A (see Fig. 10.1) and also in section A-B; and $Q_s/2$ in section B-C.
Step 3. Using the pipe diameters selected in Step 2, determine the total difference in head, ΔH_T, required at A: when the laterals are both at B, i.e., $(H_T)_1$; and when one lateral is at A and the other lateral is at C, i.e., $(H_T)_2$.
Step 4. Trimming for *uphill or level main lines* (see Fig. 10.1A):

- If $(H_T)_1$ is highest, i.e., the H_T when both laterals are at B, use Eq. 10.1b to determine the length of smaller pipe to use in section B-C to increase h_{f2} by ΔH_T;
- If $(H_T)_2$ is highest, i.e., the H_T when the laterals are at A and C, but the same size of pipe was indicated by the economic pipe-selection chart for L_1 and L_2, then make no further adjustments; and
- If $(H_T)_2$ is highest and bigger pipe is indicated for L_1 than for L_2, use some smaller pipe in section A-B. The length of smaller pipe, $(L_s)_1$, should be such that h_{f1} is increased by ΔH_T plus the additional friction head loss that will occur due to this pipe size reduction when the laterals are at A and C. Thus Eq. 10.1b becomes:

$$(L_s)_1 = \frac{\Delta J(L_s)_1 + 100(h_{f1} + \Delta H_T) - J_b L_1}{J_s - J_b}$$

$$= \frac{100(h_{f1} + \Delta H_T) - J_b L_1}{0.72(J_s - J_b)} \qquad (10.3)$$

where

$(L_s)_1$ = length of smaller pipe in L_1, m (ft)
h_{f1} = friction-head loss in L_1 with all bigger pipe, m (ft)
ΔH_T = the extra pressure head required when the laterals are at A and C, compared with when both laterals are at B, i.e., $[(H_T)_2 - (H_T)_1]$, m (ft)
$\Delta J = J_s - J_b$ with $Q_s/2$, m/100 m (ft/100 ft)
J_b and J_s are head-loss gradients with Q_s

Step 4′. Trimming for *downhill main lines* (see Fig. 10.1B):

- If $(H_T)_1$ is highest, reduce the size of some pipe in section B-C (using Eq. 10.1b) so that h_{f2} is increased by $[(H_T)_1 - (H_T)_2]$;
- $(H_T)_2$ can be highest with the laterals at A and C on downhill main lines in order to satisfy the head requirement H_0 at A if $h_{f2} < (-\Delta H_{e1} - \Delta H_{e2})$. In this case use Eq. 10.1b to determine the pipe sizes for section B-C so that $h_{f2} = (-\Delta H_{e1} - \Delta H_{e2})$; and
- If $(H_T)_2$ is highest in order to satisfy the head requirement H_2 at C, use some smaller pipe in section A-B as determined by Eq. 10.3, providing different sizes of pipe were indicated for L_1 and L_2 by the Economic Pipe-Selection Chart.

Sample Calculation 10.2. *Economic design for uphill main line with twin, lateral split-line operation.*

GIVEN: The design information given in Sample Calculation 10.1 and the Economic Pipe-Selection Chart, Fig. 8.4, for the portable aluminum pipe and specific economic conditions.

FIND: The most economic design for the supply and main line from A to C in Fig. 10.1A.

CALCULATIONS: Entering Fig. 8.4 with $Q_s = 500$ gpm, select:

8-in. pipe for the supply line
8-in. pipe for section A-B with $Q_1 = 500$ gpm
6-in. pipe for section B-C with $Q_2 = 250$ gpm

From Table 8.4, $J_b = 0.56$ ft/100 ft for 8-in. pipe with 500 gpm. Therefore, the head loss h_{f1} when both laterals are at B in Fig. 10.1A is:

$$h_{f1} = 0.56 \frac{600}{100} = 3.4 \text{ ft}$$

And the total head required at A is:

$$(H_T)_1 = H_2 + \Delta H_{e1} + h_{f1} = 125.0 + 7.0 + 3.4 = 135.4 \text{ ft}$$

From Table 8.4, with a flow rate of 250 gpm, $J_b = 0.15$ ft/100 ft for 8-in. and $J_s = 0.63$ for 6-in. pipe. Therefore, when one lateral is at A and the other lateral is at C in Fig. 10.1A:

$$h_{f2} = 0.15 \frac{600}{100} + 0.63 \frac{600}{100} = 4.7 \text{ ft}$$

And the total head required at A is:

$$(H_T)_2 = H_2 + \Delta H_{e1} + \Delta H_{e2} + h_{f2}$$

$$= 125.0 + 7.0 + 7.0 + 4.7 = 143.7 \text{ ft}$$

According to Step 4, since the $(H_T)_2$ is largest when the laterals are at A and C, h_{f1} should be increased by ΔH_T:

$$\Delta H_T = 143.7 - 135.4 = 8.3 \text{ ft}$$

And from Table 8.4, $J_s = 2.27$ ft/100 ft for 6-in. pipe with 500 gpm; therefore, by Eq. 10.3 the length of the smaller 6-in. pipe, $(L_s)_1$, to use in section A-B is:

$$(L_s)_1 = \frac{100(3.4 + 8.3) - 0.56(600)}{0.72(2.27 - 0.56)} = 677 \text{ ft}$$

Since $L_1 = 600$ ft, use all 6-in. pipe.

The final total head required at A will now be highest when the laterals are at A and C. The final h_{f1} with all 6-in. pipe is:

$$h_{f2} = 0.63 \frac{1200}{100} = 7.6 \text{ ft}$$

And the final $(H_T)_2$ is:

$$(H_T)_2 = 125.0 + 7.0 + 7.0 + 7.6 = 146.6 \text{ ft}$$

A quick check shows that the final $(H_T)_1$ is less:

$$h_{f1} = 2.27 \frac{600}{100} = 13.6 \text{ ft}$$

And:

$$(H_T)_1 = 125.0 + 7.0 + 13.6 = 145.6 \text{ ft}$$

If the computed $(L_s)_1 < L_1$, then $(H_T)_1 = (H_T)_2$.

Design with Multiple Laterals in Rotation

Where more than two laterals are operated and the flow in the main line is split, part taken out at the first lateral and the rest continuing to serve other laterals, the design problem becomes more complex. Figure 5.1 shows an example of four laterals in rotation along main line B-G.

No simple mathematical formulas can be used to determine the most economic pipe sizes for such layouts. Approximations of optimum sizes, however, can be made by inspection and by trial-and-error calculations, or computer-assisted design techniques can be employed.

The recommended design strategy for multiple laterals in rotation is similar to the economic split-line design strategy:

Step 1. Construct an economic pipe-selection chart for the system operating conditions (see Fig. 8.4), or use Fig. 8.7 if appropriate.

Step 2. Select a set of most economic pipe sizes assuming the maximum flow rate that can occur in each reach of main line, as was done in Step 2 for the economic split-line design with only two laterals. For example, with four laterals in rotation, select pipe sizes as follows:
For 1st quarter use Q_s;
For 2nd quarter use $\frac{3}{4}\ Q_s$;
For 3rd quarter use $\frac{1}{2}\ Q_s$; and
For 4th quarter use $\frac{1}{4}\ Q_s$.

Step 3. Using the pipe diameter selected in Step 2, determine the ΔH_T required when one lateral is at the distal end of the main line; and one-half of an irrigation cycle later when two laterals are opposite each other and near the distal end.

Step 4. The trimming strategy is the same as was described in Steps 4 or 4′ for the split-line design; however, it is more complex, because there are more nodes to be considered. For economic reasons it is especially important to trim downhill main lines where the head-loss gradient based on economic pipe sizes is less than the ground slope.

For a computer-assisted design strategy, follow Steps 1, 2, and 3. Then trim the system so that the ΔH_T found in Step 3 approaches zero, and the farthest (distal) outlet is at the minimum inlet pressure head required for the laterals unless velocity restrictions override economic considerations. This trimming process can be carried out so that the pipe size reductions are economically ordered. To do this, start by reducing the pipe size in the section where the value of the reduction, RV, will be greatest:

$$RV = \frac{\Delta(\text{Cost/unit length of next smaller pipe})}{\Delta J \text{ with smaller pipe at same flow rate}} \tag{10.4}$$

Pipe sizes should be reduced in the order of decreasing RV until ΔH_T approaches zero.

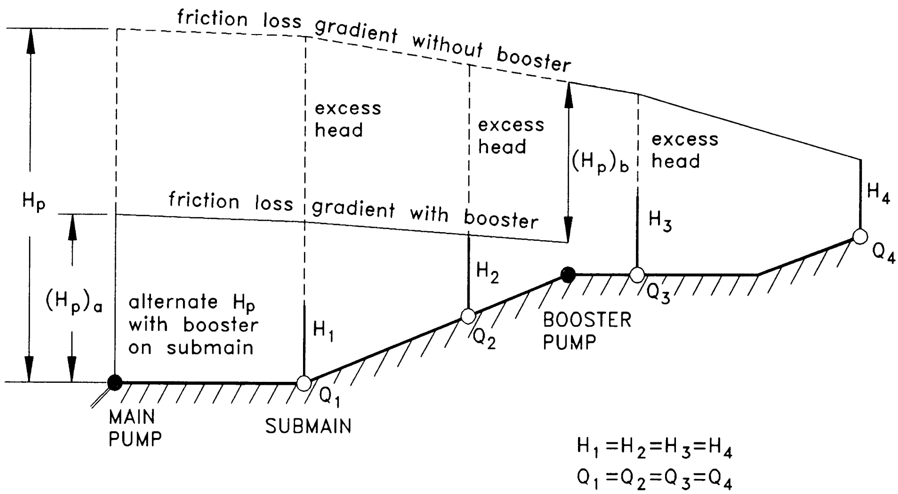


FIG. 10.2. Use of a Booster Pump to Reduce Maximum Head Requirement at the Main Pump and Save Energy.

DESIGN OF MAIN AND SUBMAIN LAYOUT

Before designing a main line that feeds several submains, the maximum operating head for each submain must be computed. The solution for minimum pipe sizes consistent with the allowable head loss in each submain is similar to the main line design problems in Sample Calculations 10.1 and 10.2. Figure 10.2 shows a cross-sectional view of a pump (and booster pump) serving four submains requiring equal inlet pressure heads and flows. The figure illustrates that without a booster pump the maximum head requirement at the main pump, H_p, is determined by the head requirement, H_4, for submain 4. However, if a booster pump is installed at the top of the hill, H_p can be reduced by half. This will save approximately one-fourth of the energy, for only half the water needs to be pressurized to meet the demands of Q_4, H_4 rather than the entire flow rate. When there is a booster pump, the alternate main pump $(H_p)_a$ depends on the maximum head requirement H_2 at submain 2. The booster pump $(H_p)_b$ depends on the difference between the head H_4 required at submain 4 and the residual head at the booster pump.

Sample Calculation 10.3. *Main line pipe selection for a system with submains.*

GIVEN: A project with four small center pivots, as shown in Fig. 10.3, where the flow rate to each center pivot is 200 gpm.

FIND: The most economical pipe sizes for the system based on the Economic Pipe-Selection Chart presented in Fig. 8.4 for aluminum pipe.

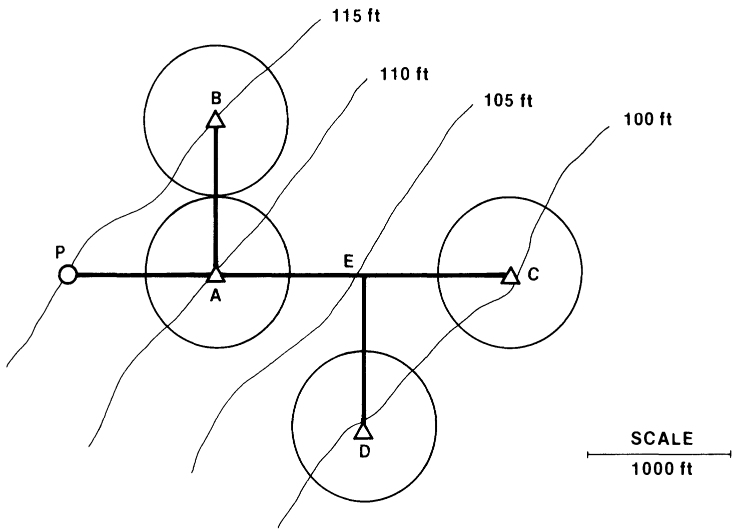


FIG. 10.3. Layout of Project with Four Small Center-pivot Laterals.

CALCULATIONS: First select the pipe sizes from Fig. 8.4, and compute the friction loss in each pipe section as in Table 10.1. Then locate the critical pivot lateral inlet, as demonstrated in the top portion of Table 10.2. The critical point is the inlet requiring the largest $h_f + \Delta H_e$, which in this case is point B. Excess pressure along the path from the pump to the critical inlet cannot be reduced by pipe-size reductions. However, the excess pressure in all other branches may be reduced, providing the velocity limitations are not exceeded. The excess

Table 10.1. Friction head loss calculations in each section for Sample Calculation 10.3

Pipe section	Flow, gpm	D -in.	J, ft/100 ft	L, ft	$h_f = J \times L/100$ ft
		Pipe sizes selected from economic chart			
P-A	800	10	0.45	1000	4.5
A-B	200	6	0.42	1000	4.2
A-E	400	8	0.37	1000	3.7
E-C	200	6	0.42	1000	4.2
E-D	200	6	0.42	1000	4.2
		Next smaller set of pipe sizes			
A-E	400	6	1.50	1000	15.0
E-C	200	5	1.01	1000	10.1
E-D	200	5	1.01	1000	10.1

Table 10.2. Location of critical pivot lateral inlet and trimming sequence for Sample Calculation 10.3

Pipe sections	h_f, ft	ΔH_e, ft	$h_f + \Delta H_e$, ft	Excess ft
Using pipe sizes selected from economic chart:				
P-A	4.5	−5	−0.5	0[1]
P-A-B	4.5 + 4.2 = 8.7	0	8.7	0[2]
P-A-E-C	4.5 + 3.7 + 4.2 = 12.4	−15	−2.6	11.3
P-A-E-D	4.5 + 3.7 + 4.2 = 12.4	−15	−2.6	11.3
Replacing 6-in. with 5-in. pipe between E-C and E-D:				
P-A-E-C	4.5 + 3.7 + 10.1 = 18.3	−15	3.3	5.4
P-A-E-D	4.5 + 3.7 + 10.1 = 18.3	−15	3.3	5.4
Replacing 478 ft of 8-in. pipe with 6-in. pipe between A-E:				
P-A-E-C	4.5 + 9.1 + 10.1 = 23.7	−15	8.7	0
P-A-E-D	4.5 + 9.1 + 10.1 = 23.7	−15	8.7	0

[1]Excess pressure at lateral inlets along critical path cannot be reduced by pipe-size reductions.
[2]The critical lateral inlet is at B.

head at C is equal to the difference between the $h_f + \Delta H_e$ values for P-B and for P-C, which is $8.7 - (-2.6) = 11.3$ ft. The same amount of excess head occurs at D.

Replacing the 6-in. pipe in sections E-C and E-D with 5-in. pipe still results in excess heads of 5.4 ft at C and D (see the center section of Table 10.2). Therefore, a portion of the 8-in. pipe in section A-E may be reduced to 6-in. pipe. The length, L_s, of the smaller 6-in. pipe that will increase the head loss by $\Delta h_f = 5.4$ ft can be computed by:

$$L_S = \frac{100\ \Delta h_f}{J_s - J_b} \tag{10.5}$$

where Δh_f = desired increase in head loss (by using L_s of smaller pipe in a given pipe section), m (ft)

And substituting into Eq. 10.5 gives:

$$L_s = \frac{100\ (5.4)}{1.50 - 0.37} = 478 \text{ ft}$$

Replacing 478 ft of 8-in. with 6-in. pipe in section A-E eliminates the excess head at inlets C and D, as indicated in the bottom portion of Table 10.2.

Portable Versus Buried Main Line

Buried main lines are restricted to areas that are to be irrigated permanently, whereas portable main lines can be used on all areas. Aside from this restriction on the use of main lines, the choice between portable and buried mains and between different pipe materials is largely a matter of economics.

Portable main lines have a potential advantage over buried main lines in that they can be moved about, and, in many cases, a greater area can be covered with the same length of pipe. For example, in Fig. 7.2B the required length of portable main line pipe is only half the length that would be required if buried pipe were used. Another advantage of portable lines is that installation costs are much less than for buried lines.

Nevertheless, buried main lines have some distinct advantages over portable main lines. Typical materials, such as plastic, used in buried main line pipe and the fact that the pipe is not handled after initial installation contribute to a much longer life. Thus, for the same length and size of main line, the annual fixed cost for buried main lines is usually lower than for portable lines. Buried pipe also affords a considerable saving in operating costs by eliminating the labor required to move portable lines within the design area and to and from the place of storage at the start and end of the irrigation season. Furthermore, buried lines do not present obstacles to planting, cultural, or harvesting operations.

The procedure for making an economic comparison between two main line pipe materials is as follows: first develop a layout and select sets of pipe diameters using the economic methods described earlier for each pipe material under consideration, and then determine the total annual cost (fixed, energy, maintenance, labor) of the main line portion of each system.

Design for Continuous Operation

Most irrigators prefer systems that may be operated continuously while each lateral line is uncoupled and moved to its next position. To achieve this with portable main lines, valve-tee couplers are placed at each lateral position, and each lateral is equipped with a quick-coupling, valve-opening elbow or tee (see Figs. 5.3B and 10.4). The elbow provided with each lateral is used to open and close the valves along the main line as required, without interrupting the rest of the system. For buried main lines the takeoff or hydrant valves are placed above ground on risers and serve the same purpose as the valve-tee couplers along portable lines.

One or more extra lateral lines are often used, so that lateral lines may be moved from one position to another while others are in use, thereby permitting uninterrupted operation. This type of operation takes only one or two people, moving one lateral line while the other lines are running, to keep a relatively large system operating continuously.

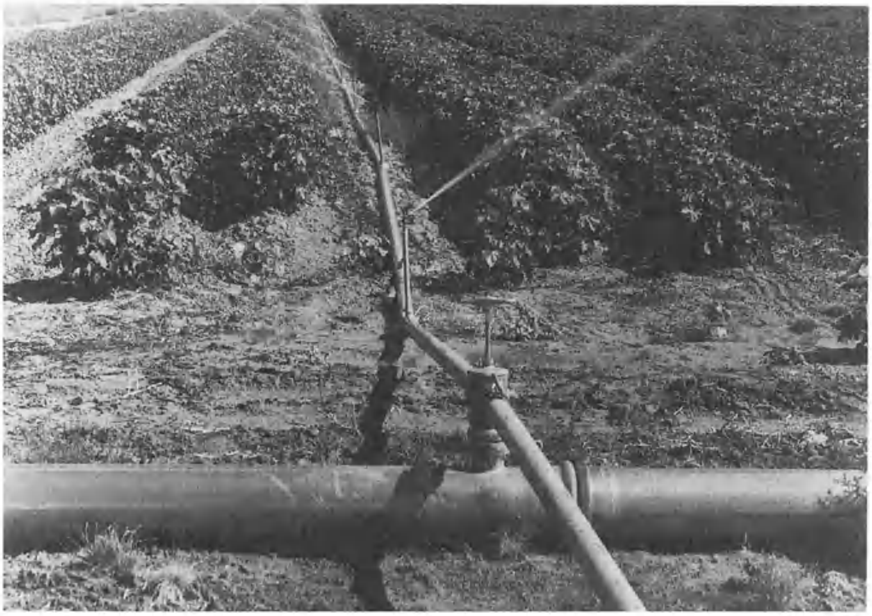


FIG. 10.4. Hand-move Sprinkler Lateral Operating from a Quick-coupling, Valve-opening Tee.

SPECIAL GRAVITY-PRESSURE CONSIDERATIONS

For the most convenient management of irrigation systems with set sprinklers, it is desirable to hold the application rate constant. However, for gravity-pressured systems the elevation differential may not be sufficient to produce the desired operating pressure in the areas adjacent to the water source. Sprinkler discharges will be below normal in the low-pressure areas, and to obtain a constant average application rate the sprinkler spacing must be decreased accordingly.

As pressure decreases, the diameter of the sprinkler coverage decreases at a slower rate than does the discharge (see Table 5.2). Therefore, fairly good coverage and uniformity of application may be maintained at lower pressure by reducing the sprinkler spacing. Lateral spacing may be reduced in proportion to the square root of the drop in pressure (as explained below); however, neither spacing nor pressure should be decreased below normally accepted values. The alternatives to operating at low pressures are to add a booster pump to the system or not to water the high-elevation areas of the fields.

Because the sprinkler spacing on the lateral line is fixed, the lateral spacing (or set time) along the main line must be adjusted to compensate for the lower sprinkler discharge. If constant set times are assumed, an equation for determining the appropriate lateral spacing along the main line may be derived as follows. First, note that the nozzle discharge can be expressed by Eq. 5.1b, and the average application rate for a given sprinkler discharge at a given sprinkler spacing is given by Eq. 5.5. Then combine Eqs. 5.1b and 5.5 and rearrange the terms to obtain:

$$(S_l) = \frac{K\,K_d(H_a)^{0.5}}{I \times S_e} \tag{10.6a}$$

where

(S_l) = variable lateral spacing along main, m (ft)
K = conversion constant, 60 for metric units (96.3 for English units)
K_d = discharge coefficient for the sprinkler and nozzle combined
H_a = average operating pressure head of the sprinklers along the lateral, m (ft)
I = average application rate, mm/hr (in./hr)
S_e = sprinkler spacing on lateral, m (ft)

By holding I and S_e constant and noting that the available lateral inlet pressure head, H_l, establishes H_a, Eq. 10.6a may be reduced to:

$$(S_l) = K_{ds}(H_l)^{0.5} \tag{10.6b}$$

where K_{ds} = lateral discharge-spacing coefficient, which is a function of I, S_e, K, and K_d; and H_l = lateral inlet pressure head, m (ft).

The coefficient, K_{ds}, may be theoretically derived; however, a simpler method for determining it is as follows. First, select the desired operating conditions for that portion of the field where sufficient lateral inlet pressure is available. In selecting the desired operating conditions, S_l and H_l are automatically set and K_{ds} can be solved very simply from Eq. 10.6b. (The lateral inlet pressure, P_l, may be substituted for H_l, as K_{ds} will assume the necessary conversion factors.)

The spacing between lateral moves that will give a constant average application rate can easily be determined where below-normal operating pressures are encountered. This is accomplished by solving Eq. 10.6b, using the K_{ds} as determined above and the actual H_l available at each lateral position. Care must be taken, however, that the pressures utilized will be sufficient to rotate the sprinklers and provide adequate jet breakup.

Sample Calculation 10.4. *Design of variable lateral spacing to give an equal application rate throughout a gravity-pressured system.*

GIVEN: A gravity-pressured system with hand-move sprinkler laterals is to be designed for a standard inlet pressure P_l = 345 kPa (50 psi) at a standard lateral spacing along the main line S_l = 18.3 m (60 ft).

The main line passes downhill through some steep irrigable land before the available lateral inlet pressure reaches P_l. The ground slope in this upper region $s = -11\%$, and the pipe friction head-loss gradient or slope along the main line is only $J = 1\%$.

FIND: Determine the lateral spacing that will give the desired standard application rate for the three lateral positions uphill from where the standard lateral spacing can begin.

CALCULATIONS: Converting P_l to H_l:

$$H_l = 345/9.8 = 35.2 \text{ m } (115 \text{ ft})$$

By Eq. 10.6b:

$$K_{ds} = 18.3/(35.2)^{0.5} = 3.08$$

The difference in the ground and friction slopes is 10%. Therefore, the pressure head will be approximately $0.10 \times 18.3 = 1.8$ m less than the standard $H_l = 35.2$ m or $(H_l)_1 = 33.4$ m at the *first uphill lateral position*. Thus, by Eq. 10.6b the first uphill spacing from the standard part of the field should be:

$$(S_l)_1 = 3.08(33.4)^{0.5} = 17.8 \text{ m}$$

At the second uphill lateral position the available pressure head will be approximately:

$$(H_l)_2 = 35.2 - 0.10(2 \times 17.8) = 31.6 \text{ m}$$

And the spacing between the first and second uphill laterals should be:

$$(S_l)_2 = 3.08(31.6)^{0.5} = 17.3 \text{ m}$$

At the third uphill lateral position the available pressure head will be approximately:

$$(H_l)_3 = 35.2 - 0.10(17.8 + 2 \times 17.3) = 30.0 \text{ m}$$

And the spacing between the second and third uphill laterals should be:

$$(S_l)_3 = 3.08(30.0)^{0.5} = 16.9 \text{ m}$$

Sample Calculation 10.5. *Selecting the main supply-network pipe sizes for a small project serving 20 farm units from a well.*

GIVEN: A 100-ha square, small project area served from a well, as shown in Fig. 10.5. The project area is subdivided into 20 5-ha farm units, each with its own sprinkler lateral. Other relevant site-specific and design data include the following.

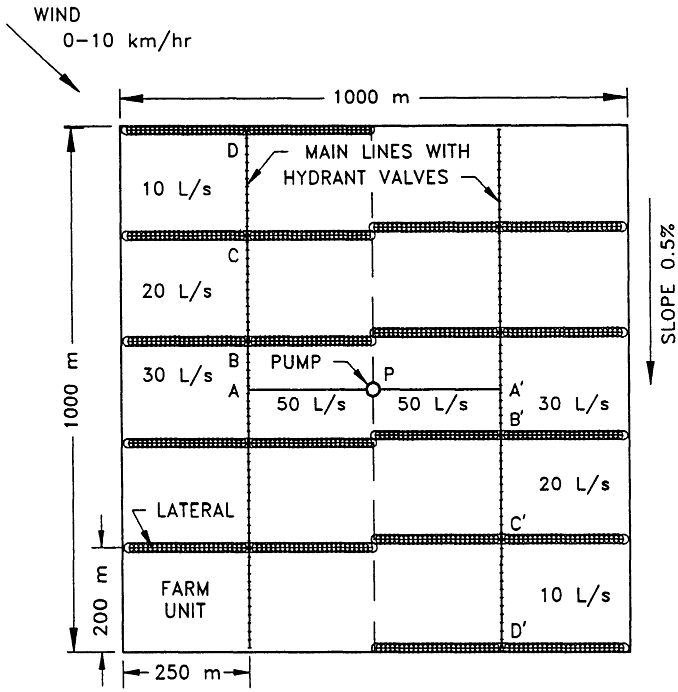


FIG. 10.5. Layout of Small Sprinkler Irrigation Project Serving 20 Farm Units.

Soil-water-plant relationships:

$W_a = 120$ mm/m; $U_d = 6$ mm/day; and $U = 1000$ mm/year

Economic relationships:

Diesel at \$0.20/L; producing 4.0 hp-hr/L; $E_p = 75\%$; Cp = \$0.75/lb.; $n = 10$ years; $i = 10\%$; and $e = 9\%$

Sprinkler and lateral configurations:

$H_a = 29.5$ m, $q_a = 11.1$ L/min, and $S_e \times S_l = 9.1 \times 15.2$ m, so $I = 4.8$ mm/hr; with two 11-hr sets/day and $E_h = 80\%$, $d = 53$ mm, $d_n = 42$ mm, and $f' = 7$ days; and with 27 sprinklers/lateral, $Q_l = 5.0$ L/s.

FIND: The most economical main line pipe sizes for the system layout based on the Universal Economic Chart presented in Fig. 8.7.

CALCULATIONS: The strategy for determining the most economical pipe sizes is simple but the process is involved. Therefore, the steps are numbered for convenience.

1. Compute the equivalent annual escalating cost of energy, E', by Eq. 8.19.
   a. First determine the operating hours per year, O_t:
   Each lateral must be moved

$$\frac{200 \text{ m}}{15.2 \text{ m}} \simeq 13 \text{ times/irrigation}$$

so:

$$O_t = \frac{1000 \text{ mm}}{42 \text{ mm}/(13 \text{ sets} \times 11 \text{ hr/set})} = 3400 \text{ hr}$$

b. From Table 8.9; $EAE(9) = 1.420$ and $CRF = 0.163$ for $n = 10$, $i = 0.10$, and $e = 0.09$

c. Then by Eq. 8.19:

$$E' = \frac{3400 \text{ hr/year} \times \$0.20/\text{L} \times 1.420}{75/100 \text{ pump eff} \times 4 \text{ hp-hr/L}} = \$322/\text{hp-year}$$

2. Next compute the adjustment factor A_f by Eq. 8.20:

$$A_f = \frac{0.0001 \times \$322/\text{hp-year}}{0.163 \times 0.75/\text{lb for PVC}} = 2.63$$

3. Use Fig. 8.7 to select the pipe sizes for the uphill branch from P to D.

a. First convert the section discharges to gpm:

$$50.0 \text{ L/s} = 793 \text{ gpm}$$

$$30.0 \text{ L/s} = 475 \text{ gpm}$$

$$20.0 \text{ L/s} = 317 \text{ gpm}$$

$$10.0 \text{ L/s} = 159 \text{ gpm}$$

b. Because the system is symmetrical, both in terms of flows and slopes, enter Fig. 8.7 with the adjusted left-hand branch flow:

$$Q_s' = A_f Q_s = 2.63 \times 793 = 2090 \text{ gpm}$$

c. And select the pipe sizes for the uphill and most difficult to serve branch, as:

Section	Flow, gpm	Pipe size
P-A	793	10-in.
A-B	475	8-in.
B-C	317	8-in.
C-D	159	6-in.

4. Determine the pressure head required at P to overcome main line friction losses and elevation from P to D as:

Section	Length, m	J, m/100 m	h_f – m
P-A	250	0.27	0.7
A-B	100	0.30	0.3
B-C	200	0.15	0.3
C-D	200	0.15	0.3
		Total h_f =	1.6 m
		$\Delta El = 0.005 \times 500$ m =	2.5 m
		Total $H_e + h_f$ =	4.1 m

5. Determine the pressure head available for friction losses between A' and D' so that the resulting pressure head at the uphill point D will be the same as at the downhill point D′:

Pressure head available at P	4.1 m
Friction loss in section P-A′	(0.7)
ΔEl between D′ and A′	2.5
Available for h_f between A′ and D′ =	5.9 m

Section P-A′ should also be 10-in. pipe because it also serves the uphill branch on the right side of the field.

6. Select pipe sizes for each downhill segment to give h_f = 5.9 m from A′ to D′:

Section	Length, m	Pipe size	Flow, gpm	J, m/100 m	h_f, m
A′-B′	100	6-in.	475	1.10	1.10
B′-C′	200	6-in.	317	0.52	1.04
C′-D′	200	4-in.	159	0.95	1.90
				Total h_f =	4.04 m

To take full advantage of the 5.9 m, some of section B′-C′ can be reduced from 6- to 4-in. pipe. Eq. 10.5; letting $\Delta h_f = (5.9 - 4.04) = 1.86$ m, $J_s = J_4 = 3.17$ m/100 m, and $J_b = J_6 = 0.52$:

$$L_4 = \frac{100 \times 1.86}{3.17 - 0.52} = 70 \text{ m}$$

Therefore, in Section B′-C′ use 130 m of 6-in. pipe and 70 m of 4-in. pipe.

RECOMMENDED READING

Benami, A, and A. Ofen. 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).

Benami, A. 1982. Sprinkler irrigation. In *CRC Handbook of Irrigation Technology*, ed. H. J. Finkel, vol. I, pp. 193-245. Boca Raton, Florida: CRC Press.

11

Pressure Requirements for Set Sprinkler Systems

To select a pump and power unit that will operate a system efficiently, it is necessary to determine the desired lateral inlet pressure head, head differences due to elevation, and pressure losses in the delivery system. The sum of a properly selected set of these is the total dynamic head, TDH, against which water must be pumped. Where operating conditions will vary considerably with the movement of laterals and main lines or due to changes in the number of sprinklers operated, both the maximum and minimum TDH should be computed.

The pressure head required to operate laterals and to overcome friction losses in main lines and submains has already been discussed. Other head losses for which pressure at the pump must be provided are discussed in succeeding paragraphs.

FITTING AND VALVE LOSSES

Allowance must be made for friction losses in all elbows, tees, crossings, reducers, increasers, adapters, and valves that are in series between the water supply and critical sprinkler. These losses must be taken into account in laterals, main lines, submains, and in the suction line. Where deep-well turbine pumps are used, losses in the column (between the ground surface and the pump bowels) must also be considered. Pump manufacturers make allowances for losses in the pump itself.

Losses in fittings and valves can be computed by:

$$h_f = K_r \frac{V^2}{2g} \tag{11.1}$$

where

h_f = friction-head loss due to pipe fitting, m (ft)
K_r = resistance coefficient for the fitting or valve

$\frac{V^2}{2g}$ = velocity head for a given discharge and pipe or fitting diameter, m (ft)

g = accumulation due to gravity, 9.81 m/s^2 (32.2 ft/s^2)

Values of the resistance coefficient, K_r, may be taken from Table 11.1 for irrigation pipe or Table 11.2 for standard pipe fittings and valves. Figure 11.1 shows a typical aluminum, lateral pipe, hook-latch coupler with a sprinkler riser outlet. Figure 10.4 shows a typical portable pipe main line with ring-lock couplers and a hydrant valve with an opener supplying a pair of aluminum lateral lines.

The velocity head may be computed by:

$$\frac{V^2}{2g} = K\frac{Q^2}{D^4} \tag{11.2}$$

Table 11.1. Values of resistance coefficient, K_r, for plastic and portable aluminum irrigation pipe fittings and valves[1]

	Nominal diameter – in.							
Fitting of valve	2	3	4	5	6	8	10	12
Couplers								
ABC (Ames)	1.2	0.8	0.4	0.3				
Hook-latch	0.6	0.4	0.3	0.2	0.2			
Ring-lock				0.2	0.2	0.2	0.2	0.2
Elbows								
Long radius	0.4	0.3	0.3	0.3	0.2	0.2	0.2	0.2
Mitered	0.8	0.7	0.6	0.6	0.6	0.6	0.6	0.5
Tees								
Hydrant (off)		0.6	0.5	0.4	0.3	0.3	0.3	0.3
Side outlet	1.6	1.3	1.2	1.1	1.0	0.9	0.8	0.8
Line flow	0.8	0.7	0.6	0.6	0.5	0.5	0.4	0.4
Side inlet	2.4	1.9	1.7	1.5	1.4	1.2	1.1	1.1
Valves								
Butterfly	1.2	1.2	1.1	1.0	0.8	0.6	0.5	0.5
Plate type	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
Check	2.2	2.0	1.8	1.6	1.5	1.3	1.2	1.1
Hydrant with opener	–	8.0	7.5	7.0	6.7	–	–	–
Special								
Strainer	1.5	1.3	1.0	0.9	0.8	0.7	0.6	0.5
"Y" (long rad.)	0.8	0.6	0.6	0.6	0.4	0.4	0.3	0.3

1 in. = 25 mm

[1]Adapted from McCulloch et al. (1957).

Table 11.2. Values of resistance coefficient, K_r, for standard pipe fittings and valves

Standard fitting or valve	Nominal diameter – in. 3	4	5	6	7	8	10	12	14
Elbows flanged:[1]									
Regular 90°	0.34	0.31	0.30	0.28	0.27	0.26	0.25	0.24	0.23
Long radius 90°	0.25	0.22	0.20	0.18	0.17	0.15	0.14	0.13	0.12
Long radius 45°	0.19	0.18	0.18	0.17	0.17	0.17	0.16	0.15	0.15
Elbows screwed:[1]									
Regular 90°	0.80	0.70							
Long radius 90°	0.30	0.23							
Regular 45°	0.30	0.28							
Bends[1]									
Return flanged	0.33	0.30	0.29	0.28	0.27	0.25	0.24	0.23	0.23
Return screwed	0.80	0.70							
Tees flanged:[1]									
Line flow	0.16	0.14	0.13	0.12	0.11	0.10	0.09	0.08	0.08
Branch flow	0.73	0.68	0.65	0.60	0.58	0.56	0.52	0.49	0.47
Tees screwed:[1]									
Line flow	0.90	0.90							
Branch flow	1.20	1.10							
Valves:[1]									
Globe flanged	7.0	6.3	6.0	5.8	5.7	5.6	5.5	5.40	5.40
Globe screwed	6.0	5.7							
Gate flanged	0.21	0.16	0.13	0.11	0.09	0.075	0.06	0.05	0.04
Gate screwed	0.14	0.12							
Check flanged	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.00	2.00
Check screwed	2.1	2.0							
Angle flanged	2.2	2.1	2.0	2.0	2.0	2.0	2.0	2.00	2.00
Angle screwed	1.3	1.0							
Foot	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80	0.80
Basket strainer	1.25	1.05	0.95	0.85	0.80	0.75	0.67	0.60	0.53
Other fittings									
Inlets or entrances:[2]									
Inward	0.78								
projecting		All diameters							
Sharp cornered	0.50	All diameters							
Slightly rounded	0.23	All diameters							
Bell-mouth	0.04	All diameters							
Sudden enlargements	$K_r = [1 - (D_r)^2]^2$ where D_r = ratio of small to large inside diameter								
Sudden contractions	$K_r = 0.7\,[1 - (D_r)]^2$								

1 in. = 25 mm.

[1]Values recommended in *Pipe Friction Manual*, Hydraulic Institute.

[2]Values recommended in King's handbook.

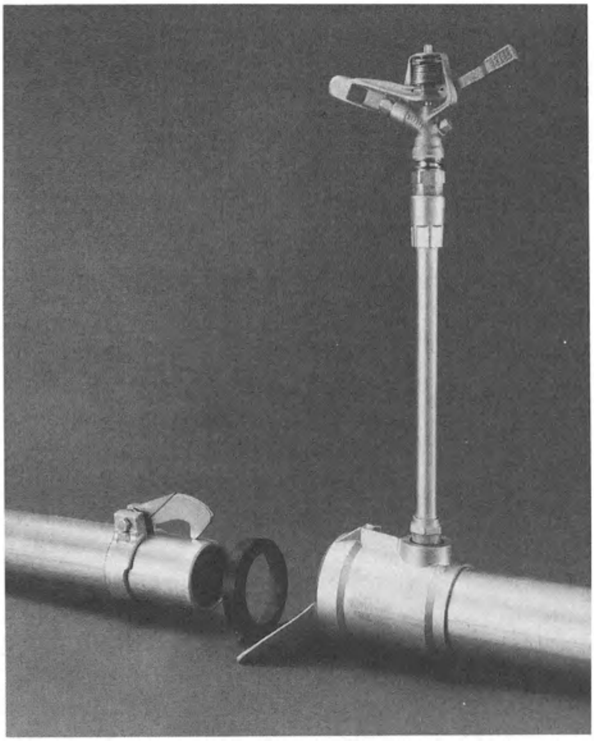


FIG. 11.1. Typical Hook-latch Coupler (with Impact Sprinkler on Riser) for Portable Aluminum Pipe (Source: Hastings Irrigation Pipe Co.).

where K = conversion constant, 8.26×10^4 for metric units (2.59×10^{-3} for English units); Q = flow rate, L/s (gpm); and D = inside diameter of pipe, mm (in.).

Table 11.3 gives velocity heads in feet for inside diameters in whole-inch increments. Actual inside diameters of pipes and fittings are usually different, but the table values give satisfactory results, because the values of K_r for the fittings are based on using the nominal diameters of the fittings as well.

When determining the velocity head at a reducing fitting, the diameter and flow that give the highest head should be used. As an example assume a 200- by 150- by 150-mm (8- by 6- by 6-in.) reducing side-outlet tee has an inflow of 63 L/s (1000 gpm) and outflows of 25 L/s (400 gpm) from the side outlet and 38 L/s (600 gpm) through the body. The three respective velocity heads from Table 11.3 are 0.20 m (0.64 ft) for the inlet with a flow of 63 L/s (1000 gpm), 0.10 m (0.32 ft) for the side outlet with 25 L/s (400 gpm), and 0.22 m (0.71 ft) for the remaining line flow through the fitting body. Therefore, when estimating h_f for the side-outlet flow, use the velocity head of 0.20 m (0.64 ft), since it is larger than 0.10 m (0.32 ft), and K_r = 1.0 from Table

Table 11.3. Values of velocity head, $V^2/2g$, for different diameters and flow rates

Flow	Inside diameter – in.								
	2	3	4	5	6	7	8	10	12
gpm	Velocity head – ft								
100	1.62	0.32	0.10	0.04	0.02				
150	3.64	0.72	0.23	0.09	0.04				
200	6.48	1.28	0.40	0.17	0.08				
250		2.00	0.63	0.26	0.12				
300		2.88	0.91	0.37	0.18	0.10	0.06		
350		3.92	1.24	0.51	0.24	0.13	0.08		
400		5.12	1.62	0.66	0.32	0.17	0.10		
450		6.48	2.05	0.84	0.40	0.22	0.13		
500		8.00	2.53	1.04	0.50	0.27	0.16	0.06	
550		9.68	3.06	1.25	0.60	0.33	0.19	0.07	
600			3.64	1.49	0.71	0.39	0.23	0.09	
650			4.28	1.75	0.84	0.46	0.27	0.11	
700			4.96	2.03	0.98	0.53	0.31	0.13	0.06
750			5.69	2.33	1.12	0.61	0.36	0.15	0.07
800			6.48	2.65	1.28	0.69	0.41	0.17	0.08
850			7.31	2.99	1.44	0.78	0.46	0.19	0.09
900			8.20	3.36	1.62	0.87	0.52	0.21	0.10
1000				4.15	2.00	1.08	0.64	0.26	0.13
1100				5.02	2.42	1.31	0.77	0.31	0.15
1200				5.97	2.88	1.55	0.92	0.37	0.18
1300				7.01	3.38	1.82	1.07	0.44	0.21
1400				8.12	3.92	2.11	1.25	0.51	0.25
1500				9.33	4.50	2.43	1.42	0.58	0.28
1600					5.12	2.75	1.64	0.66	0.32
1800					6.48	3.49	2.01	0.84	0.41
2000					8.00	4.32	2.53	1.04	0.50
2200					9.68	5.22	3.06	1.25	0.61
2400						6.22	3.64	1.49	0.72
2600						2.29	4.28	1.75	0.84
2800						8.49	4.96	2.03	0.98
3000						9.71	5.69	2.33	1.12
3200							6.47	2.65	1.28

1.0 in. = 25.4 mm; 100 gpm = 6.31 L/s; 1.0 ft = 0.305 m

11.1 to obtain $h_f = 1.0 \times 0.20 = 0.20$ m ($1.0 \times 0.64 = 0.64$ ft) (by Eq. 11.1). For the line flow $h_f = 0.5 \times 0.22 = 0.11$ m ($0.5 \times 0.71 = 0.36$ ft).

Figure 11.2 is a nomograph that can be used in conjunction with Tables 11.1 and 11.2 to estimate losses in fittings and valves. Sample Calculation 11.1 is presented to demonstrate the use of the nomograph.

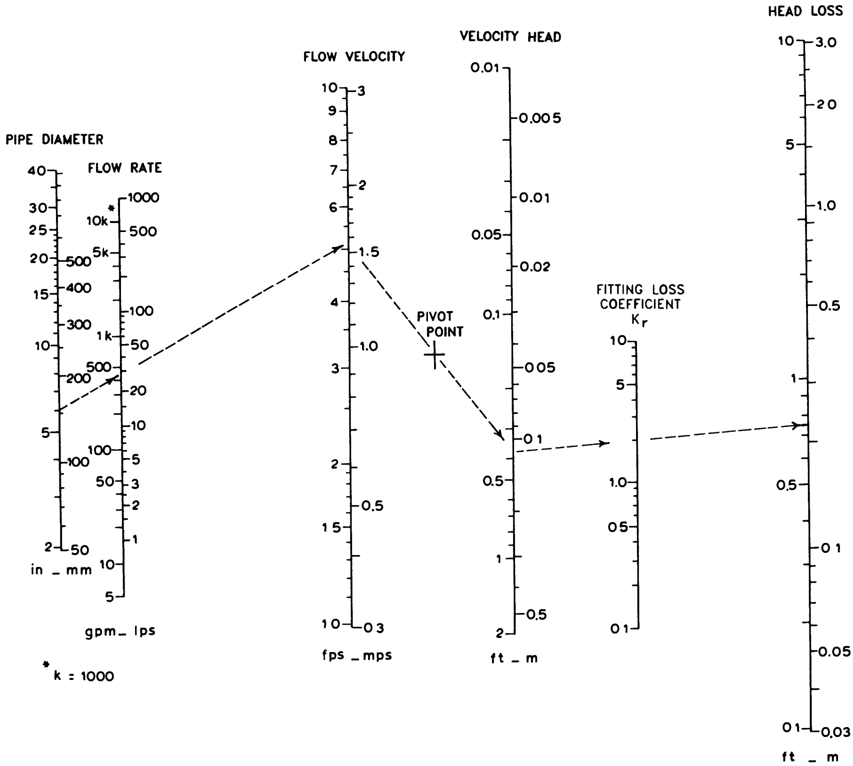


FIG. 11.2. Nomograph for Estimating Friction Losses Due to Pipe Fittings (By: Safa N. Hamad).

Sample Calculation 11.1. *Use of nomograph for estimating fitting losses.*

GIVEN: A plate valve for irrigation pipe with $Q = 28.4$ L/s (450 gpm); and $D = 152$ mm (6 in.).

FIND: The head loss resulting from the valve based on the nomograph for estimating fitting losses, Fig. 11.2.

CALCULATIONS: From Table 11.1, $K = 2.0$.

Start from the left on the nomograph with 152-mm (6 in.) pipe diameter passing through 28.4 L/s (450 gpm) on the flow-rate line.

This line interests the flow-velocity scale at 1.55 m/s (5.1 ft/s).

Draw a line from the flow velocity through the pivot point to intersect the velocity head at 0.12 m (0.38 ft).

Draw a line from this velocity head through $K = 2.0$ to the head loss (right-hand) scale to find $h_f = 0.23$ m (0.76 ft).

BERNOULLI THEOREM

The well-known Bernoulli theorem is helpful in understanding the relationship between the components of head that are discussed at the beginning of Chapter 8. The theorem states that the sum of pressure head, H, velocity head, $V^2/2g$, and elevation, El, at any point in a pipeline is equal; to the sum of the corresponding quantities at any point upstream, less the friction head loss, h_f, between the two points. Referring to Fig. 11.3, in which the velocity head is negligible except in the jets from the sprinklers, the relationships can be expressed as:

$$
\begin{aligned}
H_4 + \frac{V_4^2}{2g} + El_4 &= \frac{V_j^2}{2g} + El_5 + (\text{head loss in sprinkler}) \\
&= H_2 + \frac{V_2^2}{2g} + El_2 - h_{f(\text{lateral})} \\
&= H_1 + \frac{V_1^2}{2g} + El_1 - h_{f(1\text{ to }4)} \\
&= TDH + El_0 - h_{f(\text{total})} \qquad (11.3)
\end{aligned}
$$

The following observation can be made in reference to Fig. 11.3:

- Between 0 and 1 the difference between TDH produced by the pump and H_1 is the suction lift and friction;
- Between 1 and 2 the difference between H_1 and H_2 is mostly due to the $\Delta El(1\text{ to }2)$;
- At 1 and 2 the head-loss-curve drops are caused by the pump discharge valving and the hydrant valve, respectively; and
- At the last sprinkler most of the remaining pressure head, H_4, is converted to jet velocity head, $V_j^2/2g$, at the sprinkler nozzle.

TOTAL DYNAMIC HEAD

Total dynamic head, TDH, is the head required for an irrigation system to deliver the specified total system discharge, Q_s. The accurate estimate of TDH is very important when selecting a pumping plant. A low estimate will result in the system delivering less water than specified and possibly a low irrigation efficiency, and a high estimate will waste energy and result in a higher pumping cost than necessary.

The TDH is the sum of the pressure, elevation, and friction-loss heads during normal operation. It is computed as the sum of the following (see Fig. 11.3):

- Pressure-head required to operate the critical lateral, H_l, m (ft)
- Friction-head losses in main line and submains, h_f, m (ft)

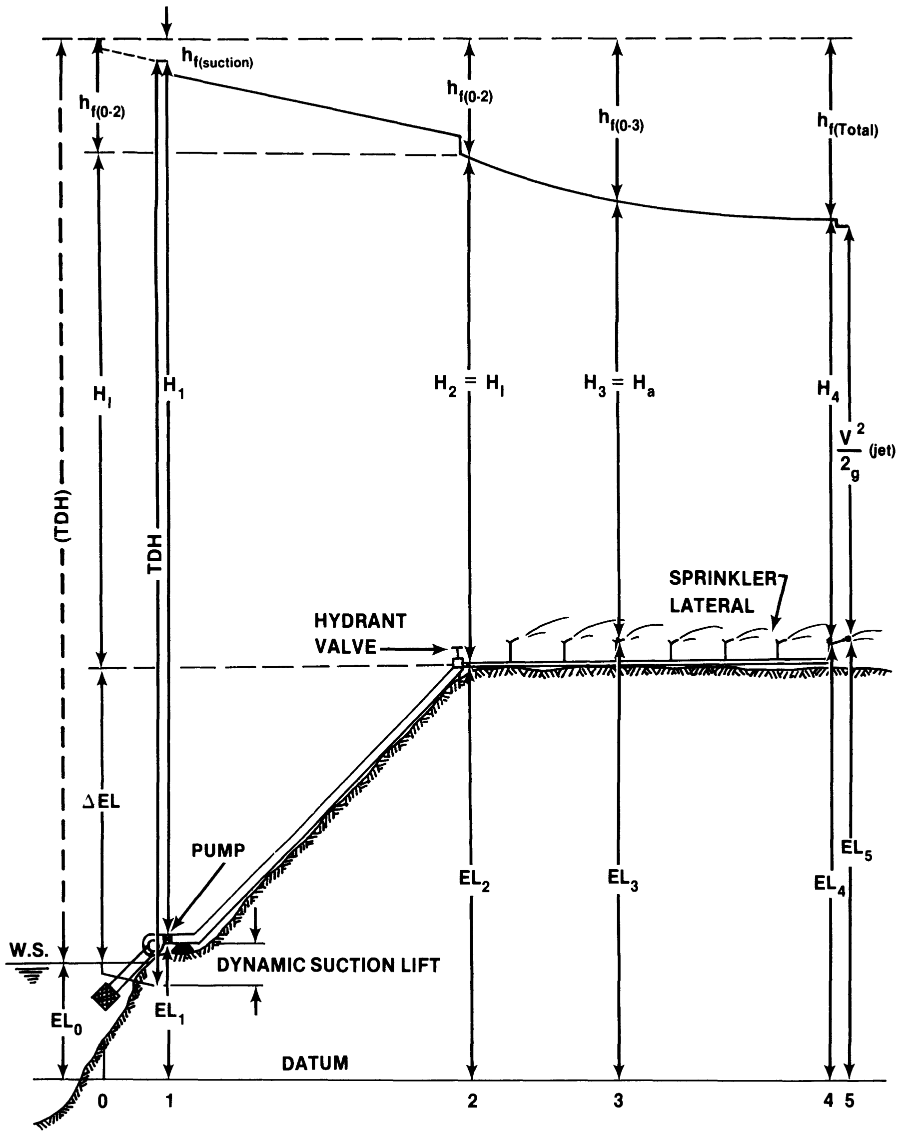


FIG. 11.3. Schematic Representation of Total Dynamic Head and Bernoulli Equation (Assuming Velocity Head is Negligible Except in Sprinkler Jet). The Extra (Dashed) Suction Friction and TDH Losses are Presented to Graphically Show the Friction Loss in the Whole System.

- Friction-head losses in fittings an valves, Σh_f, m (ft)
- Difference in elevation from water surface, ΔEl, which is the static head, H_e, m (ft)
- Suction assembly friction-head losses, h_f, m (ft)
- Miscellaneous losses (for safety), usually taken as 20% of the pipe and fitting friction losses, h_f, m (ft)

Sample calculation 11.2 is presented to demonstrate the computation of the TDH for a typical periodic-move sprinkler system.

Sample Calculation 11.2. *Determining the total dynamic head for a sprinkle system.*

GIVEN: The system layout as shown in Fig. 11.4 with three laterals having flow rates of $Q_l = 300$ gpm at $P_l = 50$ psi. Mainlines are PVC plastic pipe, IPS-SDR-41, and the system capacity is $Q_s = 900$ gpm. The suction lift 8.0 ft, and the friction loss in the suction assembly and screen is $h_f = 3.2$ ft.

FIND: Determine the total dynamic head, TDH, required.

CALCULATIONS: By inspection the critical lateral is at C, and the pressure head, H_l, required at the inlet is:

$$H_l = P_l \times 2.31 = 50 \times 2.31 = 115.5 \text{ ft}$$

The friction loss in the main line between P and C using J values from Table 8.5 is:

Section P-A	$0.98 \times 1000/100 =$	9.8 ft
Section A-B	$1.67 \times 500/100 =$	8.4 ft
Section B-C	$0.47 \times 500/100 =$	2.4 ft
	Total $h_f =$	20.6 ft

The total friction-head loss due to the fittings based on K_r values from Table 11.1 and velocity head values from Table 11.3 is computed as follows, using Eq. 11.1:

Velocity heads from Table 11.3 are:

Section P-A	0.52 ft
Section A-B	0.71 ft
Section B-C	0.18 ft
4-in. hydrant	0.91 ft

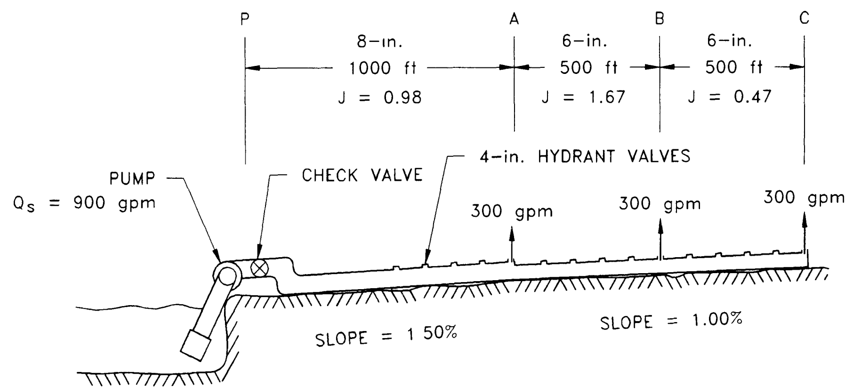


FIG. 11.4. Sprinkle System Main Line Layout with Both Open and Closed Hydrant Valves.

The fitting losses in Section P-A:

1 8-in. check valve	1.3×0.52	= 0.7 ft
2 8-in. mitered elbows	$2\,(0.6 \times 0.52)$	= 0.6 ft
4 8-in. hydrants (off)	$4\,(0.3 \times 0.52)$	= 0.6 ft
1 8-in. line-flow tee (flow in main past open hydrant)	0.5×0.52	= 0.3 ft

The fitting losses in Section A-B:

4 6-in. hydrants (off)	$4\,(0.3 \times 0.71)$	= 0.9 ft
1 6-in. line-flow tee	0.5×0.71	= 0.4 ft

The fitting losses in Section B-C:

4 6-in. hydrants (off)	$4\,(0.3 \times 0.18)$	= 0.2 ft

The fitting loss at C:

1 4-in. hydrant with opener	7.5×0.91	= 6.8 ft
	Total fitting h_f	= 10.5 ft

The static head between water surface elevation and D is:

Suction lift (given)		= 8.0 ft
Section P-A	$1.5\% \times 1000/100$	= 15.0 ft
Section A-C	$1.0\% \times 1000/100$	= 10.0 ft
	Total ΔEl	= 33.0 ft

The miscellaneous losses are typically estimated as 20% of the total friction losses. Thus, they may be computed as:

$$\frac{20}{100}\,(20.6 + 10.5 + 3.2) = 6.9 \text{ ft}$$

Summing the above values gives the total dynamic head, TDH, required at the pump discharge as follows:

Lateral inlet pressure head	$H_l =$	115.5 ft
Total pipe friction head	$h_f =$	20.6 ft
Total fitting friction head	$h_f =$	10.5 ft
Static head	$H_e = \Delta El =$	33.0 ft
Suction assembly friction	$h_f =$	3.2 ft
Miscellaneous losses		6.9 ft
	TDH =	190.0 ft

SYSTEM *H-Q* CHARACTERISTICS

Systems will operate at the design Q_s only if the exact TDH is available. When more (or less) pressure head is available the system will discharge more (or less) water than the design Q_s unless pressure or flow regulation is provided.

Head-Discharge Relationships

The subtotal of the pressure head required to overcome pipeline and fitting friction losses and to provide the necessary (fixed-nozzle or orifice) discharge pressure can be expressed by:

$$H_{tp} = (h_f)\text{ pipe} + (h_f)\text{ fittings} + H_a$$

Noting the relationships between H and Q in Eqs. 8.1, 11.1, and 5.1, and assuming the system characteristics are fixed:

$$H_{tp} = K_{sp}Q_s^{1.85} + K_{sr}Q_g^2 + K_{sd}Q_s^2 \tag{11.4a}$$

which can be closely approximated by:

$$H_{tp} = K_s(Q_s)^2 \tag{11.4b}$$

where

H_{tp} = subtotal of pressure heads required to overcome pipeline and fitting friction losses and to provide sprinkler operating pressure, m (ft)
K_{sp} = system pipeline friction coefficient
K_{sr} = system fitting friction coefficient
K_{sd} = system sprinkler (or orifice-type emitter) discharge coefficient
K_s = system discharge coefficient
Q_s = total system discharge, L/s (gpm)

The exponent for the last term in Eq. 11.4a is the reciprocal of the discharge exponent of the emission devices, i.e., $1/x$, which is $1/0.5 = 2$ for fixed nozzles and orifices without pressure- or flow-regulating devices (see Eq. 5.1). However, for systems with outlets that have discharge exponents other than $x = 0.5$, the exponent in the outlet (farthest to the right) term in Eq. 11.4a must be changed accordingly. Furthermore, the three head terms cannot be consolidated as in Eq. 11.4b.

System Head-Discharge Curves

The relationship between sprinkle system discharge (without flow regulation) and the total dynamic head required to produce the discharge is:

$$\text{TDH} = K_s(Q_s)^2 + H_e \tag{11.5a}$$

which can be arranged to give:

$$Q_s = K_s'(\text{TDH} - H_e)^{0.5} \tag{11.5b}$$

The K_s or K_s' for the system can be found by entering the design Q_s, TDH, and H_e in Eq. 11.5a or 11.5b. Furthermore, the system discharge resulting from any given total pumping head, H_T, can be expressed by:

$$Q = Q_s \left[\frac{H_T - H_e}{\text{TDH} - H_e}\right]^{0.5} \tag{11.5c}$$

or

$$Q = K_s'(H_T - H_e)^{0.5} \tag{11.5d}$$

where H_T = any total dynamic pumping head, m (ft), and $H_e = \Delta El$ and is positive (+) for uphill and negative (−) for downhill.

Sample Calculation 11.3. *Determining the system head versus discharge curves.*

GIVEN: The system layout as shown in Fig. 11.4 and the design discharge and head characteristics determined in Sample Calculation 11.2.

FIND: Draw curves of the *system discharge* versus various pumping *heads* for all three laterals operating and with laterals only at B and C operating, assuming fixed-nozzle sprinklers and no other flow or pressure regulation. Also show the effect of flow (or pressure) regulation after the design system's discharge of Q_s = 900 gpm or 600 gpm has been reached.

CALCULATIONS: At system design specifications when all three laterals are operating, Q_s = 900 gpm. TDH = 190 ft and ΔEl = 33 ft. Therefore by Eq. 11.5c, when the total dynamic pumping head is increased to H_T = 220 ft:

$$Q = 900\left(\frac{220 - 33}{190 - 33}\right)^{0.5} = 982 \text{ gpm}$$

when it is reduced to H_T = 160 ft:

$$Q = 900\left(\frac{160 - 33}{190 - 33}\right)^{0.5} = 809 \text{ gpm}$$

and in a similar manner, the Q for other H_T values is:

$$H_T = 120 \text{ ft}; Q = 670 \text{ gpm}$$
$$H_T = 80 \text{ ft}; Q = 492 \text{ gpm}$$
$$H_T = 40 \text{ ft}; Q = 190 \text{ gpm}$$

and when $H_T = H_e = 33$ ft; $Q = 0$ gpm, according to Eq. 11.5c; however, since the outlet at A is 10 ft lower than C, there would actually be some discharge when $H_T = 33$ ft.

The points connected by the solid line in Fig. 11.5 show a plot of the above data. The vertical line extending above the point representing the design Q_s = 900 gpm at a TDH = 190 ft representing the system characteristics with flow (or pressure) regulation to cancel the effect of excess head on discharge. The lower solid curve (beginning at $H_T = 23$ ft) is an estimate of the actual head-discharge relationship accounting for the fact that water will begin to discharge from the laterals at A and B in Fig. 11.4 before it will from C. These lower portions of the system head-discharge curve are of academic interest only, for the system should be operated somewhere near the design point.

With the lateral at A turned off, the suction assembly plus section P to A

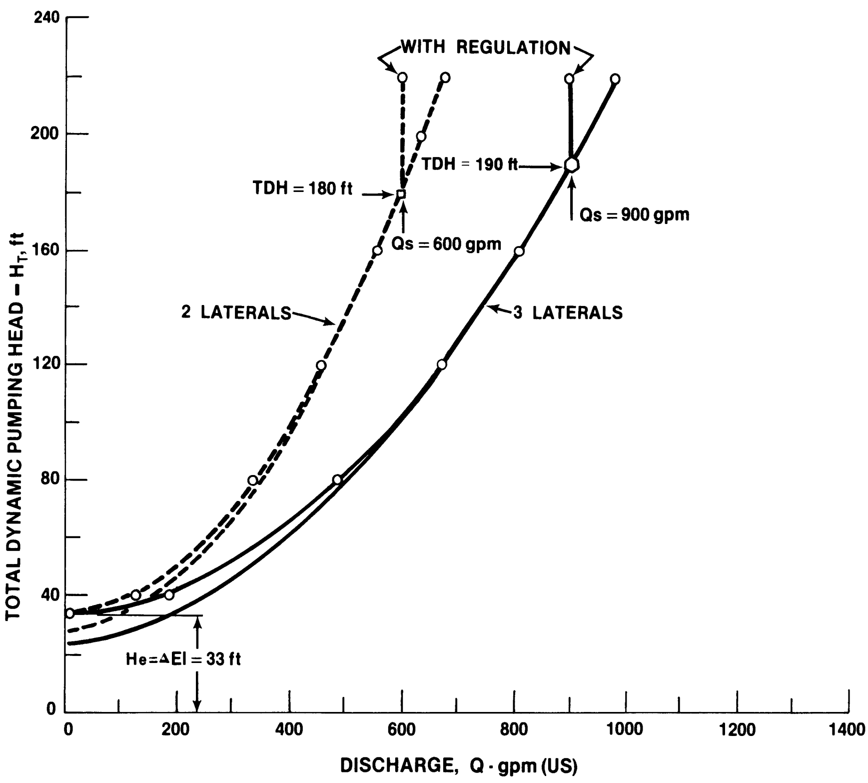


FIG. 11.5. System Head Versus Discharge Curves for both Three and Two (Sprinkler) Laterals in Operation.

friction losses will be reduced by approximately the square of the discharge ratios to give:

$$\left(\frac{600}{900}\right)^2 (3.2 + 9.8 + 2.2) = 6.8$$

Recalculating the TDH for the two lateral operations gives:

Lateral inlet pressure head	$H_l = 115.5$ ft
Total friction head to point A	$h_f = 6.8$ ft
Pipe friction head from A to C	$h_f = 10.8$ ft
Fitting friction head from A through C	$h_f = 8.3$ ft
Static head	$H_e = El = 33.0$ ft
Miscellaneous	$0.2\,(6.8 + 10.8 + 8.3) = 5.2$ ft
	TDH $= 180.0$ ft

With only the laterals at B and C operating when $H_T = 220$ ft, the discharge is:

$$Q = 600\left(\frac{220 - 33}{180 - 33}\right)^{0.5} = 677 \text{ gpm}$$

And in a similar manner, the Q for other H_T values is:

$$H_T = 200 \text{ ft};\ Q = 640 \text{ gpm}$$
$$H_T = 160 \text{ ft};\ Q = 558 \text{ gpm}$$
$$H_T = 120 \text{ ft};\ Q = 462 \text{ gpm}$$
$$H_T = 80 \text{ ft};\ Q = 340 \text{ gpm}$$
$$H_T = 40 \text{ ft};\ Q = 130 \text{ gpm}$$
$$H_T = 33 \text{ ft};\ Q = 0 \text{ gpm}$$

The points connected by the dotted line in Fig. 11.5 show a plot of the above data. The vertical line extending above the point representing the design capacity $Q_s = 600$ gpm, with laterals at B and C operating at TDH = 180 ft represents the system characteristics with flow regulation. The lower dotted curve (beginning at $H_T = 28$ ft) is an estimate of the actual system head-discharge relationships at lower heads.

Sample Calculation 11.4. *Analysis of the hydraulics for a complete sprinkle irrigation system serving a small farm unit.*

GIVEN: A sprinkle system designed for a tomato field, as shown in Fig. 11.6, with:

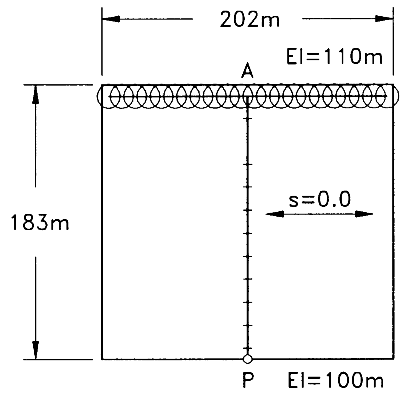


FIG. 11.6. Simple Hand-moved Sprinkle System Layout for a Small Farm.

The root depth, $Z = 1.00$ m;
The soil water-holding capacity, $W_a = 122$ mm/m;
The management allowable deficit, MAD = 40%;
The irrigation efficiency, $E_h = 74\%$;
The consumptive use rate, $U_d = 7.0$ mm/day; and
The sprinkler spacings, $S_e = 9.2$ m and $S_l = 15.25$ m.

FIND: Determine or verify the following:

1. The recommended gross depth of application per irrigation, d;
2. The irrigation frequency, f';
3. Demonstrate that with two 11-hr lateral line sets per day it will take $f = 6$ days to complete an irrigation;
4. Demonstrate that $q_a = 14.0$ L/min and $H_a = 29.1$ m assuming $K_d = 2.595$ for (single-nozzle) sprinklers with 3.6-mm nozzles;
5. Show that the lateral inlet pressure head, H_l, = 31.5 m for 2-in. aluminum pipe with an inside diameter, ID = 48.3 mm and 0.75-m-high sprinkler risers;
6. Assuming $A_f = 1.23$, what size main line is recommended from P to A based on the Universal Economic Pipe-Selection Chart shown in Fig. 8.7; and
7. The elevation at P is 100 m, the elevation of the top lateral position is 110 m, and, assume the minor plus valve friction losses equal 4.0 m. For a 3-in PVC main line (ID = 3.284 in.) between P and A, show that the pump discharge or system inlet pressure head $H_S = 48.5$ m when an extra 20% of the main line, minor, and valve friction losses is allowed for safety.

CALCULATIONS: The following calculations are numbered in the same order as above:

1. Gross depth by combining Eqs. 3.1 and 5.3:

$$d = \frac{Z \times W_a \times \text{MAD}/100}{E_h/100}$$

$$= \frac{1.0 \times 122 \times 40/100}{74/100} = 66 \text{ mm}$$

2. Frequency by combining Eqs. 3.2 and 5.3:

$$f' = \frac{d \times E_h/100}{U_d} = \frac{66 \times 74/100}{7} = 7 \text{ days}$$

3. Operating days to complete an irrigation:

$$\text{The number of lateral sets} = \frac{183 \text{ m}}{15.25 \text{ m}}$$

and with two sets/day, it will take $f = 6$ days.

4. Average sprinkler discharge and head:

$$I = \frac{d}{I_a} = \frac{66 \text{ mm}}{11 \text{ hr}} = 6.0 \text{ mm/hr}$$

$$q_a = \frac{S_e \times S_l \times I}{60} = \frac{9.2 \times 15.25 \times 6.0}{60} = 14.0 \text{ L/min} \tag{5.5}$$

$$H_a = \left(\frac{q_a}{K_d}\right)^2 = \left(\frac{14.0}{2.595}\right)^2 = 29.1 \text{ m} \tag{5.1}$$

5. Lateral inlet pressure head:

$$Q_l = q_a \times \frac{L}{S_e} = 14.0 \times \frac{101}{9.2} = 154 \text{ L/min} = 2.57 \text{ L/s}$$

$$J = 1.212 \times 10^{12} \left(\frac{Q}{C}\right)^{1.852} D^{-4.87}$$

$$= 1.212 \times 10^{12} \left(\frac{2.57}{130}\right)^{1.852} (48.3)^{-4.87} \tag{8.1}$$

$$= 5.33 \text{ m/100 m}$$

or interpolating from Table 8.1, $J = 5.4$ m/100 m, and with $F = 0.40$ from Table 8.7 for 11 outlets:

$$h_f = J \times F \times \frac{L}{100} = 5.4 \times 0.40 \times \frac{101}{100} = 2.2 \text{ m} \qquad (8.8)$$

$$H_l = H_a + 3/4\, h_f + 1/2\, \Delta H_e + H_r \qquad (9.2)$$
$$= 29.1 + 3/4(2.2) + 0.0 + 0.75 = 31.5 \text{ m}$$

6. System inlet pressure head:

$$Q_s = 2 \times Q_l = 2 \times 2.57 \text{ L/s} = 5.14 \text{ L/s} = 81.5 \text{ gpm}$$

Enter Fig. 8.7 with $Q_s' = 1.23 \times 81.5 = 100$ gpm on the vertical axis and $Q = 81.5$ gpm on the horizontal axis to find a point that falls in the 3-in. pipe region.

7. The total pressure head required to operate the system is the sum of the following:

H_l	$= 31.5$ m
$h_{f(\text{P-A})} = J_3 \times \text{L}/100 = 1.03 \times 175/100$	$= 1.8$ m
$\Delta EL = (EL)_\text{A} - (EL)_\text{P} = 110 - 100$	$= 10.0$ m
Minor losses (given)	$= 4.0$ m
Safety, 20% of (1.8 + 4)	$= 1.2$ m
Total system pressure head	$H_S = 48.5$ m

RECOMMENDED READING

Cuenca, R. H. 1989. *Irrigation system design*. Englewood Cliffs, New Jersey: Prentice-Hall.

Finkel, H. J. 1982. Pipe flow. In *CRC Handbook of Irrigation Technology*, ed. H. J. Finkel. vol. I, pp. 171–191. Boca Raton, Florida: CRC Press.

Hydraulic Institute. 1954. Pipe Friction Manual. New York.

King, H. W. and E. F. Brater. 1963. *Handbook of Hydraulics*, 5th ed., New York: McGraw-Hill Book Co., Inc.

McCulloch, A. W., J. Keller, R. M. Sherman, and R. C. Mueller. 1957. Revised by Keller, 1967. *Irrigation Handbook*. Milpitas, California: W. R. Ames Company.

12

Pump and Power Unit Selection

All sprinkle and trickle irrigation systems require energy to move water through the pipe distribution network and discharge it through the sprinklers or emitters. In some instances this energy is provided by gravity as water flows downhill through the delivery system. However, in most irrigation systems, energy is imparted to the water by a pump that in turn receives its energy from either an electric motor or an internal combustion engine. The combination of pump and engine is central to the performance of most irrigation systems. Therefore, it is important that both the pump and the engine be well-suited to satisfy the requirements of the irrigation system.

This chapter is intended to serve as an introduction to centrifugal and turbine pumps. These are the types of pumps most commonly used in sprinkle and trickle irrigation systems. Some important characteristics of centrifugal and turbine pumps and of electric and internal combustion power units are described, as are some of the advantages and drawbacks of various pump and power units.

CENTRIFUGAL PUMPS

Centrifugal pumps operate when water is drawn into the central chamber of a spinning impeller. It is then engaged by vanes that drive the water to the outside of the pump volute case (see Fig. 12.1). This process transforms the kinetic energy of the spinning impeller into the pressure head used to discharge water from sprinklers or emitters located throughout the area being irrigated.

One drawback of centrifugal pumps is that before starting the pump the impeller case and intake pipe must be filled with water, or primed. This is necessary so that the pressure differential needed to draw water into the pump will be created when the pump is turned on. As water flows from the impeller into the delivery system, an area of low pressure is created at the impeller's center. This draws a continuous stream of water from the source into the impeller. Moreover, water enters the pump because of the difference between the atmospheric pressure on the supply and the lower pressure created by the impeller. Thus, a centrifugal pump may be used only in installations where the lift to the pump is considerably less than the equivalent atmospheric pressure head, or no more than about 20 ft. Excessive lifts cause cavitation (vaporization of the water

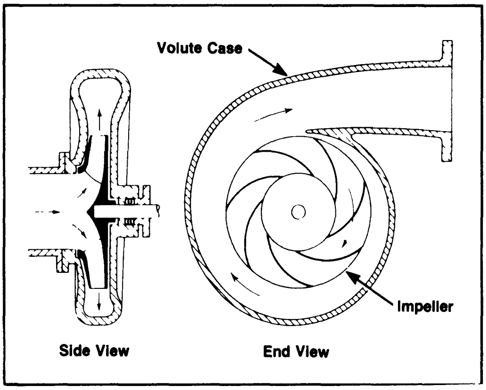


FIG. 12.1. Cross Sections of a Centrifugal Pump.

near where it enters the impeller), which reduces pumping efficiency and may damage the pump.

Suction Lift

A more precise estimate of the maximum vertical distance a centrifugal pump can lift a column of water without cavitating is determined from the net positive suction head (NPSH) required by the pump. At sea level, water at a temperature of 10°C (50°F) will rise 10.2 m (33.6 ft) up a column having a complete vacuum. Pump impellers, however, do not create a perfect vacuum, so that, even under ideal conditions, the potential suction lift of a pump will generally be less than 70% of the theoretical maximum. Further reductions in this potential suction lift are necessary to account for water temperature which affects the vaporization pressure of water, and altitude, which affects the atmospheric pressure, at the pump site. Table 12.1 gives data needed for estimating how both the water temperature and altitude at a site affect suction lift.

The maximum rise of a column of water having a complete vacuum above it is the barometric pressure head minus the vapor pressure of the water. In addition, losses in pipes and fittings on the suction side of the pump, the velocity head of the water at the entrance to the pump, and inclusion of a safety factor further lower the potential suction lift. Sample Calculation 12.1 includes the computation of the maximum suction lift for a typical centrifugal pump installation.

Occasionally cavitation problems occur even when it appears that the NPSH requirements have been met. In centrifugal pump installations the suction conditions are very important. Conditions that can cause the available NPSH to drop below that required and cause cavitation are: extra turbulence in the suction pipe; an introduction of air on the suction side of the pump; or wear between the impeller inlet and the stationary wear ring, which increases circula-

Table 12.1. Vapor pressure of water at different temperatures and barometric pressure heads at different altitudes

Temperature		Vapor pressure		Altitude		Barometric head	
(°C)	(°F)	(m)	(ft)	(m)	(ft)	(m)	(ft)
0.0	32	0.06	0.2	0	0	10.4	34.0
4.4	40	0.09	0.3	150	500	10.2	33.0
10.0	50	0.13	0.4	300	1000	10.0	32.8
15.6	60	0.18	0.6	460	1500	9.8	32.2
21.1	70	0.26	0.8	610	2000	9.6	31.6
26.7	80	0.36	1.2	760	2500	9.5	31.0
32.2	90	0.49	1.6	910	3000	9.3	30.5
37.8	100	0.67	2.2	1220	4000	9.0	29.4
43.3	110	0.90	2.9	1520	5000	8.6	28.3
48.9	120	1.19	3.9	1830	6000	8.3	27.3
54.4	130	1.56	5.1	2130	7000	8.0	26.2
60.0	140	2.03	6.7	2440	8000	7.7	25.2
65.6	150	2.62	8.6	2740	9000	7.4	24.3
71.1	160	3.34	11.0	3050	10000	7.1	23.4

tion and turbulence. The result is reduced pump performance and possible damage to the impeller.

Cavitation damage results from vapor bubbles forming and then collapsing. They form in the vicinity of where the incoming water turns from an axial to a radial direction, which causes an additional pressure drop within the impeller (see Fig. 12.1). The vapor bubbles travel to regions of high pressure where they collapse. As the bubbles collapse they induce small, rapid shock waves that fatigue the impeller and over a length of time can erode its vanes.

Pump Sump and Suction Assembly

To assure good centrifugal pump performance, use the following suction assembly and pump sump design recommendations (see Fig. 12.2):

1. Use a suction assembly pipe diameter large enough to keep the flow velocity $V < 3.3$ m/s (10 ft/s);
2. Use smooth tube bends (no mitered joints);
3. Use a bell-mouth suction inlet or screen with a plate on top, and submerge the inlet by a depth equal to four pipe diameters to discourage vortex formation. If this is not possible, install a floating cover around the suction pipe;
4. Allow at least two pipe diameters for clearance between the suction pipe inlet and the floor and walls of concrete sumps (use three or more pipe diameters' for clearance in earthen sumps);

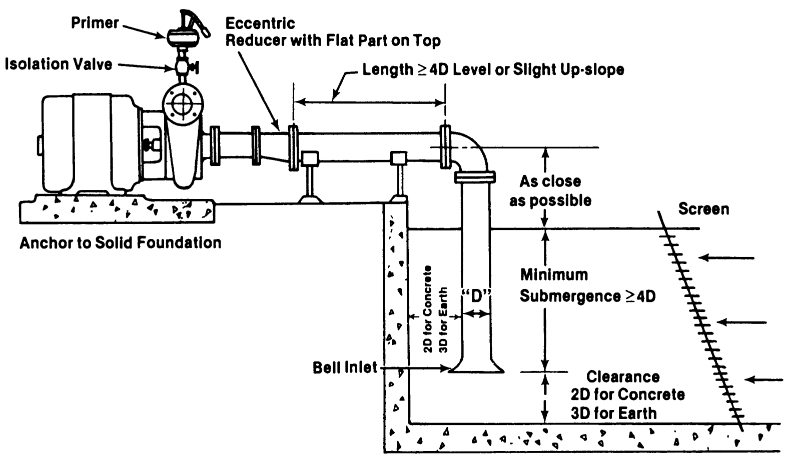


FIG. 12.2. Recommended Guidelines for a Centrifugal Pump Installation Using: a Rigid Suction Assembly With Supports and Airtight Joints; a Smooth Elbow; an Eccentric Reducer; and a Primer on the Discharge, Which Will Require a Check Valve.

5. Have a minimum of six diameters of straight pipe ahead of the pump suction; and
6. Use an eccentric reducer adjacent to the pump inlet, and make sure the pump inlet is as high as or higher than any point in the suction assembly to eliminate air pockets.

Pump Characteristic Curves for Fixed Conditions

In choosing a pump, aim at selecting a unit that will operate at or near peak efficiency while meeting the Q_s and TDH demands of the irrigation system. Pump characteristic curves are a useful tool in this selection process. They show the head and volume range of a given pump, as well as the efficiencies at which the pump operates within this range. Pump curves usually include a power curve showing the horsepower required to drive the pump and should give the required NPSH of the unit. Figure 12.3 is a set of representative characteristic curves for a centrifugal pump with a fixed impeller diameter and speed of operation. The basic information for developing the curves must be obtained from hydraulic laboratory tests.

All the pump characteristic curves are related to the discharge. The efficiency at any given discharge gives the relationship between the useful energy transferred from the pump to the water, *WP*, to the energy input needed to drive the pump, *BP*. The *WP*, or power output, is:

$$WP = \frac{H\,Q}{K} \tag{12.1}$$

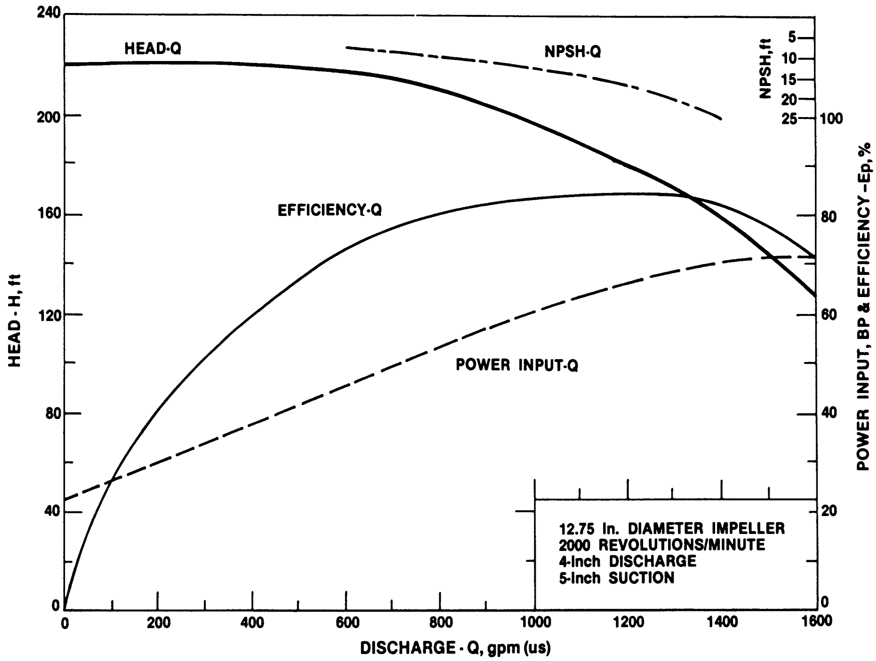


FIG. 12.3 Typical Centrifugal Pump Characteristic Curves.

And the *BP*, or power input, is:

$$BP = \frac{WP}{E_p/100} \tag{12.2a}$$

$$= \frac{H\,Q}{KE_p/100} \tag{12.2b}$$

where

WP = water power output, kW (hp)
K = conversion constant, 102 for metric units (3960 for English units)
H = total operating head, m (ft)
Q = discharge, L/s (gpm)
BP = brake power input, kW (hp)
E_p = pump efficiency, %

The head, efficiency, and power curves are interrelated in accordance with Eq. 12.2b. For example, at 1000 gpm, H = 1197 ft, E_p = 83%, and BP = 60 hp from the curves (see Fig. 12.3). Solving for the *BP* by Eq. 12.2b also gives:

$$BP = \frac{197 \times 1000}{3960 \times 0.83} = 60 \text{ hp}$$

Pump Selection

After the Q_s and TDH have been determined, the next step in the irrigation system design process is to select a suitable pump for the operation. The selection process involves finding an economical pumping plant that will provide the required Q_s and TDH and also operate at a high efficiency. Figure 11.3 shows a schematic of the dynamic suction lift for a pump and the pressure head added (TDH) at the pump. Sample Calculation 12.1 is presented to demonstrate the pump selection process.

Sample Calculation 12.1. *Pump selection, power and suction requirements, and system performance.*

GIVEN: The system shown in Fig. 11.4 with the design Q_s = 900 gpm at TDH = 190 ft and system head-discharge curves presented in Fig. 11.5, based on computations in Sample Calculations 11.2 and 11.3.

The altitude of the site is 5000 ft above sea level and the highest expected water temperature is 60°F.

FIND: Select a pump that will operate at a high efficiency while satisfying the designed Q_s and TDH. Then determine the power required to operate the pump, the operating efficiency; and check the site and design conditions to verify that the NPSH required by the pump will be satisfied.

CALCULATIONS: One centrifugal pump available for use with an engine drive has the characteristic curves shown in Fig. 12.3. Superimposing the system curves Fig. 11.5 on Fig. 12.3 gives Fig. 12.4, which shows that the pump provides a good match for meeting the system requirements. With all *three laterals* operating and no pressure or flow regulation, the system will have a discharge Q = 935 gpm at a head H_T = 202 ft; it will be operating at an efficiency E_p = 83%, requiring a brake power input BP = 58 hp.

From Fig. 12.4 the required NPSH = 11 ft. The available NPSH for the set condition is:

Barometric pressure at 5000 ft (Table 12.1)	28.3 ft
Vapor pressure at 60°F (Table 12.1)	−0.6 ft
Suction assembly friction (Samp. Calc. 11.2)	−3.2 ft
Suction lift (Samp. Calc. 11.2)	−8.0 ft
Velocity head 935 gpm; 5-in. inlet (Eq. 11.2)	−3.6 ft
Available NPSH =	12.9 ft

The required NPSH < available NPSH, i.e., 11.0 < 12.9; therefore, the site and design conditions are satisfactory. (Note that the velocity head is computed using the suction inlet diameter of the pump, not the diameter of the suction pipe, which is usually larger.)

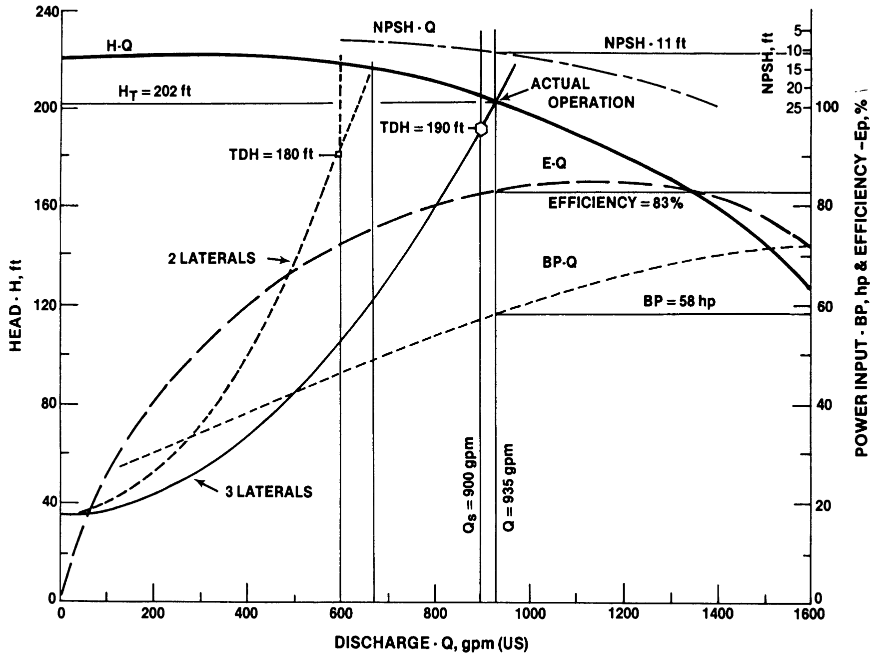


FIG. 12.4 System Head Versus Discharge Curves (Fig. 11.5) Superimposed on Pump Characteristic Curves (Fig. 12.3) for a 12.75-in. Impeller at 2000 rpm.

Automatic or manually controlled flow-regulation valves could be used to hold the system discharge at $Q_s = 900$ gpm. This would require increasing the systems friction losses by 12 ft to $H_T = 202$ ft when $Q = 900$ gpm (see Fig. 12.4). This would decrease the input power to $BP = 57$ hp, but increase the energy per unit of water pumped. For example, without regulation, the BP per 1000 gpm is:

$$\frac{58}{0.935} = 62.0 \text{ hp}/1000 \text{ gpm}$$

and with regulation, it is:

$$\frac{57}{0.900} = 63.3 \text{ hp}/1000 \text{ gpm}$$

The best way to save energy would be to change the speed or diameter of the impeller, so that the pump H-Q curve would pass through the design Q_s and TDH point. Assuming this would not materially change the pump efficiency, the required power input (Eq. 12.2b) would become:

$$BP = \frac{190 \times 900}{3960 \times 0.83} = 52.0 \text{ hp}$$

and the *BP* per 1000 gpm would be:

$$\frac{52.0}{0.900} = 57.8 \text{ hp}/1000 \text{ gpm}$$

The *two lateral* operations can also be analyzed without flow regulation. This will create no problem from the operational standpoint, but will consume considerably more energy per unit of water pumped. For example, without regulation from Fig. 12.4, $Q_s = 670$ gpm, and $BP = 49$ hp; thus, the $BP/1000$ gpm is:

$$\frac{49}{0.670} = 73.1 \text{ hp}/1000 \text{ gpm}$$

and with regulation, $Q_s = 600$ gpm and $BP = 47$ hp; thus, the pumping energy is:

$$\frac{47}{0.600} = 78.3 \text{ hp}/1000 \text{ gpm}$$

This extra energy per unit of pumping results from the combined effects of operating at a lower efficiency, E_p, on the pump curve and the poor match between the pump characteristics and system requirements.

Matching the Pump to the System

The impeller speed or diameter can be adjusted to tailor the selected pumping plant to fit the system design Q_s and TDH. Changing either the speed or the diameter of the impeller changes its rim velocity proportionately. The pump discharge is directly proportional to the impeller rim velocity, assuming the cross-sectional flow area remains constant, and the pressure head is proportional to the square of the rim velocity. Therefore, the effect of impeller speed and/or diameter changes on pump discharge is:

$$\frac{Q_1}{Q_2} = \frac{\text{RPM}_1 \times D_{i1}}{\text{RPM}_2 \times D_{i2}} \tag{12.3}$$

and the effect on the discharge head is:

$$\frac{H_1}{H_2} = \left(\frac{\text{RPM}_1 \times D_{i1}}{\text{RPM}_2 \times D_{i2}}\right)^2 = \left(\frac{Q_1}{Q_2}\right)^2 \tag{12.4}$$

Consequently, assuming the pump efficiency remains nearly constant, the effect on the power input required is proportional to $H \times Q$ (see Eq. 12.2b); thus:

$$\frac{BP_1}{BP_2} = \left(\frac{\text{RPM}_1 \times D_{i1}}{\text{RPM}_2 \times D_{i2}}\right)^3 = \left(\frac{Q_1}{Q_2}\right)^3 \tag{12.5}$$

where

RPM = speed of the impeller in revolutions per minute, rpm
D_i = impeller diameter, mm (in.)
1 and 2 are subscripts denoting different sets of RPM and D_i and corresponding Q, H, and BP values

The above equations, which are sometimes called the *pump affinity or similarity laws*, can be used to predict pump characteristics to extend the information obtained from hydraulic tests.

Sample Calculation 12.2. *Tailoring the selected pump to fit the system design Q_s and TDH.*

GIVEN: The pump characteristic curve Fig. 12.3 and the system design Q_s = 900 gpm at TDH = 190 ft

FIND: Determine the impeller speed that will match the system design with the full 12.75-in. diameter impeller. Also find the trimmed impeller diameter that will match the system design when operated at a speed of 2000 rpm.

CALCULATIONS: Equation 12.4 can be used to construct what is sometimes called an *equal efficiency curve* through the point representing the system design Q_s and TDH. This line will pass through the origin of the pump curve, as shown in Fig. 12.5. However, only the portion of the curve that passes through the design values and intersects the pump *H-Q* curve is of interest. The full curve is presented to show that it is not the same as a system *H-Q* curve except where the static head component of the TDH, $H_e = \Delta El = 0$ and the system has fixed orifices or nozzles.

Following is an example calculation for a point on the equal-efficiency curve. First select an H_T, for example, H_T = 160 ft, then determine the corresponding discharge by Eq. 12.4 as:

$$Q = Q_s \left(\frac{H_T}{\text{TDH}}\right)^{0.5} = 900 \left(\frac{160}{190}\right)^{0.5} = 825 \text{ gpm}$$

Other points plotted on Fig. 12.5 are obtained in a similar manner.

The impeller speed or diameter ratio that will give the desired system performance is the same as the ratio of the Q_s and the Q, where the equal-efficiency

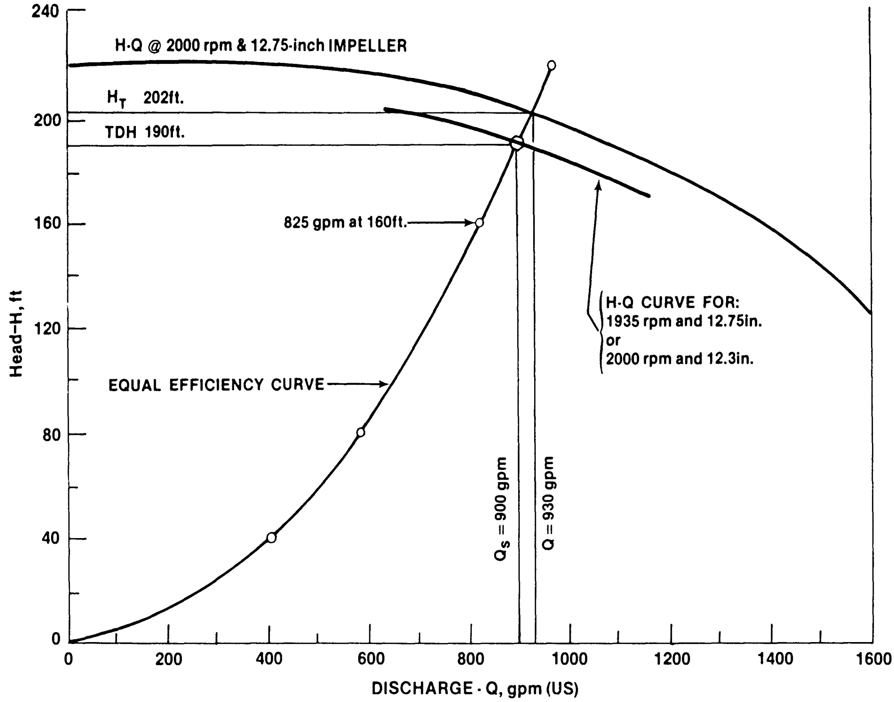


FIG. 12.5. Procedure for Matching Pump Curve from Fig. 12.3 to Fit System Design Q_s = 900 gpm at TDH = 190 ft.

curve intersects the pump *H-Q* curve. From Fig. 12.5, with the diameter held constant in Eq. 12.3, the required impeller speed is:

$$\text{RPM} = \frac{900}{930} \times 2000 = 1935 \text{ rpm}$$

And with the speed held constant at 2000 rpm:

$$D_i = \frac{900}{930} \times 12.75 = 12.3 \text{ in.}$$

These same values can be determined from Fig. 12.5 and Eq. 12.4. For example, working with pressure head values instead of discharges to find the required impeller speed gives:

$$\text{RPM} = \left(\frac{190}{202}\right)^{0.5} \times 2000 = 1940 \text{ rpm}$$

The minor discrepancy between 1935 and 1940 rpm is due to graphical and roundoff errors.

Excess Trimming

The above calculations are acceptable for rather large changes in speed, but must be used with caution when computing impeller trims. This is because changing the impeller diameter alters the internal pump geometry, so that the predicted trimmed diameter is less than that required for efficient pumping. Because of the above, and because it is impractical to correct for overtrimming, the amount of material trimmed from the impeller should be reduced to 80% of the computed value. Therefore, for trimming purposes, where the speed is to remain constant, Eq. 12.3 should be adjusted to give:

$$D_{t1} = \left(0.2 + 0.8\,\frac{Q_1}{Q_2}\right) D_{t2} \tag{12.6}$$

which for Sample Calculation 12.2 gives:

$$D_i = \left(0.2 + 0.8\,\frac{900}{930}\right) 12.75 = 12.4 \text{ in.}$$

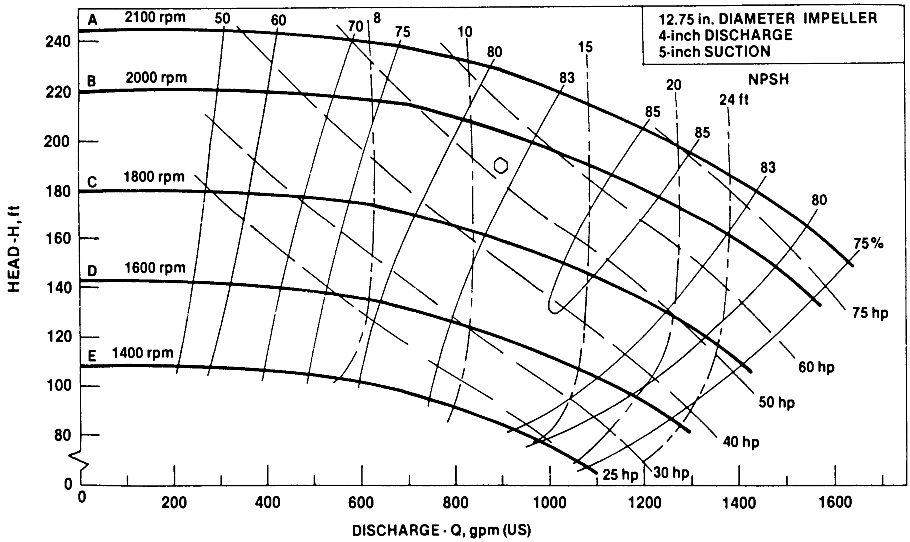
Pump Characteristic Curves for Variable Speed and Trim

A limited quantity of hydraulic test data can be expanded using Eqs. 12.1 through 12.6 to generate families of pump characteristic curves. However, it is important that the expanded data include sufficient test data to assure their validity. Figure 12.6 shows a family of pump characteristic curves for various impeller speeds with efficiency, power input, and NPSH contours. (The *B* curve for 2000 rpm is the same as the *H-Q* curves in Figs. 12.3, 12.4, and 12.5.) With a family of *H-Q* curves, equal efficiency, equal power input and equal NPSH contours can be constructed. This is preferable to having a separate set of (efficiency, power input and NPSH) curves for each *H-Q* curve.

The table of values above the caption in Fig. 12.6 gives combinations of impeller speeds and trim diameters that will produce the *A*, *B*, *C*, *D*, and *E* head-discharge (*H-Q*) curves. Speed and trim values can be interpolated from the adjacent curves to tailor the pump for the desired Q_s and TDH. The design system Q_s = 900 gpm and TDH = 190 ft (see the hexagon in Fig. 12.6) is approximately one-third of the distance from the *B* to the *C H-Q* curve. Thus, for a full-diameter impeller the speed should be approximately $2000 - \frac{1}{3}(200) \simeq 1935$ rpm.

Pumps in Series

Two or more pumps may be connected in series, so that the discharge of one pump (or stage) is piped into the inlet of the next pump. Each pump imparts



H-Q	TRIM inch	SPEED rpm	H-Q	TRIM inch	SPEED rpm	H-Q	TRIM inch	SPEED rpm	H-Q	TRIM inch	SPEED rpm
A	FULL	2100	C	11.1	2100	D	10.2	2100	E	9.6	2000
B	12.2	2100		11.7	2000		10.6	2000		10.4	1800
	FULL	2000		FULL	1800		11.3	1800		11.4	1600
							FULL	1600		FULL	1400

FIG. 12.6. Typical Family of Pump Characteristic Curves for Various Impeller Speeds with Efficiency, Power Input, and NPSH Contours (See Table for H-Q Curve (A, B, C, D or E) to Use with Various Other Impeller Trims and Speeds).

more energy to the same flow of water. This type of installation is used where the same discharge rate is needed, but a larger head is required than can be provided by a single pump (or stage). In an installation with two pumps in series, the second pump is sometimes called a *booster pump*. In some booster pump installations, the second pump may be a considerable distance from the first pump and/or some water may be discharged from the system before entering the second (or booster) pump (see Fig. 10.2).

For two pumps operating in series the combined head is equal to the sum of the individual heads imparted to the flow of water, as indicated in Fig. 12.7. Furthermore, if all the water flows through both of them, the combined efficiency of the pumps in series, $(E_p)_s$, is:

$$(E_p)_s = 100 \frac{Q(H_1 + H_2)}{K(BP_1 + BP_2)} \tag{12.7a}$$

or the combined input power for the pumps in series BP_s is:

$$BP_s = 100 \frac{Q}{K} \left[\frac{H_1}{E_{p1}} + \frac{H_2}{E_{p2}} \right] \tag{12.7b}$$

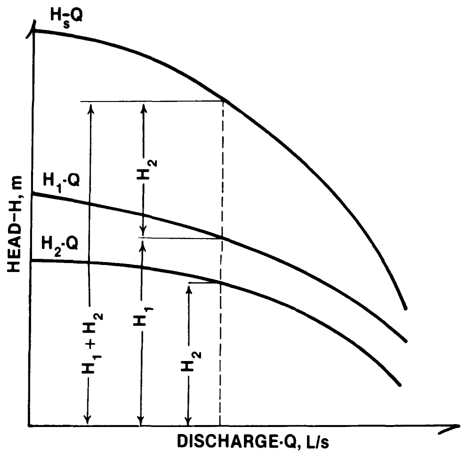


FIG. 12.7. Characteristic *H-Q* Curves for Two Pumps Operating Individually and in Series.

where Q, H, BP, and K have the same definitions and units as in Eq. 12.2; and the subscripts 1 and 2 represent the first and second pumps.

If two pumps have the same design specifications they will have the same characteristic curves, $H_1 = H_2$ and $BP_1 = BP_2$ at any given Q. Therefore, simply double the head, H, and brake power, BP, values on the plot of the pump characteristic curves to obtain the characteristic curves for the pumps in series. For example, if two pumps with the same characteristic curves, as shown in Fig. 12.3, were directly connected in series, they would produce $Q = 1200$ gpm at $H_s = 2 \times 180 = 360$ ft.

If the pumps do not have the same characteristic curves, and/or part of the flow is discharged before entering the second pump, the two H-Q curves and corresponding BP-Q curves must be added graphically, as demonstrated in Fig. 12.7. Furthermore, the combined efficiency $(E_p)_s$ must be computed by Eq. 12.7a for several points and plotted. However, the NPSH-Q curve for the pumps in series is the same as the NPSH-Q curve for the first pump, because the second pump will be operating with excess pressure at its intake. Moreover, the manufacturer of the second pump should be consulted to verify that the extra pressure due to the series operation will not damage the volute case or shaft seals.

Pumps in Parallel

Two or more pumps may be operated in parallel, so that the individual flows of the pumps are discharged into the same pipeline. Typical examples where parallel pumps would be appropriate are: where two or more pumps draw water from the same sump and supply a large pipeline network requiring a relatively constant TDH with varying water demands; where two or more small wells are pumped and their flows are discharged into a common pipeline; or where large volumes of water are required. When water must be supplied to a group of fields with a mixture of crops, the water demand during different times of the year will vary greatly. In such cases, it is practical to use two or more low-volume pumps in parallel rather than one large pump, in order to conserve energy. Moreover, using several smaller pumps provides an element of safety in the event of a pump or engine breakdown.

The combined characteristic curves for pumps in parallel are developed in a similar manner to that described for pumps in series, except the Q rather than the H values must be added (see Fig. 12.8). The combined efficiency for the pumps in parallel, $(E_p)_p$ is:

$$(E_p)_p = 100 \frac{H(Q_1 + Q_2)}{K(BP_1 + BP_2)} \tag{12.8a}$$

or the combined input power for the pumps in parallel, BP_p, is:

$$BP_p = 100 \frac{H}{K}\left[\frac{Q_1}{E_{p1}} + \frac{Q_2}{E_{p2}}\right] \tag{12.8b}$$

where Q, H, BP, and K have the same definitions and units as in Eq. 12.2; and the subscripts 1 and 2 represent the first and second pumps.

If both pumps have the same characteristic curves and are pumping water from the same source, then $Q_1 = Q_2$, and $BP_1 = BP_2$ at any H. Therefore, simply multiply the discharge, Q, and brake power, BP, values on the pump curve plot by 2 to obtain the characteristic curves for the pumps in parallel. For example, when two pumps having the same characteristic curves as in Fig. 12.3 are placed in parallel, the discharge will be $Q_p = 2 \times 1200 = 2400$ gpm against a head $H = 180$ ft.

Where the pumps have different characteristic curves, the characteristic curves

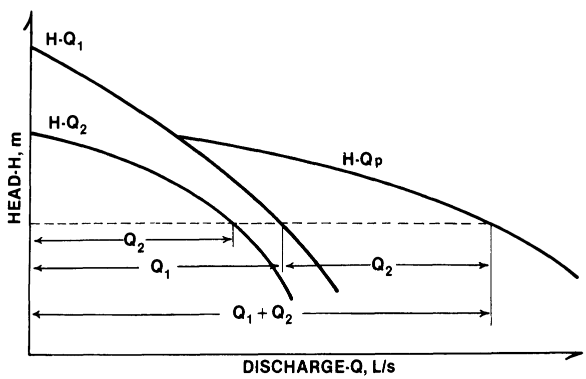


FIG. 12.8. Characteristic H-Q Curves for Two Pumps Operating Individually and in Parallel.

representing the pump in parallel must be determined graphically. Where the pumps are taking water from the same source, they will be discharging against the same pumping head. Therefore, the individual discharges from each pump at any given H can be added, i.e., $Q_1 + Q_2 = Q_p$, and plotted to obtain the H-Q_p curve for the pumps in parallel (see Fig. 12.8).

The BP_p-Q_p curve can be plotted from $BP_1 + BP_2 = BP_p$ and $Q_1 + Q_2 = Q_p$ values for a series of different H values. Data for the $(E_p)_p$-Q_p curve can be determined by Eq. 12.8a. However, the individual NPSH-Q curves will still be needed for each pump.

Where pumps in parallel take water from different sources, each pump may be working against a different pumping head. In such cases the analysis becomes more difficult. If the pumps are not properly selected, they may oppose each other, resulting in fluctuating discharges. One pump may provide most of the water or even pump water backward through the other pump. For example, assume two identical pumps with characteristic curves, as in Fig. 12.3, are pumping into a common pipeline from two different sources. If the difference in elevation between the water levels and the friction losses were such that $TDH_1 \approx 160$ ft and $TDH_2 \approx 220$ ft, then $Q_1 \approx 1400$ gpm and Q_2 might vary from 0 to 600 gpm because the H-Q curve is flat in that range. The exact solution for such problems requires a trial-and-error procedure.

TURBINE PUMPS

A deep-well turbine pump is a specialized multistage application of the centrifugal design. It operates on the same principle as a centrifugal pump except that the housing, called the *bowl case*, is designed to direct water from the discharge of one stage to the inlet of the next stage. The most common application of turbine pumps is in cased wells where the water surface is below the practical limits of suction into a centrifugal pump. However, turbine pumps are often used to deliver water from surface sources to eliminate the problems associated with priming centrifugal pumps. While turbine pumps are generally more expensive to buy and to maintain than centrifugal pumps, they are as efficient and will give long and dependable service if properly cared for.

Turbine pumps are composed of four major assemblies: the pump head, the pump bowls, the impeller drive shaft, and the discharge column (see Fig. 12.9). Power generated at the pump head is transmitted by a shaft down to the pump bowls where the impellers are located. Because of the small diameter of most turbine pumps, it is usually necessary to install a series of impellers in the well to generate the discharge and head required by the system. Water leaving the pump bowls flows up the discharge column and out through the pump head into the irrigation pipe network. Figure 12.9 shows a pump head with a vertical, hollow-shaft electric motor drive. For diesel-engine-powered installations, right-

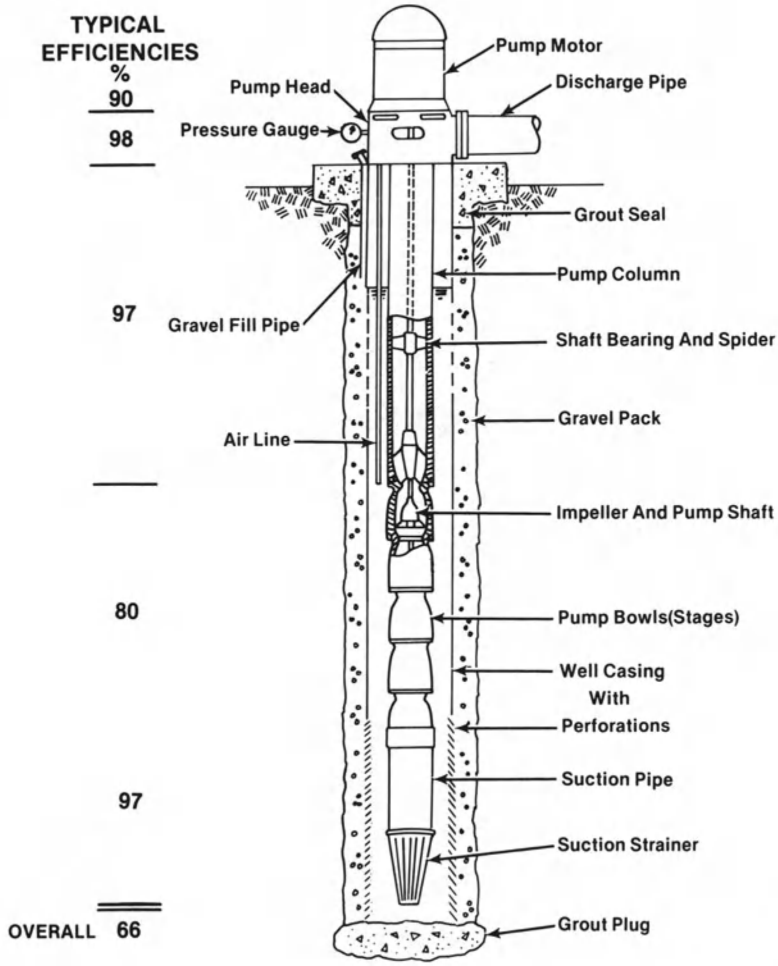


FIG. 12.9. Deep-well Turbine Pump with a Vertical, Hollow-shaft Electric Motor Drive and Water-lubricated Shaft Bearings.

angle gear heads are required to connect the horizontal crankshaft and the vertical impeller shaft (see Fig. 12.10).

Characteristic curves of turbine pumps are similar to those of centrifugal pumps. Figure 12.11 shows a typical set of deep-well turbine pump curves. The shape of the *H-Q* curves is similar to that of a centrifugal pump. Turbine pump selection is considerably more complicated than is selection of a centrifugal pump. Determining how best to fit a pump to a given well and irrigation system requires judgments on: the proper size of pump column; size, type, and number

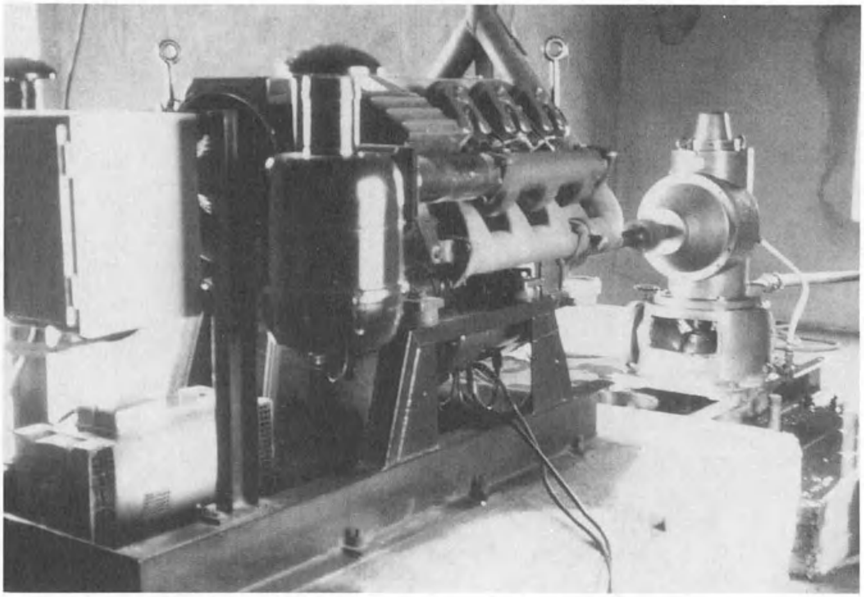


FIG. 12.10. Diesel-engine-driven, Deep-well Turbine Pump Installation.

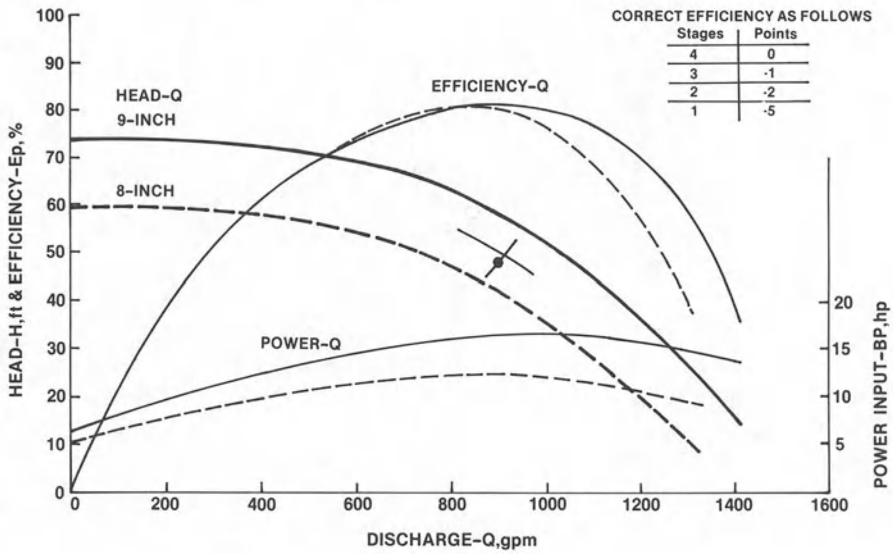


FIG. 12.11. Deep-well Turbine Pump Characteristic Curves for One Stage of a 12-in.-Diameter Bowl Assembly with Impeller Operating at 1760 rpm.

of bowls; impeller shaft diameters and spacing of bearings; and the depth at which the pump will be set.

Shaft Selection

Turbine pump manufacturers should provide guidelines for properly selecting the impeller shaft and column diameters. The recommended shaft diameter is a function of the *BP* (torque) it must transmit and/or the maximum thrust it will be subject to. Table 12.2 gives typical maximum input power, BP_x, and power loss values per 100 ft of shaft length, *PL*, for different diameters and turning speeds for steel pump shafts (Scott and Scolmanini, 1978). The table also gives the maximum thrust mass that each shaft diameter can handle. The shaft diameter should be selected so that neither the BP_x nor the maximum thrust is exceeded.

The actual thrust for a deep-well turbine pump installation can be approximated by:

$$\text{THRUST} \simeq K_H\,\text{TDH} + K_L\text{LS} + K_N\,\text{NS} \qquad (12.9)$$

where

$$K_H = K_h \frac{\pi}{4}(0.23\,D_b)^2, \qquad \text{kg}_\text{f}/\text{m}\ (\text{lb}_\text{f}/\text{ft})$$

in which K_h is a conversion constant equal to 5.3×10^{-3} for metric units (2.3 for English units) and D_b is the nominal diameter of the pump, mm (in.).

$$K_L = K_l d_s^2, \qquad \text{kg}_\text{f}/\text{m}\ (\text{lb}_\text{f}/\text{ft})$$

in which K_l is a conversion constant equal to 2.0×10^{-3} for metric units (2.85 for English units) and d_s is the diameter of the pump shaft, mm (in.).

$$K_N = K_n D_b^3, \qquad \text{kg}_\text{f}\ (\text{lb}_\text{f})$$

in which K_n is a conversion constant equal to 2.76×10^{-7} for metric units (0.01 for English units).

THRUST = thrust for deepwell turbine pump, kg_f (lb_f)
TDH = total dynamic head, m (ft)
LS = length of the pump shaft, m (ft)
NS = number of pump stages or impellers

The head factor, K_H, and the stage factor, K_N, both depend on the design of the impeller, as well as the nominal diameter of the pump. Therefore, K_H and K_N

Table 12.2. Typical values for maximum input power, BP_x, and power loss, PL, hp/1000 ft of shaft for different turbine pump shaft diameters and speeds

Shaft				Shaft turning speed – rpm[1]							
			Max	3450		2900		1760		1460	
Diameter, in.	Mass, lb per 100 ft	Tube diameter, in.	thrust mass, lb_f	BP_x, hp	PL, hp	BP_x, hp	PL, hp	BP_x, hp	PL, hp	BP_x, hp	PL, hp
$\frac{3}{4}$	1.6	$1\frac{1}{4}$	2,000	38	0.7	32	0.6	19	0.4	16	0.3
1	2.8	$1\frac{1}{2}$	3,700	96	1.2	80	1.0	48	0.6	40	0.5
$1\frac{3}{16}$	4.0	2	5,400	163	1.7	135	1.4	82	0.9	67	0.7
$1\frac{7}{16}$	5.8	$2\frac{1}{2}$	7,900	290	2.1	241	1.8	145	1.2	121	1.1
$1\frac{11}{16}$	8.1	3	11,700	530	2.8	440	2.4	265	1.6	220	1.3
$1\frac{15}{16}$	10.6	3	14,700	740	3.6	610	3.1	365	2.0	305	1.7
$2\frac{3}{16}$	13.6	$3\frac{1}{2}$	19,200			900	3.9	545	2.6	455	2.2
$2\frac{7}{16}$	17.0	4	24,400			1290	4.8	790	2.9	645	2.4
$2\frac{11}{16}$	21.0	4	30,000					1060	3.4	890	2.9
$2\frac{15}{16}$	25.0	5	36,200					1400	4.1	1170	3.5
Thrust bearing loss per 10,000 lb of thrust				2.6 hp		2.2 hp		1.3 hp		1.1 hp	

[1] BP_x values shown are absolute maximums assuming total thrust does not exceed the maximum thrust mass values shown and standard steel shafts are used.

NOTE: Enclosed lineshafts should be used for BP_x values below heavy lines. Open or enclosed lineshafts can be used for values above heavy lines. 1.0 in. = 25.4 mm; 1.0 lb = 0.453 kg; 1.0 ft = 0.305 m; 1.0 hp = 0.746 kW.

may be in error by over ±25% and should be used only as first approximations for preliminary planning purposes.

Column Selection

The friction head loss in the pump column is a function of: the diameter of the column pipe and lineshaft; the pump discharge; and the length of column. Table 12.3 gives typical pump head, suction assembly, and column pipe friction losses for various diameters of lineshaft tubes (Scott and Scolmanini, 1978). Standard practice is to limit the friction loss gradient in the pump column to approximately 5 m/100 m (5 ft/100 ft). However, an economic analysis similar to that described for selecting pipe diameters in Chapter 8 gives a more rational basis for selecting the column diameter. Normally, the column, pump head, and suction assembly should all be the same nominal diameter.

Shaft Lubrication

The shafts of deep-well turbine pumps must be supported; thus, shaft bearings and lubrication for them are required. One method of lubrication is to enclose the shaft in a tube filled with oil. Table 12.2 gives typical shaft and corresponding tube diameters for oil-lubricated enclosed shafts. The other method of lubrication is to leave the shaft exposed to the pumped water in the column and use rubberlike water-lubricated bearings (see Fig. 12.9).

When water lubrication is used, if the static water lift is greater than 10 m (33 ft), the bearings should be prelubricated from a special water tank at the surface before starting the pump. The quantity of prelubrication water required is proportional to the diameter and length of the column. A 300-mm (12-in.) column requires approximately 6 L/m (0.5 gal/ft). Thus, for a 150-mm (6-in.) pump column and a static water table 60 m (200 ft) from the ground surface, approximately 180 L (50 gal) of prelubrication water would be required.

Table 12.3 gives column pipe friction losses for enclosed (oil-lubricated) lineshafts. The table can also be used for open (water-lubricated) lineshafts. When using the table for open lineshafts, enter Table 12.3 with the tube diameter from Table 12.2 that would fit the shaft. (The reason the friction loss values are similar is because of the extra bearing-spider supports required for water-lubricated pumps.)

Mechanical Losses

In addition to the hydraulic losses discussed above, the shaft and thrust bearings cause mechanical losses. For example, assume an electric-motor-driven pump has a $1\frac{3}{16}$-in. diameter, 200-ft long shaft operating at 1760 rpm. From Table

Table 12.3. Typical turbine pump friction head losses in the head and suction assemblies and column pipe

Diameter, in.		Discharge – gpm										
Column, head, and suction	Tube	100	200	400	600	800	1000	1500	2000	3000	4000	5000
		Friction losses: head and suction assemblies – ft; column – ft/100 ft										
	$1\frac{1}{4}$	0.17	0.59	2.05	4.20	7.00						
	$1\frac{1}{2}$	0.21	0.73	2.55	5.30	9.00						
6	2	0.28	0.96	3.30	6.80							
	$2\frac{1}{2}$	0.40	1.40	4.75	9.60							
	3	0.65	2.20	7.60								
Head	–			0.30	0.75	1.30	2.00					
Suction	–			1.08	2.33	4.05	6.20					
	$1\frac{1}{4}$				0.64	1.08	1.60	3.37	5.65			
	$1\frac{1}{2}$			0.35	0.73	1.30	1.90	3.90	6.70			
	2			0.61	1.28	2.17	3.22	6.80				
8	$2\frac{1}{2}$			0.74	1.52	2.60	3.90	8.20				
	3		0.29	1.00	2.10	3.60	5.40					
	$3\frac{1}{2}$		0.42	1.47	3.05	5.20	7.80					
	4	0.16	0.57	2.00	4.15	7.00						
Head	–				0.23	0.40	0.65	1.35	2.35			
Suction	–				0.46	0.84	1.31	2.89	5.14			

	1½		0.31	0.51	0.77	1.59	2.69	5.65		
	2		0.33	0.57	0.85	1.80	3.02	6.40		
	2½	0.19	0.38	0.65	0.99	2.06	3.50	7.25		
10	3	0.22	0.45	0.77	1.17	2.47	4.25	8.95		
	3½	0.27	0.56	0.95	1.40	3.00	5.20			
	4	0.35	0.74	1.30	1.90	3.90	6.70			
Head	–				0.24	0.53	0.95	2.00		
Suction	–				0.50	1.12	2.01	5.37		
	2				0.34	0.71	1.20	2.50	4.20	6.40
	2½			0.25	0.38	0.80	1.36	2.87	4.80	7.30
	3			0.29	0.44	0.93	1.57	3.30	5.65	8.55
12	3½			0.33	0.50	1.06	1.80	3.85	6.60	9.90
	4		0.24	0.40	0.60	1.30	2.10	4.55	7.85	
	5		0.34	0.57	0.85	1.80	3.00	6.40		
Head	–					0.25	0.45	1.05	1.60	2.50
Suction	–					0.63	1.10	2.42	4.23	6.58
	2					0.46	0.77	1.59	2.70	4.05
	2½				0.23	0.48	0.81	1.68	2.90	4.35
	3				0.27	0.55	0.92	1.90	3.20	4.80
14	3½				0.29	0.60	1.00	2.10	3.50	5.25
	4			0.21	0.32	0.66	1.10	2.30	3.85	5.80
	5			0.27	0.40	0.82	1.38	2.87	4.85	7.20
Head	–						0.35	0.78	1.30	2.00
Suction	–						0.60	1.31	2.29	3.54

1 in. = 25.4 mm; 1.0 ft = 0.305 m; 1.0 gpm = 0.063 L/s.

12.2, the shaft bearing losses would be approximately $2 \times 0.9 = 1.8$ hp and the thrust bearing loss about 1.3 hp for each 10,000 lb of thrust.

Design Procedure

Sample Calculation 12.3 presents the procedure for designing a turbine pump installation for a surface water supply. The steps in the procedure include: determining the TDH at Q_s; selecting a suitable pump and bowl configuration; trimming the impellers; and determining the power required to operate the pump.

Sample Calculation 12.3. *Turbine pump selection, number of stages, impeller diameters and power requirements.*

GIVEN: The sprinkle system shown in Fig. 11.4 with the design $Q_s = 900$ gpm at TDH = 190 ft, as computed in Sample Calculation 11.2.

FIND: Select and design an electric-motor-driven (1760-rpm), close-coupled turbine pump to replace the centrifugal pump and suction assembly.

CALCULATIONS: From Sample Calculation 11.2, $h_f = 3.2$ ft for the centrifugal pumps' suction assembly, and the power input required is between 50 and 60 hp. From Table 12.2 for 1760 rpm and between 48 and 82 hp, a shaft diameter of $1\frac{3}{16}$-in. with a 2-in. oil-tube is required. From Table 12.3 select an 8-in. column to limit the head-loss gradient to 5 ft/100 ft. Assuming 100 ft of column for computational purposes and noting that the head loss in the column varies approximately as the 1.8 power of the discharge, the column friction loss is:

$$3.22 \times \left(\frac{900}{1000}\right)^{1.8} = 2.7 \text{ ft}$$

Furthermore, assuming an 8-in. discharge head and suction assembly and noting that their head loss varies approximately as the square of the discharge, the pump head plus suction assembly friction loss is:

$$(0.65 + 1.31) \times \left(\frac{900}{1000}\right)^{2} = 1.6 \text{ ft}$$

The TDH for the close-coupled turbine pump can now be computed as:

TDH from Sample Calculation 11.2	190.0 ft.
Centrifugal suction assembly friction	−3.2
Pump column friction	2.7
Pump head and suction assembly function	1.6
TDH =	191.0 ft

Figure 12.11 gives the characteristic curves for an available turbine pump bowl assembly that will discharge the Q_s = 900 gpm at an E_p = 81% when operating at 1760 rpm. From Fig. 12.11 a full diameter 9-in. impeller will produce H = 58 ft at Q = 900 gpm, and when trimmed to an 8-in. diameter, it will produce H = 42 ft at Q = 900 gpm. Thus, a four-stage pump is called for, and the combination of 9- and 8-in. impellers that comes closest to satisfying the system requirements is two of each, giving an H_T = 200 ft at Q = 900 gpm.

The actual system operating point without pressure or flow regulation can be determined by plotting the characteristic curves for the series of stages and using the graphical procedures presented in Sample Calculation 12.1. Furthermore, the required trimming that would produce a more exact fit could be computed as in Sample Calculation 12.2.

A rapid procedure for estimating the actual operating point is to plot the average TDH = 191/4 = 48 ft added per stage at Q = 900, as indicated by the dot in Fig. 12.11. Then sketch a small section of the average *pump H-Q* curve, which will pass through H = 50 at Q = 900. Then sketch a small section of the *system H-Q* curve representing the average H added by each of the four stages which will pass through at Q = 900 at H = 48, as shown in Fig. 12.11. The intersection of these curves gives the actual operating point, which is approximately H = 49.5 ft at an actual Q_s = 910 gpm. The TDH = 49.5 × 4 = 198 ft for the four stages in series.

With an electric drive the speed is fixed; therefore, more precise trimming would be required to obtain a more exact fit. One possibility would be to trim one of the 9-in. impellers to give H = 49 ft at 900 gpm. The required diameter can be estimated from the proportionate distance between the desired operating point and the 8- and 9-in. *H-Q* curves at the desired 900 gpm by:

$$9 - (9 - 8)\,\frac{58 - 49}{58 - 42} = 8.44 = 8\tfrac{7}{16}\text{ in.}$$

In the upper right-hand corner of Fig. 12.11 there is a small table indicating efficiency corrections in percentage points to subtract for various numbers of stages. For example, if there were only two 9-in. stages at Q = 900 gpm, the efficiency would be E_p = 81 − 2 = 79%. However, since we have four stages, E_p = 81 and 79% for the 9- and 8-in., respectively, as no efficiency reduction is necessary. Thus, the input power required using two 8-in. and two 9-in. impellers and no flow regulation by Eq. 12.7b is:

$$BP = \frac{100 \times 910}{3960}\left(\frac{2 \times 58}{81} + \frac{2 \times 41}{79}\right) = 57\text{ hp}$$

or the BP can be estimated directly from Fig. 12.11 as:

$$BP = (2 \times 16.5) + (2 \times 12) = 57\text{ hp}$$

PUMPING FROM WELLS

Historically, well designers have attempted to design wells for maximum production per foot of drilled depth. This condition occurs when the drawdown in the well, $DD = (1 - y)AP$, is equal to one-half the penetration of the saturated sediments (see Fig. 12.12). For this condition, $y = 0.5$. Designing for this condition tends to minimize initial investment, but ignores operating costs. Actually, the most economical well would be designed using life-cycle cost techniques. It would consider, in addition to aquifer characteristics and desired flow rate, the cost of well construction, cost of the pumping unit, and annual energy costs for pumping. When these costs are included in the analysis, a deeper well with more screen length and less drawdown results. Referring to Fig. 12.12, AP and y would both be greater, giving a well with higher initial cost and lower energy cost for the least total annual cost. The optimum well design occurs at discrete values of y and AP.

Design Strategy

A good design strategy is to assume steady-state artesian conditions, uniform and continuous aquifer characteristics, full penetration of the well into the saturated aquifer and a screen length equal to yAP. Full penetration may not ac-

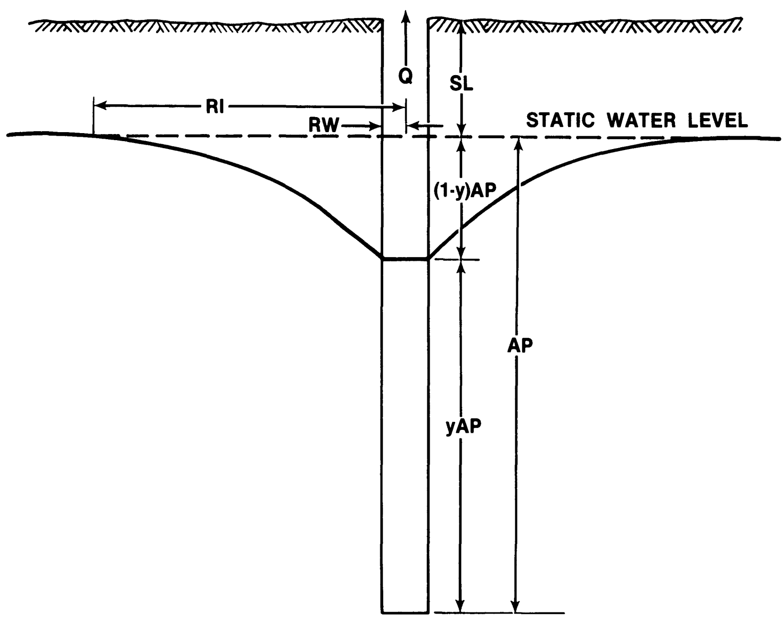


FIG. 12.12. Schematic Diagram of Well Parameters Showing Static and Dynamic Water Levels.

tually be achieved, but this assumption simplifies calculations and results in conservative designs (Driscoll, 1986). For these conditions the well discharge is given by:

$$Q_w = \frac{HC(AP)^2(y - y^2)}{K \log (RI/RW)} \tag{12.10a}$$

where, referring to Fig. 12.12:

Q_w = well discharge, L/s (gpm)
HC = hydraulic conductivity of the aquifer, L/day per m^2 (gal/day per ft^2)
AP = depth of saturated aquifer penetration, m (ft)
y = fraction of aquifer penetration below the dynamic water level that is available for contribution, i.e., yAP = screen length, m (ft)
K = conversion constant, 3.17×10^4 for metric units (528 for English units)
RI = radius of the cone of influence, m (ft)
RW = radius of the well, m (ft)

When Q_w is known, then AP must be solved for.

A well test is required to obtain estimates of HC and RI; however, procedures for doing this and dealing with the various types of boundary conditions is beyond the scope of this text. Typical HC values for aquifers that produce good irrigation wells might range from 4000 to 80,000 L/day per m^2 (100 to 2000 gal/day per ft^2), and RI values range from 100 to 1000 m (330 to 3300 ft). More often than not, irrigation wells are simply installed in accordance with the conventional local wisdom. In any event a well test should at least be conducted to estimate the drawdown at the desired discharge for the system, Q_s.

To demonstrate the principles involved in economic well design, we will assume Q_s, HC, RI, and RW are known and the AP must be solved for. Rearranging the terms in Eq. 12.10a gives:

$$\begin{aligned} AP &= \left(\frac{Q_w K \log (RI/RW)}{HC}\right)^{0.5} \cdot (y - y^2)^{-0.5} \\ &= K_w(y - y^2)^{-0.5} = K_w Y \end{aligned} \tag{12.10b}$$

where K_w is the specific well constant, and Y is the y factor.

The total annual cost of the well is the sum of the annualized well construction and pumping unit costs and the annual power cost. These components must be further divided and defined in terms of their relationship to well depth and pumping lift. With these cost factors defined, determine the total annual cost of owning and operating the well as:

$$\text{Annual cost} = K_1[SL + (1 - y)K_wY] + K_2 + K_3(SL + K_wY) + K_4[SL + (1 - y)K_wY] + K_5[SL + (1 - y)K_wY]$$

where

SL = static lift (see Fig. 12.12), m (ft)
K_1 = annual power cost, with inflation adjustment, per unit of total dynamic lift, \$/m (\$/ft)
K_2 = annual fixed cost for mobilization and cleanup during well constructions, \$
K_3 = annual fixed cost per unit of depth for well construction, \$/m (\$/ft)
K_4 = annual fixed plus maintenance cost per unit length of column, tube, and shaft, \$/m (\$/ft)
K_5 = annual fixed plus maintenance cost per unit of total dynamic lift for pump head and bowls and power plant, \$/m (\$/ft)

The most economic well design is when the total annual cost is minimum. The y value that gives the minimum cost can be found by setting the derivative of the total cost equal to zero and solving for y to obtain:

$$y = \frac{K_1 + K_3 + K_4 + K_5}{K_1 + 2K_3 + K_4 + K_5} \tag{12.11}$$

The optimum well depth can now be obtained by substituting y in Eq. 12.10b, solving for AP, and noting that the total well depth is ($SL + AP$), as shown in Fig. 12.12.

The above procedure appears to give a discrete solution for the optimum well depth. However, the per unit values of K_1, K_3 and K_4 are all subject to some increase with depth, and K_5 may vary with depth. For example, drilling costs, K_3, increase with depth, and well drillers usually increase the per unit depth charge beyond some fixed depth. For greater lifts, more power is required which may require a larger diameter shaft and tube, thus increasing K_4 and K_1. Moreover, K_5 varies with the power required and the relative fit of available pump bowls and power units. Therefore, for more accurate estimates, an alternative solution, which lends itself to computer-assisted design, is required.

Design Procedure

The procedure for optimizing well design plus selecting and designing a suitable pump is presented in Sample Calculation 12.4. The primary steps in the procedure are:

1. Determine the aquifer parameters from test well data, and compute the optimum y value by Eq. 12.11;

2. Compute the optimum aquifer penetration and well depth by Eq. 12.10;
3. Determine the dynamic pumping lift when the well is discharging Q_s;
4. Determine a trial TDH for the complete irrigation system using the turbine pump friction-loss values from Table 12.3 for an assumed column and tube size;
5. Select the pump and number of bowls and impeller configuration required;
6. Check the trial shaft diameter against power and thrust limitations from Table 12.2, and repeat Steps 4 and 5 if a larger shaft (and tube) is required; and
7. Determine the total input power required to drive the pump.

Sample Calculation 12.4. *Well and pump design.*

GIVEN: The sprinkle system shown in Fig. 11.4 with the surface water supply replaced by a well. The $Q_s = 900$ gpm, and the discharge head at the well is $190 - 3.2 = 186.8$ ft. The following were determined from a nearby test well:

Static water level is $SL = 50$ ft;
Hydraulic conductivity of aquifer is $HC \approx 1000$ gal/d per ft^2; and
Radius of influence of well is $RW \approx 1000$ ft.

FIND: Determine the optimum depth for a 2-ft diameter well ($RW = 1.0$ ft) when, by Eq. 12.11, $y = 0.828$. Then plot the drawdown curve and select a suitable set of pump impeller/bowls for supplying the sprinkle system.

CALCULATIONS: The optimum penetration of the well into the aquifer, AP, for supplying $Q_s = Q_w = 900$ gpm may be computed by Eq. 12.10b as:

$$AP = \left[\frac{900 \times 528 \times \log(1000/1.0)}{1000}\right]^{0.5} \times (0.828 - 0.828^2)^{-0.5}$$

$$= 37.76 \times 2.65 = 100 \text{ ft}$$

Because the static lift $SL = 50$ ft, the optimum well depth is $50 + 100 = 150$ ft

The well discharge Q_w at different drawdown levels, DD, can be computed by noting that $DD = (1 - y)AP$ from Fig. 12.12 and substituting various values of y in Eq. 12.10a. For example, for $DD = 30$ ft:

$$y = \frac{AP - DD}{AP} = \frac{100 - 30}{100} = 0.70$$

and by Eq. 12.10a:

$$Q_w = \frac{1000 \times 100^2 \times (0.7 - 0.7^2)}{528 \log(1000/1.0)}$$

$$= 1326 \text{ gpm}$$

Figure 12.13 shows a plot of the expected Q_w at different DD values. Also shown are the relationships among the static water level, the drawdown, and the dynamic pumping lift at various well discharges.

Figure 12.13 includes a sketch depicting the proposed well discharging $Q_s = Q_w = 900$ gpm. The total dynamic pumping lift $DL = 67$ ft can be determined graphically or by noting that for the optimum conditions when $Q_w = 900$ gpm, with $AP = 100$ ft, $y = 0.828$; thus:

$$DD = (1 - 0.828)\ 100 = 17.2 \text{ ft}$$

Therefore, for $SL = 50$ ft, $DL = 67.2$ ft, as shown on Fig. 12.13.

A trial TDH for the irrigation system supplied from the well can be computed using the friction loss values for the 8-in. pump column, head and suction assemblies, as computed in Sample Calculation 12.3 to obtain:

Discharge head	186.8 ft
Pump column friction*	2.7
Pump head and suction assembly friction	1.6
Dynamic lift	67.2
TDH =	258.3 ft

*This is the friction loss for 100 ft of 8-in. column with a shaft diameter of $1\frac{3}{16}$-in. and 2-in. oil tube.

When discharging 900 gpm, a pump assemblage with three 9-in. and two 8-in. impellers having the characteristics depicted in Fig. 12.11 will provide the required TDH as:

$$H = 3 \times 58 + 2 \times 42 = 258 \text{ ft}$$

From Fig. 12.11 the required power input to drive the impellers will be:

$$BP = 3 \times 16.5 + 2 \times 12 = 73.5 \text{ hp}$$

This is less than the maximum power rating of 82 hp given in Table 12.2 for the $1\frac{3}{16}$-in. shaft at 1760 rpm. However, the actual thrust must also be checked against the maximum allowable thrust of 5400 lb for the $1\frac{3}{16}$-in. shaft before finalizing the design. By Eq. 12.9, the actual thrust with the pump assemblage of five 12-in. diameter bowls set 70 ft deep and supplying a TDH = 258 ft is:

$$\begin{aligned} \text{THRUST} &= K_H\,\text{TDH} + K_L\,LS + K_N\,NS \\ &= 13.76 \times 258 + 4.0 \times 70 + 17 \times 5 = 3900 \text{ lb} \end{aligned}$$

Based on the above, the trial design is acceptable. The final step in the design is to determine the total input power required to drive the pump, BP_T. For a direct electric drive the BP_T is the sum of the power losses in the thrust and shaft bearings plus the BP required to drive the impellers. From Table 12.2,

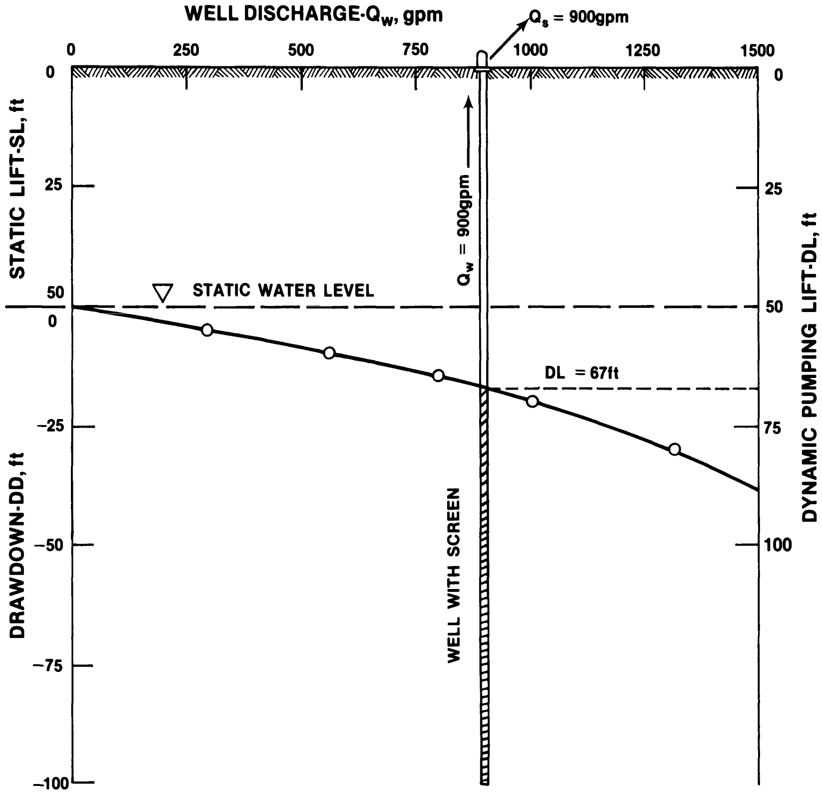


Fig. 12.13. Schematic of Drawdown Curve for the Well Analyzed in Sample Calculation 12.4.

the shaft bearing loss LP = 0.9 hp/100 ft, and the thrust bearing loss is 1.3 hp/10,000 lb; thus, the BP_T is:

$$BP_T = 0.9 \times (70/100) + 1.3 \times (3900/10{,}000) + 73.5 = 74.6 \text{ hp}$$

POWER UNIT SELECTION

After selecting the pump, a power unit must be chosen to drive it. Then the base and housing for the pumping plant can be designed. The goal in selecting a power unit for an irrigation system is to choose one that is reliable and able to drive the pump so it delivers the required Q_s and TDH and is cost-effective. Thus, the performance and expected life-cycle cost of owning and operating the power unit are the two key items to consider. The choice between electric or the various types of internal combustion power units should be based on a life-cycle cost analysis of the available units and sources of energy (see Chapter 8).

Electric Motors

Electric motors offer the advantage of long life (25 years), ease of maintenance, and dependability. Other advantages of electric motors are that they deliver full power throughout their life and are not easily damaged by fluctuations in pump loading. A major consideration in choosing between internal combustion and electric power is the accessibility to and cost of electricity at the pump site. Rate structures for electricity vary widely, so it is important to apply the appropriate electric rates when estimating the cost of installing and operating an electric motor.

When an electric motor is used to drive the pump, the rotation speed of the pump will be fixed. For 60-cycle electricity it is usually approximately 1760 or 3450 rpm. For 50-cycle electricity it is approximately 1465 or 2875 rpm.

Electric motors are capable of continuous operation at their rated horsepower provided they are adequately ventilated (or cooled). Furthermore, they are available for discrete power outputs only, such as 1, 2, 4, 7.5, 15, 30, 60, 75, and 150 kW. However, when they are operating at half or more of their rated output the energy input per unit of power output, kW / hp-hr, is nearly constant.

In view of the above, selecting a suitable motor for a direct or indirectly coupled electric-powered pumping plant is relatively straightforward and simple. The motor merely needs to have a rated power output at least equal to the power required to drive the pump at the necessary or desired speed.

Submersible Pumps. As well as being used in installations powered by motors or engines mounted at the surface, a turbine pump may also be close-coupled to a submersible electric motor to form a unit known as a *submersible pump*. The submersible motor is mounted directly below the bowl assembly. Therefore, the long shaft and bearings between the pump impellers and surface are eliminated, and the motor is not as susceptible to physical abuse.

Internal-Combustion Engines

Internal-combustion engines used in irrigation are generally higher in initial cost and more difficult to maintain than electric motors, but their fuel cost is usually lower. In some instances it is necessary to carefully weigh the advantages and disadvantages of electricity as opposed to internal-combustion fuels before deciding upon a source of power. However, where portability is desired or where providing access to an adequate source of electricity would be prohibitively expensive, internal-combustion engines are the only option.

Selecting an engine to drive a pump is considerably more complicated than selecting a motor. Engine speed can vary, and as it varies, the power-output capability of the engine also varies. Therefore, matching the power requirements of the pump when it is meeting the irrigation system needs involves more complicated procedures for engine selection and design.

Choice of Fuel. A factor to consider when selecting an internal-combustion engine is the choice between compression-ignition (diesel) engines and spark-ignition (natural gas, LPG, gasoline) engines. As is the case with many decisions in irrigation design, this one should also be made on the basis of economics. The choice should be the type of engine that will give the lowest life-cycle cost when the engine and expected fuel and maintenance costs are combined.

Engine Performance Curves. Information on an engine's output and fuel consumption at varying speeds, altitudes, and ambient temperatures should be available from the manufacturer. It is usually presented in the form of performance curves (see Fig. 12.14) that give power ratings and fuel consumptions for a range of engine speeds.

The fuel-consumption curve gives the corresponding fuel consumed per unit of power output, i.e., L/hp-hr, when the engine is operating at the full rated load for that speed. Down to roughly 80% of the rated load, the L/hp-hr curve will be about the same as for a full load. As the relative load is reduced further, the engine efficiency drops so the fuel consumed for each unit of power output becomes considerably greater. Therefore, for best fuel efficiency, engines should be selected so they will be operating close to their *derated* load capacity.

Some manufacturers present performance curves for continuous engine op-

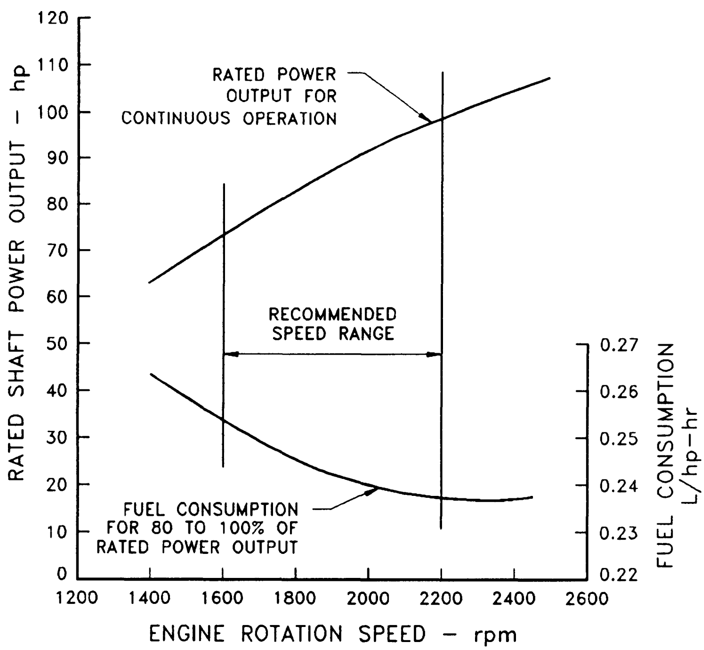


FIG. 12.14. A Diesel Engine's Performance Curves for Continuous Duty (with No Accessories) at Sea Level and 15°C (60°F).

erating with or without the necessary accessories attached, as in Fig. 12.14. However, other manufacturers present performance curves for intermittent engine operation with them stripped of all accessories that sap output power. Such curves need to be derated by 15 to 20% to allow for continuous operation. In addition 2 to 4% of the power output must be subtracted to allow for an air cleaner, generator, and muffler.

Engine Cooling. Engines produce excess heat and they must be kept cool to operate properly. One way to dispose of the excess heat is to use the ambient air to carry it away. This requires circulating the air by fan, which consumes 5 to 10% of the engine's output power to operate. The energy required by the fan depends on the ambient air temperature and either the size of the radiator, for engines designed to use a liquid coolant, or the configuration of the cooling fins (or surfaces), for engines designed to be cooled by air directly. For liquid-cooled engines there is a tradeoff between the higher initial cost of larger radiators and the higher operating cost of smaller radiators.

The most efficient way to cool an engine is to utilize the irrigation water being pumped to carry away the excess heat. This consumes very little of the engine's power output. The installation should be provided with a heat exchanger that isolates the engine's coolant liquid from the irrigation water, rather than allowing it to flow through the engine's liquid cooling system directly. This should be done because if the irrigation water is used directly it is too difficult to keep the engine temperature in the proper operating range. Furthermore, minerals or debris in the water may deposit in and clog the cooling system.

Available Pumping Power. Engine-performance curves are usually reported for sea level operating at an ambient temperature of 15°C (60°F). The density of the ambient air and thus the quantity of oxygen available for the combustion process decrease as either elevation or temperature increases. Therefore, it is necessary to derate engine performance to adjust for the ambient temperature and elevation of the pump installation.

General rules for altitude and temperature derating of naturally aspirated engines are:

1. Reduce continuous load rating by 3% for every 300 m (1,000 ft) above sea level.
2. Reduce continuous load rating by 1% for every 5°C (10°F) above 15°C (60°F).

For engines equipped with turbochargers that pump a greater volume of air into the cylinders, the above values can be reduced to 1% for every 300 m (1,000 ft) and 0.5% for every 5°C (10°F).

After derating for altitude and temperature, remember that some of the power output will be required for the accessories. The air cleaner, generator, and muffler will consume roughly 3% of it, and if a forced-air cooling system is used the fan will consume another 5 to 10% of it as mentioned earlier. Furthermore, for some installations the pump engine is also used to operate a generator (or hydraulic fluid pump) to provide power to rotate a center-pivot system. The generator usually requires 15 hp or more (see Chapter 14) to operate.

Matching Engines with Pumps. Engines are most efficient in terms of fuel consumed per unit of power output (L/hp-hr) when they are operating within the speed range that gives the highest fuel efficiency (see Fig. 12.14). They also should be loaded to near their rated power output as mentioned earlier.

Both the engine and pump will run at the same speed when they are directly coupled. This adds to the complexity of matching them, because both the engine output power (see Fig. 12.14) and the power required by the pump (see Fig. 12.6) depend on the speed of rotation. The power output of an engine increases almost directly with the rotation speed (see Fig. 12.14) while the power required by the pump increases as the cube of the speed (see Eq. 12.5).

In view of the above, the following procedures can be used to select an engine to drive a directly coupled pump that has been selected to accommodate a given irrigation system's Q_s and TDH requirements:

1. Find an engine that appears to have sufficient power when operating at a suitable pump speed for the required Q_s and TDH within its (the engine's) efficient speed range. There is some flexibility here, because the pump impeller can be trimmed to give the necessary Q_s and TDH at different speeds (see Fig. 12.6).
2. Derate the engine's rated continuous-power output versus rotation-speed curve for the site elevation and highest anticipated ambient air temperature surrounding the air filter inlet.
3. Subtract the power required to operate the necessary accessories and auxiliary equipment plus an additional 10% (of the derated curve) from the derated curve to obtain the available power curve. This new curve represents the power available to operate the pump across the acceptable speed range with a 10% safety factor.
4. Check the pump's characteristic curves (see Fig. 12.6) to determine if there is an impeller diameter and speed combination that has a power requirement with $\pm$ 5% of the available power curve from Step 3. If there is such a combination within the pump's efficient operating range that will give the required Q_s and TDH, the power unit is well-suited for driving it.

There is also a graphical procedure for determining the locus of suitable operating ranges for pumps directly coupled to engines (Bliesner and Keller, 1982).

However, when the Q_s and TDH requirements are being considered for only one or two systems, the above procedure is quicker and simpler.

Where the power is indirectly transferred from an engine to a pump through gears or belts, their rotation speeds can be different. The ratio between the speeds can be adjusted by using different gear or belt-pulley diameters. This increases the flexibility of matching engines with pumps. With gears like those in a typical right-angle drive for a turbine pump, there are usually only a few ratios available. However, for belt-pulley drives the available ratios are practically unlimited.

The procedure for selecting an engine to drive an indirectly coupled pump that has been chosen to provide a given system's Q_s and TDH is relatively simple. For driving a pump indirectly the procedure for direct-coupled reduces to:

1. Find an engine that appears to have sufficient power when operating at a suitable speed and within its efficient speed range.
2. Derate the engine's rated power output at that speed for altitude and temperature.
3. Subtract the power required to operate the necessary accessories and auxiliary equipment plus an additional 10% (of the derated power output) from the derated power output at the desired speed.
4. Check whether the required power to drive the pump is within $\pm$ 5% of the value determined in Step 3. If it is, the power unit is well-suited to drive the pump at the designated speed for satisfying the irrigation system's Q_s and TDH requirement.

Sample Calculation 12.5. *Matching an engine with a pump.*

GIVEN: An engine with the performance curve shown in Fig. 12.14 and the information given or developed in Sample Calculation 12.1 in which the: brake power required by the pump, $BP = 58$ hp; pump speed is 2000 rpm; and elevation is 5000 ft above sea level. The maximum temperature expected in the pump house is ~ 100°F and the engine will be air-cooled.

FIND: The suitability of the engine for directly driving the pump, and estimate the expected rate of diesel fuel consumption.

CALCULATIONS: From Fig. 12.14 the rated continuous daily power output of the engine is 92 hp when operating at 2000 rpm, sea level, and 60°F. Therefore, the power output when derated for elevation and temperature will be:

$$92\left[1.0 - 0.03\left(\tfrac{5000}{1000}\right) - 0.01\left(\tfrac{40}{10}\right)\right] = 74.5 \text{ hp}$$

Assuming 3% of this will be required for the accessories and 7.5% will be required to operate the cooling fan, the remaining power to drive the pump is:

$$74.5(1.0 - 0.03 - 0.075) = 66.7 \text{ hp}$$

If a 10% factor for safety is allowed, the remaining power available to drive the pump is:

$$66.7 - (0.01 \times 74.5) = 59.2 \text{ hp}$$

This is within $\pm$ 5% of the 58 hp required to drive the pump, and 2000 rpm is within the engine's efficient operating range. Therefore, the diesel engine should be well-suited for directly driving the pump.

From the fuel-consumption curve in Fig. 12.14, the rate of diesel consumption will be 0.24 L/hp-hr when the engine is over 80% loaded. Therefore, when producing 58 hp to drive the pump plus the power required by its accessories and cooling fan, the engine's diesel fuel consumption rate will be about:

$$0.24\,[58 + 74.5(0.03 + 0.075)] = 16 \text{ L/hp-hr}$$

REFERENCES

Bliesner, R. D., and J. Keller. 1982. *Diesel Powered Pumping for Irrigation.* 2nd ed. Detroit, Michigan: Detroit Diesel Allison Division of General Motors Corp.

Driscoll, F. G. 1986. *Groundwater and Wells,* 2nd ed., pp. 213–214. St. Paul, Minnesota: Johnson Division.

Scott, V. H., and J. C. Scolmanini. 1978. Water wells and pumps: Their design, construction, operation, and maintenance. *California Division of Agricultural Sciences Bulletin* 1889. University of California, Berkeley.

RECOMMENDED READING

Boswell, M. J. 1985. *Micro-Irrigation Design Manual.* El Cajon, California: James Hardie Irrigation.

Cuenca, R. H. 1989. *Irrigation System Design.* Englewood Cliffs, New Jersey: Prentice-Hall.

James, L. G. 1988. *Principles of Farm Irrigation System Design.* New York: Wiley.

Longenbaugh, R. A., and N. R. Dube. 1980. Farm pumps. In *Design and Operation of Farm Irrigation Systems,* ASAE Monograph 3, ed. M. E. Jensen, pp. 347–391. St. Joseph, Michigan: American Society of Agricultural Engineers.

Nir, Z. 1982. Pumps and pumping. In *CRC Handbook of Irrigation Technology,* ed. H. J. Finkel, vol. I, pp. 299–338. Boca Raton, Florida: CRC Press.

13

Traveling Sprinkler System Design

A typical traveling sprinkler system consists of the following major components: pumping plant, main line, flexible hose, traveler unit, and gun sprinkler. Two types of hose-fed traveling sprinkler systems are briefly described in Chapter 4. One drags the hose behind and the other uses the hose to pull the sprinkler.

The *cable-drawn* or *self-propelled* type pulls itself along by winding in a cable as it drags the hose behind itself (see Figs. 2.3 and 4.9). It was developed in the United States, where it is most often used in the more humid areas. With water supplied at the middle of the towpaths, as shown in Fig. 13.1, these cable-drawn travelers can traverse the length of the towpath unattended. The cable must be as long as the towpath and anchored at one end, but the hose needs to be only half as long. Drag-type hoses are usually flexible enough to lay flat when drained. They are constructed with layers of fabric for strength and artificial rubber, which makes them watertight and tough.

The other type of hose-fed traveling sprinkler was developed in Europe and is extensively used there. It is pulled along by winding up the hose. Some of these *hose-drawn* travelers have the hose reel at the water supply end, and others have the reel at the sprinkler end of the hose. In either case, the unit can travel unattended only for a distance equal to the length of the hose. Furthermore, the hose reel must be very large and strong to accommodate the entire length of hose full of water and under high pressure. The pull-type hose is made of thick and relatively rigid plastic that retains its pipelike shape when drained. Detailed descriptions of both types of traveling sprinklers and their operation are presented by Rolland (1982).

Many of the concepts used in designing systems with traveling sprinklers are common to most all sprinkle irrigation systems. The general design procedure, system capacity requirements, depth of application, optimum application rates, and irrigation efficiency criteria are presented in Chapter 5, "General Sprinkler Irrigation Planning Concepts." Information on main line design pressure requirements and the selection of pumping plants is presented in Chapters 10, 11, and 12 for set sprinkler systems. The following sections cover those concepts that are unique to traveling sprinklers. They include system layout, sprinkler and traveler selection, towpath spacing, the relation between travel speed and rate of application, and friction losses in the hose and in the traveler mechanism

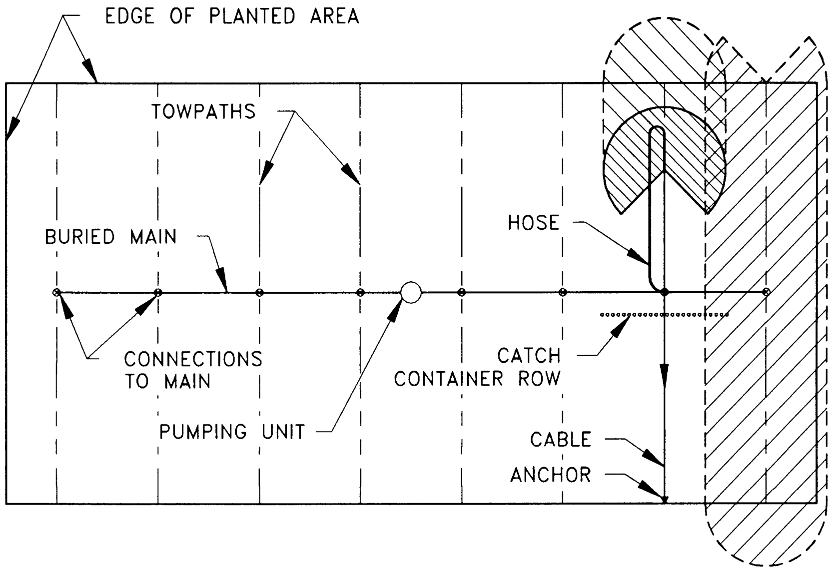


FIG. 13.1. Typical Layout for a Cable-drawn Traveling Sprinkler Showing Location of the Line of Catch Containers Used for Evaluating the Distribution Uniformity.

itself. The design and operating procedures are nearly the same for both the cable-drawn and hose-drawn travelers described above. Therefore, to save time and space, the following discussions will focus on the cable-drawn travelers.

SYSTEM LAYOUT

Figures 4.9 and 13.1 show the layout of a typical cable-drawn traveling sprinkler system, as described in Chapter 4. The following general criteria should be considered when designing such systems:

1. With unrestricted water supplies, it is usually desirable to design the system for at least 20 hr of operation per day during peak-use periods.
2. Traveling sprinklers should normally be designed to require only one or, at most, two setups per day. (Cable-drawn travelers operate unattended until they reach the end of a towpath, at which time the traveler and hose must be moved and set up for a new run in the next towpath, as shown in Fig. 13.1.)
3. At least 1 hr should be allotted for each setup. Thus, the maximum operating time should not exceed 23 hr per day for systems requiring only one setup per day.
4. Cable-drawn travelers are often designed for the traveler to begin and end at the edges of the field, as shown in Fig. 13.1. But this applies only about 50% of the desired application at the outermost ends of the towpaths

and, in addition, leaves a strip equal to the sprinkler's wetted radius with an average deficit of approximately 25%. The quantity of water represented by this deficit is the water wasted beyond the ends of the towpaths (except where the adjacent field is also irrigated by a traveler).

Sometimes it is not advisable or practical to irrigate over the edges of the field at the ends of the towpaths, and the sprinklers must be started and stopped one full wetted radius, R_j, inside the boundaries. Providing sufficient stand time is allowed (as discussed later), this produces tolerable but greater irrigation deficits, as well as overages at the outer reaches of the towpaths. However, it reduces the length of hose required considerably. A compromise that saves on hose and gives practically the same quality of irrigation (as starting and stopping travel at the edges of the field) is to start and stop at points approximately $(2/3)R_j$ from the field edges.

When a cable-drawn traveler is started and/or stopped inside the field edges, it should be allowed to stand for a time at each end of the towpaths. The initial and final standing times should be approximately equal. The total stand time should be equal to the added time that would have been required to travel the full length of the towpath from edge to edge of the field.

Hose-drawn travelers are often designed to start at the outer ends of the towpaths and be pulled inward with the water supply at the center. For such designs the starting point and initial stand-time criteria described above are appropriate. However, the sprinklers should always be pulled to the center of the towpath and shut off immediately for the best uniformity.

5. If practical, where prevailing winds exceed 8 km/hr (5 mph), towpaths should be laid out so they do not line up with the prevailing wind direction. However, towpaths should be laid out in the same direction as the rows, usually following the contours of steeply sloping fields.
6. The actual application rate from full-circle, traveling-gun sprinklers ranges from about 7.5 to 15 mm/hr (0.3 to 0.6 in/hr) for sprinklers discharging 20 to 60 L/s (300 to 1000 gpm), respectively. Therefore, where infiltration is apt to be a problem, it is best to use several low-discharge traveling sprinklers instead of a few large units.
7. The field should be divided into a series of equal strips to obtain a potential set of towpath spacings, provided irrigation beyond the field edges is permitted (see Fig. 13.1). To stay within the outside field edges (parallel to the towpaths), subtract the wetted sprinkler diameter from the length (or width), and divide the remainder into a series of equal strips.
8. The final design layout will be a compromise between the above factors such that: the number of towpaths is an integral multiple of the number of sprinklers; the overlapping between towpaths gives reasonable uni-

formity under the expected wind conditions with the sprinkler nozzle size, angle of trajectory, and pressure selected; and the depth and frequency of irrigation fall within acceptable limits using one or two setups per day.

9. Figure 13.1 shows the traveler moving in opposite directions in adjacent towpaths. This is the easiest way to operate, as it requires less towing of the traveler than would be required for traveling in the same direction all the time. However, it is sometimes best to have the traveler move in the same direction to improve application uniformity and have dry operating strips under certain windy conditions. The best overall uniformity of application is usually achieved when winds affect the sprinkler pattern the same along adjacent towpaths.

SPRINKLER AND TRAVELER SELECTION

Sprinkler characteristics that need to be considered are: nozzle size and type, operating pressure, jet trajectory, and sprinkler body design. The operating conditions that enter into the selection process are: soil infiltration characteristics; desired depth and frequency of irrigation; towpath length; potential towpath spacings and number of paths for each potential spacing; wind conditions; crop characteristics; and the mechanical properties of the soil.

Sprinkler Variables

Gun sprinklers used on most travelers have trajectory angles ranging between 18 and 32°. Higher trajectory angles increase the altitude of the jet, which allows the stream to exhaust its horizontal velocity before the water droplets reach the soil surface. Therefore, the higher angles give maximum coverage in low winds, and droplet impact is minimized. Low angles give more uniform coverage in winds above 16 km/hr (10 mph), but drop impact can be quite severe, especially when operating pressures are low, and may be detrimental to all but the sturdiest crops and coarsest soils. For average conditions, trajectories between 23 and 25° are satisfactory. These midrange trajectories give reasonable uniformity in moderate winds, produce gentle enough drop impacts for use on most crops and soils, and are suitable for operation on varying slope conditions where there will be some riser tilting.

Nozzles. The majority of gun sprinklers used on travelers can be fitted with either tapered or orifice-ring-type nozzles. The tapered nozzles normally produce a compact water jet that is less susceptible to wind distortion than the more diffuse stream from an orifice nozzle. Therefore, for a given discharge the tapered nozzles will also provide a greater distance of throw, which may permit wider spacings between towpaths and lower application rates. However, orifice nozzles produce better stream breakup at lower operating pressures. This in turn produces smaller drops, which is an important factor on delicate crops and unstable soils. Furthermore, the orifice nozzles are relatively inexpensive, and thus offer considerable discharge flexibility at low cost.

Typical nozzle discharges and diameters of coverage are presented in Table 13.1 for gun sprinklers with 24° angles of trajectory and tapered nozzles. The wetted diameter would increase, or decrease, about 1% for each 1° change in trajectory angle. Orifice-type nozzles sized to give similar discharges at the same pressures would produce wetted diameters about 5% smaller than those presented in Table 13.1.

Wetted Sector. Both full- and part-circle gun sprinklers are available in all nozzle types and size ranges. Some traveling sprinklers need to be operated with part-circle coverage to give more uniform water distribution. Another important reason for part-circle operation is to leave a dry path for vehicle travel (as shown in Figs. 4.9 and 13.1) However, the use of part-circle sprinklers increases the application rate. For example, half-circle coverage will double the full-circle application rate of the same sprinkler operating under similar conditions.

Figure 13.2 shows the application depth profile to either side of the towpath for (or across the strip irrigated by) a traveling sprinkler from a single sprinkler pass. The catch containers are perpendicular to the direction of travel, as in Fig. 13.1. The standing sprinkler profile has a hypothetical uniform shape, and the wetted sector angle $\omega = 270°$. Figure 13.3 shows the profile variations produced by wetted patterns with sector angles of 180 (or 360), 210, 240, 270, and 330°.

Computations for these application depth profiles were made easy by using a sprinkler that provided a uniform application rate over the entire wetted ra-

Table 13.1. Typical discharges and wetted diameters for gun sprinklers with 24° angles of trajectory and tapered nozzles operating when there is no wind

Sprinkler pressure, psi	Diameter of tapered nozzle, in.									
	0.8		1.0		1.2		1.4		1.6	
	Sprinkler discharge and wetted diameter									
	gpm	ft	gpm	ft	gpm	ft	gpm	ft	gpm	ft
60	143	285	225	325	330	365	–	–	–	–
70	155	300	245	340	355	380	480	435	–	–
80	165	310	260	355	380	395	515	455	675	480
90	175	320	275	365	405	410	545	470	715	495
100	185	330	290	375	425	420	575	480	755	510
110	195	340	305	385	445	430	605	490	790	520
120	205	350	320	395	465	440	630	500	825	535

10 psi = 68.95 kPa; 1 in. = 25.4 mm; 100 ft = 30.48 m; 100 gpm = 6.31 L/s.

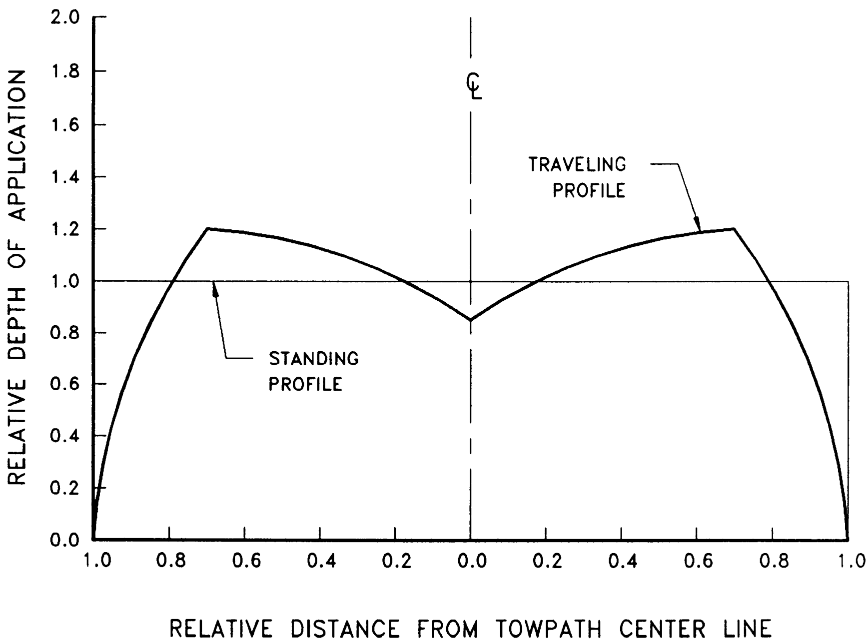


FIG. 13.2 Comparison Between the (Uniform) Application Rate Profile When Standing and the Application Depth Profile a Gun Sprinkler Produces When Traveling with $\omega = 270°$.

dius. This is because the application depth at any catch point is equal to the length of time the point is watered times the uniform sprinkler application rate. The length of watering time will then be equal to the distance (parallel to the towpath) across the wetted sector divided by the travel speed.

Figure 13.3 demonstrates the effect of changing the wetted sector angle. Either full (360° wetted pattern) or half-circle (180° wetted pattern) operation produces the same application depth profiles. The profile will be a semicircle when the maximum depth and distance from the centerline are equal lengths on a dimensionless plot. The reason half- and full-circle operations produce the same application depth profiles across the irrigated strip is because the relative watering times for all the catch points are the same for each. Comparing half-circle with full-circle patterns, the watering times are half as long, but the application rates are double.

The application profiles in Fig. 13.3 all have the same average depth. This is because the uniform full-circle application rate of this sprinkler was multiplied by $360/\omega$. As ω is increased from 180 to 210°, the peak relative depth decreases from approximately 1.3 to 1.1. The most uniform profile is with $\omega = 210°$; however, the profile produced with $\omega = 240°$ is almost as good. It may even be better when considering overlapping application depth profiles, which are produced in adjacent towpaths. The profile for $\omega = 270°$, which is

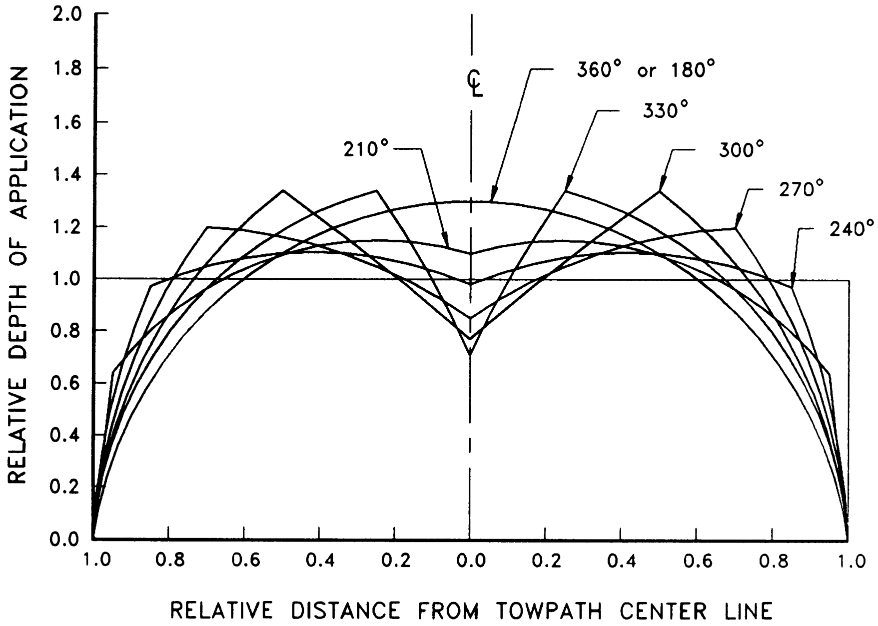


FIG. 13.3. Application Depth Profiles for Wetted Angles Between 180 (or 360) and 330° Produced by a Traveling Part-circle Gun Sprinkler Having a Uniform (Christiansen's *F* Type) Application Rate Profile.

shown in Fig. 13.2, is still fairly uniform; as ω is increased further, the uniformity of the profile becomes poorer.

Sample Calculation 13.1. *Application depth profile computations.*

GIVEN: A part-circle gun sprinkler that produces a uniform application rate over the entire wetted radius, thus giving a uniform standing profile, as shown in Fig. 13.2. The sprinkler is being operated with a wetted sector angle of 330°.

FIND: Determine the relationship between the depth of water collected by a catch container along the centerline of the towpath with the depths in catch containers at 20 and 30% of the wetted radius to each side of the centerline. (See Fig. 13.4 and Fig. 13.1.)

CALCULATIONS: The operating situation is depicted in Fig. 13.4.

Assume the travel speed, $V_t = 1.0R_j$ per hour, and the average application rate for full-circle operation, $I = 10$ mm/hr.

Note that the leading angles to either side of the dry sector in Fig. 3.4 are:

$$90 \frac{360 - 330}{2} = 75^\circ$$

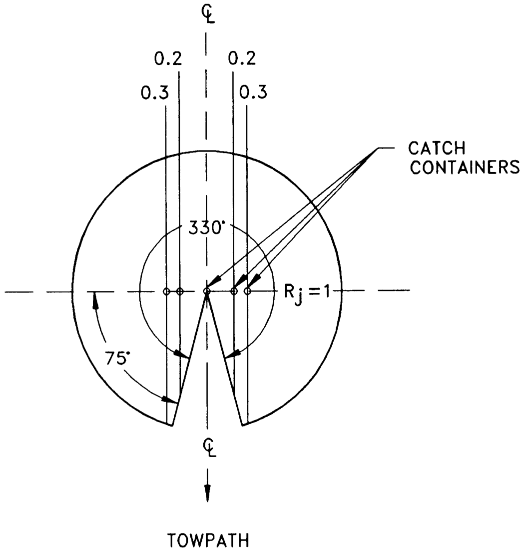


FIG. 13.4. Layout of Catch Containers at 0, 20, and 30% of the Wetted Radius from Towpath Centerline for a Traveling Sprinkler.

and that

$$\cos 75° = 0.26$$

The points at $0.3R_j$ to either side of the towpath will be outside the dry sector, but the points at $0.2R_j$ will fall inside the strip affected by the dry sector.

The application time along the centerline is:

$$T_a = \frac{R_j}{V_t} = \frac{R_j}{1.0R_j \text{ per hour}} = 1.0 \text{ hr}$$

and the depth applied is:

$$d = \frac{360}{\omega} IT_a = \frac{360}{330} \times 10 \times 1.0 = 10.9 \text{ mm}$$

The application time at $0.3R_j$ is:

$$T_a = 2\frac{R_j}{V_t} \cdot \left(1 - (0.3)^2\right)^{0.5} = 1.91 \text{ hr}$$

and the depth applied is:

$$d = \frac{360}{330} \times 10 \times 1.91 = 20.8 \text{ mm}$$

The application time at $0.2R_j$ is:

$$T_a = \frac{R_j}{V_t} \cdot \left[\left(1 - (0.2)^2\right)^{0.5} + \tan 75° \, (0.2) \right]$$

$$= 1.0(0.98 + 0.75) = 1.73 \text{ hr}$$

and the depth applied is:

$$d = \frac{360}{330} \times 10 \times 1.73 = 18.9 \text{ mm}$$

Application Rates. Traveling sprinklers, like periodic-move systems, are usually managed to apply relatively deep irrigations. Furthermore, drop sizes tend to be large, so when Table 5.4 is used as a guide to selecting maximum application rates for traveling sprinklers, the values should be reduced by 25%.

Some irrigators may prefer to begin the irrigation season with small nozzles operating at high pressures to generate smaller droplets during the critical germination or blossom stages. As the season progresses, they will install larger nozzles to increase the discharge in order to meet greater crop demands during the peak moisture-consumption period. At that time, the ground is normally covered with foliage, and larger water droplets are not as apt to adversely affect crop production or soil tilth.

Gun sprinklers tend to produce Christiansen's *D* or *E* type application rate profiles (see Fig. 5.8). Table 13.2 shows the actual application rates for a typical small gun sprinkler operating at a standstill in low winds. These data were used in plotting Fig. 13.5, which is a *D* type profile except for the excess application near the sprinkler caused by the driving arm (see Fig. 4.4).

The average application rate over the entire wetted circle is computed by weighting each radial value by its wetted area and taking the mean weighted value. A simple and efficient way to do this is to multiply each rate value by

Table 13.2. Catch container data from a stationary traveling sprinkler

Item	Locations and application rates									
Radius, m	1	2	3	4	5	6	7	8	9	10
Rate, mm/hr	13.8	13.5	11.0	10.0	13.3	12.6	10.5	8.7	7.3	6.2
Radius, m	11	12	13	14	15	16	17	18	19	20
Rate, mm/hr	6.2	6.2	6.7	7.0	7.0	7.4	7.6	7.4	7.4	6.9
Radius, m	21	22	23	24	25	26	27	28	29	30
Rate, mm/hr	7.4	7.1	7.1	7.0	6.9	7.0	7.1	7.1	6.8	7.1
Radius, m	31	32	33	34	35	36	37	38	39	40
Rate, mm/hr	7.7	7.2	6.4	7.0	6.9	6.4	5.5	3.8	2.1	0.9

1 mm = 0.0394 in.; 1 m = 3.28 ft

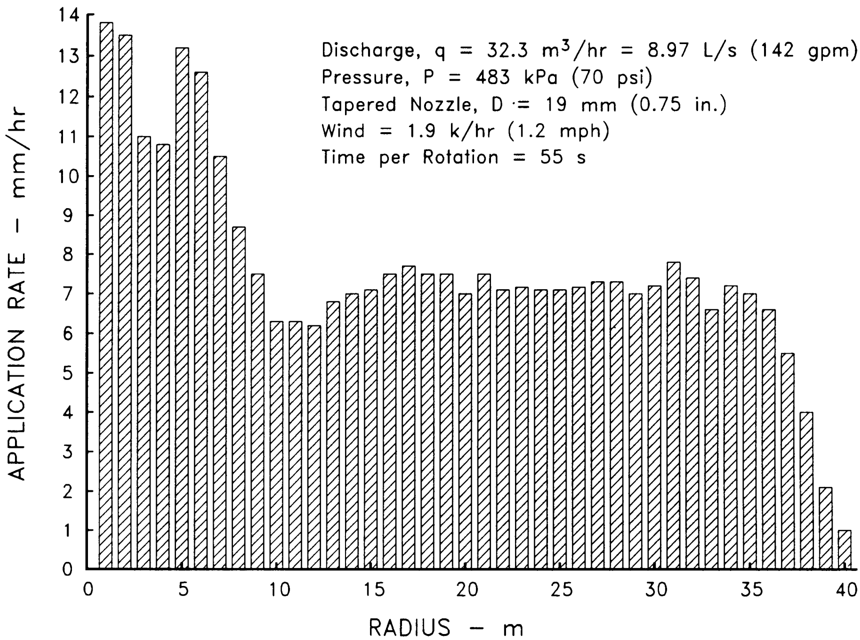


FIG. 13.5. Typical Application Rate Profile of a 24° Trajectory Gun Sprinkler in Very Low Winds (1 mm = 0.039 in.; 1 m = 3.28 ft).

its radius; then divide the sum of the weighted values by the sum of their radii. For the above data this is:

$$\frac{\text{Sum of weighted rates}}{\text{Sum of radii}} = \frac{5268}{820} = 6.42 \text{ mm}$$

Since traveling sprinklers operate independently, the application rate at which water must infiltrate into the soil to eliminate runoff is useful to know. For a full- or part-circle gun sprinkler spaced to give sufficient overlap between tow-paths, this rate is approximately:

$$I_t = \frac{K q}{\pi (0.9 R_j)^2} \cdot \frac{360}{\omega} \tag{13.1}$$

where

I_t = approximate average application rate to be infiltrated under or from a traveling sprinkler, mm/hr (in./hr)
K = conversion constant, 3600 for metric units (96.3 for English units)
q = sprinkler discharge, L/s (gpm)

R_j = wetted radius of sprinkler, m (ft)
ω = portion of circle receiving water (degrees)

This is similar to Eq. 5.6. The wetted area is based on only 90% of the radius of throw to give the approximate application rate over the major portion of the pattern rather than the average rate over the whole wetted area.

Taking data from Fig. 13.5, for full-circle operation:

$$I_t = \frac{3600 \times 8.97}{\pi(0.9 \times 40)^2} \times \frac{360}{360} = 7.9 \text{ mm/hr}$$

This is on the conservative (high) side of the values in the major part of the sprinkler profile. Runoff will not occur if the infiltration capacity is greater than I_t for the maximum application time, T_a.

The average application rate under the profile for the full diameter is:

$$I = \frac{3600 \times 8.97}{\pi\ 40^2} = 6.4 \text{ mm/hr}$$

This is the same as the average weighted catch for the 1.0-hr application test presented in Fig. 13.5, which is unusual because it implies that the profile represents the average and there was no drift or evaporation loss.

Based on data from Table 13.1 in Eq. 13.1, the actual application rates from 20- and 40-mm (0.8- and 1.6-in.) nozzles operating full-circle at 550 kPa (80 psi) are 6.6 and 11.2 mm/hr (0.26 and 0.44 in./hr), respectively. Using orifice nozzles, which would reduce the wetted diameters by about 5%, would increase the application rate to approximately 7.4 and 12.4 mm/hr (0.29 and 0.49 in./hr), respectively. For a tapered nozzle operating with a 75° dry wedge, as in Fig. 13.1, the application rates would be increased to 8.4 and 14.2 mm/hr (0.33 and 0.56 in./hr), respectively.

Traveler Selection

When a hose-fed traveler is selected it must be capable of supporting a gun sprinkler with the required flow rate. It must also have the power to drag the hose and/or sprinkler at the travel speeds necessary to meet the design criteria (see Fig. 13.6). Constant travel speed is required for uniform water distribution over the irrigated area. Therefore, controls that will not allow the speed to vary more than ±10% as the traveler moves from one end of the field to the other and will provide positive shutoff at the end of travel are essential.

Cable-drawn and hose-drawn travelers differ most in their winching and hose-handling mechanisms. Thus, they present different (but compatible) problems related to maintaining a constant speed of travel. Some of the factors that affect

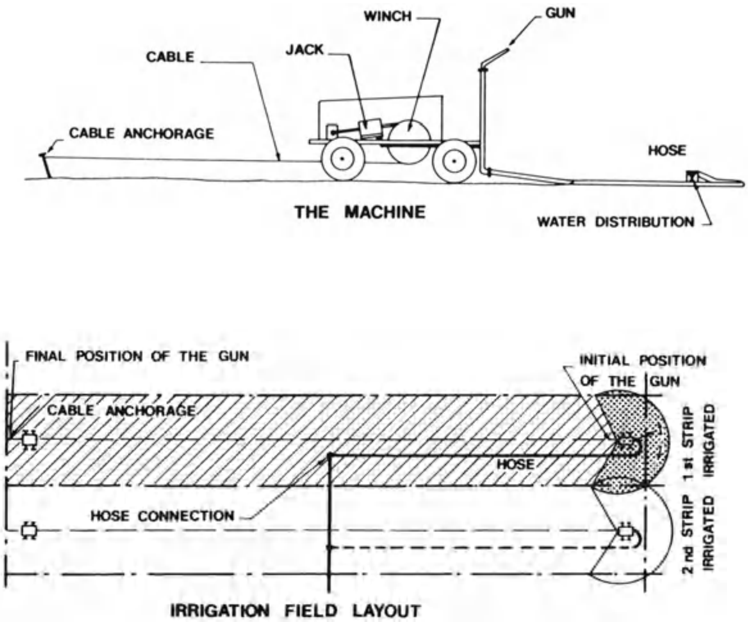


FIG. 13.6. Cable-drawn Traveler and Field Setup (Source: Rolland, 1982 (Fig. 32)).

the ability of a cable-drawn (see Fig. 13.6) traveler to maintain constant speed are:

1. Variations in the drag resistance, which is a function of conditions along the towpath, and the length of trailing hose, which varies from almost none to its full length during a given travel run.
2. Changes in water pressure and flow rate.
3. The continuously increasing diameter of the winch reel as the cable is wound up. This must be compensated for in the design of the winch mechanism, so the machine will not speed up through the travel run.

The characteristics of the power unit on the traveler must be matched to the requirements of hose drag resistance, which may vary by as much as 200 to 300%, depending upon location. Thus, the design and operation of the traveler must include the capability to handle a wide range of conditions.

The end pull required to drag a given length of hose depends on the soil texture, soil moisture conditions, and crop cover. Drag is greatest on wet-bare-sticky soils and least on wet vegetation or on bare-sandy soils. For sticky soils, the towpaths should be left in grass or other vegetation.

Sprinkler performance is adversely affected by turbulence created within the traveler mechanism. Such turbulence can be caused in the internal plumbing by protrusions inside the pipe, changes in pipe size, sharp elbows, and obstacles near the base of the gun sprinkler. Thus, the smoothness of the plumbing should be considered in the selection of a traveler.

When moving a lay-flat, drag-type hose from one location to the next, a hose reel should be used. The mechanism should be designed to squeeze the water out of the hose and flatten it as it is reeled in. It should also be designed so the hose may be placed on it without first removing the pull coupler. Such a reel provides a good means of storing the hose in the off-season, and it provides a better method of moving the hose than dragging it from one field to another.

The differences in winching and handling the more rigid hoses used with hose-drawn, compared with the cable-drawn, travelers discussed above are obvious. They depend upon whether the reel is at the inlet (fixed) or sprinkler (moving) end of the hose (see Figs. 13.7 and 13.8). But there are similar problems with maintaining constant travel speeds.

TOWPATH SPACING[1]

The application uniformity of traveling sprinklers is affected by: wind velocity and direction; jet trajectory, nozzle type, and wetted sector angle; sprinkler profile characteristics and overlap; and variations in operating pressure and travel speed (Shull and Dylla, 1976; and Collier and Rochester, 1980). For wind speeds near 16 km/hr (10 mph), typical CUs reach only 70 to 75% in the central portions of fields when recommended towpath spacings are used. Obviously, these values would decrease proportional to travel-speed variations from one part of the field to another.

The continuous movement of the traveler is equivalent to having periodic-move sprinklers very closely spaced along the lateral. The effect is to improve the uniformity compared with periodic-move gun sprinkler installations. Thus, for a traveler, the expected application uniformity in the central portion of the field should be considerably higher than the uniformity for periodic-move gun or boom sprinklers. Figure 13.9 shows the application profile measured across the towpath for a traveling gun sprinkler, derived from the standing application rate profile shown in Fig. 13.5. From this figure, it is evident that a towpath spacing of 80 to 90% of the wetted diameter will produce excellent uniformity under very calm wind conditions, and closer spacings would produce excessive application midway between adjacent towpaths.

Figure 13.10 shows the application depth profile across a 100-m (330-ft) irrigated strip between two adjacent towpaths. The plot is based on field data from a single pass of a sprinkler traveling 0.305 m/min (1.0 ft/min) and discharging 31.56 L/s (500 gpm) while operating at 690 kPa (100 psi). Catch containers were laid out as in Fig. 13.1. There was a crosswind of 2.2 to 4.5

[1]This section was developed in 1986 with the assistance of Prof. Dov Nir, while on sabbatical at the Agric. and Irrig. Eng. Dept., Utah State Univ., Logan, UT.

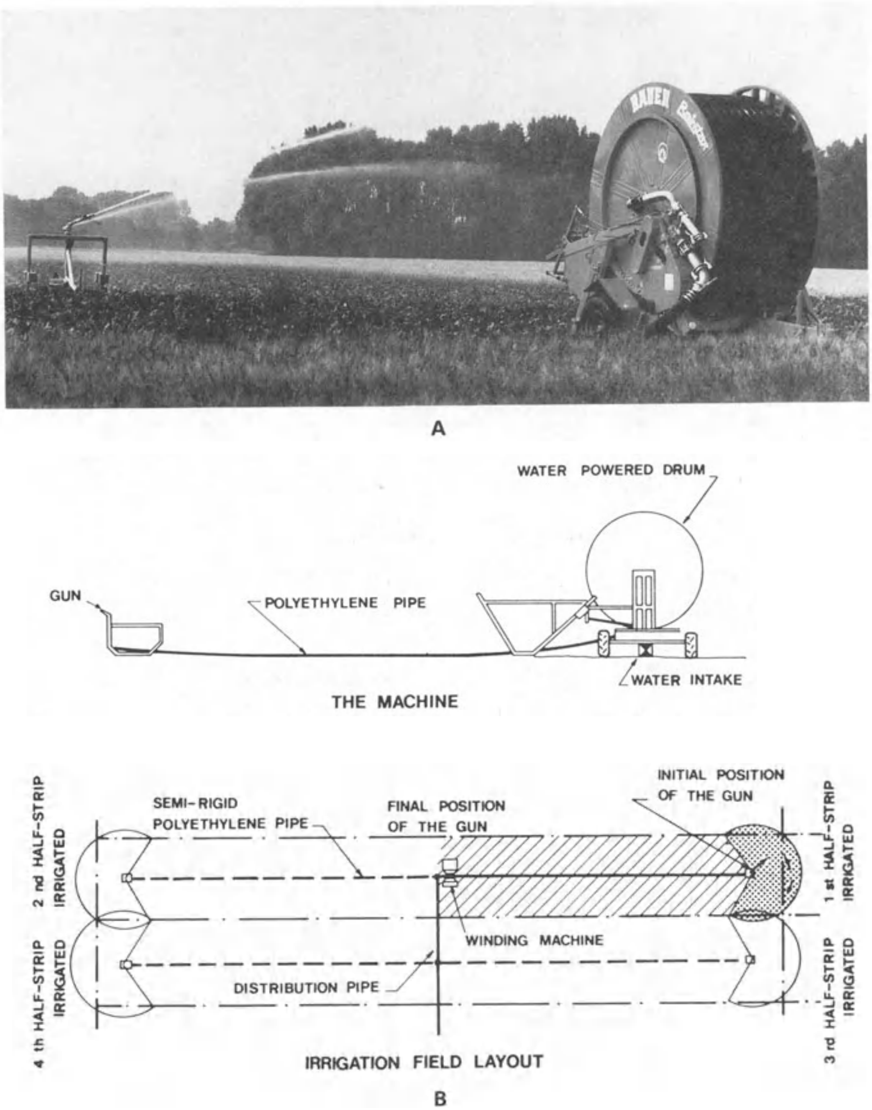


FIG. 13.7. Hose-drawn Traveler (with Reel at the Inlet End). A. Photograph of Operating System (Source: Valmont Industries Inc.). B. Schematics of the Machine and Field Layout (Source: Rolland, 1982 (Fig. 33)).

m/s (5 to 10 mph), and the wetted sector angle was set at $\omega = 330°$. The data from the single pass was overlapped to simulate the results from an identical traveler operating along an adjacent towpath.

Some interesting comments about Fig. 13.10 and the information and data contained therein are:

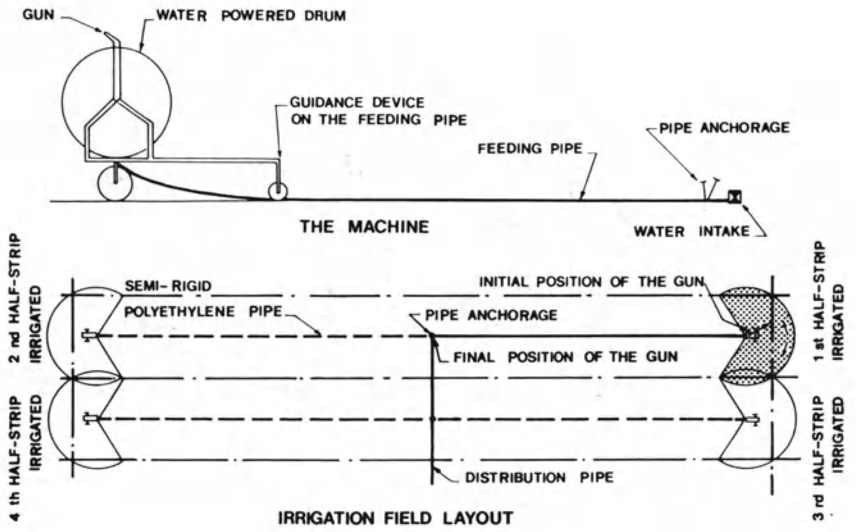


FIG. 13.8. Hose-drawn Traveler (with Reel at the Sprinkler End) and Field Setup (Source: Rolland, 1982 (Fig. 34)).

1. Water reached 76 m (250 ft) in the downwind direction from the sprinkler. But it reached only 49 m (160 ft) in the upwind direction, even though the winds were relatively light and at approximately 45° to the direction of travel (see the wind arrow in Fig. 13.10)

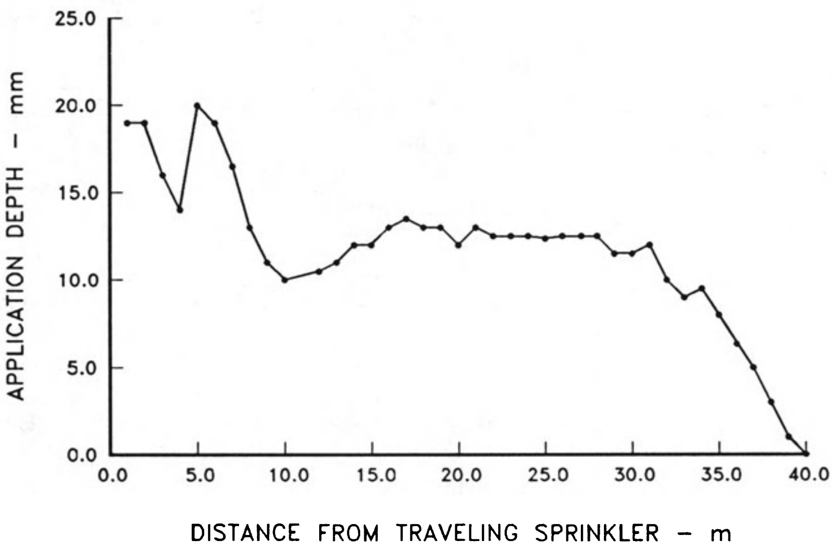


FIG. 13.9. Application Depth Profile Produced by a Traveling Gun Sprinkler with the Stationary Application Rate Profile Shown in Fig. 13.5 when ω = 270°, and V_t = 40 m/hr.

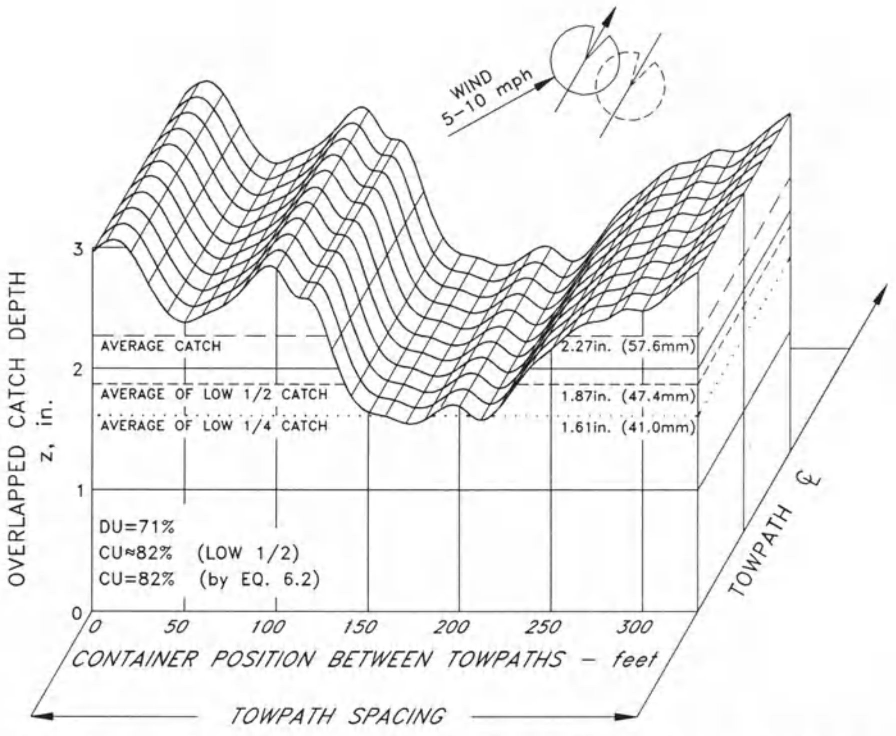


FIG. 13.10. Application Depth Profile Between Towpaths Based on Field-Test Data from a Traveling Sprinkler.

2. The overlapped (or sum of the) catch depths representing traveling along right and left towpaths is minimum midway between the towpaths. This could have been corrected in part by decreasing the wetted sector angle by 90° (see Fig. 13.3 for $\omega = 240°$); however, the higher effective application rate may be a problem.
3. In spite of the way the application depth profile across the wetted strip between towpaths appears, the CU = 82% is fairly good. In part this is because the application depths are assumed to be uniform in the direction of travel (as depicted in Fig. 13.10).
4. The average of the low half of the catch data values divided by the mean catch ($1.87/2.27 = 0.82$) expressed as a percentage is the same as the CU = 82% computed by Eq. 6.2.

Table 13.3 gives recommended towpath spacings for travelers with gun sprinklers having 23 to 25° trajectory angles. The spacings are given as a function of the diameter wetted by the sprinklers and average wind velocities anticipated. By using these spacings, full coverage midway between towpaths is essentially assured. The higher values given for each wind range should be used for tapered

Table 13.3. Recommended towpath spacings for traveling gun sprinklers under various wind conditions

Sprinkler wetted diameter, ft	Wind speed ranges, mph						
	Over 10		5 to 10		2 to 5		0 to 2
	Spacing as a percentage of wetted diameter[1]						
	50	55	60	65	70	75	80
	Towpath spacing, ft[1]						
200	100	110	120	130	140	150	160
250	125	137	150	162	175	187	200
300	150	165	180	195	210	225	240
350	175	192	210	227	245	262	280
400	200	220	240	260	280	300	320
450	225	248	270	292	315	338	360
500	250	275	300	325	350	375	400
550	275	302	330	358	385	412	440
600	300	330	360	390	420	-	-

100 ft = 30.48 m; 1 mph = 1.61 km/hr = 0.447 m/s.
[1]Use lower values for ring-type nozzles and higher values for tapered nozzles.

nozzles and the lower values for ring nozzles. Where average winds are expected to exceed 16 km/hr (10 mph), sprinklers with jet trajectory angles of 20 to 21° should be used. Where winds are negligible, 26 to 28° trajectories will give the best results.

APPLICATION RELATIONSHIPS AND TRAVEL SPEED

The rate of application is unaffected by travel speed, but the depth of application is a function of speed. This makes optimizing a traveler system design somewhat of a trial-and-error process in order to minimize the flow rate and downtime. To optimize the design the travel speed must be selected to satisfy two interrelated functions. The speed should be selected so that the design application depth is applied and the total travel time to traverse the set of towpaths is almost as long as the irrigation interval. This would be difficult without an efficient design strategy, because the design depth, irrigation interval, gun sprinkler discharge rate, towpath length, and towpath spacing are all interrelated.

Application Depth

The average gross depth of water applied per irrigation by a traveling sprinkler can be computed by:

$$d = K \frac{q}{V_t W} \tag{13.2}$$

where

d = gross depth of application, mm (in.)
K = conversion constant, 60 for metric units (1.604 for English units)
q = sprinkler discharge, L/s (gpm)
W = towpath spacing, m (ft)
V_t = travel speed, m/min (ft/min)

During the design process, Eq. 13.2 is also rearranged to compute q, given d and V_t, and to compute V_t, given d and q, as will be demonstrated in Sample Calculation 13.2.

Water-application efficiencies for traveling gun sprinklers are relatively low. This is true even in the central portions of the field where the overlap and performance are best. Typical ranges of application uniformities and efficiencies for well-designed traveler systems are:

For very low winds of 0 to 8 km/hr (5 mph), CU $\approx$ 82%, and $E_h \approx$ 77%; and

For moderate winds approaching 16 km/hr (10 mph) CU $\approx$ 70%, and $E_h \approx$ 65%.

The equivalent range for E_q values is from 67 to 55%. However, the E_h values are more typically used in Eq. 5.3 (than E_q) to compute the irrigation depth, d_n, from d, or vice versa, since travelers are most often used for supplemental irrigation and/or on lower value field and forage crops.

Travel Speed

Travel speed, V_t, should be set so the traveler moves the length of the towpath in approximately 23 hr for one setup per day (or 11 hr for two setups per day). This will allow 1 hr to set up the traveler between runs while maintaining the same daily schedule. It will also minimize downtime, providing the number of towpaths is equal to the irrigation interval for one setup per day (or twice the interval for two setups per day). For example, where the traveler starts and stops at the field boundaries, as shown in Fig. 13.1, the travel speed, V_t, for one setup per day on 400-m (1320-ft) long towpaths should be approximately $V_t = 400/(23 \times 60) \approx 0.3$ m/min (1.0 ft/min).

Standing Positions and Times

When the traveler is started (and stopped) inside the field edges, the travel speed should be the same as for traveling the full length of the towpath. But an appropriate standing time, during which the sprinkler is operating but not traveling, should be allowed at each end of the towpath. The sum of the standing times should be equal to the time it would have taken to travel the extra distance to the ends of the towpath. The strategy used for computing beginning and ending standing positions and times depends on whether the traveler moves from end to end (see Fig. 13.6) or from the ends to the center (see Fig. 13.7 or 13.8) of the towpaths. It also depends on the angle wetted, ω, which may be anywhere from $180 \geq \omega \geq 360°$, and is different for isolated than adjacent traveler-irrigated fields. First, the strategies for both end-to-end and end-to-center operations on isolated fields will be considered.

End-to-End. Cable-drawn travelers are usually operated continuously for the full length of the towpath in the same direction. The distance from the initial starting point to the field edge should be approximately:

$$ID_e = (2/3)R_j \tag{13.3}$$

and the distance from the final stopping point to the opposite edge of the field should be approximately:

$$FD_e = (2/3) \cdot \left(1 - \frac{360 - \omega}{180}\right)R_j = (2/3) \cdot \left(\frac{\omega - 180}{180}\right)R_j \tag{13.4}$$

where

ID_e = initial or starting point distance from end of towpath for end-to-end operation, m (ft)
FD_e = final or stopping point distance to end of towpath for end-to-end operation, m (ft)
ω = portion of circle receiving water, $180° \geq \omega \leq 360°$
R_j = wetted radius of sprinkler, m (ft)

Figure 13.11 shows a layout of the initial (starting) and final (stopping) positions for two setups of a traveling sprinkler in which the towpath spacing $W = 1.5R_j$ and $ID_e = (2/3)R_j$. The sprinkler patterns cross at the field's edge nearest to where the traveler begins its journey for each setup as indicated by point X, because:

$$\left(R_j^2 - (0.75R_j)^2\right)^{0.5} = 0.66R_j \simeq (2/3)R_j$$

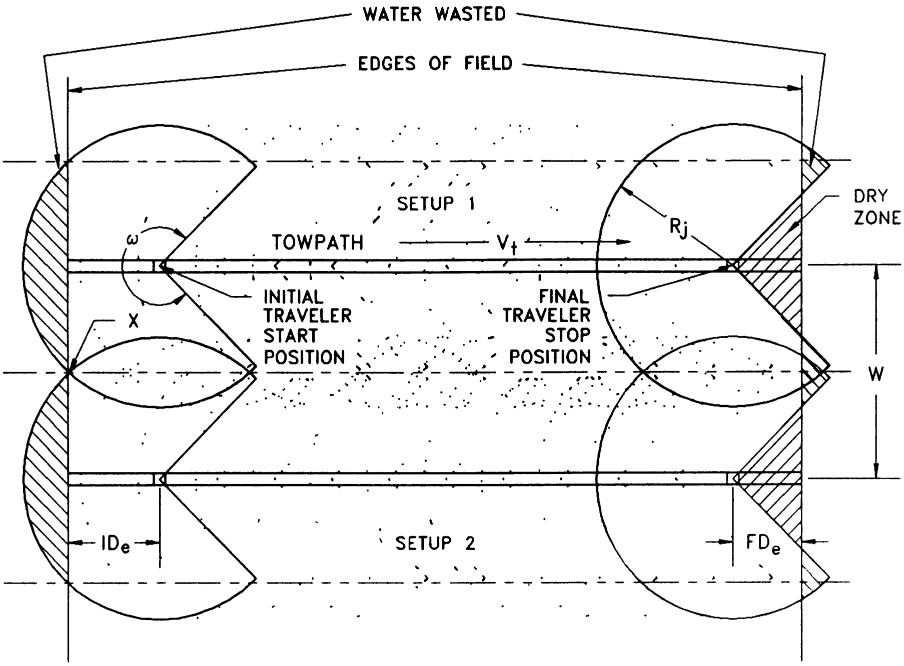


FIG. 13.11. Plan View Showing Watering Patterns for Adjacent Setups of a Cable-drawn Traveler and Terminology Used in Defining Initial and Final Traveler Positions.

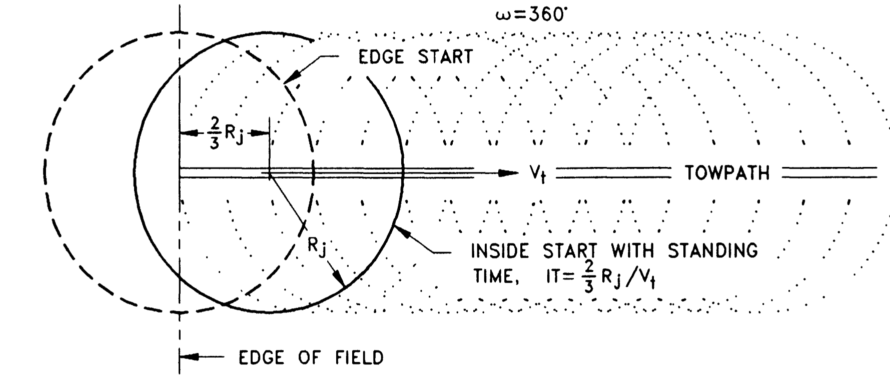
The total standing time should be equal to the additional time that would have been required to travel the full length of the towpath at V_t. It should be equally divided between the two ends thus:

$$IT_e = FT_e = \frac{ID_e + FD_e}{2V_t} = (2/3)\frac{\omega}{360} \cdot \frac{R_j}{V_t} \tag{13.5}$$

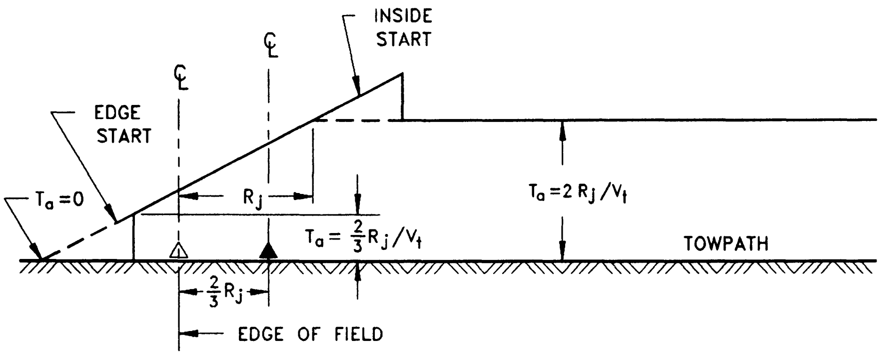
where IT_e = initial standing time for end-to-end operation, min, and FT_e = final standing time for end-to-end operation, min.

Based on the above strategy of operation, the average depths of water applied at the two ends of the towpaths will be $(1/2)d$ at each edge of the field when averaged together. The average depth applied will increase to d at a distance R_j inside each edge and remain constant thereafter. This is true for either edge-to-edge travel with no standing times or as suggested above.

To visualize this, assume a full-circle gun sprinkler ($\omega = 360°$) with a uniform application rate profile so that the depth of application is directly proportional to the application time, $d \propto T_a$. Figure 13.12 shows the application pattern for a traveling sprinkler starting at the edge of the field or inside the field (see Part A). Part B shows the corresponding application time profiles along the



A. PLAN VIEW OF WATERING PATTERNS FOR EDGE AND INSIDE STARTS



B. APPLICATION TIME PROFILES FOR EDGE AND INSIDE STARTS

FIG. 13.12. Application Pattern for a Traveling Sprinkler and Corresponding Relative Application Time Profiles Along the Towpath with $\omega = 360°$.

towpath. At the starting end of the towpath $T_a = R_j/V_t$, and at any point along the towpath greater than R_j from the field edges $T_a = 2R_j/V_t$. This is obvious for starting at the edge of the field with no standing time. It is also true for inside starts with standing time. For example, by Eq. 13.3:

$$ID_e = (2/3)R_j$$

and by Eq. 13.5 assuming full-circle operation:

$$IT_e = (2/3)\frac{360}{360} \cdot \frac{R_j}{V_t} = (2/3)\frac{R_j}{V_t}$$

Therefore, the total application time at the end of the towpath is:

$$T_a = IT_e + \text{time required to travel } (R_j - ID_e)$$

$$= (2/3)\frac{R_j}{V_t} + \frac{[R_j - (2/3)R_j]}{V_t} = \frac{R_j}{V_t}$$

End-to-Center. Hose-drawn traveling sprinklers are usually started at the ends of the towpath and pulled to the center, as in Figs. 13.7 and 13.8. The distances from the initial starting points to the edge of the field at each end of the towpaths should be the same as for end-to-end operation. Thus, ID_c should be computed by Eq. 13.3. Furthermore, the sprinkler should be drawn as close to the halfway point along the towpath as practical, so $FD_c \approx 0$.

The total standing time for the setup on each half of the towpath should be ID_c/V_t using the same logic as before. However, for end-to-center operation the standing time should be proportioned as follows:

$$IT_c = \frac{\omega}{360} \cdot \frac{ID_c}{V_t} = (2/3)\frac{\omega}{360} \cdot \frac{R_j}{V_t} \tag{13.6a}$$

and

$$FT_c = \left(1 - \frac{\omega}{360}\right) \cdot \frac{ID_c}{V_t} = (2/3) \cdot \left(1 - \frac{\omega}{360}\right)\frac{R_j}{V_t} \tag{13.6b}$$

where IT_c = initial (end) standing time for end-to-center operation, min, and FT_c = final (center) standing time for end-to-center operation, min.

Adjacent (or Large) Fields. Sometimes travelers are used on adjacent fields that can be irrigated as a unit without a dry strip in between or on long fields requiring multiple setups along each towpath (see Fig. 13.13). With end-to-end operation the discontinuity in the watering patterns between the end regions of the first setup and the beginning of the next can be practically eliminated by simulating uninterrupted operation across the interface. The following design and operating criteria are required to achieve this result with end-to-end operation across adjacent fields (see Figure 13.13):

1. The towpaths should be continuous between adjacent fields or within a large field, and the traveler should move in the same direction in any given towpath. Furthermore, the q, R_j, ω, d, and consequently, V_t should be approximately the same for the travelers in adjacent fields or within a large field.
2. Let Sections (1) and (2) (numbered in the direction of travel) be equal in length and form a continuous towpath $(1+2)$ with interface $(1|2)$. The

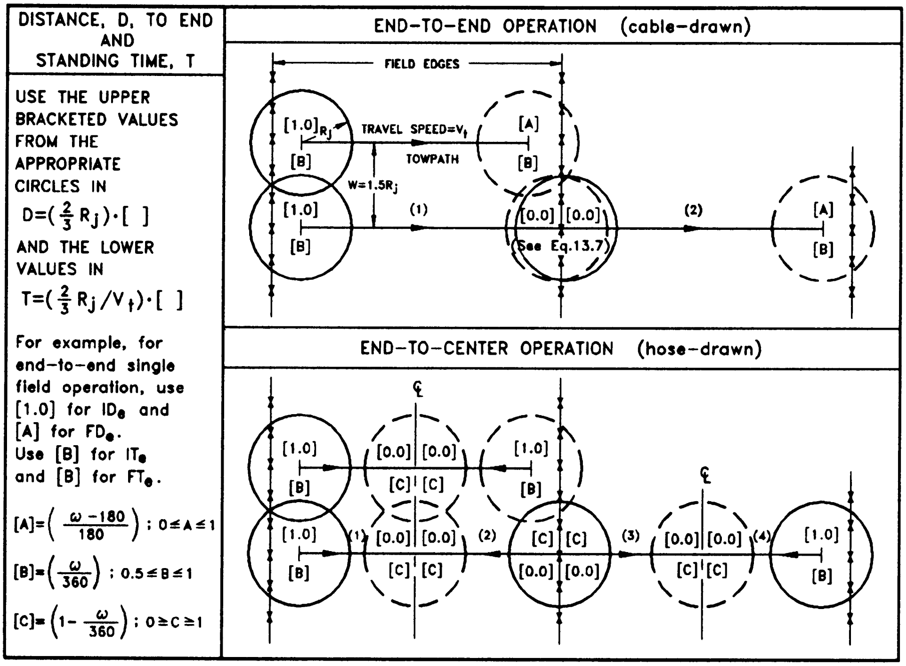


FIG. 13.13. Standing Position Distances to Ends of Towpath Sections and Standing Times for Individual and Adjacent Files with Wetted Sector Angles Ranging From 180° ≤ ω ≤ 360°.

starting points $(ID_e)_1$ should be computed by Eq. 13.3 and $(FD_e)_2$ by Eq. 13.4; and $(FD_e)_1$ and $(ID_e)_2$ should be as close to zero as possible. Ideally, the sprinkler should travel up to the interface during the setup in Section (1) and continue on during the setup in Section (2). However, sometimes this is impossible.

3. The standing times $(IT_e)_1$ and $(FT_e)_2$ should be computed by Eq. 13.5, and the standing times at the interface should be:

$$(FT_e)_1 = (IT_e)_2 = \frac{(FD_e)_1 + (ID_e)_2}{2V_t} \tag{13.7}$$

For end-to-center operation, avoiding the watering discontinuity at the interface between fields or multiple setups is more difficult. This is because there are also discontinuities at both the centers of each field and pair of setups. The following design and operating criteria are required to achieve the best uniformity with *end-to-center* operation in adjacent fields (see Fig. 13.13):

1. The towpaths should be continuous between adjacent fields or within a large field, and the q, R_j, ω, d, and consequently V_t should be approximately the same for the travelers in adjacent fields or within a large field.

2. Let Sections (1+2) and (3+4) form a continuous towpath (1+2+3+4) with equal length subsections. Water is supplied at the interfaces (1|2) and (3|4) within each section and the interface between fields or pairs of setups is at (2|3). Then, noting the symmetry along the towpath, the starting points $(ID_c)_1$ and $(ID_c)_4$ should be computed by Eq. 13.3. Furthermore, the sprinkler(s) should be pulled as close to the center of each section as possible during each setup so that $(FD_c)_1$, $(FD_c)_2$, $(FD_c)_3$, and $(FD_c)_4 \approx 0$. To minimize the watering discontinuity between fields at interface (2|3):

$$(ID_c)_2 = (ID_c)_3 = (2/3) \cdot \left(1 - \frac{\omega}{360}\right) R_j \qquad (13.8)$$

3. The initial standing times for the outside subsections $(IT_c)_1$ and $(IT_c)_4$ should then be computed by Eq. 13.6a, and for the inside subsections $(IT_c)_3$ and $(IT_c)_4 = 0$. The final standing times for all subsections should be computed by Eq. 13.6b. It is obvious that the final standing times for the outside subsections should be the same as for individual fields. The reason it is also the same for the inside subsections is because the initial standing time is zero; therefore:

$$(FT_c)_2 = (FT_c)_3 = \frac{(ID_c)_2}{V_t} = (2/3) \cdot \left(1 - \frac{\omega}{360}\right) \frac{R_j}{V_t} \qquad (13.9)$$

Summary of Standing Positions and Times. A summary of the standing positions and times for end-to-end and end-to-center operation in individual and adjacent fields is presented in Fig. 13.13. The standing distance, D, and time, T, values are presented as functions of any wetted sector angle from $180 \leq \omega \leq 360$ and are in dimensionless form. The unit of distance used for locating the standing positions in relation to the ends of the towpaths is $(2/3)R_j$, and the unit of standing time is $(2/3)R_j/V_t$.

The [1.0], [0.0], or [A] = $[(\omega - 180)/180]$ in the upper brackets in each circle is the recommended portion of a unit distance from the edge of the field to the standing points at the respective locations. The values in the lower brackets in each circle, which are: [B] = $(\omega/360)$; [C] = $(1 - \omega/360)$; [Eq. 13.7]; or [0.0] give the recommended portion of a unit standing time at the respective locations. The solid-line circles represent the initial or starting positions, and the dashed-line circles represent the final or stopping positions for the traveling sprinklers for full-circle operations. In Figure 13.13 the radius, R_j, of the wetted areas and lengths of the towpath sections are drawn more or less to scale for $R_j \approx 60$ m (200 ft) and field widths of approximately 400 m (1320 ft) between edges.

Rate of Irrigation Coverage

Knowing the rate at which the traveler is irrigating the land area is often a useful parameter for use in the planning and design process. The rate of irrigation coverage is a function of travel speed and towpath spacing, which can be computed by:

$$RIC = \frac{W\,V_t}{K} \tag{13.10}$$

where

RIC = rate of irrigation coverage, ha/hr (A/hr)
K = conversion constant, 166.7, for metric units (726 for English units)
W = towpath spacing, m (ft)

HOSE AND TRAVELER FRICTION LOSSES

Hose-fed traveling sprinklers must have hoses that are long, flexible, tough-skinned, and capable of withstanding high pressures. The hoses work best if there are no couplers or repair sleeves to foul the winding mechanisms, and contiguous lengths up to 400 m (1320 ft) are typical. High-pressure traveler hoses are approximately five times as expensive as pipe, are easily damaged during operation by sharp objects in the towpaths, and are difficult to repair. Furthermore, the end pull required to drag a hose is approximately proportional to the square of the diameter. Therefore, relatively small-diameter hoses are used for rather large flow rates. In Table 13.4 friction-loss values are given for the normal range of flow rates used for each size of lay-flat hose.

The inside diameter of lay-flat hose (used for cable-drawn travelers) increases by almost 10% under normal operating pressures. Thus, for a given friction-loss gradient, a lay-flat hose will have about 20% more carrying capacity than the same diameter rigid plastic hose (used for hose-drawn travelers). The friction-loss values given in Table 13.4 are estimated for lay-flat hose operating at approximately 690 kPa (100 psi).

Friction loss can be estimated by Eq. 8.7 when the actual inside hose diameter during operation is known. The more rigid, thick-walled, polyethylene (PE) plastic hoses used for hose-drawn travelers do not lay flat and have calibrated inside diameters that do not change appreciably due to pressure. Thus, Eq. 8.7 can be used directly to estimate friction-head losses for such plastic hoses. Two popular sizes of PE hoses used for hose-drawn travelers are: 110-mm (4.3 in.) with inside diameter, D = 90 mm (3.54 in.); and 90-mm (3.5-in.) with D = 73.6 mm (2.90 in.)

Traveler vehicles are powered by water turbines, water pistons, or engines.

Table 13.4. Estimated friction loss gradients in both psi/100 ft and *J* values in m/100 m (ft/100 ft) for lay-flat irrigation hose operating at approximately 690 kPa (100 psi)

		Nominal lay-flat hose diameter, in.									
		2.5		3		4		4.5		5	
Flow rate,		Friction-loss gradients, psi/100 ft and *J* values									
L/s	(gpm)	psi	*J*	psi	*J*	psi	*J*	psi	*J*	psi	*J*
6.3	100	1.6	3.7								
9.5	150	3.4	7.9	1.4	3.2						
12.6	200	5.6	12.9	2.4	5.5						
15.8	250			3.6	8.3	0.9	2.2				
18.9	300			5.1	11.8	1.3	3.1	0.6	1.4		
25.2	400					2.3	5.3	1.3	2.9		
31.5	500					3.5	8.1	2.1	4.8	1.1	2.5
37.9	600					4.9	11.3	2.7	6.1	1.6	3.7
44.2	700							3.6	8.2	2.1	4.9
50.5	800							4.6	10.5	2.7	6.2
56.8	900									3.4	7.9
63.1	1000									4.2	9.7

1 psi/100 ft = 22.6 kPa/100 m.

When determining system pressure requirements, both the pressure loss through the traveler mechanism and riser height should be considered. This is especially important for turbine-driven travelers where the pressure difference between the vehicle inlet and sprinkler base typically exceeds 70 kPa (10 psi). Friction-loss data should be available from the manufacturer for operating their travelers at various flow rates and travel speeds.

SYSTEM DESIGN PROCEDURE

The simplest way to present the procedure for designing traveling sprinkler systems is through use of a comprehensive sample calculation. Sample Calculation 13.2 is presented with this in mind. Rather than outlining a step-by-step procedure beforehand, the design strategy is presented, and each step is discussed along the way.

Sample Calculation 13.2. *Traveling sprinkler irrigation system design for cable-drawn traveler.*

GIVEN: The $\frac{1}{2}$-mile long by $\frac{1}{4}$-mile wide 80-A field with a well in the center, shown in Fig. 13.1. Assumed winds are low, ranging between 0 and 5 mph, and the irrigation efficiency of the low half is $E_h = 70\%$. The peak moisture-use rate is 0.22 in./day for the corn crop to be grown. For the soil, which is sandy, the allowable application rate and moisture depletion are 1.0 in./hr and 3.0 in., respectively. Irrigation over the edges of the field is both permissible and practical.

FIND: Determine: the system layout and required sprinkle discharge; nozzle size; travel speed; and the pressure required at the hose inlet for a cable-drawn traveling sprinkler system.

CALCULATIONS: The strategy that will be used to optimize the design is as follows. First, the minimum allowable system flow rate, Q_s, will be determined, and then a sprinkler configuration will be selected to closely match this minimum Q_s; next, an optimum towpath spacing will be selected from a practical set of spacings; and then, the travel speed that applies the required depth of application consistent with a practical travel schedule will be selected and tested.

Minimum system capacity: The simplest way to obtain the minimum system capacity is to assume a 1-day irrigation interval. Using Eq. 5.3a with an $E_a = E_h = 70\%$ and a peak moisture-use rate of 0.22 in./day, the average gross depth of application for a 1-day irrigation interval during the peak-use period must be:

$$d' = \frac{0.22}{70/100} = 0.32 \text{ in.} \tag{5.3a}$$

The minimum system capacity required for the 80-A field can now be determined using Eq. 5.4 and letting $f = 1$ day and $T = 23$ hr/day:

$$Q_s = \frac{453A\,d}{f\,T}$$

$$= \frac{453 \times 80 \times 0.32}{1 \times 23} = 504 \text{ gpm} \tag{5.4}$$

Sprinkler selection: For the corn crop and sandy soil, no special consideration need be given to application rate or droplet impact. Therefore, either tapered or orifice-type nozzles can be used, along with relatively low pressures. From Table 13.1, a gun sprinkler with a 1.4-in. tapered bore nozzle operating at 80 psi will discharge $q = 515$ gpm. This q closely matches the minimum Q_s determined above; thus a single traveler should be sufficient to irrigate the entire 80-A field.

Towpath spacing: From Table 13.1, which is based on 24° jet trajectory angles, the 1.4-in. nozzle can be expected to produce a wetted diameter of approximately 455 ft when operating at 80 psi. From Table 13.3, for winds up to 5 mph, the towpath spacing can be 75% of the wetted diameter, which is $0.75 \times 455 = 341$ ft.

The practical towpath spacings for the $\frac{1}{2}$-mile (2640-ft) dimension of the field rounded to the nearest 10 ft are:

Number of Towpaths	Spacing, ft	Number of Towpaths	Spacing, ft
7	380	10	260
8	330	11	240
9	290	12	220

Based on the above, the nearest acceptable practical towpath spacing for the design at hand is 330 ft. Thus, eight towpaths will be required, as shown in Figure 13.1.

Travel speed: It is desirable to have only one setup per day. Assuming an 8-day irrigation interval, the gross depth of water required per irrigation is $d = 8 \times 0.32 = 2.56$ in. Therefore, from Eq. 13.2, the required travel speed is:

$$V_t = \frac{1.604\, q}{W\, d} \tag{13.2}$$

$$= \frac{1.604 \times 515}{330 \times 2.56} = 0.98 \text{ ft/min}$$

The time required to travel the 1320-ft length of each towpath is:

$$\frac{1320}{0.98 \times 60} = 22.4 \text{ hr}$$

This is a reasonable design. In practice, the travel speed would probably be adjusted to as close to 1.0 ft/min as possible. This would decrease the depth of application slightly and reduce the travel time to 22 hr.

Design adjustment: An example of a small adjustment would be to make the time to travel 1320 ft exactly 23 hr by setting $V_t = 0.96$ ft/min. By Eq. 13.2, the required sprinkler discharge would then be:

$$q = \frac{2.56 \times 330 \times 0.96}{1.604} = 506 \text{ gpm}$$

This approximates the minimum system capacity $Q_s = 504$, which was found earlier. In accordance with Eq. 5.2, the sprinkler operating pressure could be reduced from 80 to approximately 77 psi. This would reduce the actual sprinkler discharge from 515 to 504 gpm.

Hose inlet pressure: An economic analysis using life-cycle costing (see Chapter 8) was made assuming a hose life of 7 years and selecting travelers capable of dragging the different-sized hoses. The 4.5-in.-diameter lay-flat hose proved to be the most economical for the 515 gpm design flow rate. From Table 13.4 the estimated pressure gradient for the 4.5-in. lay-flat irrigation hose is

2.1 psi/100 ft for a flow rate of 500 gpm. Using Eq. 8.7a as a basis for interpolation, the expected pressure loss, P_h, through a 660-ft-long hose with a flow rate of Q_s = 515 gpm is:

$$P_h = 2.1\left(\frac{515}{500}\right)^{1.75} \times \frac{660}{100} = 14.6 \text{ psi}$$

A turbine drive traveler was selected that, according to the manufacturer's charts, would have a friction plus drive turbine pressure loss of 7.5 psi when V_t = 1.0 ft/min. Therefore, at 515 gpm the total pressure loss through the traveler unit, P_t, would be:

$$P_t = 7.5\left(\frac{515}{500}\right)^2 = 8.0 \text{ psi}$$

In addition, the pressure loss through the automatic shutoff valve, P_{cv} = 3.5 psi, and the height of the nozzle is 10 ft above ground level.

The hose inlet pressure required for the traveling sprinkler, assuming there is no elevation difference along the towpath, is:

Component	Pressure, psi
Sprinkler pressure, P	80.0
Pressure loss in hose, P_h	14.6
Pressure loss in traveler, P_t	8.0
Automatic shutoff valve, P_{cv}	3.5
Riser height (10 ft), P_r	4.3
Required hose inlet pressure, P_l	110.4 psi

Sample Calculation 13.3. *Standing position and time computations for cable-drawn traveling sprinkler.*

GIVEN: The system specifications and design parameters in Sample Calculation 13.2. The gun sprinkler is to be operated with a wetted sector angle of ω = 285°, giving a dry wedge of 75°.

FIND: The initial and final standing positions and times and the required hose length and inlet pressure.

CALCULATIONS: Because cable-drawn travelers operate from end to end, from Eq. 13.3 (or Fig. 13.13) the distance from the edge of the field to the initial standing position should be:

$$ID_e = (2/3)R_j = (2/3) \times (455/2) = 152 \text{ ft}$$

and to the final standing position by Eq. 13.4 or from Fig. 13.13 it is:

$$FD_e = (2/3) \times \frac{(285 - 180)}{180} \times (455/2) = 88 \text{ ft}$$

The total standing time should be equal to the additional time that would have been required to travel the full length of the towpath at $V_t = 0.98$ found in Sample Calculation 13.2. It should be equally divided between the two ends to give (see Eq. 13.5):

$$IT_e = FT_e = \frac{ID + FD_e}{2V_t} = \frac{152 + 88}{2 \times 0.98} = 122 \text{ min}$$

or by using the information presented in Fig. 13.13:

$$IT_e = FT_e = 2/3(R_j/V_t) \cdot \left(\frac{\omega}{360}\right)$$

$$= (2/3) \times \left(\frac{455/2}{0.98}\right) \times \left(\frac{285}{360}\right) = 123 \text{ min}$$

The hose length could be reduced by a maximum of:

$$1/2(152 + 88) = 120 \text{ ft}$$

which would not only reduce the initial system cost, but also the pressure loss in the hose from $P_h = 14.6$ psi found in Sample Calculation 13.2 to:

$$P_h = 14.6 - 14.6(120/660) = 14.6 - 2.7 = 12.9 \text{ psi}$$

and the required hose inlet pressure to:

$$P_l = 110.4 - 2.7 = 107.7 \text{ psi}$$

Sample Calculation 13.4. *Traveling sprinkler irrigation system design for small hose-drawn travelers.*

GIVEN: The system specifications and design parameters in Sample Calculation 13.2.

FIND: Determine the system layout, required sprinkler discharge, and travel speed for the 80-A field using two hose-drawn travelers.

CALCULATIONS: For this design, the travel distance will be one-half the towpath length, or 660 ft between setups. From Table 13.1, two travelers, each equipped with a 1.0-in. tapered nozzle discharging 260 gpm at 80 psi could be used. This will give $Q_s = 520$ gpm, which is close to the minimum $Q_s = 504$ gpm found in Sample Calculation 13.2.

From Table 13.1, the expected wetted diameter will be 355 ft. Using a maximum towpath spacing of 75% of the wetted diameter as before, the maximum spacing should not exceed $0.75 \times 355 = 266$ ft. With two travelers it is best to use an even number of towpaths. Thus, the closest acceptable towpath spacing is 260 ft, and 10 towpaths will be required.

Assuming each sprinkler will travel a distance equal to one-half the length of a towpath each day, the irrigation interval will be 10 days. Thus, the gross depth of water required per irrigation is $d = 10 \times 0.32 = 3.2$ in., and by Eq. 13.2:

$$V_t = \frac{1.604 \times 260}{260 \times 3.2} = 0.50 \text{ ft/min}$$

and the time required to travel the 660 ft from the edge of the field to the center along each towpath is:

$$\frac{660}{0.50 \times 60} = 22 \text{ hr}$$

Sample Calculation 13.5. *Depth of water applied at towpath ends using different management criteria for a hose-drawn sprinkler.*

GIVEN: The system specifications and design parameters in Sample Calculations 13.2 and 13.4 for a hose-drawn traveler with a sprinkler producing a wetted radius of $R_j \approx 180$ ft.

FIND: Compare the relative depth of water applied along the towpath for the hose-drawn traveler layout for starting on the edge of the field with no standing times and for inside standing positions and standing times using $\omega = 180$ and $360°$.

CALCULATIONS: The application time along the inner portions of the towpaths with $V_t = 0.5$ ft/min for half-circle operation is:

$$T_a = R_j/V_t = 180/0.5 = 360 \text{ min} = 6 \text{ hr}$$

From the information in Fig. 13.13 for end-to-center operation of a hose-drawn traveler with $\omega = 180°$, the initial standing position, ID_c, and time, IT_c, for inside starts are:

$$ID_c = (2/3)R_j \cdot (1.0) = (2/3)180 = 120 \text{ ft}$$

and:

$$IT_c = [(2/3)R_j/V_t] \cdot \left(\frac{\omega}{360}\right) = \left((2/3) \times \frac{180}{0.5}\right) \times \frac{180}{360}$$

$$= (240) \times \frac{180}{360} = 120 \text{ min} = 2.0 \text{ hr}$$

and from Fig. 13.13 the final standing position FD_c and time FT_c are:

$$FD_c = 0$$

and:

$$FT_c = [(2/3)R_j/V_t] \cdot \left(1 - \frac{\omega}{360}\right) = 240 \times \left(1 - \frac{180}{360}\right)$$
$$= 120 \text{ min} = 2.0 \text{ hr}$$

For full-circle operation with $\omega = 360°$, the values are:

$$ID_c = 120 \text{ ft}$$

$$IT_c = (240) \times \left(\frac{360}{360}\right) = 240 \text{ min} = 4.0 \text{ hr}$$

and:

$$FD_c = 0$$

$$FT_c = (240) \times \left(1 - \frac{360}{360}\right) = 0$$

Figure 13.12 shows a comparison between the relative application time profiles along the towpath at the edge of the field for the edge and inside start positions when $\omega = 360°$. By substituting $R_j = 180$ ft and $V_t = 0.5$ ft/min, $T_a = 12$ hr along the midpositions of the towpath. With $\omega = 360°$, there would be little discontinuity at the field's center (midway between the edges), assuming the traveler was drawn very close to the center from each edge and turned off.

Figure 13.14 shows the comparable application time profiles when $\omega = 180°$. The edge start profile is shown by the dashed lines until it reaches the constant portion where $T_a = 6$ hr. With half-circle operation there is considerable dis-

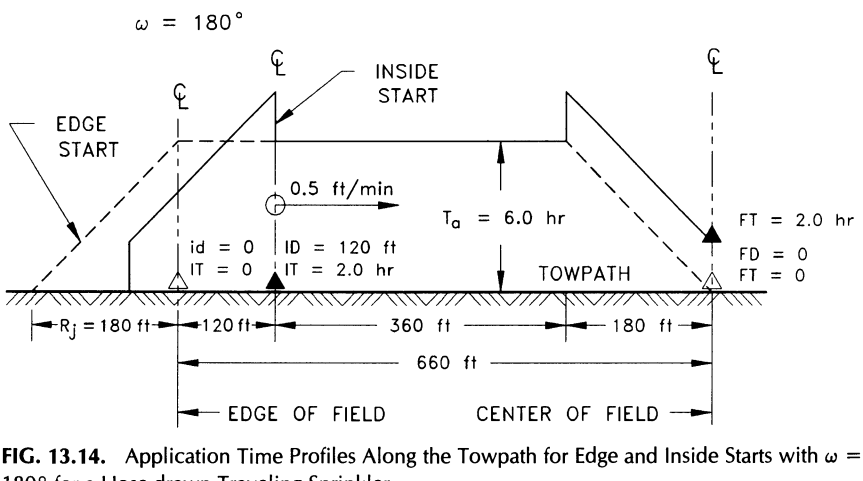


FIG. 13.14. Application Time Profiles Along the Towpath for Edge and Inside Starts with $\omega = 180°$ for a Hose-drawn Traveling Sprinkler.

continuity in the application time and consequently in the application depth in the vicinity of the center of the field. This discontinuity would diminish with increases in ω and is eliminated, as mentioned above, with full-circle operation. The application rate is double for half-circle operation compared with full-circle operation. Therefore, the depth applied in 12 hr with $\omega = 360°$ is the same as with 6 hr with $\omega = 180°$.

REFERENCES

Collier, R. L., and E. W. Rochester. 1980. Water application uniformity of hose towed traveler irrigations. *ASAE Transactions* 23(5):1135–1138.

Rolland, Lionel. 1982. Mechanized sprinkler irrigation. *Food and Agriculture Organization of the United Nations Irrigation and Drainage Paper* 35.

Shull, H., and A. S. Dylla, 1976. Traveling gun application uniformity in high winds. *ASAE Transactions* 19(2):254–258.

RECOMMENDED READING

Anon. No date. Traveling sprinkler system planning guide. Nelson Irrigation Corp.: Sales brochure TS 270. Walla Walla, Washington.

Pair, C. H., W. H. Hing, K. R. Frost, R. E. Sneed, and T. J. Schilty (Eds.). 1983. *Irrigation*, 5th ed. Arlington, Virginia: The Irrigation Association.

14

Center-Pivot System Design

Center-pivot irrigation machines are among the most popular systems for irrigating general field crops and are used on over half of the sprinkle-irrigated land in the United States. They have made it easy to efficiently irrigate many areas where surface or conventional sprinkle irrigation methods are not adaptable. Being relatively easy to manage, they have also opened new horizons in crop production. Farmers with center-pivots can apply light and frequent irrigations as needed to best fit crop water requirements and maximize production. This is practical for there is little labor associated with each irrigation. Thus, the applications can be scheduled without considering labor regimes or being tied to soil moisture-holding capacity or content.

Over the past two decades, center-pivot machinery has been quite well perfected. It is mechanically reliable and simple to operate, although, like any machinery, systematic routine maintenance is necessary. Furthermore, various nozzling packages that provide very uniform water applications have been developed. Low-pressure nozzling packages are available for use where soil infiltration capacities and/or surface storage are sufficiently high (see Fig. 2.1). For steep or hilly topography, flow- or pressure-control devices are available. Some of the main advantages of center-pivot irrigation machines are:

- Water delivery is simplified through the use of a stationary pivot point.
- Guidance and alignment are controlled at a fixed pivot point (see Fig. 4.10).
- Relatively high water application uniformities are easily achieved under the continuously moving sprinklers.
- After completing one irrigation, the system is at the starting point for the next irrigation.
- Achieving good irrigation management is simplified because accurate and timely application of water is made easy.
- More accurate and timely applications of fertilizer and other chemicals are possible by applying them through the irrigation water.
- Flexibility of operation makes it feasible to develop electric-load-management schemes.

The above advantages eliminate the most difficult mechanical and operational problems associated with other sprinkle irrigation systems.

As with all irrigation machines, to reduce the cost per unit of area irrigated, it is advantageous to irrigate as large an area as possible with a minimum amount of equipment. In the case of center-pivots, this is accomplished by irrigating as large a circle as possible because the cost of equipment is proportional to the radius, but the area irrigated is proportional to the square of the radius. The most common radius of center-pivot machines is approximately 400 m (1320 ft), which fits on a square 65-ha (160-A) field commonly called a *quarter-section* in the western United States.

From a water application standpoint, center-pivots have the following disadvantages:

- Where the pivot point is in the center of the square field, only 50 to 53 ha (125 to 132 A) will be irrigated, depending on how far water is thrown from the end sprinklers. This leaves about 20% of the area unirrigated, unless special equipment is provided for the corners, but this adds considerably to the system's cost and complexity (see Fig. 4.11).
- The average application rate at the outer edge of the irrigated circle (see Fig. 4.10) is usually quite high. In some systems it may be over 100 mm/hr (4 in./hr) with certain nozzle configurations.
- Relatively, light and frequent applications must be used on all but the more sandy soils (or cracked clays) to reduce or eliminate runoff problems associated with these high application rates. In extreme cases to avoid runoff, it may even be necessary to set the travel speed so a center-pivot lateral cycles faster than one revolution per day. This increases evaporation losses and center-pivot maintenance costs and may decrease crop yield.
- Because each additional increment of radius irrigates a large concentric band, most of the water must be carried toward the outer end of the lateral. This results in relatively high pipe-friction losses.
- On sloping fields the average lateral operating pressure will vary significantly depending on whether it is pointing up- or downhill. This can result in large variations in discharge unless sprinklers with pressure- or flow-controlled nozzles are used.

At first glance it may appear that pivot machines are so complete and automatic there is little left for the field designer to do. But this is not true. To optimize the performance of these rather exquisite irrigation machines, considerable finesse is needed in the selection, design, and management of them.

Working with a fast and continuously moving lateral that pivots around a fixed end presents many unique design challenges. Almost every facet of the design process developed in the preceding chapters requires some modification.

GENERAL SYSTEM LAYOUT, CONCEPTS AND HARDWARE

Figure 14.1 shows the general layout and components of a center-pivot irrigation system with a special end-gun that is operated only while the lateral is passing the corner areas. If the radius of the basic circle L = 402 m (1320 ft) and the effective radius irrigated by the end-gun, $(R - L)$ = 30 m (98 ft), the total irrigation area would be approximately, A = 55 ha (136 A). This is 85% of the total field area, leaving 15% unirrigated. The crescent-shaped areas in the four corners irrigated by the end-gun are each a little larger than 1.0 ha (2.5 A). Therefore, without the end-gun, A = 50.8 ha (125.7 A), which is only about 80% of the 65-ha (160-A) field.

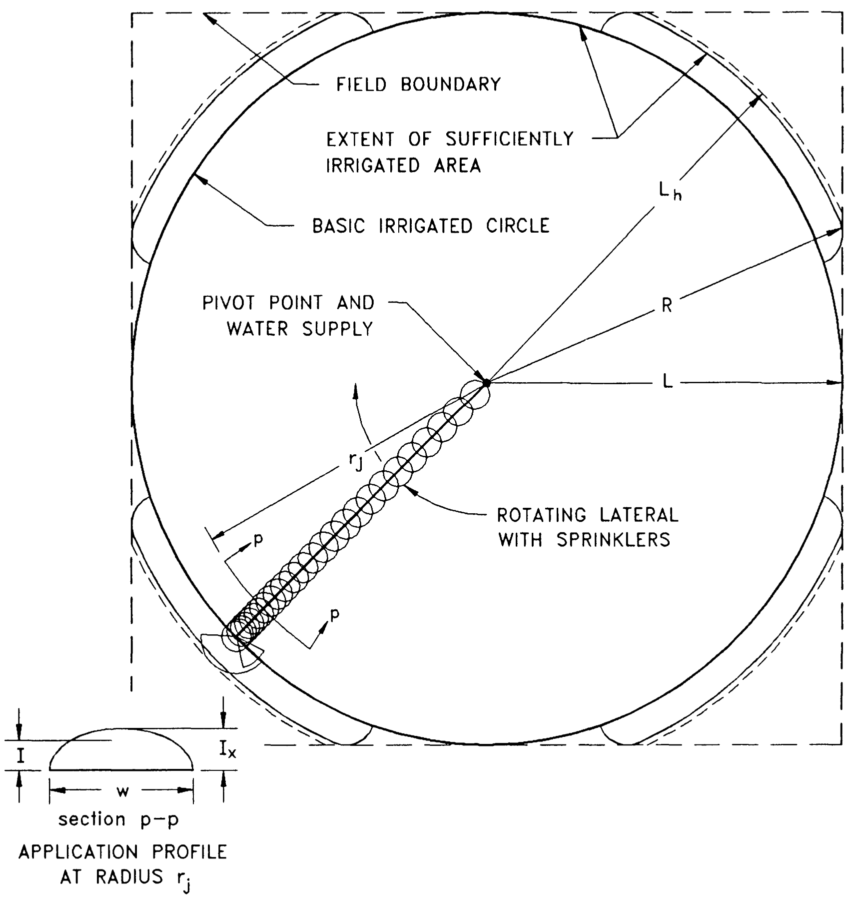


FIG. 14.1. General Layout and Components of a Center-pivot Sprinkle Irrigation System with End-gun Operating In Corners.

Section *p-p* of Fig. 14.1 shows a typical "stationary" water-application-rate profile. Stationary profiles are plots of application rates generated by the overlapping watering patterns of the individual sprinklers. They can be computed from catch data obtained by placing rows of containers along radial arcs and operating the sprinklers without rotating the lateral.

An observer standing under a moving lateral would experience the profile as a continuously varying application rate as the lateral passed overhead. Regardless of how fast the lateral moved overhead (rotated), the wetted width, w, the maximum application rate, I_x, and the average application rate, I, would remain the same. But, the length of time and, consequently, the total amount of water falling on the observer would be directly proportional to the speed of rotation.

Typical Hardware Configurations

The center-pivot lateral rotates around the fixed pivot point. The lateral is supported above the crop by a series of A-frame towers (see Fig. 2.1). Each tower is supported on two rubber-tired wheels (see Fig. 4.10) powered by electric (or hydraulic) motors. Typical span lengths are between about 30 to 50 m (100 to 165 ft), and the lateral pipe diameters typically used are between 100 and 250 mm (4 and 10 in.). The most common or "standard" machines in the United States have a lateral pipe diameter of 168 mm, which is the size of standard IPS 6 $\frac{5}{8}$-in. iron pipe). The laterals are supported on 4-m (13-ft) high towers spaced approximately 40 m (130 ft) apart. On level or uniformly sloping ground, this gives about 3 m (10 ft) of clearance between the ground surface and bottom of span trussing. However, higher (4-m or 13-ft), and lower (2-m to 6-ft) clearance machines are also available.

The inlet end of each span is usually provided with a flexible joint to allow the lateral to articulate and handle slope changes (in the radial direction) of up to 30% on rolling ground. Most standard machines can travel up and down slopes as steep as 20% (in the circular direction) on fields with shallow furrows. However, where furrows are more than 0.15 m (0.5 ft) deep, they experience difficulty on slopes exceeding about 15%. For machines fitted with larger pipe or longer spans than the above-mentioned standard machine configurations, maximum negotiable slopes are somewhat less.

In view of the above, it is important for field design engineers to select system configurations carefully to ensure reliable operation and long system life. The slopes along the tower's wheel tracks need to be taken into consideration where pronounced slope changes are apparent. Furthermore, on rolling fields there may be some high areas between tower-wheel paths that reduce the clearance between the soil surface and the bottom of span trussing. In some fields this creates a crop, or worse yet, a ground clearance problem. To facilitate operation, high spots can be trimmed and low ones filled in; span lengths can be changed where needed; or a combination of the two can be used. (There is no problem with using different span lengths for a given machine.)

The minimum rotation time for the standard center-pivot laterals produced by most manufacturers is a little less than 24 hr with 60-Hz motors. However, special high-speed drive mechanisms are available that reduce the rotation time to less than 12 hr for special situations. The rotation speed is controlled by varying the average travel speed of the end tower. For machines with electric motor drives, this is done by cycling the power on and off using a percentage-timer mounted at the pivot end. Typically the on-off cycle time is 1 min. For example, setting the on-time at 25% would turn the end tower drive motor on for 15 s every minute. This would increase the time per revolution for a standard machine from about 24 to 96 hr, or 4 days.

The alignment of the lateral is maintained by special control devices activated by deflections created due to misalignment. But for most electric-drive machines the instantaneous traveling speed of all towers is essentially the same. Therefore, each tower's drive unit is activated for only a fraction of the time the outer tower is in motion. The fraction is equal to the respective ratio of the lengths of their travel paths. Thus, the forward motion of electric-drive machines is unsteady, This causes problems with the uniformity of individual watering cycles, which fortunately are mostly mitigated by subsequent cycles. This is not a problem with hydraulic-drive machines for which speed and alignment are achieved by valves. The valves control the instantaneous as well as the average speed, rather than on-off cycling, which controls only the average speed.

GENERAL DESIGN CONCEPTS

Because of the completely automatic nature of center-pivot systems, it is relatively easy to carefully manage soil moisture levels. Theoretically, for the same irrigation interval, center-pivot systems would have about the same capacity requirements as fixed systems, which also provide excellent water control. However, mechanical breakdown is more likely and periodic maintenance is necessary, so it is advisable to allow for more downtime. As a general rule it is advisable to base center-pivot system capacity requirements on operating only 90% of the time or about 22 hr/day for 7 days a week during the peak-crop-water-use period.

Application Frequency and Depth

Many center-pivot systems must be operated to apply light and frequent irrigations. This is necessary because of water-intake limitations or low water-holding capacities of soils, to maintain sufficient soil water storage capacity for in-season rainfall, for seed germination, and/or maintain uniform soil water levels. It is not unusual for systems to be designed that require irrigating as frequently as every 1 or 2 days during peak water-use periods. But, when this is done it significantly increases the evaporation component of evapotranspiration. This must be taken into account during the design process when determining the application depths and system capacity.

To compensate for high-frequency irrigation, the conventionally computed average daily-consumptive-use rate during the peak-use month, U_d (or seasonal use, U) must be adjusted upward. In addition, the application efficiency and leaching requirements must be taken into account. The gross daily application depth can be determined by combining Eqs. 5.3 and 6.9 and multiplying by a factor to adjust for frequency. Where the leaching requirement $LR \leq 0.1$ (as computed by Eq. 3.3) then:

$$d' = \frac{k_f U_d}{E_{pa}/100} = \frac{100 k_f U_d}{DE_{pa} R_e O_e} = \frac{d_i'}{R_e O_e} \tag{14.1a}$$

Where $LR > 0.1$ and rainfall makes up less than 25% of the net water requirements, then ordinary deep-percolation losses will not be sufficient to satisfy LR, and:

$$d' = \frac{90 k_f U_d}{(1.0 - LR)\, DE_{pa} R_e O_e} = \frac{d_i'}{R_e O_e} \tag{14.1b}$$

where

d' = average daily gross depth of water application required during the peak-water-use period, mm (in.)

d_i' = average daily gross depth of infiltration required during the peak-water-use period, mm (in.)

k_f = frequency factor from Table 14.1 to adjust standard crop water-use values for high-frequency irrigation, decimal

U_d = average daily crop water-use rate during peak-use month, mm (in.)

E_{pa} = application efficiency based on adequately irrigating a given percentage, pa, of the field area, %

DE_{pa} = sprinkler distribution efficiency based on adequately irrigating a given percentage, pa, of the field area, %

R_e = effective portion of water discharged from sprinklers, most of which reaches the irrigated soil-plant surface, decimal

O_e = ratio of water effectively discharged through sprinklers to total system discharge, decimal

LR = leaching requirement ratio as computed in Eq. 3.3, decimal

The average depth of application that must infiltrate into the soil to avoid runoff is less than the gross depth required, $d_i' < d'$. This difference results from system leakage plus drift and evaporation losses that are accounted for by R_e and O_e.

Crop water requirements are made up of a combination of evaporation from the soil surface and transpiration from the plants. The water requirements for a specific crop are usually determined by field experiments using irrigation intervals that are relatively long, but still do not restrict productivity. Thus, the soil surface has time to dry between applications, which reduces the evaporation component of evapotranspiration. Typically, for the convenience of transferring crop water-use information, the daily water-use rate, U_d, of each specific crop is directly related to the evapotranspiration of alfalfa, ET, by a crop coefficient, k_c. Thus, U_d equals the specific crop's k_c times the ET (of alfalfa, which is used as a reference crop, i.e., $U_d = k_c\, ET$).

The frequency factors for both peak daily, k_f, and seasonal, k_{fs}, given in Table 14.1 were developed using concepts from (Hill, 1989; and Hill et al., 1983) to provide a simple means to account for the increase in k_c (and consequently U_d) due to high-frequency irrigation. In developing the frequency factors it was assumed that conventional k_c values already take into account evaporation from the soil surface that would occur with normal periodic-move irrigation systems. Furthermore, it was assumed that crops could be conveniently grouped into categories relative to their transpiration percentage, PT, compared to the transpiration portion of ET for alfalfa with a full canopy.

Table 14.1. Peak-use period, k_f, and seasonal k_{fs} irrigation frequency factors for different crops and irrigation intervals, f

Crop type and	PT, %[1]	Type period	Irrigation interval, f', days[2]							
			≤1	2	3	4	5	6	7	>10
Vegetable and fruit[3]	80	Peak	1.15	1.09	1.05	1.02	1.01	1.01	1.00	–
	80	Seasonal	1.15	1.09	1.05	1.02	1.01	1.01	1.00	–
Field and row[4]	90	Peak	1.09	1.06	1.04	1.03	1.02	1.02	1.01	1.00
	90	Seasonal	1.09	1.06	1.04	1.03	1.02	1.02	1.01	1.00
All small grain[4]	100	Peak	1.02	1.01	1.00	1.00	1.00	1.00	1.00	1.00
	100	Seasonal	1.02	1.01	1.00	1.00	1.00	1.00	1.00	1.00
Forage[4] cut	100	Peak	1.02	1.01	1.00	1.00	1.00	1.00	1.00	1.00
	90	Seasonal	1.09	1.06	1.04	1.03	1.02	1.02	1.01	1.00
Pasture:	90	Peak	1.09	1.06	1.04	1.03	0.02	0.02	1.01	1.00
	80	Seasonal	1.20	1.14	1.10	1.07	1.05	1.04	1.03	1.00

[1]Transpiration percentage relative to alfalfa during the full growth stage.
[2]Assumes drying time, DT, of 3 days for coarse, 5 days for medium, and 7 days for fine-textured soils.
[3]Assumes conventional values of U_d or U based on irrigation intervals, f', for shallow-rooted crops of 4 days for coarse, 5 days for medium, and 7 days for fine-textured soils.
[4]Assumes conventional values of U_d or U based on f' for deep-rooted crops of 7 days for coarse, 10 days for medium, and 14 days for fine-textured soils.

The k_f and k_{fs} values in Table 14.1 represent composite averages for coarse, medium, and fine-textured soils that would normally be considered suitable for center-pivot irrigation. They take into account: different drying times associated with the three soil types, as given in Table 14.1, footnote 2, and evaporation from the soil surface that is presumably already accounted for in k_c for the normal irrigation intervals associated with the crop and soil types, as given in table footnotes 3 and 4. The frequency factors computed for all three soil textures were close enough so the average values presented in Table 14.1 are sufficiently representative for them all. Frequency coefficients k_f' for similar crop types with transpiration percentages PT' that are different than in Table 14.1 can be computed by:

$$k_f' = 1.0 + \frac{(100 - PT')/PT'}{(100 - PT)/PT}[k_f - 1.0]$$

where k_f and the corresponding PT are taken from Table 14.1 and set PT' or $PT = 98$ if either is greater than 98%.

Additional research and analysis are still needed to verify the k_f and k_{fs} values given in Table 14.1 and categorize the important crops more specifically, but the magnitude and general trends of the values should be realistic for design purposes. For crops with PT values of 80 and 90%, $k_f \approx k_{fs}$, providing irrigations are scheduled so the depth of each application, d, is the same and the interval is varied. If the interval were constant and d adjusted to schedule irrigations throughout the growing season, k_{fs} would be very much larger than k_f. For small grains $k_f \approx k_{fs}$, but they differ significantly for a forage crop that is harvested periodically throughout the season. This is because when the crop canopy is cut back, the soil is more exposed and PT is consequently lowered. Pastured forage crops are normally harvested (by the grazing animals) more often than machine-harvested ones, and thus they have a lower PT and higher k_{fs} values.

Limited Irrigation

For a crop-soil system with a 125-mm (5.0-in) or greater available water-storage capacity, it may be practical in both arid and humid regions to use limited irrigation during the peak-use period without appreciably affecting yield. Frequent irrigations make it practical to gradually deplete the moisture stored in the deep soil profile during peak crop-water-use periods. Thus, the system capacity need not be sufficient to fully meet peak crop-water-withdrawal rates. However, where subsoil moisture is inadequate, applying light and frequent irrigations is an inefficient way to use a limited supply of water and may also increase salinity due to increased evaporation. Under such conditions, deeper, less frequent irrigations usually produce better yields.

Under certain conditions, system capacities as low as 60% of those recommended for ordinary periodic-move systems may be adequate. However, to determine the maximum area that can be irrigated from a given water supply based on the limited irrigation concept, a soil moisture budget should be developed for the entire irrigation season. To maintain near-optimum crop yields, the available soil water content should not be allowed to fall too far below the recommended management-allowed deficit, MAD, which is usually about 50%.

In regions where the probability of rainfall is significant during the irrigation season, an additional benefit can be derived from limited irrigation. Allowing the moisture stored in the deep soil profile to gradually deplete leaves storage space for in-season rainfall. This is discussed in the section on scheduling and operations.

Design Procedure

The main field factors to be considered when designing center-pivot irrigation systems are: the seasonal and peak water-use rate of the cropped area; soil infiltration and moisture holding characteristics; crop characteristics and their water-versus-yield relationship; anticipated effective rainfall; field topography and boundaries; water supply quality and quantity; equipment and operating costs; and various other economic parameters. In ordinary practice the system designer specifies the maximum required travel speed, lateral length, system discharge, nozzling configuration, lateral pipe diameter, span lengths, and tire sizes, and perhaps the available inlet pressure and topographic extremes. The supplier provides the center-pivot machine that meets the specifications. Ordinarily field engineers are not required to design nozzling packages or any of the machine's mechanical aspects. However, field engineers are sometimes responsible for retrofitting existing machines with new sprinkler/nozzle packages.

System Capacity

The required system capacity, Q_s, for periodic-move irrigation systems can be computed by Eq. 5.4. It can also be used to compute the unit system capacity required for different consumptive use rates. For center-pivots it is useful to modify Eq. 5.4 to obtain Q_s directly without first determining the irrigated area, such that:

$$Q_s = Q_b + Q_g = \frac{L^2 d'}{K\,T} + Q_g \tag{14.2}$$

where

Q_s = total system discharge capacity, L/s (gpm)
Q_b = discharge to basic circular field, L/s (gpm)

Q_g = discharge from end-gun (or corner system), L/s (gpm)
K = conversion constant, 1146 for metric units (30.6 for English units)
T = average daily operating time, hr
L = radius irrigated in basic circle when end-gun or corner system is not operating, m (ft)

The Q_s is separated into Q_b and Q_g, because the operating characteristics, application efficiency, and design procedures are quite different for the basic circle and the area irrigated by the end-gun. To allow for breakdowns and periodic maintenance the value of T is normally set at 22 or at most 23 hr/day. To allow time to move the system when one machine is used to irrigate two fields or for shutdowns associated with electric load management, the value of T must be reduced accordingly.

APPLICATION INTENSITY

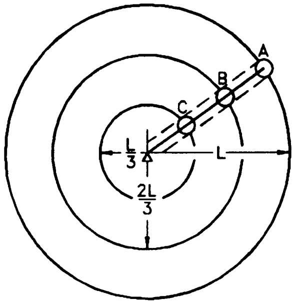
The distance traveled by each sprinkler along a center-pivot lateral is equal to $2\pi r$, where r is the radial distance from the pivot point. Thus, the application rate must increase with r to obtain a uniform depth of application. This is demonstrated in Fig. 14.2 for a center-pivot lateral with variably spaced sprinklers that produce a uniform wetted width, as depicted in Fig. 14.1. As a result, application rates, especially near the lateral's moving end, often exceed the soil's infiltration capacity. The resulting runoff may cause considerable erosion and severely reduce the uniformity of irrigation, which causes a combination of water, energy, and crop production losses.

Elliptical water-application rate profiles, as shown in Figs. 14.1 and 14.2, are usually assumed to be representative. A stationary profile taken at any radius r_j from the pivot (as shown in section *p-p* of Fig. 14.1) can be transformed into a moving application rate profile (see Fig. 14.2B) by dividing the profile's base width by the speed of the moving lateral at the same r_j. The average water application rate is obtained by dividing the depth applied, which is represented by the area under the moving profiles in Fig. 14.2B, by the length of time water is applied to the soil. Thus, for an elliptical profile:

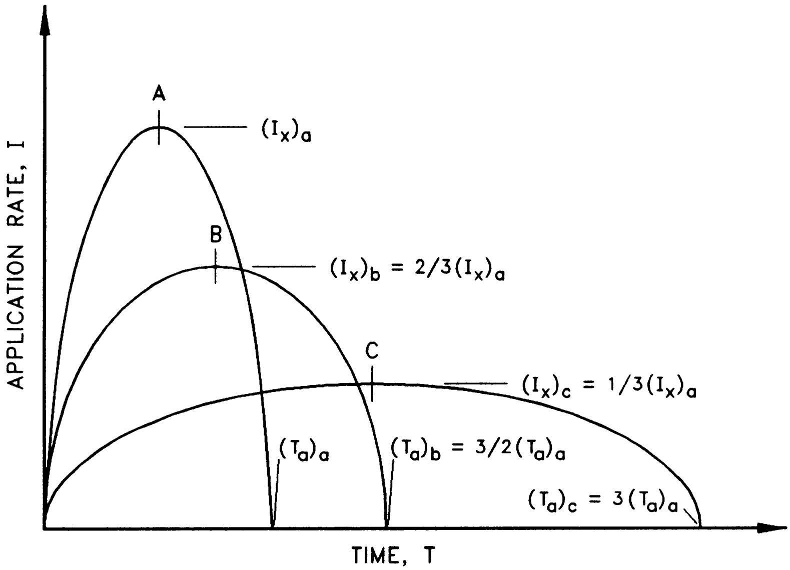
$$I_j = \frac{d\, R_e O_e}{T_{aj}/60} = \frac{\pi T_{aj} I_{xj}}{4\, T_{aj}} = \frac{\pi}{4} I_{xj} \tag{14.3}$$

where

I_j = average application rate required at radial distance r_j, mm/hr (in./hr)
d = gross depth of water required per irrigation, mm (in.)
T_{aj} = application time at radial distance r_j, min
I_{xj} = peak application rate at radial distance r_j, mm/hr (in./hr)



A. PLAN VIEW OF CENTER–PIVOT FIELD WITH UNIFORM WIDTH OF WETTED STRIP



B. WATER APPLICATION RATE PROFILES AT DIFFERENT POINTS ALONG PIVOT LATERAL

FIG. 14.2. Water-application Rates at Different Points Along a Center-pivot Lateral with a Uniform Width of Wetted Strip.

Theoretically, the depth of water reaching the soil does not include the drift and evaporation losses (from the droplets); however, this is very difficult to measure in practice.

Application Rate

The average application rate that should be received on the solid surface is a function of the radial distance r_j from the pivot point. It is proportional to the length of the circular travel path at r_j of the pivoting lateral divided by the width of the stationary application profile. It is given by:

$$I_j = \frac{2\pi r_j}{w_j} \cdot \frac{d_i'}{T} \tag{14.4a}$$

Combining Eqs. 14.1, 14.2, and 14.4a gives:

$$I_j = \frac{2\pi r_j}{w_j} \cdot \frac{K\, Q_b\, R_e O_e}{L^2} \tag{14.4b}$$

where

I_j = average application rate at radius r_j, mm/hr (in./hr)
r_j = radial distance from pivot to point under study, m (ft)
d_i' = average peak gross infiltration required per day, mm (in.)
w_j = width of stationary application patterns at r_j, m (ft)
T = average daily operating time, hr
K = conversion constants as in Eq. 14.2

Infiltration Rate and Surface Storage

Suggested maximum application rates for periodic-move sprinkle systems operating on different soils and slopes are presented in Table 5.4. Values in the table can be increased greatly without causing runoff when applying light irrigations with a center-pivot system.

Short-term soil and leaf-surface storage, although small, plays an important role in minimizing runoff from pivot systems. This is because when irrigating low-intake soils, individual irrigation applications can be kept small. In many cases surface storage can temporarily hold a third or more of the water applied during each application. Thus, the application rate can exceed the infiltration rate by a considerable amount without causing runoff. For example, assume an irrigation of 7.5 mm (0.3 in.) is applied and the surface storage is 2.5 mm (0.1 in.). In this case only 5 mm (0.2 in.) would need to infiltrate to prevent runoff while the system passes overhead.

For surface storage to be most effective, water that does not immediately infiltrate must be held on leaf and soil surfaces and in micro and small depressions or pockets uniformly spaced throughout the field. It is important that excess water remain near where it falls and soak in or be consumed. If it travels very far, the uniformity of irrigation will be adversely affected, as high areas will be under- and low areas overirrigated. Furthermore, the accumulation of the running water into increasingly large streams may cause serious erosion on slopes (especially in wheel tracks) and waterlogging in field depressions.

The average effective depth of naturally occurring surface storage on all but essentially level fields seldom exceeds 7.5 mm (0.3 in.). The amount of surface storage primarily depends on the general slope, microtopography, and roughness of the soil surface. The following equation for estimating the average depth of surface storage is based on the authors' field observations, review of literature, and logic:

$$SS \approx K(SS_m + SS_f + kSS_d)$$

Substituting a factor to relate the storage capacity of the small depressions, SS_d, to slope, s, gives:

$$SS \approx K\left(SS_m + SS_f + k\frac{(6 - s)(12 - s^2)}{144}\right) \tag{14.5}$$

where

SS = average surface storage capacity, mm (in.)
K = conversion constant, 1.0 for metric units ($\frac{1}{25.4}$ for English units)
SS_m = storage capacity of microdepressions, mm (in.)
SS_f = storage on plant foliage, mm (in.)
SS_d = storage capacity of the small depressions, mm (in.)
k = 0.5 for furrowed field and 1.0 for smooth fields
s = general field slope in area under study, %

The value of SS_m depends on the microcharacteristics of the soil surface and is essentially independent of s. Reasonable values for $(SS_m + SS_f)$ are in the order of:

1 mm for weathered soil surfaces without a closely spaced cover crop;
2 to 3 mm for newly tilled land;
3 mm for pastures and grass cover crops; and
2 mm for closely spaced crops, such as alfalfa or small grain.

The SS_d term in Eq. 14.5 is based on the assumptions that: the macro portion of SS is held in inverted pyramid-shaped depressions; they average approximately 0.6 m (2.0 ft) in length (in the direction of the slope); and the typical maximum depth of an average depression is 18 mm (0.71 in.). Because the volume of a pyramid is $\frac{1}{3}$ of its height times the area of its base, on a level field the average depth of storage in such a depression would be approximately 6 mm (0.24 in.). The SS_d term is an approximation of how slope would affect the storage in such depressions. For fields with furrows it is assumed the depressions are only in furrow bottoms, or cover about half of the area: thus, $k = 0.5$ for furrowed fields.

Equation 14.5 has not been verified, but it does give *SS* values that seem reasonable and perhaps on the conservative side. The predicted *SS* values suggested in most literature related to infiltration under center-pivots are considerably higher. Typically, a value of $SS = 12.5$ mm (0.5 in.) is suggested for all level fields. This is decreased by 2.5 mm (0.1 in.) for each 1.0% slope; so $SS = 0.0$ when $s = 0.05$.

Tillage implements are available that have ripping shanks followed by a trailing spade to provide what is called *reservoir tillage*. Reservoir tillage is the process where each furrow is ripped to a depth of approximately 0.3 m (12 in.). This is followed by a revolving spade that implants a series of small dikes (or depressions) at equal spacings. The purpose of reservoir tillage is to increase both the intake rate of the furrows (by the ripping) and the *SS* by incorporating a series of small reservoirs. Reservoir tillage has proven successful under certain conditions. However, this practice should be used with some caution, for it is fairly expensive and complicated to do. Furthermore, as the irrigation season progresses, the dikes tend to break down, and once this begins to happen a chain reaction may take place that can cause severe erosion problems especially on steep slopes.

Another factor that allows center-pivots to apply high application rates without causing runoff is that soil infiltration capacity is high initially and decreases with time. Light, frequent applications take maximum advantage of this phenomenon. For example, in Fig. 14.3, the shaded portion depicts the potential runoff from an elliptical application rate profile. If the system were speeded up, the peak rate of the application pattern would remain the same, but the breadth (time of application) would decrease. This would decrease the amount of excess

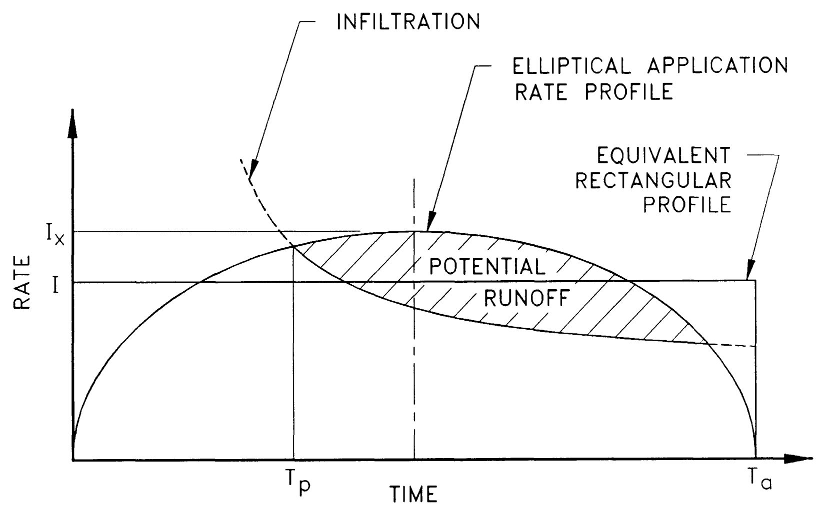


FIG. 14.3. Intersection Between an Elliptical Moving Application Rate Profile Under a Center-pivot Lateral and a Typical Infiltration Curve.

precipitation (represented by the shaded area) somewhat and consequently the *SS* needed to eliminate runoff. However, the infiltration curve will also shift to the left, and this will reduce the benefit from light, frequent applications.

Figure 14.4 shows the soil-texture-classification triangle with an overlay of general infiltration-rate contours and anticipated center-pivot performance criteria for bare soils with average structure. It can be used as a simple guide for identifying where center-pivots may be appropriate and for anticipating management difficulties. Therefore, it is useful where field experience (based on actual center-pivot operation with various nozzling configurations or on infiltrometer tests) is not available. Obviously, Fig. 14.4 should be used only as a first approximation, because factors other than soil texture, such as soil structure, chemistry, slope, and surface cover, also affect a soil's water-infiltration capacity.

In general, the anticipated performance of center-pivot irrigation on soils having the various textural classifications is as follows:

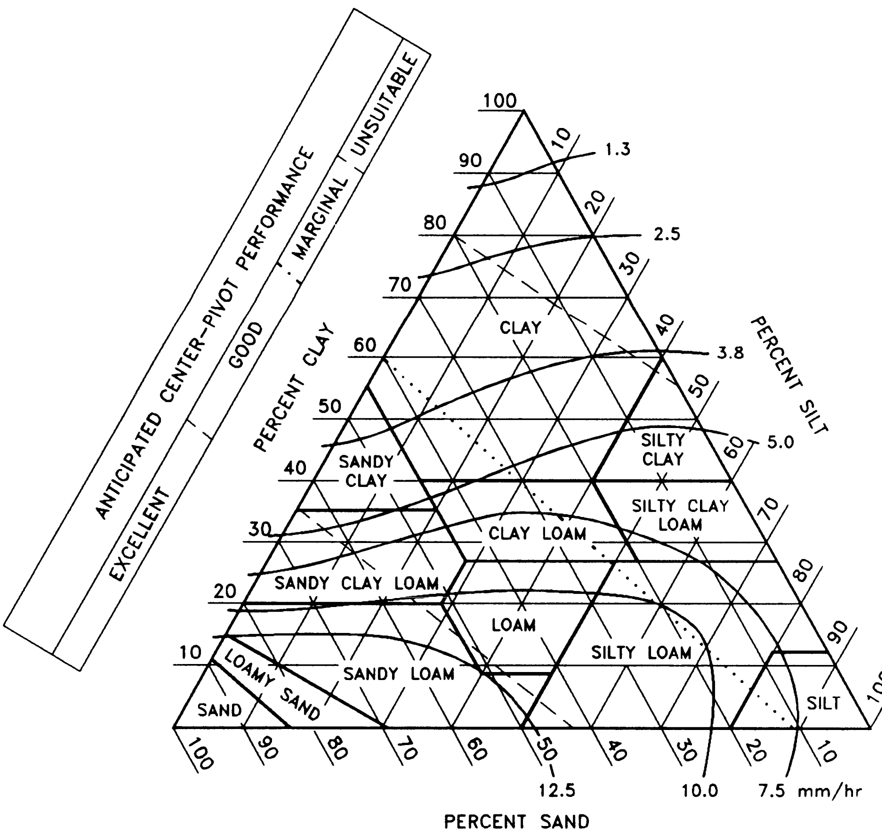


FIG. 14.4. Soil-texture Triangle with Overlay of General Infiltration-rate Contours in mm/hr and Anticipated Center-pivot Performance Criteria.

- *Excellent to good* on soils bounded by the 10.0 mm/hr (0.3 in./hr) contour. Successful performance is relatively independent of drop size, application rate, and surface storage.
- *Good to marginal* on soils between the 10.0 and 5.0 mm/hr (0.4 and 0.2 in./hr) contours. Successful performance requires having low application rates with small drops, and surface storage becomes increasingly important.
- *Marginal to unsuitable* on soils that fall outside the 5-mm/hr (0.2-in./hr) contour. Successful performance will depend heavily on surface storage even when small application depths are applied at low application rates.

Where there are center-pivot systems operating nearby and under similar conditions (soils, slopes, application rates, and nozzling packages), observing their performance is very useful for developing design and management guidelines. Systematic field infiltration tests are very useful where sufficient local experience with center-pivot system is lacking and will be covered in a later section.

SPRINKLER TYPE AND SPACING

A major consideration when designing center-pivot laterals is properly selecting the sprinkler package. The two major variables are the spacing and the type of sprinklers.

Sprinkler Spacing

The three most typical sprinkler-spacing configurations used along center-pivot laterals are:

Uniform sprinkler spacing with 9 to 12 m (30 to 40 ft) between sprinklers along the lateral and their discharge increasing in direct proportion to their distance from the fixed or pivot end. Their wetted diameter also increases, but not in direct proportion to their discharge.

Semiuniform sprinkler spacing in which the lateral is divided into three or four reaches, and a different uniform spacing is used in each reach beginning with the widest spacing near the pivot.

Uniform sprinkler discharge with only small discharge variations necessary. This requires beginning with the sprinklers spaced about 12 m (40 ft) apart near the pivot end decreasing it to about 1.5 m (5.0 ft) at the moving or outer end of the lateral. The sprinkler spacing along the lateral is in inverse proportion to the radial distance from the pivot, so the spacing times the radial distance is a constant (see Fig. 14.1).

A uniform spacing between outlets is most commonly used. This is done for simplicity of manufacture and ease of field assembly. However, with uniform sprinkler spacing, relatively large sprinkler nozzles requiring high operating pressures are needed, and this results in higher energy costs. Furthermore, when irrigating fragile soils without a cover crop the droplets from large nozzles may cause surface crusting and sealing, which decrease intake and may increase runoff.

To avoid problems associated with large nozzles, semiuniform sprinkler spacings are often used. A typical strategy is to have a 12-m (40-ft) sprinkler spacing along the first third of the lateral, a 6-m (20 ft) spacing along the middle third, and a 3-m (10-ft) spacing along the last third of the lateral. With such a configuration the outlets can be uniformly spaced at 3-m (10-ft) intervals along the entire lateral, and the unused outlets are simply closed off with pipe plugs. For example, sprinklers are installed in every fourth outlet along the first third, every other outlet along the middle third, and every outlet along the last third of the lateral.

Types, Pressures, and Pattern Widths

The ranges of pressures and wetted widths for different sprinkler type and spacing configurations commonly used on center-pivot laterals are presented in Table 14.2. Figure 14.5 shows a section of a center-pivot lateral with low-pressure, fixed-spray sprinklers (spray heads). The sprinklers are on short booms to increase the wetted width. Figure 14.6 shows a typical center-pivot lateral with medium-pressure-impact sprinklers. Standard-impact sprinklers operating at the low end of the pressure range produce large droplets that may cause excessive soil sealing. To remedy this, special noncircular and/or untapered nozzles are used. However, these reduce the wetted width of their patterns compared with those produced by standard nozzles that are designed to maximize throw.

The principle reason for using *low-pressure spray heads* on center-pivot laterals is to conserve energy. However, the desired savings may be more than offset by poor application efficiency because of excessive runoff, wind drift, and pressure variations due to elevation differences across sloping fields. The jets of most spray heads used on center-pivots impinge on plates to divert the water and produce a 360° wetting pattern. They produce narrow pattern widths and consequently high application rates (see Eq. 14.4) unless placed on short booms, as shown in Fig. 14.5, to increase the width of coverage. Their use even with short booms is limited to high-infiltration soils or to nearly level fields where the surface water storage, *SS*, is high enough to eliminate runoff. Nozzles impinging on smooth plates produce small drops that cause only limited surface sealing, but are subject to high wind-drift losses. However, spray heads are available with serrated plates that produce coarser sprays to reduce wind drift and increased pattern widths.

Table 14.2. Range of normal operating pressures and associated pattern widths for different sprinkler type and spacing configurations most commonly used on center-pivot laterals

Sprinkler type and spacing configuration	Pressure range[1] kPa	(psi)	Pattern width range[2] m	(ft)
Low-pressure spray[3]:				
1. Single row drop[4]	70–205	(10–30)	3–9	(10–30)
2. Single row top	70–205	(10–30)	6–14	(20–45)
3. On short booms[5]	70–140	(10–20)	12–18	(40–60)
4. On long booms[6]	105–170	(15–25)	20–26	(65–85)
Low-pressure impact:[7]				
5. Variable spacing	140–240	(20–35)	18–23	(60–75)
6. Semiuniform spacing	205–275	(30–40)	21–24	(70–80)
Medium-pressure impact:				
7. Variable spacing	275–345	(40–50)	27–34	(90–110)
8. Semiuniform spacing	275–380	(40–55)	30–37	(100–120)
High-pressure impact:				
9. Uniform spacing	380–450	(55–65)	40–50	(130–160)

[1]Values are rounded to nearest 5 kPa or psi and are sprinkler or boom inlet pressures that include typical pressure regulator losses at the high end of the range.
[2]Pattern width with full-circle sprinklers at the moving end of lateral with $L \approx 400$ m (1300 ft).
[3]For all sprayers the range depends on height of sprayers above crop, configuration of spray plate, and pressure.
[4]Sprayers on drops should be at least 1 m (3.3 ft) above the crop, and their maximum spacing should not exceed 1.5 m or 2.4 m (5 or 8 ft) when operated at 70 or 140 kPa (10 or 20 psi), respectively.
[5]Booms 3 to 6 m (10 to 20 ft) long with two or three spray heads.
[6]Booms up to 14 m (45 ft) long with up to seven spray heads available for use with lateral outlets up to 9 m (30 ft) apart.
[7]The sprinklers have nozzles that diffuse the jets for better breakup and distribution.

The sprinkler pressure variations that occur as a lateral rotates on a sloping field cause discharge variations that are proportional to the square root of the design operating pressure. Therefore, the water distribution uniformity from center-pivots with low-pressure sprinklers operating on uneven topography may be very poor unless the sprinklers are fitted with flexible-orifice nozzles (see Fig. 6.6) or pressure regulators.

Low-pressure impact sprinklers with *variable or semiuniform spacings* provide a more economical sprinkler package than spray heads on long booms. They produce similar pattern widths, but require somewhat higher operating pressures.

The *high-pressure impact sprinklers* with large-diameter nozzles usually used with *uniform spacing* along the lateral produce wide wetting patterns and large drops. The wide patterns produce a relatively low application rate, but the large drops may cause surface sealing that can significantly reduce the soil's infiltration capacity and cause runoff.

The use of *medium-pressure impact sprinklers* (see Fig. 14.6) with a *variable or semiuniform spacing* is perhaps the best compromise for most soils. Where

FIG. 14.5. Center-pivot Lateral With Low-pressure, Fixed-spray Sprinklers on Short Booms (Source: Nelson Irrigation Corp.).

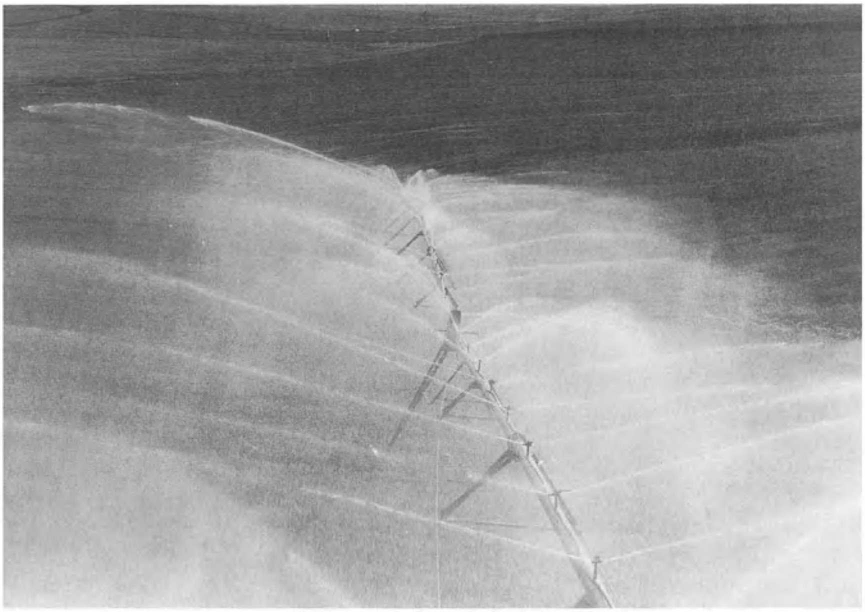


FIG. 14.6. Center-pivot Lateral with Medium-pressure Impact Sprinklers (Source: Nelson Irrigation Corp.).

soil sealing and lower infiltration rates are not likely to be problems, the semiuniform spacing with 25-kPa (40-psi) pressure can be used to save energy. For soils that are more difficult to manage the variable-spacing configuration with 345-kPa (50-psi) pressure should be used. On undulating topography, flexible-orifice nozzles or pressure regulators can be used to maintain the desired sprinkler discharges while their pressures vary due to elevation changes.

Jet Trajectory and Spray Losses

Impact (rotating) sprinklers with various jet trajectory angles are available. The use of high-trajectory (23 to 27°) sprinklers normally used for periodic-move and fixed sprinkle systems often results in excessive drift losses when placed high above the ground along center-pivot laterals. To minimize drift losses, recommended trajectory angles for center-pivot sprinklers range between 6 and 18° with the low end of the range being preferable in high winds.

Table 14.3 shows drift-plus-spray losses from field can test data for center-pivots with different nozzle configurations. (The numbers preceded by a # in

Table 14.3. Drift-plus-spray losses based on field evaluations of center-pivots with different sprinkler configurations

Sprinkler type and spacing configuration[1]	Trajectory angle[2]	Wind, mph	Temp., °F	Loss, %
Spray, #2	High	3	80	20
Semiuniform spacing, #8	6°	7	80	15
Semiuniform spacing, #8	6°	7	88	10
Semiuniform spacing, #8	23°	7	90	18
Spray, #2	High	9	95	25
Semiuniform spacing, #8	6°	10	86	3
Uniform discharge, #7	6°	10	88	17
Semiuniform spacing, #8	6°	12	86	7
Spray, #1	Low	8	80	4
Semiuniform spacing, #8	6–18°	4	83	6
Spray, #2	High	6	90	14
Spray, #2	High	8	92	10
Spray, #1	Low	7	85	5
Uniform spacing, #9	23–27°	9	86	13
Semiuniform spacing, #8	6–18°	12	90	19
Uniform spacing, #9	23–27°	13	91	36
Uniform discharge, #7	6–18°	16	91	16
Spray, #2	High	8	85	12

NOTE: 1.0 mph = 1.6 km/hr; °C = (°F − 32)/1.8.

[1]Refer to sprinkler type/configuration numbers in Table 14.2 for details, i.e., #2 is single row top spray, #7 is variable spacing medium-pressure impact, etc.

[2]High spray is on top of lateral; low spray is on drop pipe close to crop canopy.

Table 14.3 correspond to the numbers designating each of the nine nozzle configurations in Table 14.2.) These should be used only as indicative values, because all tests are unreliable for accurately estimating spray-and-drift losses. Such tests often significantly overestimate losses. The data indicate that where average wind speeds are expected to exceed 10 km/hr (6 mph) low-trajectory impact sprinklers or drop pipes to lower spray heads (to about 1 m (3 ft) above the crop canopy) should be used.

All estimated drift-plus-spray losses determined from can catch data are not actual losses. Some of the drifting water eventually becomes available for crop use (outside the wetted pattern area) and/or reduces the effective evapotranspiration of the crop. Equation 6.8 can be used for estimating the effective portion of the applied water, R_e, for low-angle impact sprinklers and fixed-spray sprinklers on top of the lateral or on drop pipes or booms.

Coarseness index (CI) values for the use in Eq. 6.8 may be determined using the information in Table 14.4. The table gives the suggested CI for fixed-spray sprinklers. For impact sprinklers recommended operating pressure, P, and nozzle diameter, B, adjustments are given for use in Eq. 6.7 to compute CI values. Such adjustments of P and B are needed to compensate for the differences be-

Table 14.4. Guidelines for determining the coarseness index, CI, when computing R_e for center-pivots

For low-pressure, fixed-*spray* sprinklers:

With smooth plates let CI = 17;
With narrow groove serrated plates let CI = 12; and
With wide groove serrated plates let CI = 7.

For *impact*-sprinklers use Eq. 6.7 to compute CI. To compensate for the different nozzle configurations use adjusted values for the nozzle operating pressure, P, and nozzle diameter, B, as given below.

For diffusing nozzles:
P = 1.5 (actual P); and
B = the equivalent B for a standard nozzle.

For ordinary nozzles:
P = actual P; and
B = actual B.

For nozzles with flexible flow-control orifices:
P = 1.1 (actual P); and
B = the equivalent B at normal operating pressure.

For nozzles with straightening values:
P = 0.9 (actual P); and
B = actual B.

For orifice-type (or ring) nozzles:
P = 1.2 (actual P); and
B = the equivalent B for a tapered nozzle or about 0.85 (actual B).

tween anticipated CI values for standard-tapered nozzles and the special diffusing and other nozzles used with impact sprinklers for center-pivots.

The R_e values obtained by using Eq. 6.7 are based on ordinary sprinklers typically used on periodic-move laterals. Such sprinklers produce pattern widths of 21 to 34 m (70 to 110 ft), as indicated in Table 5.2. For the narrow patterns produced by a single row (or double row using short booms) of sprayers on drop pipes, the losses will be considerably less than computed by Eq. 6.8. To adjust for this, for patterns 6 m (20 ft) wide or less, the computed R_e values should be reduced by about 50%. Thus, the adjusted $R_e = [1 - 0.5\,(1 - R_e)]$ where $w < 6$ m.

SELECTING SPRINKLER CONFIGURATIONS

The main problems encountered in the design of water-application packages for center-pivot systems involve trying to eliminate runoff and still use relatively slow lateral speeds. The first design step, relative to selecting the sprinkler configuration, is to determine the peak daily gross depth of water that must be infiltrated, d_i'.

Gross Infiltration Requirements

The value of d_i' can be determined by Eq. 14.1; however, first a value for DE_{pa} must be determined. Because center-pivot laterals are continuously moving and the sprinklers along them are closely spaced, application uniformities are high. Furthermore, because irrigations are usually light and frequent, the composite uniformity based on the sum of several irrigations is appropriate to use. Thus, uniformity problems caused by wind and uneven lateral motion tend to be ironed out. The resulting composite uniformity for well-designed systems is: $90 <$ CU < 94 or $84 <$ DU < 90.

Sample Calculation 14.1. *Determining the system capacity and application rate for a center-pivot system.*

GIVEN: A square 65-ha (160-A) corn field in a moderate to hot climate is to be irrigated with an electric-drive, center-pivot lateral that is 402 m (1320 ft) long and fitted with low-pressure diffusing nozzles in impact sprinklers at variable spacings.

The average peak daily water requirement, $U_d = 7.0$ mm/day.
The desired percentage adequacy, $pa = 80\%$.
The anticipated average uniformity of application, CU = 90%.
The anticipated irrigation interval, $1.5 < f < 2$ days.
The daily operating time, $T = 22.0$ hr.

The width of the sprinkler pattern at the outer end of the lateral for $r_j = L$, $w_j = 20$ m.

Average wind speeds during the peak-use period range below 6 and 16 km/hr.

The average equivalent nozzle size and operating pressure along the lateral are $B = 5$ mm and $P = 205$ kPa.

The leaching requirement, $LR < 0.1$.

FIND: The system capacity and the average application rate at the moving end of the lateral (assuming there is no end-gun or other type of corner system).

CALCULATIONS: To determine the system capacity, Q_s, the gross daily application depth, d', must first be determined by Eq. 14.1. This requires knowing the anticipated O_e, DE_{80} and R_e. For electric-drive machines $O_e \approx 1.00$; however, for a water-drive machine $O_e < 1.0$. Assuming CU = 90%, then $DE_{80} = 90\%$ (see Chapter 6). To determine the value of R_e, the coarseness index of the spray, CI, must first be computed by Eq. 6.7 using the information given in Table 14.4. Assuming an average nozzle diameter of 5 mm in Eq. 6.7 gives:

$$\mathrm{CI} = K\frac{P^{1.3}}{B} \tag{6.7}$$

$$= 0.032\,\frac{(1.5 \times 205)^{1.3}}{5} = 11.0$$

and from Fig. 6.8, $(R_e)_f = 0.90$ and $(R_e)_c = 0.97$. So by Eq. 6.8a:

$$R_e = \frac{(11.0 - 7)}{10} \times 0.90 + \frac{17 - 11.0}{10} \times 0.97 = 0.94$$

Entering Table 14.1 with $1.5 < f < 2$ days for corn, which is a row crop, $k_f = 1.07$, and substituting into Eq. 14.1a gives:

$$d' = \frac{100 \times 1.07 \times 7.0}{90 \times 0.94 \times 1.00} = 8.9 \text{ mm}$$

Equation 14.1a can also be rearranged in two different ways to give the average peak daily gross depth of infiltration required:

$$d_i' = \frac{100 \times 1.07 \times 7.0}{90} = 8.3 \text{ mm}$$

or

$$d_i' = 0.94 \times 1.00 \times 8.9 = 8.3 \text{ mm}$$

The system capacity Q_s can now be determined by Eq. 14.2 for $T = 22$ hr/day and no end-gun, so that:

$$Q_s = Q_b + Q_g = \frac{(402)^2 \times 8.9}{1146 \times 22.0} + 0.0 = 57.0 \text{ L/s}$$

The average application rate, I', at the moving end of the lateral ($r_j = L$) can be computed by Eq. 14.4a as:

$$I' = \frac{2\ \pi\ 402}{20} \times \frac{8.3}{22.0} = 48\ \text{mm/hr}$$

Assuming the sprinklers all produce about the same width pattern, $I' = 48$ mm/hr will be the highest average application rate encountered along the lateral.

Both Q_s and I' could be decreased if the operating time during the peak use period were increased to $T = 24$/hr. This would remove the safety factor for downtime, but the potential for having any serious production loss would be minimal, assuming that repair services were available locally. Using $T = 24$ hr in Eq. 14.2 gives $Q_s = 52.3$ L/s, and using $T = 24$ hr in Eq. 14.4 gives $I' =$ 44 mm/hr.

Determining Infiltration Capacity

The average application rate $I' = 48$ mm/hr (1.89 in./hr) determined in Sample Calculation 14.1 is quite high. This is typical under the moving end of center-pivot laterals fitted with low-pressure sprinklers, even in relatively mild climates. Such high application rates often cause runoff on all but the most permeable soils. To reduce the potential for runoff, surface storage, *SS*, can be increased and/or light, frequent irrigation can be used. But the practical applications of these remedies is limited.

Figure 14.4 provides a useful guide for predicting how well a center-pivot system might perform in an area where there are no operating center-pivots to observe. However, a more rigorous approach for tailoring application rates to fit soil-intake capacities is needed except for systems on very sandy soils. To do this with a reasonable level of confidence requires specific infiltration data for the soils to be irrigated. This, in turn, requires conducting infiltration studies on the field to be irrigated.

Infiltration characteristics can be determined either by ponding or by sprinkling the soil surface. The ponding method is easier, but sprinkling appears more promising for simulating center-pivot irrigations. Conducting field infiltration tests using either method is time-consuming and tedious, so selecting test sites that can provide the most useful data is important. Infiltration tests should normally be conducted in the circular area served by the moving end of the lateral, where application rates will be highest. Two or three complete infiltration tests should be sufficient. The tests should be conducted on the dominant soils that appear (from past experience or from their textural composition, see Fig. 14.4) to be most problematic.

The methods for conducting ponded infiltration tests are well known and documented. Sprinkler infiltrometers, however, are less known or utilized, but they should give more realistic and useful data for center-pivot system design. Because they are not well known and are still in somewhat of an experimental or developmental stage, a brief description of them is warranted. But first a note of caution: whatever the method, field infiltration test data can represent only antecedent soil surface and moisture conditions at the time of the test and at best on a few spots in the field. Therefore, infiltration data are useful only as an indication of what might be encountered under actual operating conditions.

Two types of sprinkler infiltrometers have been developed. One developed by Beggs (1981) attempts to give a uniform application rate over a small area directly beneath a small rotating spray head. However, it has proven difficult to obtain uniform application rates over the small area wetted with this type of apparatus. The application rate is controlled by cycling a valve to give different percentages of on-time. A complete field infiltration test at a specific site consists of first preparing the soil surface so it appears as it would under an operating system. Then the apparatus is set up and operated to give some constant application rate, I, and the depth applied, and time at which the first signs of ponding begin to appear, T_p, is recorded. The water can be applied longer to obtain some impression of surface storage, and attempts can be made to measure runoff. But accurately measuring the runoff is very difficult.

After each data point (T_p, I) is obtained, either the apparatus must be moved to a nearby spot and the process repeated, or a few days of drying time are required before repeating the process at the same location. A minimum of three data points is needed for a complete test. Obviously, there will be problems with the consistency of the data, because either the soil will be different if the apparatus is moved, or the antecedent conditions will be different if repeated irrigations are applied to the identical test site.

Figure 14.7 shows a schematic of a lightweight and portable "revolving-sprinkler infiltrometer" developed by Reinders and Louw (1985). It has a self-

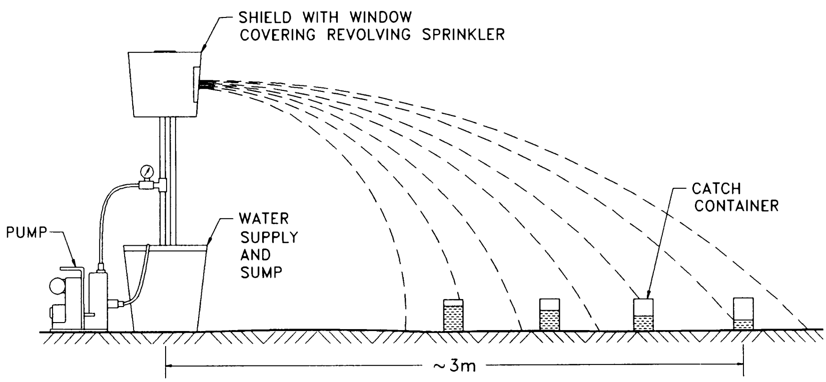


FIG. 14.7. Schematic of the Revolving-sprinkler Infiltrometer.

contained water supply and pumping system and a small revolving sprinkler that is covered by a shield to collect most of the water discharged. The shield has a narrow window through which the jet passes during each revolution. Because of the window, the jet produces a wedge-shaped application pattern when viewed from above. Viewed from the side, the distribution profile is triangular, with the application rate decreasing more or less uniformly as the distance from the nozzle increases. The radius of throw is a little over 3 m (10 ft), and the maximum effective $I > 100$ mm (4 in.) per hour.

A full infiltration test requires: preparing the test plot; setting up the apparatus with the shield's window facing downwind; placing a radial row of catch containers in front of the window; and recording the T_p and corresponding depth of catch, z, which is also equal to the depth infiltrated, d_i, as the "ponding front" progresses outward. The number of data points, (T_p, d_i) obtained depends on the number of catch containers. (The containers can be covered when the ponding front first reaches them and z recorded later.)

With the revolving-sprinkler infiltrometer all the data points needed for a complete infiltration test are conveniently obtained in rapid sequence from a single setup. But, each point is at a different location and thus representative of a slightly different soil condition.

The major problems with obtaining reliable sprinkler infiltration data are associated with preparing the test site to simulate field conditions under an operating center-pivot system. Experienced practitioners suggest elaborate test-site-preparation procedures. For example, it is recommended that cultivated soil be rather deeply wetted by hand-watering, irrigation, or rain at least twice and time allowed for the top 10 to 20 mm (0.4 to 0.8 in.) to dry before commencing infiltration tests. This whole process can take a week or longer. The reason for this procedure is to try to simulate whatever crusting might occur and typical antecedent soil moisture conditions that will be encountered by the center-pivot system.

Matching Application and Intake Rates

A plot of actual sprinkler-infiltrometer data obtained on a soil with approximately 30% sand, 55% silt, and 15% clay is shown in Fig. 14.8. Part A shows the data plotted as a function of accumulated depth infiltrated d_t at T_p for three different constant application rates (represented by the dotted lines). The curved line in part A represents the *envelope* of (T_p, d_i) data points resulting from different constant-application rates. This is not the same as the classic plot of time versus the depth of infiltration from ponded infiltration tests.

Part B of Fig. 14.8 is a plot of T_p for the three different constant application rates. The curved line in part B represents the (T_p, I) envelope resulting from the same three application rates as in Part A. The envelope is not the same as the classic plot of time versus the instantaneous or average infiltration rate, I,

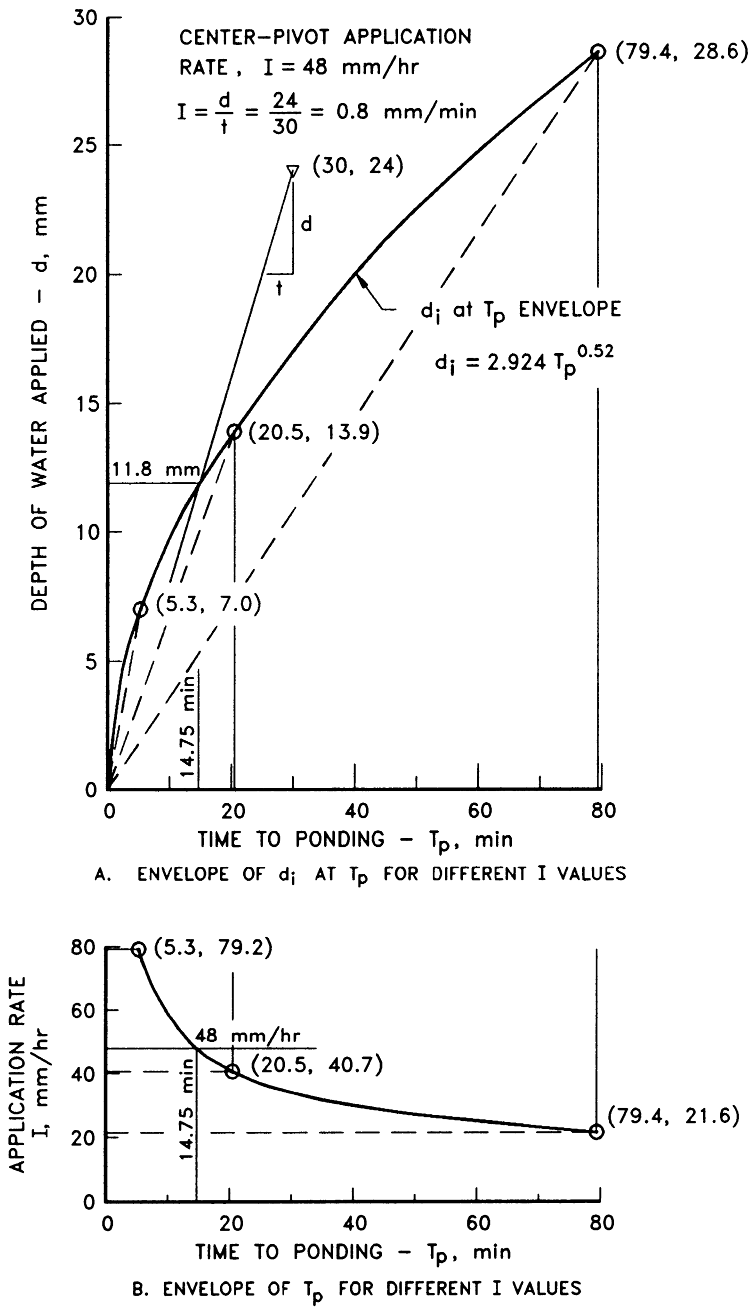


FIG. 14.8. Sprinkler Infiltrometer Time-to-ponding Envelopes.

data from ponded infiltration tests. Using "I" from ponded infiltration tests does not adequately account for "time" effects, which are related to soil swelling.

When designing center-pivot systems, estimating the depth, d_i, that will infiltrate while the lateral passes by is the main concern. Dealing with d_i gets directly to the heart of the problem. From field experience it has been demonstrated that for most soils the (d_i at T_p) envelope can be described by a simple exponential relationship:

$$d_i = k_p (T_p)^p \tag{14.6}$$

where

d_i = depth infiltrated for average sprinkle application rate, I, at time of ponding, T_p, mm (in.)
k_p = time-to-ponding coefficient dependent on soil and water characteristics at the time of the test and the measurement units used
p = time-to-ponding exponent dependent on soil and water characteristics at the time of the test

The most effective way to explain how to match center-pivot system application rate with expected soil-intake rates is by an example.

Sample Calculation 14.2. *Testing the suitability, based on soil-infiltration data, of a sprinkler configuration for a center-pivot system.*

GIVEN: The design information from Sample Calculation 14.1 and related findings plus the revolving-sprinkler infiltrometer test data plotted in Fig. 14.8. Assume the sprinkler pattern can be represented by its average application rate, I, and width, w.

FIND: The maximum depth of water, d_i, that can be applied without ponding and the corresponding travel speed, V_e, and center-pivot lateral cycle time, T_c.

CALCULATIONS: Figure 14.8A shows the depth infiltrated at the time-of-ponding (d_i at T_p) envelope for the three revolving-sprinkler infiltrometer test data points. These points are carefully plotted to scale. A linear regression of the logarithms of the data to determine the coefficients for Eq. 14.6 was made to define the d_i at T_p envelope curve. The regression coefficients for Eq. 14.6 are $k_p = 2.924$ and $p = 0.52$.

Next a line was drawn representing the center-pivot's average application rate $I' = 48$ mm/hr at the $r_j = L$ found in Sample Calculation 14.1. To position this line, the depth $d = 24$ mm that would be applied in 0.5 hr = 30 min was plotted, and a line was drawn from the origin to it. The $I = 48$ mm/hr application rate line intersects the d_i at T_p envelope at:

$$d_i = 11.8 \text{ mm}$$

$$T_p = 14.75 \text{ min}$$

For the pattern width $w = 20$ m to pass in $T_p = 14.75$ min, the travel speed must be:

$$V_e = \frac{20}{14.75} = 1.36 \text{ m/min}$$

And for a lateral with the moving end traveling 1.36 m/min at a point $L = 402$ m from the pivot, the cycle time is:

$$T_c = \frac{2\pi 402}{1.36 \times 60} = 31.0 \text{ hr}$$

Sample Calculation 14.2 was carried out graphically to help conceptualize the procedure. It can also be done numerically by first using linear regression (of the logarithms of the data) to determine the coefficients for Eq. 14.6. Then, let $d_i = T_p I$ in Eq. 14.6, and solve for T_p to obtain:

$$T_p = \left(\frac{60k_p}{I}\right)^{1/(1-p)} \tag{14.7}$$

Sample Calculation 14.3. *Comparison between the graphical and numerical computation of depth applied at time of ponding.*

GIVEN: Data given and developed in Sample Calculation 14.2.

FIND: Time of ponding and depth of water applied using the numerical method.

CALCULATIONS: First determine the coefficients for use in Eq. 14.6. This can be done by a linear regression (using hand calculators) of the logarithms of the sprinkler-infiltrometer data points to obtain $k_p = 2.924$ and $p = 0.52$. Substituting into Eq. 14.7 with $I = 48$ mm/hr gives:

$$T_p = \left(\frac{60 \times 2.924}{48}\right)^{1/(1-0.52)}$$

$$= 14.9 \text{ min}$$

and

$$d_i = \frac{T_p I}{60} = 11.9 \text{ mm}$$

These values are almost the same as those obtained graphically in Sample Calculation 14.2

Designing to Utilize Surface Storage

When sprinkling at a constant rate I is continued for longer than the T_p for the rate I, then ponding or runoff must occur (see Fig. 14.8). The amount of surface storage necessary to eliminate runoff is:

$$SS = \text{depth applied} - \text{depth infiltrated}$$

The shaded area depicted in Fig. 14.3 represents the SS needed to avoid runoff.

It is assumed that as long as I is within the infiltration envelope, as depicted in Figs. 14.3 and 14.8B, the water infiltrates into the soil at the same rate it is applied. After ponding begins, which occurs at the intersection of the application rate profile and the infiltration envelope, the quantity of water that infiltrates will follow a *ponded-infiltration curve.*

A ponded-infiltration curve is not shown in Fig. 14.8. However, a reasonable assumption is that, after passing the point where ponding beings, the (d_i at T_p) envelope represents the depth that will be infiltrated. Assuming this is true, then to avoid runoff:

$$SS_j = \frac{I_j (T_a)_j}{60} - k_p (T_a)_j^p \tag{14.8}$$

where SS_j = surface storage in region along circular path at radius r_j, mm (in.), and I_j = average application rate at radius r_j, mm/hr (in./hr), and $(T_a)_j$ = application time at radius r_j that will fully utilize SS_j, min.

Sample Calculation 14.4. *Managing center-pivots to take advantage of surface storage.*

GIVEN: The information presented and computed in Sample Calculations 14.1 and 14.2. The field to be irrigated is furrowed, and in critical areas near the outer reaches it has slopes of up to $s = 3\%$. Assume there is insignificant foliar storage, $SS_f = 0$, and the storage capacity of the microdepressions $SS_m = 2$ mm.

FIND: The average surface storage capacity, SS, of the area under study; and the maximum application depth, d, and lateral cycle time T_{cx} that will not cause runoff.

CALCULATIONS: First using Eq. 14.5 determine the average depth of applied water that can be stored on the soil surface after ponding begins:

$$SS \approx K\left(SS_m + SS_f + k\,\frac{(6 - s)(12 - s)^2}{144}\right) \tag{14.5}$$

$$SS \approx 1.0\left(2.0 + 0.0 + 0.5 \times \frac{(6 - 3.0)(12 - 3.0)^2}{144}\right)$$

$$\approx 2.84 \text{ mm}$$

The application time $(T_a)_J$, that will take full advantage of the available surface storage can now be computed by Eq. 14.8, letting $SS_j = 2.84$ mm:

$$SS_J = \frac{I_J(T_a)_J}{60} - k_p(T_a)_J^p \tag{14.8}$$

$$2.84 = \frac{48(T_a)_J}{60} - 2.924(T_a)_j^{0.52}$$

Assume $(T_a)_J = 20$; then check to find:

$$2.84 \neq 16.0 - 13.88 = 2.12$$

Continue by trial and error to find:

$$(T_a)_J \approx 21.5 \text{ min}$$

For the pattern width $w = 20$ m to pass in 21.5 min:

$$(V_e)_J = \frac{20}{21.5} = 0.93 \text{ m/min}$$

Thus, with $r_J = L = 402$ m the maximum cycle time that will not cause runoff is:

$$T_{cx} = \frac{2\,\pi\,402}{0.93 \times 60} = 45.3 \text{ hr}$$

In Sample Calculation 14.2, $T_c = 31.0$ hr without taking surface storage into account. This shows the importance of a rather small amount of surface storage for significantly reducing the operating speed without causing runoff.

The PET Index

A system parameter called the PET index can be used to simplify performance analysis for transferring infiltration capacity evaluations. It is equal to the average water-application rate at the outer edge of the basic irrigated circle (see Fig. 14.1) for a specific climate, crop, and set of design configurations, so:

$$\text{PET} = I' = \frac{200\ \pi\ r_j k_f k_c ET}{w_j T\, DE_{pa}}; \qquad \text{for } r_j = L \tag{14.9}$$

where

PET = an index dependent on the center-pivot lateral length, nozzling configuration, daily operating time, and application uniformity, mm/hr (in./hr)
I' = required average application rate at radial distance $r_j = L$, mm/hr (in./hr)
r_j = radius to point under study, m (ft)
k_f = irrigation frequency factor to adjust standard crop water-use values for high-frequency irrigations (taken from Table 14.1), decimal
k_c = crop coefficient to relate the evapotranspiration of a specific crop to that of the reference crop ET, decimal
L = length of center-pivot lateral and basic irrigated circle, m (ft)
ET = reference crop evapotranspiration during peak crop-use period, mm (in.)
w_j = wetted width of water pattern at r_j, m (ft)
T = average daily operating time during peak-use period, hr
DE_{pa} = distribution efficiency based on adequately irrigating a given desired percentage, *pa*, of the field, %

The PET index is the average application rate at the moving end of a center-pivot lateral. Equation 14.9 combines the sprinkler configuration, climate, crop, and system management parameters. The maximum lateral cycle time, T_{cx}, that does not cause runoff at $r_j = L$ relates the PET index to a specific set of soil-site conditions. On soils with similar infiltration and surface storage characteristics, as long as the average application rate at all points along the lateral, $I_j \leq$ PET, runoff will not occur if $T_c \leq T_{cx}$.

The value of I_j at any radial distance r_j from the pivot for any length of lateral or nozzling configuration can be determined by Eq. 14.9. Many of the variables in Eq. 14.9 are constant near a given location. Therefore, I_j can be computed by multiplying the PET index by the appropriate ratio of any specific variable changes. For example, to find I_j for any r_j, assuming the same values of the PET index and w_j along the lateral, I_j = PET r_j/L.

Sample Calculation 14.5. *Using the PET Index and T_{cx} in a center-pivot lateral design strategy.*

GIVEN: The center-pivot system and design computations from Sample Calculations 14.1, 2, and 3, for which:

The lateral length, $L = R = 402$ m;
The daily water use rate, $k_f U_d = k_f k_c ET = 7.49$ mm;
The pattern width, $w_j = 20$ m at $r_j = L$;
The daily operating time, $T = 22$ hr; and
The distribution efficiency, $DE_{pa} = 90\%$ for $pa = 80\%$.

FIND: The point along the lateral where a single row of spray heads could be substituted for the low-pressure impact sprinklers and what type of sprinkler configuration would be required for a lateral that irrigates a basic circle with twice the area but has the same maximum lateral cycle time, $T_{cx} = 45.3$ hr.

CALCULATIONS: From Sample Calculation 14.1, $I' = 48$ mm/hr at $r_j = L = 402$ m. This is the same as the PET index, which could be computed directly by Eq. 14.9 to obtain:

$$\text{PET} = \frac{200\ \pi\ 402 \times 7.49}{20 \times 22 \times 90} = 48 \text{ mm/hr}$$

The radial distance r_j where a sprayer pattern with $w_j = 12$ m gives the same PET Index can be found by rearranging Eq. 14.9 to obtain:

$$r_j = 402 \times 12/20 = 241 \text{ m}$$

The pattern width required at the moving end of a lateral that irrigates twice as much area can be found in a similar manner by first finding the required length:

$$L_2 = \left[2(402)^2\right]^{1/2} = 569 \text{ m}$$

and then substituting into and rearranging Eq. 14.9 to obtain:

$$w_j = \frac{20 \times 569}{402} = 28 \text{ m}$$

From Table 14.2 it appears that medium-pressure impact sprinklers with either variable or semiuniform spacings would produce pattern widths ranging between 27 and 37 m and satisfy the necessary criteria.

SCHEDULING AND OPERATION

Ordinarily the irrigation interval during the peak crop-use period is determined by dividing the net depth of water applied, d_n, by the average daily crop-water-use rate during that period. For periodic-move systems it is desirable to have d_n as large as possible without exceeding some relatively high management-allowed deficit, MAD. This is done to minimize the number of lateral or sprinkler moves and thus reduce labor and management costs. The resulting maximum allowable d_n is directly proportional to the MAD times the water-holding capacity of the soil and plant root depth; see Eq. 3.1 and Tables 3.1, 3.2 and 3.4.

Irrigation Interval Limits

Management and labor costs are little affected by the number or frequency of irrigations or by what time of day center-pivot systems are started or stopped. Thus, there is more flexibility for scheduling them except for the limitations associated with getting the desired d_n infiltrated into the soil. On the coarser textured soils with high infiltration but low moisture-holding capacities, scheduling can and usually should be based on a combination of Eqs. 3.1 and 3.2.

The values of MAD given in Table 3.4 are for periodic-move systems and compromise productivity to save on labor. Under center-pivot irrigation, the soil-water deficit varies by $\pm d_n$ around the average deficit, but d_n is small because the cycle time is normally $1 < T_c < 4$ days. Therefore, for center-pivot irrigation, the concept of an average management-allowed deficit, MAD_a, will be used rather than an absolute MAD.

Crops grown on medium- and fine-textured soils with $W_a > 120$ mm/m (1.5 in./ft) should be most productive per unit of both water and land with an $\text{MAD}_a \approx 25\%$. Therefore, when designing center-pivot systems for use on *medium- and fine-textured soils* let $\text{MAD}_a = 20\%$ to compute the potential irrigation interval based on soil water shortage. However, for *coarse-textured (sandy) soils* with $W_a < 120$ mm/m (1.5 in./ft), most of the W_a is readily available, so let $\text{MAD}_a = 30\%$ to obtain a longer irrigation interval.

The maximum irrigation interval that will satisfy the desired MAD_a values can be computed by:

$$(f_x)_s = \frac{0.24\ \text{MAD}_a W_a Z}{k_f U_d} \tag{14.10}$$

where:

$(f_x)_s$ = maximum irrigation interval during peak-use period based on usable soil water storage and plant stress, hr

MAD_a = average management-allowed deficit for managing center-pivot irrigation, %

W_a = available water-holding capacity of the soil, which can be estimated from Table 3.1, mm/m (in./ft)

k_f = frequency factor to adjust standard crop water-use values for high-frequency irrigation, decimal
Z = plant root depth, which can be taken from Table 3.2, m (ft)
U_d = conventionally computed average daily consumptive-use rate during the peak-use month, mm/day (in./day)

The average gross depth of water that must be infiltrated per center-pivot lateral cycle with interval $(f_x)_s$ from Eq. 14.10 is:

$$(d_i)_s = (f_x)_s d_i'/24 \tag{14.11}$$

where $(d_i)_s$ = gross depth of water that must be infiltrated per irrigation interval based on soil water storage, mm (in.), and d_i' = average peak gross depth of infiltration required per day, mm (in.).

Often the depth of water that can be applied without causing runoff is less than $(d_i)_s$ because of limitations due to the infiltration capacity of the soil and a limited *SS*. In fact this is quite often so except for the coarser textured soils.

Where $(d_i + SS)$ is the limiting factor, the frequency of irrigation is computed by:

$$(f_x)_i = T_{cx}\, 24/T \tag{14.12}$$

where:

$(f_x)_i$ = maximum irrigation interval during the peak-use period based on infiltration capacity, hr
T_{cx} = maximum (longest) lateral cycle time that does not cause runoff, hr
T = average daily operating time during peak-use period, hr/day

Anticipating Rain

In regions where rainfall is significant during the irrigation season, center-pivot systems should be managed to allow soil water-storage space for it. Often this can easily be done with center-pivot systems without compromising yield by delaying irrigation until MAD_a has been reached. Then for the follow-up irrigations, f_x can be set by the irrigation interval as limited by infiltration, $(f_x)_i$, or one of the other criteria that follow. Thus, the soil water content can be managed to range between the lower limit set by MAD_a and an upper limit that still leaves some extra storage capacity for rainwater. However, for very shallow rooted crops or coarse-textured soils, $(f_x)_s$ may be so low that leaving capacity to store rainwater is not feasible or advisable.

The frequency of irrigation that would leave sufficient storage capacity for expected rains during the irrigation season is:

$$(f_x)_r = (f_x)_s - \frac{24R'_n}{U_d} \tag{14.13}$$

where $(f_x)_r$ = maximum irrigation interval during the peak-use period based on leaving storage for rainfall, hr, and R'_n = effective depth of anticipated rainfalls within any 2-day interval during the peak-use period, mm (in.).

Scheduling Criteria

The following scheduling criteria should be followed in setting the maximum irrigation interval during the peak-use period, f_x. This is necessary to avoid runoff as well as to more or less optimize production while being practical:

1. The f_x should not exceed the limit set by soil moisture stress, $(f_x)_s$, as computed by Eq. 14.10.
2. The f_x should not exceed the limit set by infiltration capacity, $(f_x)_t$, as determined by Eq. 14.12.
3. To allow for the effective use of rain, f_x should not exceed the limit set by the effective rainfall, $(f_x)_r$, as determined by Eq. 14.13.

Additional criteria:

4. The minimum cycle time for a standard 400-m (1300-ft.) long lateral is usually about $T_c \approx 22$ hr, so the interval cannot be less than about 24 hr.
5. Actually whenever $(f_x)_t < 36$ hr and is limiting, a different sprinkler configuration should be selected to increase the interval to more than 36 hr. This is recommended because very-high-frequency irrigation increases water loss, i.e., k_f in Eq. 14.1, and machinery wear. Furthermore, it often decreases production due to disease, poor root development, etc.
6. For various cultural, machinery service, and other reasons, many farm managers prefer to cycle their center-pivot laterals about twice per week during the peak-use period. Therefore, they may set their own upper limits on f_x.
7. The time of day a given spot receives irrigation should progressively change by at least 4 hr on each subsequent irrigation. This staggers the time of day each part of the field is irrigated. This in turn helps smooth out the effects of the weather variables on drift and evaporation losses and application uniformities.

Sample Calculation 14.6 *Determine the maximum irrigation interval for a center-pivot system during the peak crop-use period.*

GIVEN: The data presented for and findings from Sample Calculations 14.4 and 14.5.

The soil has 30% sand, 55% silt, and 15% clay.

Sweet corn with an effective root depth $Z = 0.6$ m will be grown (see Table 3.2.)

Effective rainfall events during the peak-use period are such that $R_e' = 10$ mm.

The peak average daily-water-use rate, $k_f U_d = 1.07 \times 7.0 = 7.5$ mm/day.

FIND: The longest irrigation interval that will satisfy both soil-moisture storage and infiltration limitations during the peak-use period and the recommended f_x and operational (scheduling) procedure.

CALCULATIONS: From Fig. 14.4 the soil is a silty loam and falls in the region of "good" anticipated center-pivot performance. From Table 3.1 the average $W_a = 167$ mm/m; thus; according to the scheduling criteria, $\text{MAD}_a = 20\%$. Using Eq. 14.10 to compute the irrigation interval gives:

$$(f_x)_s = \frac{0.24 \times 20 \times 167 \times 0.6}{7.5}$$

$$= 64 \text{ hr}$$

From Sample Calculation 14.4, $T_{cx} = 45.3$ hr. Using Eq. 14.12 to compute the irrigation interval with an average daily operating time $T = 22$ hr and letting $T_{cx} = 45.3$ gives:

$$(f_x)_t = 45.3 \times 24/22 = 49 \text{ hr}$$

Because $(f_x)_t < (f_x)_s$, the maximum irrigation interval should not exceed 49 hr during the peak-use period when $k_f U_d = 7.5$ mm/day. For a 49-hr interval, the system should be shut down for about 4.0 hr between cycles to avoid over-irrigation. This is necessary because the design system capacity is based on operating only 22 hr per day to allow for contingencies. During actual operation, irrigation should be scheduled to allow room for effective rainfall and other criteria. The lateral may be rotated faster, but should not be rotated slower than $T_{cx} \approx 45$ hr, to avoid runoff.

The irrigation interval that allows sufficient storage for rain is determined by Eq. 14.13 using $R_e' = 10$ mm to obtain:

$$(f_x)_r = 64 - (24 \times 10/7.5) = 32 \text{ hr}$$

Using a convenient 48-hr irrigation interval would satisfy all scheduling criteria except criteria 3 and 7. With a 48-hr interval, each part of the circular field would always receive its irrigation application at the same time of day. To leave sufficient soil-storage capacity to accommodate all the expected rain, the interval would need to be reduced to $(f_x)_r = 32$ hr. Rather than use such a short interval, a compromised value of f_x = 40 hr was selected.

To leave room to store rainfall, delay irrigation after a rainfall event until the soil moisture depletion (wherever the lateral is when it rains) equals:

$$LD \approx W_a Z \,\mathrm{MAD}_a/100 \qquad (14.14a)$$

$$\approx 167 \times 0.6 \times 20/100 \approx 20 \text{ mm}$$

A rainfall event, as depicted in Fig. 14.9, that leaves no soil moisture deficit, SMD = 0, requires a time lag of approximately:

$$LT \approx 24\, LD/(k_f U_d) \qquad (14.14b)$$

$$\approx 24 \times 20/7.5 = 64 \text{ hr}$$

where LD = lag depth of water to be consumed before commencing irrigation after rainfall events that fill the soil profile, mm (in.), and LT = lag time before commencing irrigation after rainfall events that fill the soil profile, hr.

Irrigation should begin when $LD \approx 20$ mm and the irrigation interval between subsequent lateral cycles set at approximately f_x = 40 hr. The net depth of soil water consumed during the interval between irrigations, which should be applied by each irrigation, will be:

$$d_n = k_f U_d f_x/24 \qquad (14.15)$$

$$= 7.5 \times 40/24 = 12.5 \text{ mm}$$

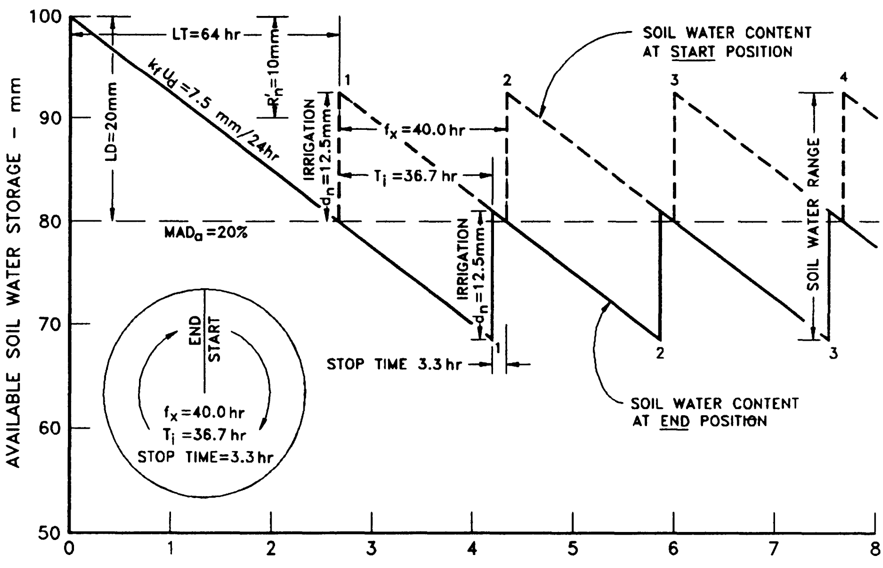


FIG. 14.9. Soil-moisture-depletion Profile for a Center-pivot Irrigation Schedule that Allows for Rainwater Storage.

This will produce a soil-moisture-depletion pattern at the beginning lateral position as depicted in Fig. 14.9. For f_x = 40 hr, the lateral cycle time, T_c, should be based on the average daily operating time T = 22 hr that was used in determining the system capacity. Therefore:

$$T_c = (T/24)f = (22/24)40 \approx 36.7 \text{ hr}$$

And the downtime between lateral cycles should be approximately 3.3 hr.

That d_n = 12.5 mm for T_c = 36.7 hr can be easily verified by the following equation:

$$d_n = \frac{K\, Q_s T_c D E_{pa} R_e O_e}{\pi R^2} \tag{14.16}$$

where K = conversion constant 36 for metric units (0.963 for English units), which from Sample Calculation 14.1 is:

$$d_n = \frac{36 \times 57.0 \times 36.7 \times 90 \times 0.94 \times 1.0}{\pi(402)^2}$$
$$= 12.5 \text{ mm}$$

The top "teeth" in Fig. 14.9 represent the soil water content at the starting position, which is replenished by four irrigations as indicated. The sloping dashed lines represent the depleting soil water content at the lateral starting position.The lower teeth in Fig. 14.9 represent the irrigation applications as the lateral approaches the end position and the subsequent depleting soil water content there. The time gap between the starting and ending positions represents the length of time between lateral rotations (or the stop time), which is equal to $(f_x - T_c)$ = 3.3 hr.

Figure 14.9 shows the range of soil water contents. It is almost equal to 2 d_n, and the average soil water deficit is approximately 19%. This is a little less than MAD_a = 20%, because $T_c < f_x$. There is about an 8-hr period at the beginning of each lateral cycle where the available water-storage capacity is not quite sufficient to hold R_e = 10 mm, but this is insignificant. For each lateral cycle there is an 8-hr time shift. Thus, each irrigation application is received at a different time of day. This progressive time shift will improve the average application uniformity significantly compared with no time shift.

Setting the Travel Speed

The lateral travel speed for electric-drive machines is controlled by the percentage on-time, PT, of the end drive unit. (For hydraulic-drive machines it is set by a flow-control valve.) For scheduling purposes it is useful to know the relationships between T_c, d_n, and V_e as a function of PT (or hydraulic fluid-flow rates). Obviously these are all proportional to the minimum lateral cycle time, $(T_c)_n$, for 100% on-time (or 100% hydraulic fluid-flow rate), such that:

$$T_c = (T_c)_n 100/PT \tag{14.17}$$

and:

$$d_n = \frac{100(T_c)_n k_f U_d}{PT\ T} = \frac{f_x k_f U_d}{24} \tag{14.18}$$

and:

$$V_e = \frac{PT \pi r_e}{3000(T_c)_n} \tag{14.19}$$

where:

PT = percentage on-time (or percentage of maximum hydraulic fluid-flow rate) at end drive unit, %
$(T_c)_n$ = minimum lateral cycle time with PT = 100%, hr
V_e = travel speed of end drive unit, m/min (ft/min)
r_e = radial distance from pivot to end-drive unit, m (ft)

Sample Calculation 14.7. *Controlling center-pivot lateral cycle time.*

GIVEN: The information in Sample Calculation 14.6 for an electric-drive center pivot.

The minimum lateral cycle time $(T_c)_n = 22$ hr.

The distance from the pivot point to the end drive unit, $r_e = 398$ m.

FIND: The required velocity, V_e of the end drive unit, such that $T_c = 36.7$ hr; and the net depth of application, d_n, when $V_e = 1.00$ m/min or when $PT = 100\%$

CALCULATIONS: By Eq. 14.17, the percentage on-time that will result in a $T_c = 36.7$ hr, as required in Sample Calculation 14.6, should be:

$$PT = (22 \times 100)/36.7 = 60.0\%$$

And the velocity of the end drive unit by Eq. 14.19 should be:

$$V_e = \frac{60\ \pi\ 398}{3000 \times 22} = 1.14 \text{ m/min}$$

The percentage on-time required to obtain $V_e = 1.00$ m/min can be determined by Eq. 14.19 as:

$$PT = \frac{1 \times 3000 \times 22}{60\ \pi\ 398} = 53\%$$

By Eq. 14.18, the net depth of application for $PT = 53\%$ is:

$$d_n = \frac{100 \times 22 \times 7.5}{53 \times 22} = 14.2 \text{ mm}$$

And for $PT = 100\%$ it would be:

$$d_n = \frac{100 \times 22 \times 7.5}{100 \times 22} = 7.5 \text{ mm}$$

SPRINKLER-SELECTION

The general strategy for selecting the sprinkler and nozzle sizes along the lateral is to:

1. Determine the discharge required for each sprinkler to apply a uniform application of water to the irrigated area;
2. Determine the minimum sprinkler operating pressure P_n (or pressure head H_n) that will give satisfactory performance for the sprinkler type and nozzling configuration selected;
3. Determine the pressure available at each sprinkler based on starting at either end with a reasonable design pressure that will satisfy P_n; and
4. From the required discharge and available pressure, select an appropriate sprinkler and nozzle size for each outlet. Because the sprinklers are closely spaced relative to their wetted pattern diameters, only the running average discharge per sprinkler needs to equal the sum of their "required discharges."

Sprinkler Discharge

The required discharge per sprinkler at any outlet along the lateral serving the basic irrigated circle is a function of r_j. It should be equal to the fraction of Q_b that is equivalent to its proportionate share of the irrigated area; thus:

$$q_j = \frac{2 r_j S_j}{L^2} Q_b \qquad (14.20a)$$

A part-circle sprinkler is often used to "square-off" and extend the basic irrigated circle up to an additional 3% beyond the end of the lateral pipeline. The discharge for such a part-circle sprinkler should be:

$$q_g = \frac{2L'\Delta L}{L^2} Q_b; \qquad \text{for } \Delta L < 0.03L \qquad (14.20b)$$

where:

q_j = the required discharge at radius r_j, L/s (gpm)
r_j = any radial distance from the pivot, m (ft)
S_j = sprinkler spacing at r_j (which is equal to the average distance to the adjacent up- and downstream sprinklers), m (ft)
Q_b = discharge to basic irrigated circle, L/s (gpm)
L = radius irrigated in basic circle (when corner system or large end-gun is either absent or not operating), m (ft)
q_g = discharge of part-circle sprinkler used to square-off or extend the basic irrigation circle, L/s (gpm)
L' = actual length of lateral pipe, m (ft)
ΔL = $(L - L')$, m (ft)

End-Gun Discharge

Figure 14.10 shows an operating end-gun on the moving-end of a center pivot lateral. The capacity of end-guns is treated separately from that of the basic irrigated circle in Eq. 14.2. This is done because their application efficiencies are normally much lower than the application efficiencies across the basic circular portion of the field. Three phenomena contribute to the lower efficiencies of end-guns: their wetting patterns are subject to distortions from winds; their application profiles are sensitive to the wetted sector angle and other mechanical

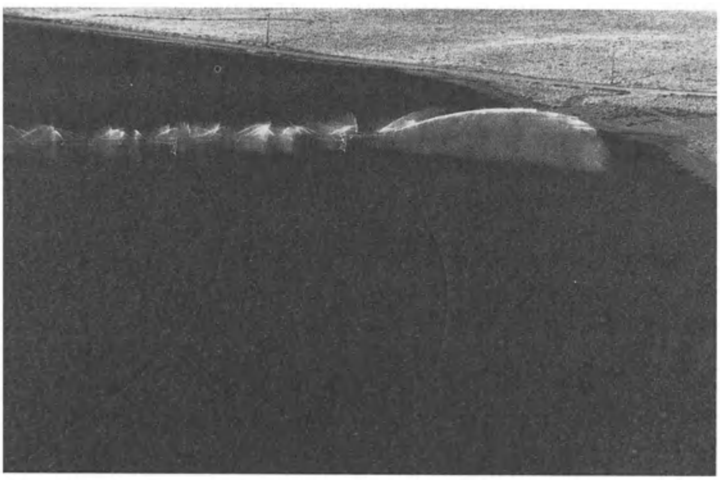


FIG. 14.10. Center-pivot End-gun in Operation (Source: Nelson Irrigation Corp.).

factors; and their application profiles taper off on the outer edge, so that a considerable quantity of water is discharged beyond the sufficiently irrigated area.

Typically the application uniformity achieved by end-guns operating with optimum wetted sector angles and pressures is equivalent to CU $\approx$ 76%. Under ideal conditions the application profile in the radial direction beyond the end-gun will be as follows: for the first 50 to 60% of the range of throw the profile will be approximately uniform; then it will taper off more or less uniformly to zero. In general it can be assumed that irrigation depths of less than half of the desired net depth are ineffective. Thus, an end-gun will increase the radius of the effectively irrigated area by about 75 to 80% of its range of throw, R_j. The water thrown beyond this effective radius, R, will be ineffective and thus wasted (see Fig. 14.1). For the above assumptions, this wasted water will be about 6 to 8% of the end-gun's discharge, Q_g, so $O_e \approx 0.93$ for the area irrigated by the end-gun.

Based on the above concepts, the desired capacity of an end-gun can be determined by:

$$(Q_g) = \frac{(R^2 - L^2)}{KT} d'_g \tag{14.21}$$

where

(Q_g) = desired discharge from end-gun, L/s (gpm)
R = radius of area sufficiently irrigated when end-gun (or corner system) is in operation, m (ft)
L = length of lateral or radius irrigated in basic circle when end-gun (or corner system) is not operating, m (ft)
K = conversion constant, 1146 for metric units (30.6 for English units)
T = average daily operating time (with or without the end-gun in operation), hr
d'_g = average daily gross depth of water application required in the area irrigated by the end-gun during the peak water-use period, mm (in.)

The d'_g can be determined using Eq. 14.1 by replacing d' with d'_g and letting $DE_{80} \approx 75\%$ and $O_e \approx 0.93$ and determining R_e using Eq. 6.8. Usually, $(R - L) \approx 0.75 R_j$, where R_j is the radius of throw of the end-gun.

End-Gun Operation

Some center-pivot systems are designed for the end-guns to operate continuously. In such cases the area irrigated by the gun sprinkler is equal to the difference between the areas of the circles with radius R and L. From the standpoint of application efficiency and operating cost, continuously operating, large end-guns are not recommended. It is preferable to use longer laterals with small end-guns so $(R - L)$ is only 6 to 12 m (20 to 40 ft).

Corner Area. Most often large end-guns are used to extend the irrigated area further into the corners of the field as depicted in Fig. 14.1. For such cases the additional area irrigated in each corner is:

$$A/C = \frac{90° - 2\cos^{-1}(L/R)}{90°} \cdot \frac{\pi(R^2 - L^2)}{4\,K} \tag{14.22}$$

where A/C = additional area irrigated in a given corner by an end gun, ha (acre), and K = conversion constant, 10,000 for metric units (43,560 for English units).

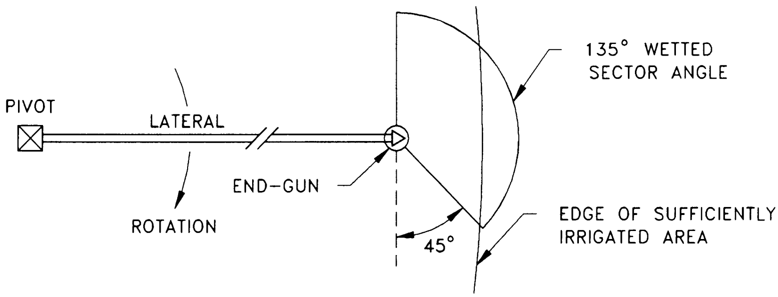
The geometry of the length and width of the crescent-shaped areas irrigated in each corner by the end-gun is shown in Figure 14.1

Wetted Sector Angle. The water-application profiles produced by gun sprinklers traveling along towpaths (see Chapter 13) and on the ends of center-pivot laterals are similar. The gradually curving path of the end-gun makes little difference to the resulting application patterns compared to traveling in a straight path. This is because the additional radius irrigated is small compared with the length of the lateral.

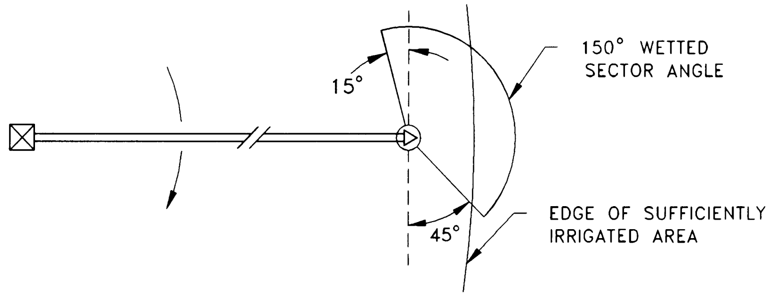
With the wetted sector angle set between 240 and 270°, traveling-gun sprinklers produce the most uniform application profile on both sides of the towpath (see Fig. 13.3). Thus, for a center pivot the wetted portion should be the equivalent, which is between 120 and 135°, because the end-gun irrigates only the strip to the outside of its curvilinear travel path. To reduce the application rate the wider 135° angle is preferred, which should be set as indicated in Fig. 14.11.

Figure 14.11A shows the recommended wetted sector angle settings for large end-guns that are operated only as the lateral passes the corners. Figure 14.11B shows the settings for small end-guns that will be operated continuously. The extra "inside" 15° segment in Fig. 14.11B is to fill in water deficits near the edge of the basic circle. The deficits result from not having downstream sprinklers to provide overlapping watering patterns. However, when a large end-gun (see Fig. 14.10) is used as in Figs. 14.1 and 14.11A, to irrigate only in corners, a smaller part-circle sprinkler mounted at the end of the lateral should be designed to operate continuously to fill in this deficit.

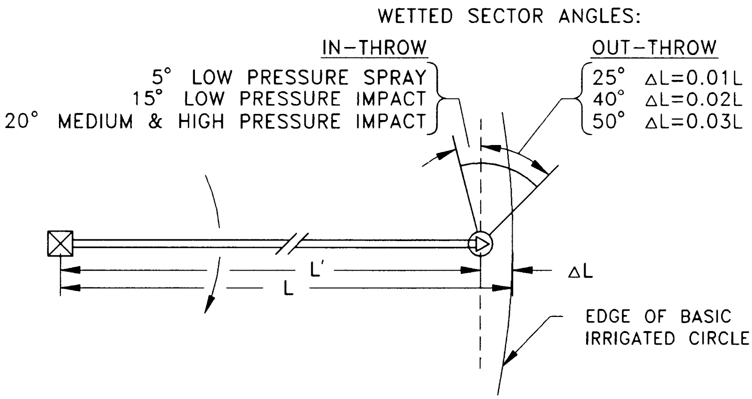
As mentioned earlier, small part-circle sprinklers are often used to square-off the radial application profile and irrigate a few meters beyond the end of the lateral pipe. Full irrigation for a short distance beyond the end of the lateral is often needed. It increases L while keeping the lateral pipe length, L', short enough to maintain sufficient clearance along fence rows, between adjacent



A. END-GUNS WHICH ARE ONLY OPERATED IN THE CORNERS



B. SMALL END-GUNS OPERATING CONTINUOUSLY



C. PART-CIRCLE SPRINKLERS USED TO SQUARE OFF BASIC IRRIGATED CIRCLE

FIG. 14.11. Plan Views of Center-pivot End-gun Wetting Patterns Showing Recommended Wetted Sector Angles.

pivots, etc. When part-circle sprinklers are used in this manner, they should be considered part of the basic irrigated circle, and their discharge, q_g, should be computed by Eq. 14.20b and included as part of Q_b. The recommended wetted sector angle settings for such sprinklers is shown in Fig. 14.11C.

Sample Calculation 14.8. *Evaluating the performance of an end-gun for a center-pivot lateral.*

GIVEN: The center-pivot specification and design information presented in Sample Calculation 14.1.

A gun sprinkler with an orifice-type nozzle having an equivalent tapered nozzle diameter of 25.4 mm (1.0 in.) operating at 415 kPa (60 psi).

From Table 13.1 the expected wetted diameter for a tapered nozzle is 99 m (325 ft). It will be about 5% less, or 94 m (308 ft), for the ring-type nozzle that is recommended to increase jet breakup.

From Table 13.1 the expected discharge is, $G_g = 14.2$ L/s (225 gpm).

The drop coarseness index CI < 7; and $R_e = 0.97$.

FIND: The total system capacity; the anticipated performance of the end-gun sprinkler; and evaluate another potential nozzle size.

CALCULATIONS: Substituting the actual sprinkler discharge of 14.2 L/s in Eq. 14.2 for Q_g and noting that $Q_b = 57.0$ from Sample Calculation 14.1 gives:

$$Q_s = Q_b + Q_g$$

$$= 57.0 + 14.2 = 71.2 \text{ L/s}$$

This is an increase of 25% over the capacity required to irrigate the basic circle.

Next, check to determine how well the actual discharge matches the *desired* discharge from the end-gun. From Sample Calculation 14.6. $(k_f U_d) = 7.5$ mm/day, and it is reasonable to assume $(R - L) \approx 0.75R_j$, $Q_e = 0.93$; and CU $\approx$ 76%. From Table 6.2, $DE_{80} = 75\%$ for a desired percentage adequacy $pa = 80\%$ over the end-gun irrigated area. Letting $d' = d'_g$ in Eq. 14.1a, the gross daily irrigation depth should be:

$$d'_g = \frac{100 \times 7.5}{75 \times 0.97 \times 0.93} = 11.1 \text{ mm}$$

The radius of the area sufficiently irrigated when the end-gun is operating is:

$$R = L + 0.75R_j$$

$$= 402 + 0.75 \times 94/2 = 437 \text{ m}$$

And by Eq. 14.21 the desired end-gun discharge is:

$$(Q_g) = \frac{[(437)^2 - (402)^2]11.1}{1146 \times 22.0}$$

$$= 13.9 \text{ L/s}$$

This is somewhat less than the actual sprinkler discharge of 14.2 L/s at the design pressure of 415 kPa. (Because only a limited selection of nozzle sizes is available, it is often impossible to exactly match the end-gun to the specific discharge required.)

The additional area sufficiently irrigated in each of the four corners by the end-gun can be computed using Eq. 14.22 to obtain:

$$A/C = \frac{90° - 2\cos^{-1}(402/437)}{90°} \cdot \frac{\pi[(437)^2 - (402)^2]}{4 \times 10{,}000}$$

$$= 0.487 \times 2.306 = 1.12 \text{ ha}$$

Therefore, the total added area in all four corners is 4.5 ha.

In summary, the end-gun will be operating only about half the time, and it will increase the irrigation area from 50.8 ha to 55.3, or about 9%. However, the additional system capacity required to operate the end-gun is about 25%. A pump will be required to boost the pressure at the end of the lateral by about 210 kPa to operate the end-gun, and this will require a 5-hp motor.

Perhaps *a better nozzle size selection* would have been an orifice-type nozzle which is equivalent to a tapered nozzle having a diameter of 22.9 mm (0.9 in.). Interpolating between the 0.8- and 1.0-in. tapered nozzles in Table 13.1 and carrying out the calculations as above, the actual discharge and irrigated radius would be $Q_g = 11.5$ L/s and $R = 434$ m. The desired discharge computed by using Eq. 14.21 would be:

$$(Q_g) = \frac{[(434)^2 - (402)^2]11.1}{1146 \times 22.0}$$

$$= 11.8 \text{ L/s}$$

This is also almost equal to the actual discharge, but on the low side. Thus, it may be a better selection as it will cause less surface sealing and reduce Q_s. The extra area sufficiently irrigated would be 4.3 ha, which is only slightly less than with the larger nozzle.

The effect of the desired discharge being slightly less than the actual discharge would be to reduce the adequacy of irrigation below $pa = 80\%$. The amount of reduction can be determined by finding the DE_{pa} value that would make the actual and desired end-gun discharge equal. Based on the relationships in Eqs. 14.1a and 14.21 it would need to be increased to:

$$DE_{pa} = 75 \times \frac{11.8}{11.5} = 77\%$$

And from Table 6.2 for CU $\approx$ 76%, the $pa \approx 78\%$ instead of the desired $pa = 80\%$, which is only a small difference. In view of the above, the smaller nozzle is recommended. This gives $Q_s = 57.0 + 11.5 = 68.5$ L/s.

CENTER-PIVOT LATERAL HYDRAULICS

The hydraulics of center-pivot laterals is similar to that of regular periodic-move laterals, the main differences being that:

1. Because the lateral irrigates a circular field, the discharge per unit length must increase proportionally with the distance, r_j, from the pivot;
2. For both the uniform discharge and semiuniform spacing sprinkler configurations, the spacing between sprinklers, S_j, is not constant;
3. For the uniform and semiuniform spacing configurations the sprinkler and nozzle size is not constant; and
4. Often the radius of the field irrigated is considerably greater than the length of the lateral pipeline, $R > L'$.

In view of these differences a special set of modified equations for multiple-outlet pipelines is necessary for center-pivot lateral design.

There are three ways for designers to analyze the hydraulic relationships along center-pivot laterals. The most intuitive way is simply by stepwise calculation. For example, to determine the friction loss along the lateral, the flow rate and friction loss in each section between outlets can be computed using Eqs. 14.24 and 8.1, respectively. The pipe-friction loss for the entire lateral can then be determined by summing up the incremental losses. Carrying out this process is very tedious and time-consuming, using hand (or manual) calculations, but it can be done quickly and simply with a computer.

A second way to carry out the analysis is to first do it stepwise using a computer as above for a standard lateral. Then find a direct dimensionless numerical solution that fits the results by using regression analysis. A third method is to find a direct numerical solution that is based on the theoretical relationships involved. For the most part this text will cover the analytical methods based directly on the theoretical relationships to enhance understanding of the subject. But in practice, the stepwise solutions are often the most straightforward and practical, assuming a computer and the skill to use it are at hand.

Flow Rate and Friction Loss

Lateral flow rate and friction loss computations are all based on relationships that include the equivalent "hydraulic length" of the lateral. This is true for all computations for manifold pipelines serving rectangular or circular fields. For laterals (without end-guns) that serve only a basic circular field, the equivalent hydraulic length, $L_h = L$. This is because by definition, L_h is the length of lateral pipe that would be required to uniformly apply Q_s over a circular field without trajecting water beyond its moving end. Thus, for center-pivot laterals with end-guns, $L_h > R > L$, and it can be computed by:

$$L_h = L\left[\frac{Q_s}{Q_b}\right]^{1/2} = L\left[\frac{Q_b + Q_g}{Q_b}\right]^{1/2} \tag{14.23}$$

where L_h = the equivalent hydraulic length of a center-pivot lateral, m (ft).

The flow rate through a center-pivot lateral pipe at any radial distance, r_j, can be computed by:

$$Q_j = Q_s\left(1 - \frac{r_j^2}{L_h^2}\right) \tag{14.24}$$

where Q_j = flow rate in lateral pipe at r_j, L/s (gpm).

A center-pivot lateral is a special type of manifolded pipeline having a uniformly increasing discharge per unit of length. Determining friction losses can be done in a manner similar to that used for laterals that are moved linearly (linear-moved) or laterals that have a uniform discharge per unit of length. Therefore, many of the same basic concepts used for periodic-move laterals can be applied to center-pivot laterals. For example, the total friction head loss along a center-pivot lateral is:

$$h_f = J\, F_p \frac{L_h}{100}; \qquad \text{for } L \geq 0.8L_h \tag{14.25a}$$

And the pressure loss due to pipe friction is:

$$P_f = K\, h_f; \qquad \text{for } L \geq 0.8L_h \tag{14.25b}$$

where

J = head-loss gradient calculated by Eq. 8.1, m/100 m (ft/100 ft)
F_p = reduction factor for center-pivot laterals to compensate for discharge along the pipe of length L_h, dimensionless
h_f = head loss due to pipe friction, m (ft)
P_f = pressure loss due to pipe friction, kPa (psi)
K = conversion constant, 9.8 for metric units (0.433 for English units)

The concept of using F_p is the same as for using F in Eq. 8.8 for linear-moved laterals. For center-pivot laterals, F_p ranges from 0.550 for 270 outlets to 0.560 for 40 outlets. Thus, for almost all standard pivots a value of F_p = 0.555 (which occurs with 73 outlets) will give results that are accurate to within ±1%. Figure 14.12 shows a plot of the number of outlets vs. F_p beginning with F_p = 0.598 for a 9-outlet lateral. For laterals with fewer than 10 outlets, the F_p values are: (9, 0.598); (8, 0.61); (7, 0.62); (6, 0.63); (5, 0.65); (4, 0.67); (3, 0.71); (2, 0.79); and (1, 1.0). The above F_p values were determined by

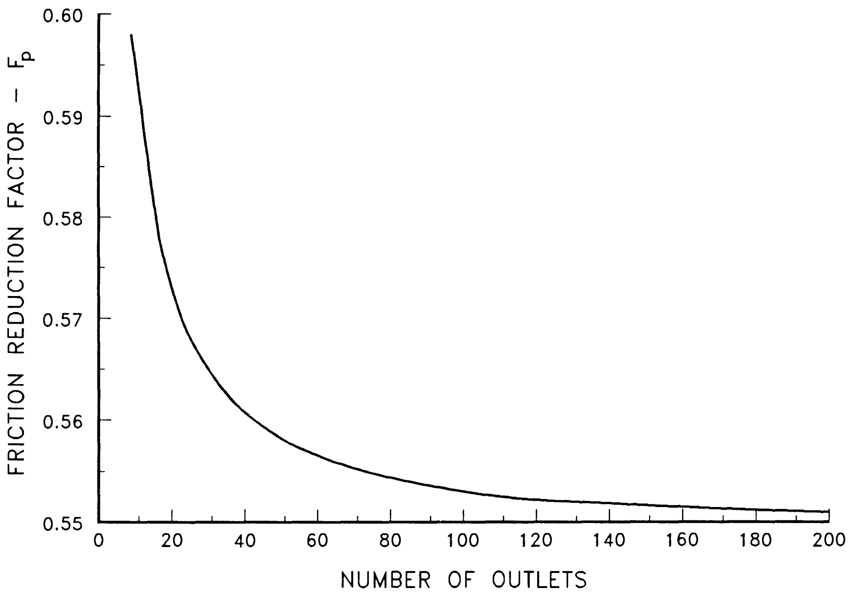


FIG. 14.12. Friction-reduction Factors for Different Numbers of Outlets Along Center-pivot Laterals.

using a stepwise (iterative) computational process and Hazen-Williams flow exponent of 1.852.

The $F_p = 0.555$ is considerably larger than the comparable $F = 0.36$ used for linear-moved laterals (see Table 8.7). The reason F_p is larger than F is because there is much less reduction in flow near the inlet end of center-pivot laterals. This results from the innermost circular areas being small compared with the average circular area irrigated per outlet.

Pressure Along the Lateral

The pressure head at any point along a center-pivot lateral can be determined by the stepwise procedure mentioned earlier, beginning at the moving end with the desired pressure or head for the end sprinkler. It can also be done more directly as follows:

1. Determine (or set) the desired pressure head for the moving end of the lateral, which will be the minimum pressure head, H_n;
2. Then determine h_f by Eq. 14.25a, and add this to H_n to obtain the inlet pressure head H_l; and
3. To find the pressure head at any radial distance r_j, add the pipe friction head loss between r_j and L_h to H_n.

This procedure is similar to the one used for linear-moved laterals. However, finding the head loss between r_j and L_h is more difficult, because the discharge is not uniform per unit length of lateral.

Two equations are available for determining the pipe-friction loss between r_j and L_h. One is based on a theoretical analysis that uses an adjusted F_p and L_h to compensate for the relatively more uniform discharge per unit of length as the moving end of the lateral is approached:

$$(h_f - h_{fj}) = \frac{J}{100}\left(0.155 + 0.4\frac{L_h}{L_h + r_j}\right)(L_h - r_j) \cdot \left[1 - \frac{r_j^2}{L_h^2}\right]^{1.852} \tag{14.26a}$$

The other, developed by Chu and Moe (1972), adjusts h_f as computed by Eq. 14.25a as follows:

$$h_{fj} = \frac{15}{8} h_f \left[\frac{r_j}{L_h} - \frac{2}{3}\left[\frac{r_j}{L_h}\right]^3 + \frac{1}{5}\left[\frac{r_j}{L_h}\right]^5\right] \tag{14.26b}$$

Combining the three steps for determining the pressure head at any point along a center-pivot lateral operating on a level plane gives:

$$H_j = H_l - h_{fj} = H_n + (h_f - h_{fj}) \tag{14.27a}$$

And the pressure at radius r_j is:

$$P_j = K(H_j) \tag{14.27b}$$

where

H_j = pressure head at any radial distance, r_j from the pivot, m (ft)
H_l = lateral inlet pressure head at the pivoting elbow on top of pivot, m (ft)
h_{fj} = pipe-friction loss between the pivot and radial distance r_j, m (ft)
H_n = desired minimum pressure head at moving end of a lateral operating on a level field, m (ft)
h_f = total pipe-friction head loss computed by Eq. 14.25a; m (ft)
$(h_h - h_{fj})$ = pipe-friction head loss between any radial distance r_j and the end of a lateral pipeline reaching to L_h computed by Eq. 14.26a, m (ft)
P_j = pressure at any radial distance, r_j from the pivot, kPa (psi)
K = conversion constant, 9.8 for metric units (0.433 for English units)

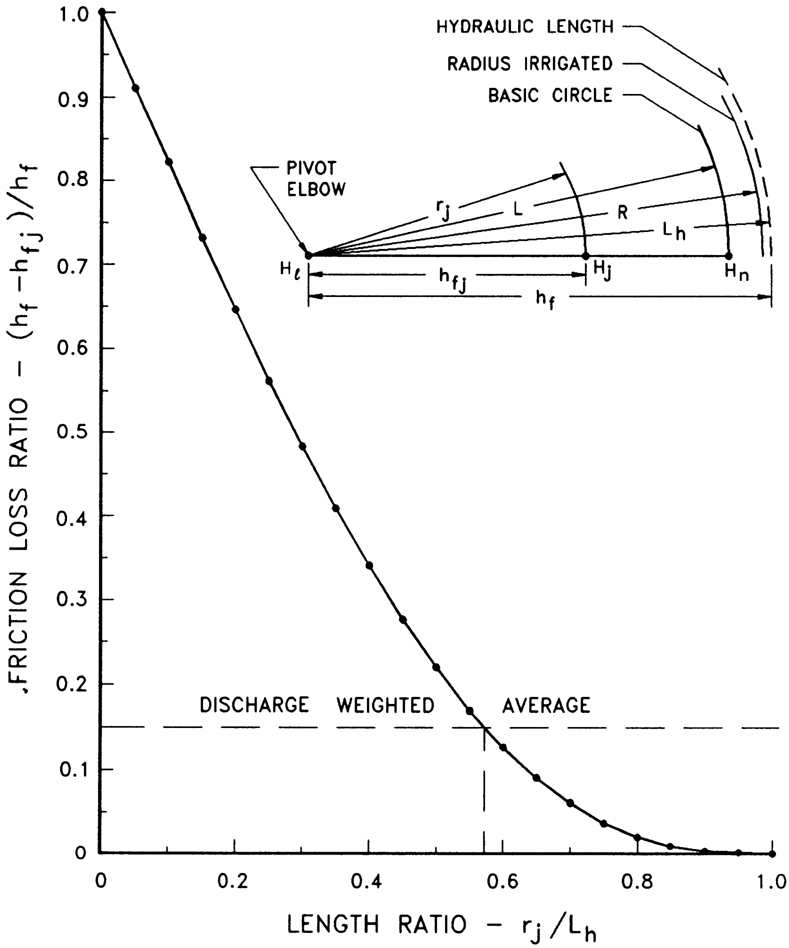


FIG. 14.13. General Dimensionless Friction Curve for a Center-pivot Lateral and a Terminology Insert.

Figure 14.13 shows a plot of r_j/L_h versus $(h_f - h_{fj})/h_f$ for a center-pivot lateral having a uniform pipe diameter. This general friction curve is comparable to Fig. 8.2 for a lateral with uniform discharge per unit length. However, it is somewhat flatter, because relatively more water is carried farther toward the closed end. The data for Fig. 14.13 were generated by a step-wise computational process for a center-pivot lateral having 100 equally spaced outlets using the flow exponent 1.852. Table 14.5 gives, in an abbreviated form, the data used to plot Fig. 14.13. The first outlet was one half-spacing from the pivot end, and the last (outlet 100) was at 0.995 L_h.

Friction-loss ratios generated by using Eq. 14.26a with 14.25a are essentially identical to those produced by the stepwise procedure. Ratios generated using

Table 14.5. Length, friction loss, area, discharge, and flow ratios for a center-pivot lateral with uniformly spaced outlets

Center-pivot lateral hydraulic ratios[1]				
Length r_j/L_h	Area[2] A_j/A_s	Discharge[3] q_j/Q_s, %	Flow[4] Q_j/Q_s	Friction[5] $(h_f - h_{fj})/h_f$
0.000	0.000	0.000	1.000	1.000
0.045	0.002	0.090	0.998	0.918
0.095	0.010	0.190	0.990	0.828
0.145	0.022	0.290	0.978	0.739
0.195	0.040	0.390	0.960	0.653
0.245	0.063	0.490	0.938	0.569
0.295	0.090	0.590	0.910	0.490
0.345	0.122	0.690	0.878	0.415
0.395	0.160	0.790	0.840	0.346
0.445	0.203	0.890	0.798	0.282
0.495	0.250	0.990	0.750	0.225
0.545	0.303	1.090	0.698	0.174
0.575	0.336	1.150	0.664	0.147
0.595	0.360	1.190	0.640	0.130
0.645	0.423	1.290	0.578	0.093
0.695	0.490	1.390	0.510	0.063
0.745	0.563	1.490	0.438	0.039
0.795	0.640	1.590	0.360	0.022
0.845	0.722	1.690	0.278	0.010
0.895	0.810	1.790	0.190	0.003
0.945	0.902	1.890	0.098	0.001
0.995	1.000	1.990	0.000	0.000

[1]For a lateral with 100 equally spaced outlets (with the first outlet one half-space from the pivot) and based on stepwise computations with flow exponent 1.852.
[2]Accumulated area irrigated at (r_j + 0.005 L_h).
[3]Discharge values are for a unit spacing of 0.01 L_h.
[4]Flow past r_j needed to irrigate area of circle outside (r_j + 0.005 L_h).
[5]Location of discharge-weighted average pressure head occurs near outlet at r_j/L_h = 0.575.

Eq. 14.26b with 14.25a range between the stepwise ratios and values as approximately 0.02 below them in the midrange. Thus, it makes little difference which procedure is used to compute pressure heads along the lateral.

Figure 14.13 shows that when $L = r_j = 0.845\ L_h$, 99% of the friction loss has occurred in a center-pivot lateral; and when $L = r_j = 0.8L_h$, 98% has occurred. End-guns on standard systems never discharge enough water so that $L < 0.8L_h$, as this would require having $Q_g > 0.56Q_b$ (see Eq. 14.23). Therefore, Eq. 14.25a can be used to compute h_f directly for all standard center-pivot systems with or without end-guns. However, for short laterals with large end-guns or corner systems using lateral extensions, a two-step procedure may be required to compute h_f.

The discharge weighted average pressure head along the lateral is $H_a \approx H_n + 0.15h_f$ for a nonsloping lateral. It occurs at $r_j \approx 0.58L_h$ (see Table 14.5 and

Fig. 14.13). These values were found by summing the products of the individual sprinkler discharges times their corresponding available pressure head and dividing by Q_s. For laterals having two or more pipe sizes, called *tapered laterals*, the functional relationship between the weighted average pressure head and h_f depends on the relative sizes and lengths of pipes used. A reasonable general value for tapered laterals is $H_a \approx H_n + 0.2h_f$, which is the exact value for a typical tapered lateral with the smaller pipe beginning at $r_j = 2L_h/3$.

Tapered Lateral Economics

Because the flow rate decreases along the lateral, it is often profitable to use tapered (multiple-pipe-size) laterals for center-pivot systems. Smaller diameter pipes not only save on pipe material costs, but also on supporting drive-unit costs, because smaller pipes are considerably lighter (especially when full of water). The procedures for selecting the most economic lateral diameters are identical to those described under Life-Cycle Costing in Chapter 8. The principle is to minimize the sum of annual fixed costs plus fuel and other operational costs that might be affected by pipe diameter, as depicted in Fig. 8.3.

The best tradeoff between fixed and operating costs can be based on a unit-length analysis, as described in the eight-step procedure leading up to Table 8.10. Where several center-pivot systems will be designed using the same economic and general hydraulic parameters, an economic pipe-selection chart, like Fig. 8.4, may save time, or the process can be set up on a computer program spreadsheet using Eqs. 8.14, 8.15, 8.17, and 8.18. For a review of the economic tradeoffs between capital cost and operating costs see Chapter 8.

Sample Calculation 14.9. *Designing a center-pivot lateral with two pipe diameters chosen from an economic pipe-selection chart.*

GIVEN: The data from Sample Calculation 14.1 for a center-pivot serving a basic irrigated circle and Sample Calculation 14.8 for sizing an end-gun to be used with it to irrigate corners.

The specification and pricing data presented in Table 14.6.

From an economic pipe-selection chart similar to Fig. 8.4, which was constructed for center-pivot laterals based on the economic conditions at the project site, the most economical lateral pipe will have a combination of 168- and 203-m ($6\frac{5}{8}$- and 8-in.) diameter galvanized pipe. (For convenience the smaller pipe will be referred to as 6-in. and the larger pipe as 8-in. in the discussions that follow.) When entering the chart for the lateral:

Without an end-gun, $Q_s = Q_b = 57.0$ L/s, and the economically neutral flow rate is at $Q_j \approx 55$ L/s; and

With an end-gun discharging $Q_g = 11.5$ L/s, $Q_s = 68.5$ L/s, and the neutral flow rate is at $Q_j \approx 50$ L/s.

Table 14.6. Some general specifications and prices for center-pivot laterals and system components[1]

Item		Nominal pipe diameter, in. (mm)[2] 4 1/2 (114)	5 9/16 (167)	6 (152)	6 5/8 (168)	8 (203)	10 (254)
Lateral pipe							
Inside	mm	108.0	135.1	146.2	162.1	197.0	247.8
Diameter[3]	(in.)	4.255	5.318	5.755	6.380	7.755	9.755
Optimum span	m		56	56	44	38	32
Length[4]	(ft)		185	185	145	125	105
Unit costs, \$/m							
Lateral pipe[5]		75	87	94	100	115	148
Overhang[6]				50			
Item costs, \$							
Pivot, generator, and controls[7]		3000	5500	6000	6000	7000	8000
End-gun packages with motor-pump and sprinkler							
2-hp (no control valve)							1800
5-hp with valve and extra generating capacity							2700
7.5-hp with valve and extra generating capacity							3600

[1]1988 prices in \$U.S.; \$1.00/m = \$0.30/ft.
[2]The 4 1/2-in. pipe is used only for small-scale lightweight systems with laterals less than 300 m (1000 ft) long.
[3]Based on "heavy duty" 11/12-gauge steel pipe. Use a pipe friction factor of $130 \leq C \leq 140$ for galvanized and $140 \leq C \leq 150$ for epoxy-coated lateral pipe.
[4]These are typical for maintaining tower/drive-unit loads in the neighborhood of 2.5 to 3.2 tons each.
[5]Typical prices including freight, installation, standard nozzling package, and standard tires.
[6]Overhangs from 4- to 25-m (15- to 85-ft) long using 152-m (6-in.) pipe are available to extend the lateral pipe beyond the end tower.
[7]Includes main system control valve and flow meter.

The standard spans used for the 203-mm (8-in.) pipe are 32 or 38 m long and for the 168-mm (6-in.) they are 38 or 44 m long. (It is interesting to note that both the 32-m × 203-mm pipe and the 44-m × 168-mm weigh almost the same when full of water, 2.6 metric tons.)

The lateral with variable spaced outlets is fitted with low-pressure impact sprinklers that have flow-controlled nozzles and are designed to operate with $P \geq 205$ kPa (30 psi).

The radius of the basic irrigated circle is $L = 402$; however, the actual length of the lateral pipe will depend on the selection of span lengths.

FIND: The h_f for all 168-mm (6-in.) pipe with and without the end-gun and for a combination of 203- and 168-mm (8- and 6-in.) pipe with the end-gun; also find the discharge, q_g, and sector angle settings for a part-circle sprinkler on the end of the lateral to extend and square-off the basic circle.

CALCULATIONS: Without an end-gun the $Q_s = Q_b = 57.0$ L/s is almost the same as the breakeven flow rate $Q_j = 55$ L/s. Therefore, a lateral with all 6-

in.-diameter pipe is the most economical design for the basic circle with L = 402 m. By Eq. 8.1, J = 4.25 m/100 m for galvanized pipe with D = 162.1 mm and C = 135. Substituting into Eq. 14.25a and letting F_p = 0.555 gives the function head loss with all 6-in. pipe when only irrigating the basic circle:

$$h_f = 4.25 \times 0.555(402/100) = 9.5 \text{ m}$$

With the end-gun on, Q_s = 68.5 L/s, and the equivalent hydraulic length of the lateral by Eq. 14.23 is:

$$L_h = 402(68.5/57.0)^{1/2} = 441 \text{ m}$$

And by Eq. 14.25a with J = 5.97 m/100 m for Q_s = 68.5 L/s in 6-in. pipe:

$$(h_f)_6 = 5.97 \times 0.555(441/100) = 14.6 \text{ m}$$

From the economic pipe-selection chart, using all 6-in. lateral pipe would not be economically efficient. For the data given, 8-in. pipe should be used when $Q_j \geq$ 50 L/s. The location where the transition from 8- to 6-in. pipe should occur can be found by solving Eq. 14.24 for r_j to obtain:

$$r_j = L_h \left[\frac{Q_s - Q_j}{Q_s} \right]^{1/2}$$

$$= 441 \left[\frac{68.5 - 50}{68.5} \right]^{1/2} = 229 \text{ m}$$

This same value can be found by entering Table 14.5 with Q_j/Q_s = 0.73 and interpolating to find $r_j/L_h \approx$ 0.53 to:

$$r_j \approx 0.53 \times 441 = 234 \text{ m}$$

The lateral spans from any given manufacturer only come in a few discrete lengths for each pipe diameter. For this design, the 8-in. is available in 32- and 38-m lengths, and the 6-in. in 38- and 44-m lengths. Therefore, the system might be configured with:

6 spans of 8-in. by 38-m	228 m
3 spans of 6-in. by 44-m	132 m
1 span of 6-in. by 38-m	38 m
No overhang	
Total length of lateral pipe = L' =	398 m

By Eq. 14.25a with J = 2.31 m/100 m for Q_s = 68.5 L/s in all 8-in. pipe:

$$(h_f)_8 = 2.31 \times 0.555(441/100) = 5.7 \text{ m}$$

To determine $(h_f)_{8,6}$ for the combination of 8- and 6-in. pipe, first calculate $(h_f - h_{fj})$ values for each pipe diameter at r_j = 228 m, using Eq. 14.26a, to obtain:

$$(h_f - h_{fj})_8 = (2.31/100)(0.419)(213)(0.562) = 1.2 \text{ m}$$

And noting the relationship between h_f and J values:

$$(h_f - h_{fj})_6 = (h_f + h_{fj})_8 J_6/J_8 = 1.16 \times 5.97/2.31 = 3.0 \text{ m}$$

The pressure head loss for the combination of 8- and 6-in. pipe can now be computed in the same manner used for linear-move laterals to obtain:

$$(h_f)_{8,6} = (h_f)_8 - (h_f - h_{fj})_8 + (h_f - h_{fj})_6$$
$$= 5.7 - 1.2 + 3.0 = 7.5 \text{ m}$$

An interesting verification of the lateral design process and hydraulic formulas is to determine the friction loss in the 8-in. pipe between the pivot and $r_j = 238$ m as:

$$(h_{fj})_8 = (h_f)_8 - (h_f - h_{fj})_8 = 5.7 - 1.2 = 4.5 \text{ m}$$

Then use Eq. 14.26b with $r_j/L_h = 228/441 = 0.517$, and find it directly:

$$(h_{fj})_8 = 4.6 \text{ m}$$

Or from Table 15.5 for $r_j/L_h = 0.52$ the friction-loss ratio $LFR \approx 0.21$ so:

$$(h_{fj})_8 = h_f(1 - LFR) = 5.7(1 - 0.21) = 4.5 \text{ m}$$

The fact that there are only minor differences among the $(h_{fj})_8$ values computed by the three different methods supports the validity of all of them.

The discharge from the part-circle sprinkler required to extend and square-off the basic circle from the end of the lateral pipe, $L' = 398$ m to the edge of the basic circle, $L = 402$ m can be determined by:

$$q_g = \frac{2\, L' \Delta L}{L_2} Q_b \tag{14.20b}$$
$$= \frac{2 \times 398(402 - 398)}{(402)^2} 57.0 = 1.12 \text{ L/s}$$

In accordance with the notations on Fig. 14.11C, the in-throw angle should be set at 15° to fill in the radial application profile for the low-pressure impact sprinklers along the lateral; and the out-throw angle should be set at approximately 25° to extend and square it off.

SYSTEM PRESSURES AND FLOWS

The inlet pressure, minimum pressure, end-gun pressure, and pressure variations because of elevation differences should be examined for center-pivot systems. The absolute minimum pressure, H_n, will normally occur at the moving end of the lateral when it is pointing uphill and if there is a corner system, when it is operating. The minimum pressure should be set in accordance with the sprinkler configuration. It should be sufficient to produce a good watering pattern that will not cause surface sealing or crop damage. Therefore, the recommended operating pressures given in Table 14.2 for the various sprinkle-type and spacing configurations should be followed. Pressures at the high end of the ranges should be used for H_n where surface sealing due to drop impact is expected.

Pressure Head Requirements

It is useful to organize the items that make up the total head required for operating a center-pivot system in the following manner. First all systems can be classed by the end-gun situation as:

A. Systems without end-guns that irrigate only the basic circle;
B. Systems with end-guns that irrigate only the basic circle; and
C. Systems with end-guns (or lateral extensions) that are used only to irrigate into the corners (see Fig. 14.1).

Next the operating pressure heads, elevation differences, and pipe and fitting friction losses information needed for design purposes can be conveniently grouped for each of the above end-gun situations. For Class A or basic circle systems:

A-1. The lowest sprinkler operating pressure head, H_n, that will be allowed, which is
- the minimum boom inlet or sprinkler nozzle pressure plus,
- for systems with pressure regulators for each lateral outlet, the additional pressure required by the regulator;

A-2. The head loss, h_f, due to pipe friction along the center-pivot lateral for $Q_s = Q_b$;

A-3. The ground elevation difference, ΔEl_p, between the pivot point and the location of the outlet having the lowest pressure head, H_n, which is usually the highest point near the perimeter of the basic irrigated circle;

A-4. The sum of the head losses, $(h_f)_s$, that are dependent on the flow rate, $Q_s = Q_b$, between the water source and pivot elbow (or lateral inlet) including
- pipe-friction losses in the supply pipeline,
- control valve and flow meter losses, and
- miscellaneous fitting losses; and

A-5. Water supply elevation difference, ΔEl_s between the water level at the source and the pivot elbow, which is made up of the elevation differences between

- the pump or pipe network outlet and the pivot elbow, plus
- (for systems supplied directly by a pump) the static water level and the pump, plus
- (for systems supplied directly from a well) the dynamic and static water levels.

) For a Class B or basic circle with end-gun systems:

B-1. The same H_n information for the sprinklers along the lateral as in A-1, and
- the desired end-gun pressure head, H_g;

B-2. The friction head loss along the lateral, h_f, for $Q_s = Q_b + Q_g$;
B-3. The ΔEl_p as described in A-3;
B-4. The $(h_f)_s$ from the water source to the pivot elbow, as described in A-4, but with $Q_s = Q_b + Q_g$; and
B-5. The elevation differences, ΔEl_s, between the water level at the source and the pivot elbow as in A-5, but with $Q_s = Q_b + Q_g$.

For a Class C basic circle with corner systems:

C-1. The same H_n information for the sprinklers along the lateral as for A-1 and
- The desired end-gun pressure head, H_g, plus
- The head loss through the end-gun's on-off valve;

C-2. The head loss along the lateral, h_f for $Q_s = Q_b$ and for $Q_s = Q_b + Q_g$;
C-3. The ΔEl_p as described in A-3;
C-4. The $(h_f)_s$ from the water source to the pivot elbow as described in A-4, but for when $Q_s = Q_b + Q_g$, as well as for $Q_s = Q_b$; and
C-5. The elevation difference, ΔEl_s, between the water source and the pivot elbow as in A-5, but for when $Q_s = Q_b + Q_g$, as well as for $Q_s = Q_b$.

Sprinkler Nozzle Selection

The sprinkler package or configuration of the nozzle sizes along the lateral should be selected to provide a constant discharge per unit area irrigated when the lateral is operating on a level plane. When pressure regulators or flow-control nozzles are used to regulate discharge, pipe friction and field topography affect the required lateral inlet pressure, but not nozzle selection.

Pipe friction and field topography should be considered when discharge regulation is not used. The individual nozzles should be sized to provide the required discharge q_j at the average operating pressure head H_j available at the corresponding radial distance r_j. Thus, in addition to the pipe-friction loss, h_{fj}, the average elevation of the circular path traveled by each outlet must be taken into account when determining the available pressure head:

$$(H_j)_{av} = H_l - h_{fj} - (\Delta El_j)_{av} \tag{14.27c}$$

for which

$$H_l = H_n + h_f + \Delta El_p$$

where

$(H_j)_{av}$ = average pressure head available at an outlet at radial distance r_j from the pivot, m (ft)

$(\Delta El_j)_{av}$ = difference between the pivot base and the average ground elevation along the circular path traveled by an outlet at radial distances r_j from the pivot, m (ft)

ΔEl_p = difference in ground elevation between the pivot point and the location of the center-pivot lateral outlet having the lowest operating pressure head (usually near the perimeter of the basic circle), m (ft)

For smooth or uniformly undulating fields with uniform general slopes, $(\Delta El_j)_{av} \approx 0$ along the entire lateral. However, where the topography is more complex, the average sprinkler elevations must be taken into account. If this is not done, the average system discharge Q_s and application uniformity will not be as expected. Thus, designing a nozzling package (without discharge-regulating devices) for a center-pivot system on complex topography requires a design strategy that includes computing $(\Delta El_j)_{av}$ values for use in Eq. 14.27c to determine $(H_j)_{av}$. The only practical way to do this is with a computer-assisted approach that involves strategic use of topographic data.

For center-pivot systems without end-guns, the nozzle sizing should be based on having a lateral inlet pressure H_l equal to the sum of pressure head items A-1, A-2, and A-3. The sprinkler/nozzle package for the basic circle should be selected so the individual sprinkler discharges, q_j, satisfy Eq. 14.20 when operating at $(H_j)_{av}$ as computed by Eq. 14.27c; their sum equals the desired Q_b; and the head-loss curve follows Fig. 14.13.

For center-pivot systems with end-guns, the required H_l depends on whether or not a booster pump will be used. (Only systems with high-pressure impact sprinklers can support end-guns without booster pumps.) For systems without boosters, let H_l equal the minimum desired H_g (plus the on-off valve loss for a corner system) plus head loss items B-2 and B-3. The sprinkler/nozzle package for the basic circle should be designed so each q_j along the lateral satisfies Eq. 14.20 when operating at $(H_j)_{av}$; their sum equals the desired Q_b; and the head-loss curve follows Fig. 14.13. The end-gun discharge Q_g should satisfy Eq. 14.21 when operating at $(H_j)_{av}$ with $r_j = L'$.

For systems with booster pumps, let H_l equal head items A-1 plus B-2 plus B-3, so the minimum pressure equals H_n. The sprinkler/nozzle package for the basic circle and end-gun should be designed as above. The booster pump should be selected to increase the pressure head from H_n to the minimum desired H_g (plus the on-off valve loss for a corner system) to obtain Q_g.

Discharge Uniformity

Pressure head changes that are due to elevation differences and on and off end-gun operation as the lateral cycles around the field are unavoidable. These in turn cause the sprinkler discharges to vary, which adversely affects the application uniformity. Where the pressure head variation is excessive, pressure regulators should be installed at each outlet along the lateral, or flow-control nozzles (see Fig. 6.6) should be used to regulate the discharge. What constitutes an excessive pressure variation is relative to the average sprinkler discharge pressure head, H_a. Fortunately, nozzle discharge will vary only about half as much as the pressure, because it is a function of the square root of the pressure.

Emission Uniformity. The concept of a design emission uniformity, EU, was first developed for trickle irrigation, where it is the major criterion for evaluating design performance. A detailed discussion of EU is presented in conjunction with Eqs. 20.9, 20.10, and 20.13. (It is recommended that this be reviewed before proceeding.) The EU depends on the coefficient of manufacturing variation, v, of the emitters or discharge-control devices and the discharge variation caused by emitter inlet pressure head differences. Where several devices serve the same area, the adverse effects of large manufacturing variations, v, are somewhat offset.

Where pressure regulators are used between the outlets and the sprinklers along a center-pivot lateral, the *discharge-control device* is the combined pressure regulator and the sprinkler(s) it serves. In other words, the device is the *regulated nozzle*.

Equation 20.13, which gives the design emission uniformity for trickle systems, must be modified for use with center-pivot sprinkle systems. For trickle irrigation the area served by each emitter is constant, but for a center-pivot lateral it increases with r_j unless the sprinkler spacing, S_j, decreases proportionally. Thus, to keep the application depth uniform the average discharge per unit area, q_{au}, must be uniform, and for center-pivots Eq. 20.13 should be modified to:

$$\mathrm{EU}_p = 100\left(1 - 1.27\frac{v_a}{\sqrt{N_p'}}\right)\frac{\Sigma q_n}{\Sigma q_a} \tag{14.28}$$

where

EU_p = design emission uniformity for center-pivots, %

v_a = weighted average of the coefficients of manufacturing variation, v, of discharge using an inlet pressure of 70 kPa (10 psi) above nominal for three (or four or five) *sample sets* of discharge-control devices representative of low, medium, and high operational discharges, decimal

N_p' = minimum number of sprinklers from which each part of the area receives water. For low-pressure sprinklers let $N_p' = 1$, and for medium- and high-pressure sprinklers let $N_p' = 2$ when $(w_j/2S_j) > 2$ for $r_j > L/2$, integer

q_{au} = average design emission rate per unit area, L/s per m^2 (gpm/ft^2)

Σq_n = sum of the minimum discharge rates obtained over the pressure range from 35 to 345 kPa (5 to 50 psi) above nominal from three (or four or five) individual discharge-control devices representative of low, medium, and high operational discharges, L/s (gpm)

Σq_a = sum of the average discharge rates obtained from the discharge-control device tests conducted to obtain Σq_n, L/s (gpm)

The weighted average of the coefficients of manufacturing variation of discharge, v_a, is computed as follows. It is determined by finding the sum of the products of the nominal discharges multiplied by the v of each of the three (or four or five) sample sets of discharge-control devices, which then must be divided by the sum of their nominal (or average) discharges. The logic of using the weighted average is that the area irrigated from the outlet is proportional to the discharge. The v_a gives a measure of the discharge variability between devices when operating at a standard pressure head.

Using $\Sigma q_n/\Sigma q_a$ effectively gives a weighted discharge ratio that is a measure of the expected performance of any given discharge device over a range of inlet pressures. This discharge ratio is a measure of the minimum expected discharge from a sprinkler that has been selected to give an average discharge of q_j, which is seldom the same as its nominal discharge.

Equation 14.28 can be applied directly for the discharge-control devices used along center-pivot sprinkler laterals. All that is needed are three (or four or five) v and q_n/q_a values that represent low, medium, and high discharges. These can be obtained from the manufacturer or determined from bench test data using Eq. 20.9 to compute the v values. The CU (or DU) expected from laterals operating without discharge-control devices on level planes can then be multiplied by $\mathrm{EU}_p/100$ to obtain the design uniformity with discharge control devices.

Discharge-control devices should not be used indiscriminately. They increase system cost by 2 or 3% and may actually lower system application uniformity. In addition, pressure regulators increase the required lateral inlet pressure by roughly 20 to 35 kPa (3 to 5 psi).

When operating a center-pivot without a corner system on a level field, the pressure head at any point along the lateral remains constant as the lateral rotates. For such systems, the use of discharge-control devices is counterproductive. The purpose of using them is to mitigate the adverse effects on discharge uniformity due to unavoidable pressure head changes as the lateral rotates. Such changes occur as an end-gun is turned on and off and the lateral travels over sloping or irregular terrain.

The pressure head variation, ΔH, at which it would be advantageous to use discharge-control devices is a function of EU_p and the average operating pressure head of the sprinklers. The EU_p represents the minimum discharge compared with the average expected discharge. Therefore, discharge-control devices should be used along the entire lateral only when the outlet discharge variations, Δq, are expected to substantially exceed the break-even point for using them. This will occur approximately when:

$$\left[\Delta q > 2.5(1 - EU_p/100)q_a\right]_{av} \tag{14.29a}$$

Because sprinkler discharge is a function of the square root of the pressure head, the devices should be used along the entire lateral only when:

$$\left[\Delta H > 2.5(1 - (EU_p/100)^2)H_a\right]_{av} \tag{14.29b}$$

where

$_{av}$ = subscript denoting the average values across the field, which for uniformly sloping laterals is at the center of mass of the discharge rates along the lateral

Δq = allowable variation in discharge $(q_x - q_n)$, L/s (gpm)

q_a, q_x, and q_n = average, maximum, and minimum sprinkler discharge rates, respectively, as the lateral cycles, L/s (gpm)

ΔH = allowable variation in pressure head $(H_x - H_n)$, m (ft)

H_a, H_x, and H_n = average, maximum, and minimum pressure heads, respectively, as the lateral cycles, m (ft)

Where the devices are used only along strategic sections of laterals that travel over isolated high spots, it would be reasonable to use them wherever the break-even point occurs. In such cases, the 2.5 in Eq. 14.29b should be reduced to 2.0, and the subscript a replaced by j representing radius r_j. Thus, flow-control devices should be used only at outlets where it is expected that:

$$\Delta H_j > 2.0\left[1 - (EU_p/100)^2\right](H_j)_{av} \tag{14.29c}$$

To use Eq. 14.29c, determine H_j by Eq. 14.27a, $(H_j)_{av}$ by Eq. 14.27c, and check $(\Delta El_p)_j$ around the circle at r_j. For center-pivots without corner systems the difference between the maximum and minimum $(\Delta El_p)_j$ will be ΔH_j, and pressure-regulating devices should be used only at specific outlets in accordance with Eq. 14.29c. For center-pivots with corner systems the difference in pressure head at r_j between when the end-gun is on and off must be taken into account along with the elevation differences around the circle at r_j.

Discharge-Control Devices

There are two types of devices available for keeping a sprinkler's discharge constant when the inlet pressure varies. One is to use a special flexible-orifice-type nozzle that is designed so the orifice diameter decreases sufficiently to keep the discharge constant as the pressure increases. The other is to use a spring-loaded pressure regulator below the sprinkler to maintain a constant inlet pressure.

Flexible-Orifice Nozzles. The discussion related to Fig. 6.6 mentions that the discharge variation of a sample set of flexible-orifice nozzles operating at the same pressure may vary by as much as $\pm$ 5% of the nominal discharge. This is caused by variations in the flexible material, temperature effects, and manufacturing tolerances. Assuming that most all (95%) of the test values from a sample set will fall within $(1 \pm 2\ v_a)q_a$, then $v_a \approx 0.025$. In addition the discharge of the best flexible-orifice nozzles varies with pressure by somewhere in the neighborhood of $\pm$ 3%, as shown in Fig. 6.6. Thus, by Eq. 14.28, a typical EU_p for *the best* flexible-orifice flow-control nozzles might be as high as 94 or 95% for $N_p' = 1$ or 2, respectively.

Some of the sales literature for flexible-orifice nozzles suggests larger variations. It is indicated that the discharge from different nozzles of the same size may vary by as much as $\pm$ 10% of the nominal discharge when operating at a standard pressure head. Furthermore, the flow-rate performance data presented indicate that nozzle discharge often varies by approximately $\pm$ 5% due to pressure differences over the normal operating range. Based on the above, the EU_p from typical flow-control nozzles would be approximately 89% for $N_p' = 1$ and 91% for $N_p' = 2$.

No extra pressure is needed to operate flexible-orifice nozzles. They are rated in terms of a normal flow or discharge rate, which is not exactly the same as their average discharge rate. Typical flexible-orifice nozzles are available in a fairly large range of nominal sizes in 0.032-L/s (0.5-gpm) increments. Specified operating pressures range between 140 and 550 kPa (20 to 80 psi). For greater design precision, rather than using the published nominal discharge, use the actual average discharge rate over the expected range of operating pressure to match the design q_j values.

For flexible nozzles with nominal discharges less than 0.5 L/s (8 gpm), the pressure range should be limited to about 450 kPa (65 psi) because of excessive jet breakup at higher pressures. For nozzles with discharge greater than 0.5 L/s (8 gpm) the pressure should be at least 240 kPa (35 psi) to assure sufficient jet breakup.

Pressure-Regulated Nozzles. Spring-loaded pressure regulators have been designed for use on center-pivot laterals. They are economic to use, operate efficiently between 0.1 and 1.0 L/s (1.6 to 16 gpm), and withstand surges quite well. Such suitable regulators are provided with preset pressure ratings of 70, 105, 140, 170, 205, 275, and 345 kPa (10, 15, 20, 25, 30, 40, and 50 psi).

When using pressure regulators, the pressure at the sprinkler inlet is equal to the pressure at the lateral outlet minus the pressure loss through the regulator itself. The purpose of the regulator is to hold the downstream pressure constant, but the downstream pressure is a function of the flow rate through the regulator. In other words the discharge pressure of a pressure regulator is flow-rate-dependent. The flow rate dependence is predictable; therefore, it can easily be included when designing the sprinkler/nozzle/regulator package.

Figure 14.14, redrawn from Kincaid et al. (1987), shows the ratio of actual outlet pressures to the nominal pressure for 70- and 140-kPa (10 and 20 psi) regulators with flows of 0.25, 0.5, and 0.75 L/s (4, 8, and 12 gpm). The hysteresis envelopes representing each flow rate are caused by mechanical friction within the valves. When the inlet pressure is on the rise, discharge pressure follows the lower leg, and when it is falling it follows the upper leg. There is also a slight increase in outlet pressure as the inlet pressure increases after the first 35-kPa (5-psi) increase over the nominal outlet pressure.

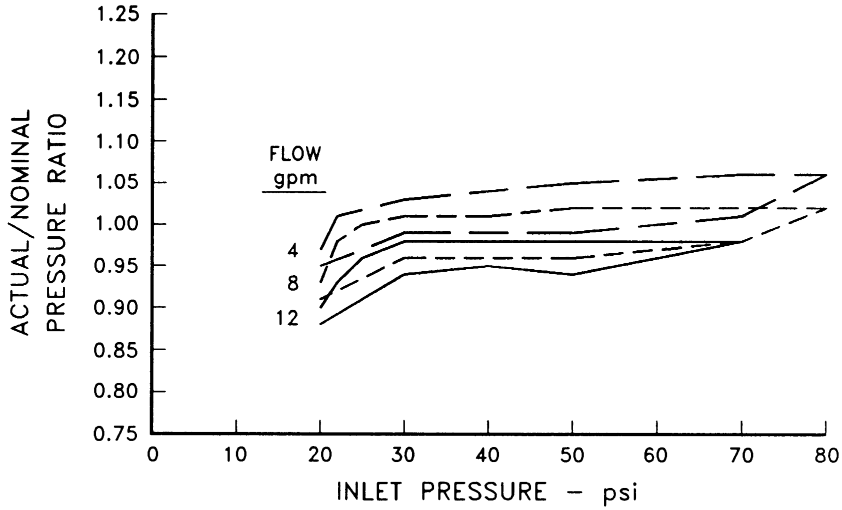
The average deviation of outlet pressure due to varying inlet pressures is about 7 kPa (1 psi) from the average pressure for both regulators at all three flow rates. Thus, the deviation in flows is about $\pm$ 5% and $\pm$ 2.5% for the 70- and 140-kPa (10- and 20-psi) regulators, respectively. There is a sharp drop in outlet pressure when the inlet pressure is less than the nominal pressure plus about 35 kPa (5 psi). Therefore, it is advisable to provide (or allow) for at least 35 kPa (5 psi) or 3.5 m (11.5 ft) of extra pressure or pressure head when computing system requirements.

The flow-rate dependency of pressure regulators should be incorporated into the design of the sprinkler/nozzle/regulator package. In this way the effects of different flow rates (see Fig. 14.14) on outlet pressure can be compensated for by adjusting the sprinkler nozzle size accordingly. To do this let the regulator outlet pressure (or sprinkler inlet pressure) be:

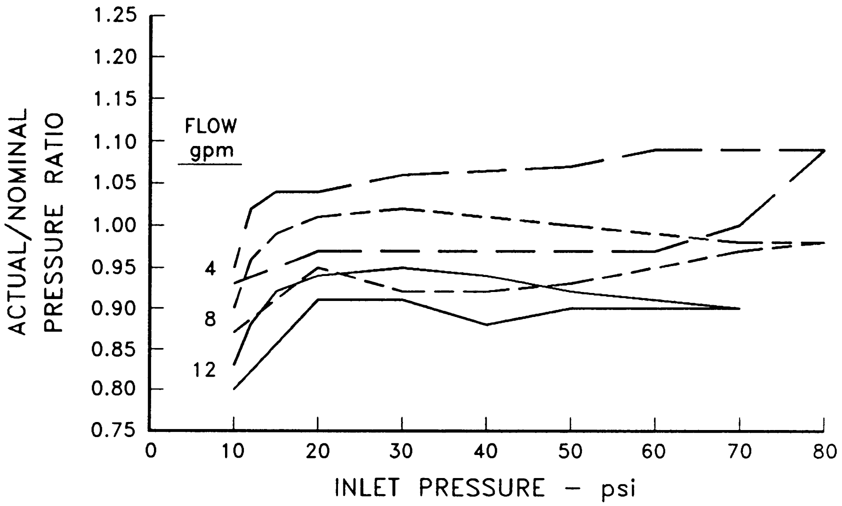
$$P_j = P_{pr} - (P_{cv})_j = P_{pr} - K(q_j/cv)^2 \tag{14.30}$$

where

P_j = available sprinkler operating pressure at radius r_j, kPa (psi)
P_{pr} = average outlet pressure (which is usually the nominal pressure rating) of the pressure regulator at low flow rates, kPa (psi)
$(P_{cv})_j$ = minimum pressure loss across the regulator for q_j at radius r_j, kPa (psi)
q_j = desired outlet discharge at radius r_j, L/s (gpm)
cv = flow coefficient that is numerically equal to the flow rate when $(P_{cv})_j$ = 1 kPa (1 psi), L/s (gpm)



A. 20 psi PRESSURE REGULATOR



B. 10 psi PRESSURE REGULATOR

FIG. 14.14. Ratio of Actual Outlet Pressure to Nominal Pressure for 10- and 20-psi Pressure Regulators with 4, 8, and 12 gpm Flows (1 gpm = 0.0631 L/s; 10 psi = 69 kPa).

K = appropriate unit pressure loss for the specific measurement units used, 1 kPa (1 psi)

The better pressure regulators available for use along center-pivot laterals have high flow coefficients. For a unit pressure loss of 1 kPa, $cv \sim 0.24$ L/s; and for a unit pressure loss of 1 psi, $cv \sim 10$ gpm. (The cv values are usually the same for regulators with the same body configuration regardless of the pressure rating.)

The average coefficient of manufacturing variation of pressure has been found to be: $v \approx 0.03$ for the better regulators. Thus, the weighted average coefficient of discharge variation for a regulator with fixed nozzle combination is:

$$v_a \approx 1 - (1 - 0.97)^{1/2} \approx 0.015$$

Assuming Fig. 14.14 is typical, then: for the 70-kPa (10-psi) regulator $\Sigma q_n / \Sigma q_a \approx 0.97$; and for the 140-kPa (20-psi) regulator it is ≈ 0.98. Computing the EU_p by Eq. 14.28 gives approximately: 95 or 96% for $N_p' = 1$ or 2, respectively, with the 70-kPa (10-psi) regulators; and 96 or 97% for $N_p' = 1$ or 2, respectively, with the 140-kPa (20-psi) regulators.

The above EU_p values are based on the assumption that Eq. 14.30 is used to determine the operating pressure to size the sprinkler nozzles. If this is not done, the $\Sigma q_n / \Sigma q_a$ would need to be replaced by q_n/(nominal rating) in Eq. 14.28. If some 0.75-L/s (12-gpm) nozzles are used, based on the data in Fig. 14.14, the q_n/(nominal rating) would be approximately 0.94 and 0.96 for the 70- and 140-kPa (10- and 20-psi) regulators, respectively. Thus, the respective EU_p values for $N_p' = 1$ would be reduced to approximately 92 and 94%, respectively; or 93 and 95% for $N_p' = 2$.

Elevation-Discharge Relationship

When a center-pivot system is operated on a sloping field, the sprinkler pressures vary as the lateral rotates. Typical nozzling packages for systems operating on uniform slopes are designed to provide a uniform application of water when there are no elevation differences along the lateral (see Eq. 14.27c). When the lateral is pointing uphill, the individual sprinkler discharges will drop. Unless discharge-control devices are used, the system discharges will decrease, and when the lateral is pointing downhill the discharge will increase.

This variation in discharge caused by elevation differences is a function of the nozzle discharge coefficients, pipe-friction loss, and pump discharge head characteristics. Estimating the effect of elevation changes on system discharge can be simplified when the lateral is on a *uniform slope*. The overall effect of elevation changes at each sprinkler can be represented by the location of the discharge-weighted average elevation for the entire lateral, which is at approximately $(\frac{2}{3})L_h$. Combining this concept with Eq. 14.23 to include systems with or without end-guns gives:

$$R_w = \frac{2\,L}{3}\left[\frac{Q_b + Q_g}{Q_b}\right]^{1/2} = \left(\frac{2}{3}\right)L_h \qquad (14.31)$$

where R_w = radius from the pivot point to the location of the discharge-weighted average elevation for laterals on uniform slopes, m (ft).

To estimate the overall effect of elevation changes, the sprinklers along the lateral can be thought of as all being at R_w. The average pressure head changes that take place as the lateral rotates will then be (± slope) × R_w. Based on this concept, the variations in Q_s can be computed by rearranging Eq. 5.1 and assuming an average discharge exponent of 0.52 (which is midway between 0.5 and 1/1.852) to account for the inclusion of pipe friction to obtain:

$$(Q_s)_{el} = Q_s[(H_a)_{el}/H_a]^{0.52}$$

$$(Q_s)_{el} = Q_s \left[\frac{H_n + k_1 \Delta El_p + k_2 h_f + k_3 (h_f)_s + \Delta H_T}{H_n + k\, h_f + \Delta El_p} \right]^{0.52} \qquad (14.32)$$

in which:

$k_1 = 1 - R_w/L = 1 - (\frac{2}{3})\, L_h/L$; for uphill position
$k_1 = 1 + R_w/L = 1 + (\frac{2}{3})\, L_h/L$; for downhill position
$k_2 = 1 - K_f + k\, K_f$
$k_3 = 1 - K_f$
$K_f = [(Q_s)_{el}/Q_s]^{1\ 852}$
k = discharge-weighted, average friction-loss ratio, which is 0.15 for uniform laterals (see Fig. 14.13) and 0.2 for tapered laterals

where

$(Q_s)_{el}$ = system capacity adjusted for elevation for laterals without pressure of flow regulation, L/s (gpm)
H_n = minimum sprinkler pressure head specified, m (ft)
h_f = head loss due to lateral pipe-friction losses for Q_s, m (ft)
$(h_f)_s$ = head loss due to pipe-friction and fitting losses that depend on the flow rate between the water source and pivoting elbow, m (ft)
ΔH_T = change in pump discharge pressure head due to difference between Q_s and $(Q_s)_{el}$, m (ft)
ΔEl_p = maximum positive difference in ground elevation between the pivot and moving end, which equals $s\, L$, m (ft)
s = uniform ground slope, (+) for uphill and (−) for downhill, decimal

Solving Eq. 14.32 requires a trial-and-error (or iterative) process. This is because the system flow-rate variations (up or down) due to the elevation effects affect the friction head losses and the pump discharge pressure head.

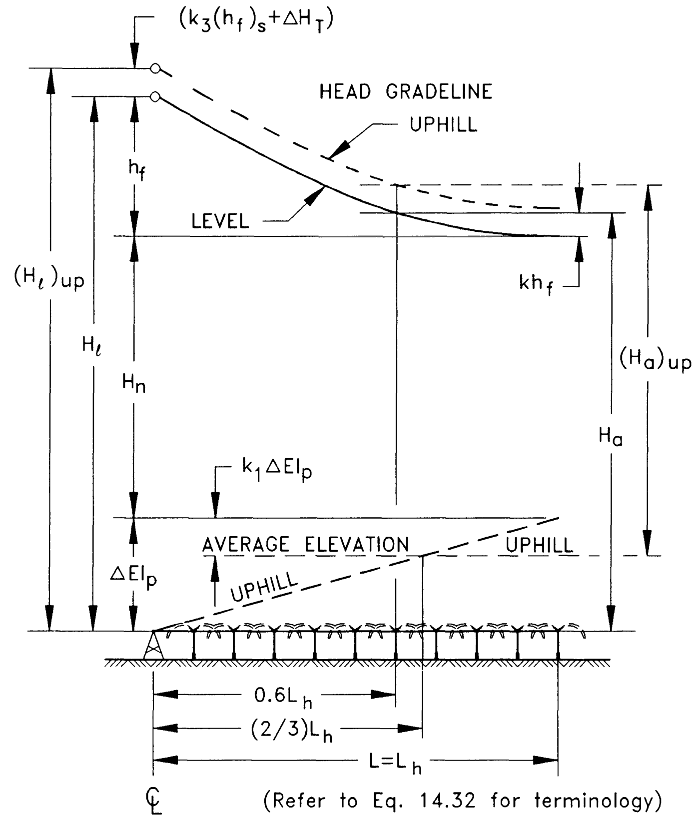


FIG. 14.15. Relationship Between Average Center-pivot Lateral Pressure Heads for Level and Uphill Operation.

Figure 14.15 shows the relationships between the average discharge-weighted pressure head and average discharge-weighted elevations for the lateral operating in the level and uphill positions. The level position is depicted by the solid lines and the uphill position by the dashed lines. The lateral irrigates a basic circle only, and the pressure loss savings and ΔH_T are somewhat exaggerated. However, the sketch depicts how the change in H_a due to elevation differences is somewhat offset by the reduction in friction loss and increase in the pump discharge pressure resulting from the reduction in Q_s.

The variations in discharges from the individual sprinklers are not uniform and obviously become greater as one moves away from the pivot point. This reduces the application uniformity and even where Q_s may be sufficient in the uphill position underirrigation may occur at the moving end of the lateral. There are two methods for reducing the uneven watering resulting from elevation-induced flow rate changes without using pressure or flow regulation for each sprinkler. One is to reduce the lateral travel speed when it is in the uphill position and increase it when in the downhill position by the ratio $(Q_s)_{el}/Q_s$. Another possibility is to increase and decrease the pivot inlet pressure when pointing uphill and downhill, respectively.

When center-pivots are fed directly from wells or individual pumping plants, the changes in Q_s will be further modified by the well and pump discharge characteristics. Therefore, a plot should be made to determine where the uphill and downhill and the with and without end-gun operating system curves intersect the pump curve to accurately determine the expected variations in Q_s.

End-Gun Pressure and Effects

End-gun pressures should be at least 345 kPa (50 psi) and preferably above 450 kPa (65 psi). The recommended pressure depends on nozzle size and type, as well as on soil, crop, and wind characteristics. To achieve the necessary pressure, a booster pump mounted on or next to the last drive unit is often needed. The pump is used to increase the pressure to the end-gun. Where low-pressure spray or impact sprinklers are used along the lateral, a booster pump is always necessary to operate an end-gun properly. With medium- and high-pressure sprinkler configurations, a booster pump may not be required.

The H_l and nozzling package for systems with end-guns should be based on (designed for) when the end-gun is operating. A major problem with using end-guns to irrigate the corners results from the change in flow rate and resulting pressure head changes when the end-gun is turned on or off. Turning the end-gun off decreases pipe friction losses. Where a pump is used to supply one or two center-pivot systems directly from a sump or well, turning the end-gun off will also cause the pump outlet pressure head to increase. To adequately design a center-pivot system, the effects of these flow and pressure changes must be taken into account.

Assuming the pump discharge (or supply) pressure head remains constant, the effect on h_f when the end-gun is turned off (whether there is a booster pump or not) can be determined by substituting Eqs. 8.1 and 14.23 into Eq. 14.25a and rearranging to obtain:

$$(h_f)_{\text{off}} = h_f K_{hf}$$

$$(h_f)_{\text{off}} = h_f \left[\frac{(Q_b)_{\text{off}}}{Q_b + Q_g} \right]^{2.352} \tag{14.33a}$$

And the effect on the supply system head loss when the end-gun is turned off is:

$$(h_{fs})_{\text{off}} = (h_f)_s K_f$$

$$(h_{fs})_{\text{off}} = (h_f)_s \left[\frac{(Q_b)_{\text{off}}}{Q_b + Q_g} \right]^{1.852} \tag{14.33b}$$

where

$(h_f)_{\text{off}}$ = head loss due to lateral pipe friction when the end-gun is not operating, m (ft)
(h_f) = design head loss due to lateral pipe friction when the end-gun is operating, m (ft)
$(Q_b)_{\text{off}}$ = discharge to the basic irrigation circle (for laterals without discharge-control devices) when the end-gun is not operating, L/s (gpm)
$(h_f)_s$ = design head loss due to friction in the supply systems, m (ft)
$(h_{fs})_{\text{off}}$ = head loss due to friction in the supply system when the end-gun is not operating, m (ft)

Equation 14.33a can be verified by computing $(h_f)_{6,\text{off}} = 9.5$ m given $(h_f)_{6,\text{on}} = 14.6$ m, as determined in Sample Calculation 14.9 as:

$$(h_f)_{6,\text{off}} = 14.6\,(57.0/68.5)^{2.352} = 9.5 \text{ m}$$

The effect on Q_b when the end-gun is turned off can now be determined in a manner similar to that used in the development of Eq. 14.32, and it can be shown that:

$$(Q_b)_{\text{off}} = Q_b\left[(H_a)_{\text{off}}/H_a\right]^{0.52}$$

$$= Q_b\left[\frac{H_n + \Delta El_p - \frac{2}{3}\,\Delta El + k_2 h_f + k_3 (h_f)_s + \Delta H_T}{H_n + k\,h_f + \Delta El_p}\right]^{0.52} \tag{14.34}$$

in which:

$k_2 = 1 - K_{hf} + k\,K_{hf}$
$k_3 = 1 - K_f$
K_{hf} = the lateral head loss coefficient for when the end-gun is turned off from Eq. 14.33a
K_f = the supply system head loss coefficient for when the end-gun is turned off from Eq. 14.33b
k = 0.15 for uniform laterals (see Fig. 14.13) and 0.2 for tapered laterals

where ΔEl = actual difference in elevation from the pivot to the moving end of the lateral for the location under study, m (ft).

The $(Q_b)_{\text{off}}$ term affects both sides of the equation; thus a trial-and-error (or iterative solution process is required to determine $(Q_b)_{\text{off}}$. This happens, because as the flow decreases so will the head loss, but as the head loss decreases the flow increases. Therefore, the solution to Eq. 14.34 converges rapidly.

Figure 14.16 shows the relationships between the average discharge-weighted

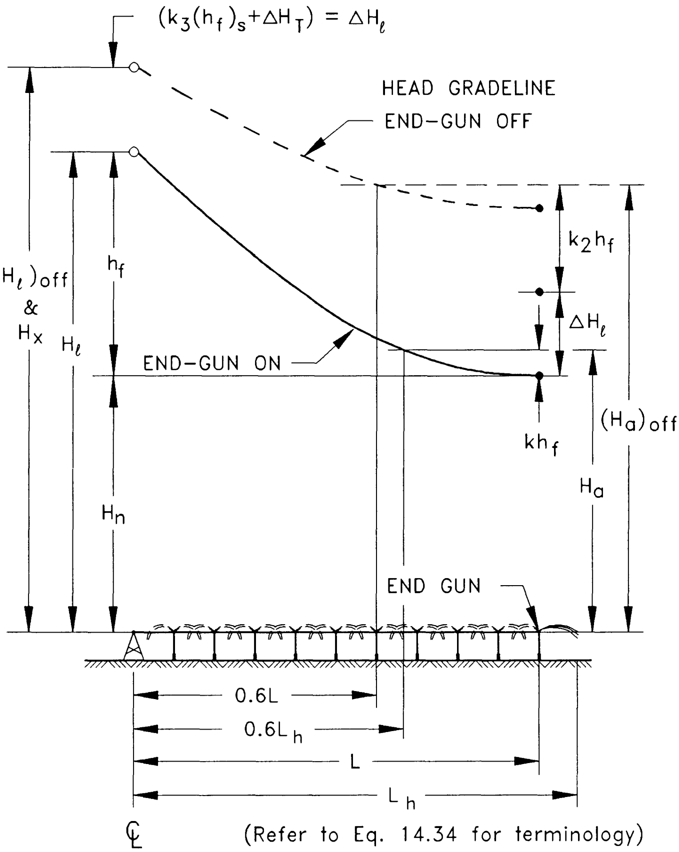


FIG. 14.16. Pressure Head Relationships Along a Center-pivot Lateral with Corner End-gun On and Off ($EL_p = 0$).

pressure heads for a lateral on a level field with the end-gun on and off. The inlet pressure head is designed for overcoming friction losses and providing the desired H_n when the end-gun is on. Therefore, there may be considerable excess pressure head when it is turned off. This is due to the combination of pump discharge head increases and pipe-friction-loss decreases. The location along the lateral where the average pressure loss occurs shifts inward when the end-gun is turned off because L_h decreases to L.

Pump discharge pressure head changes, ΔH_T, due to the large changes in Q_s when the end-gun is turned off, should be adjusted proportionately. This adds to the complexity of the iterative solution process, but fortunately it still converges rapidly. To do this let $(Q_b)_{\text{off}}$ at $(H_T)_{\text{off}}$ be a point on an appropriate pump H versus Q curve, and let $(Q_b + Q_g)$ and $(H_T)_{\text{on}}$ be another point on the curve. Then select the discharges for use in Eqs. 14.33a and 14.33b accordingly, and let $\Delta H_T = [(H_T)_{\text{off}} - (H_T)_{\text{on}}]$ in Eq. 14.34. Where the dynamic pumping lift is also affected by changes in Q_s, a similar process based on the combined well drawdown and pump curves should be incorporated into the analysis.

For situations where $(Q_b)_{\text{off}}$ is above some desired limit, say 1.05 Q_b, then:

1. The lateral cycle time could be decreased by increasing the travel speed, V_e, by the ratio of $(Q_b)_{\text{off}}/Q_b$ when the end-gun is not operating;
2. A booster pump could be added at the inlet to the lateral to offset the added pressure drop when the end-gun is operating; or an inlet control valve could be used to maintain the desired Q_b when the end-gun is not operating; or
3. Sprinkler discharge-control devices could be used along the lateral.

A strategy similar to the above could also be incorporated with Eq. 14.32 for dealing with ΔH_T due to changes in Q_s resulting from elevation differences.

When strategy 1 or 2 is used to correct for on-and-off end-gun operation (or due to elevation) the application uniformity over the basic irrigation circle will be adversely affected. However, by incorporating discharge-control devices, the application uniformity would be unaffected by the on-off operation of the end-gun or elevation changes along the lateral as it rotates.

Sample Calculation 14.10. *Estimating pressure requirements and discharge from a center-pivot lateral with an end-gun to irrigate the corners.*

GIVEN: The input information and computational results from Sample Calculations 14.8 and 14.9 and the center-pivot cost data presented in Table 14.6

The field is nearly level with some small undulations, so the difference in elevation throughout the basic circle is $\pm$ 1.0 m.

Water will be supplied from a canal by a centrifugal pump with the *H-Q* curve characteristics depicted in Fig. 12.6.

The end-gun will be operated in the corners (see Fig. 14.1); therefore the system flow rate will vary from Q_b = 57.0 L/s to $(Q_b + Q_g)$ = 68.5 L/s as the end-gun is turned on and off.

Sprinklers along the lateral are fitted with flexible-orifice, flow-control nozzles that also diffuse the jet and have an EU_p = 95%.

The absolute minimum desired pressure for the low pressure impact sprinklers, P_n' = 205 kPa (30 psi).

The desired end-gun pressure, P_g = 415 kPa (60 psi).

The height of the pivot elbow above the canal water level ΔEl_s = 4.5 m.

The sum of the friction head losses between the water supply and the pivot elbow $(h_f)_s$ = 10.0 m when Q_s = 68.5 L/s.

FIND: The required lateral inlet pressure head; the total dynamic head for the system; the centrifugal pump operating speed; the power required to operate the booster pump, assuming a 75% overall efficiency and an end-gun valve loss of 35 kPa; the cost of the center-pivot machine; and verify the need for discharge-regulating sprinklers.

CALCULATIONS: The pressure head that is equivalent to $P_n = 205$ kPa is $H_n = 20.9$ m (see Eq. 14.25b). By Eq. 14.27c the required lateral inlet pressure head for designing the nozzling package for the tapered lateral with $(h_f)_{8,6} = 7.5$ m when $Q_s = 68.5$ L/s and $\Delta El_p = 1.0$ m is:

$$H_l = 20.9 + 7.5 + 1.0 + 29.4 \text{ m}$$

The total dynamic head, TDH, is the sum of the following:

Minimum sprinkle pressure,	$H_n = 20.9$ m
Discharge regulator loss,	$H_{cv} = 0.0$ m
Friction loss in lateral,	$h_f = 7.5$ m
Pivot to high point,	$\Delta El_p = 1.0$ m
Friction loss in supply system,	$(h_f)_s = 10.0$ m
Supply to pivot elbow,	$\Delta El_s = 4.5$ m
	TDH = 43.9 m

Converting to English units for entering Fig. 12.6, TDH = 144.0 ft and Q_s = 1086 gpm; and from the pump characteristic curves the required pump operating speed is 1800 rpm. Figure 14.17 shows an expanded view of the pertinent section of the 1800-rpm *H-Q* curve.

The required pressure head for the end-gun is $H_g = 415/9.8 = 42.3$ m and the head loss though the valve is $h_{cv} = 35/9.8 = 3.6$ m. Therefore, the booster pump must increase the available head at the end of the lateral by:

End-gun operating head,	$H_g = 42.3$ m
On-off valve loss,	$h_{cv} = 3.6$ m
Head available at end,	$(H_n = 20.9$ m$)$
Booster pump	$H = 25.0$ m

For $Q_g = 11.5$ L/s and an overall motor-pump efficiency of 75%, by Eq. 12.26 the power requirement to operate the booster pump is:

$$\text{BP} = \frac{25.0 \times 11.5}{102 \times 75/100} = 3.8 \text{ kW}$$

Based on the figures in Table 14.6, the approximate cost of the center-pivot machinery with the end-gun will be:

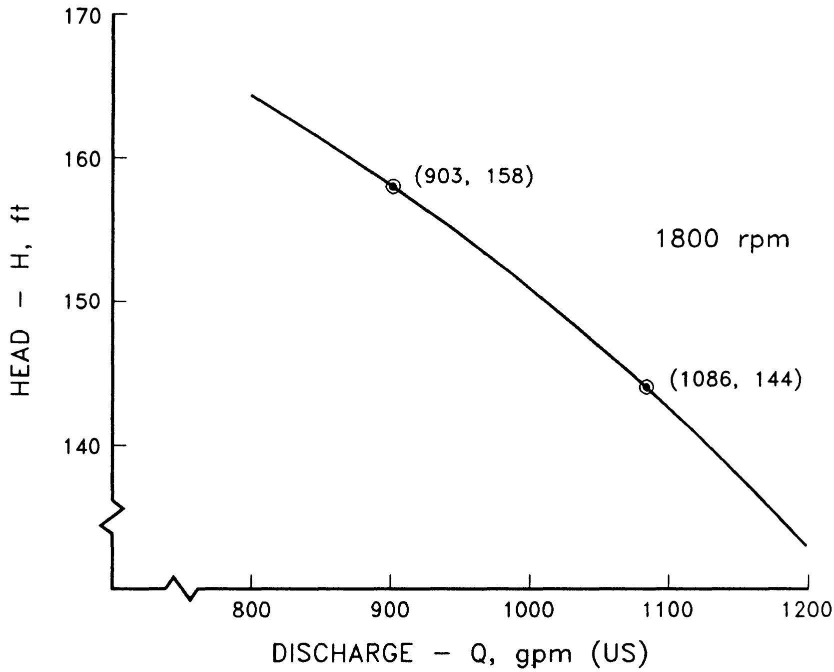


FIG. 14.17. Section of Head-discharge Curve from Pump Curve Fig. 12.6.

228 m of 8-in. @ \$115/m,	\$26,220
170 m of 6-in. @ \$100/m,	17,000
Pivot, generator, and controls,	7,000
End-gun package, 5-hp,	2,700
Special nozzles @ \$3/m,	1,200
Total	\$54,120

With flow-regulating nozzles, even though the pressure head varies as the lateral rotates, the discharge does not; thus $(Q_b)_{off} = Q_b$. When the end-gun is turned on and off, the available head at the moving-end of the lateral will vary from a low of $H_n = 20.9$ m (see Fig. 14.16) to:

Minimum sprinkler head,	$H_n = 20.9$ m
High-low elevation,	$\Delta El = 2.0$ m
Reduction in h_f,	$\Delta h_f = 2.6$ m
Reduction in $(h_f)_s$,	$(\Delta h_f)_s = 2.9$ m
Pump head difference	$\Delta H_T = 4.3$ m
Maximum head at moving end	$H = 32.7$ m

The reduction in h_f due to the change in flow as the end-gun is turned off was determined by Eq. 14.33a assuming $Q_b \approx (Q_b)_{off}$, so that:

$$(h_f)_{\text{off}} = 7.5\ (57.0/68.5)^{2.352} = 4.9 \text{ m}$$

therefore, $\Delta h_f = 7.5 - 4.9 = 2.6$ m.

The reduction in $(h_f)_s$ due to the change in flow was determined by Eq. 14.33b as:

$$(h_{fs})_{\text{off}} = 10.0\ (57.0/68.5)^{1.852} = 7.1 \text{ m}$$

therefore, $(\Delta h_f)_s = 10.0 - 7.1 = 2.9$ m.

The difference in the pump discharge head was determined from Fig. 14.17. For $Q = 57.0$ L/s (903 gpm), $H = 48.2$ (158 ft), thus:

$$\Delta H_T = 48.2 - 43.9 = 4.3 \text{ m}$$

Based on the above, the maximum sprinkler pressure head variation for any sprinkler (assuming $(Q_b)_{\text{off}} = Q_b$) would be:

$$\Delta H = 32.7 - 20.9 = 11.8 \text{ m}$$

With such a relatively large (compared with H_a) increase in pressure head, $(Q_b)_{\text{off}}$ would increase considerably above Q_b, and this would reduce the amount of excess pressure head. The first step for determining whether discharge regulation is necessary is to assume an equilibrium $(Q_b)_{\text{off}}$, for example, 63.0 L/s which is midway between 68.5 and 57.0 L/s.

From the denominator of Eq. 14.34, with $k = 0.2$ for the tapered lateral:

$$H_a = 20.9 + 0.2(7.5) + 1.0 = 23.4 \text{ m}$$

Assuming $\Delta El \approx 0$; and from Fig. 14.17 with $Q = 63.0$ L/s (999 gpm), the pump discharge head will be $H = 46.0$ m (151 ft) and $\Delta H_T = 2.1$ m (7 ft). Substituting into the numerator of Eq. 14.34 gives:

$$(H_a)_{\text{off}} = 20.9 + 1.0 + 0.34(7.5) + 0.144(10) + 2.1$$

$$= 28.0 \text{ m}$$

Therefore, from Eq. 14.34, the expected discharge with the end-gun turned off is:

$$(Q_b)_{\text{off}} = 57.0(28.0/23.4)^{0.52} = 62.6 \text{ L/s}$$

This is almost the same as the assumed equilibrium discharge of 63.0 L/s, so use an equilibrium discharge of $(Q_b)_{\text{off}} = 62.8$ L/s, for which $(H_a)_{\text{off}} = 28.2$ m.

Based on the above, the average between H_a on and off values is:

$$(H_a)_{\text{av}} = (28.2 + 23.4)/2 = 25.8 \text{ m}$$

And the average difference between the sprinkler pressure head along the lateral when the end-gun is on and off is:

$$\Delta H_{av} = 28.2 - 23.4 = 4.8 \text{ m}$$

Using Eq. 14.29b to verify the need for flow-control nozzles gives:

$$\left[\Delta H > 2.5\left(1 - (\mathrm{EU}_p/100)^2\right)H_a\right]_{av} \tag{14.29b}$$

However:

$$4.8 < 2.5\left(1 - (95/100)^2\right)25.8 = 6.3 \text{ m}$$

Because 4.8 < 6.3 m, flow-control nozzles will actually reduce the application uniformity. Therefore, they are not needed providing the excess discharge and increased application rate do not create a runoff problem.

The best strategy for operation would be to increase the travel speed when the end-gun is off, so the application depth would remain the same. Thus, V_s = 1.14 m/min (from Sample Calculation 14.7) should be increased to:

$$(V_e)_{off} = V_e\left(0.1 + 0.9(Q_b)_{off}/Q_b\right) \tag{14.35}$$

$$= 1.14(0.1 + 0.9 \times 62.8/57.0) = 1.244 \text{ m/min}$$

Use of the 0.9 factor in Eq. 14.35 is based on the assumption that application efficiency will be somewhat impaired by the pressure head change.

Because the end-gun will be off about half of the time, the cycle time, T_c, will be decreased from 36.7 hr to approximately:

$$T_c = 36.7\left[2 \times 1.14/(1.14 + 1.24)\right] = 35.2 \text{ hr}$$

Thus, the stop time should be increased from 3.3 to 4.8 hr after each cycle. This in itself would represent a decrease in pumping time and energy cost of approximately 100(36.7 − 35.2)/36.7 = 4%.

SYSTEM EVALUATION AND SELECTION

Subtopics under this heading include: field evaluation of center-pivot application uniformity; determining power requirements to operate electric-drive systems; and machine selection.

Application Uniformity Evaluation

Fully evaluating the application uniformity over the entire area irrigated provides a means for testing the nozzling package. To carry out a full evaluation requires determining the circular as well as the radial application uniformity.

Circular Uniformity. The circular uniformity is a measure of the application uniformity along concentric circular paths at constant radial distances, r_j, from the pivot end. An ideal location for measuring the circular application uniformity of the basic irrigated circle is along the circular path at $r_j = 2L/3$. This is the location of the center of mass of the discharges along the lateral.

For equally spaced collectors, along the circular row, each is representative of the same irrigated area. Therefore, the circular uniformities CU_c or DU_c can be computed in the conventional way by Eqs. 6.1 and 6.2, respectively.

The general guidelines for field evaluation of sprinkler systems (as to catch containers and accounting for evaporation losses from the containers) given in Chapter 6 should be followed. Assuming the purpose of the evaluation is to test the accuracy of the nozzling, the test should be conducted when it is calm and evaporation losses are on the low side of average. To gather the field data, the collectors should be spaced 30 m (100 ft) apart along the nearest wheel path to $2L/3$. For a standard 400-m (1320-ft) lateral, this will require approximately 55 containers. To improve the accuracy, the average catch values from a double row of collectors set about 5 m (16.5 ft) to either side of the wheel-path can be used.

To carry out the circular uniformity test, the lateral cycle time should be the regular T_c or set so $d \approx 15$ mm (0.6 in.), whichever is less. The data should be collected several times during the test period because evaporation from the containers will be significant, especially between 12:00 and 18:00 hr. A reasonable collection schedule would be at 1:00, 7:00, 10:00, 13:00, 15:00, 17:00, 20:00 hr.

Even with the above collection schedule, the data should be adjusted for evaporation losses. This can be done by placing a known amount of water in each of two control collectors set outside the test area. The remaining water in them can then be measured to determine the evaporation between scheduled readings. The collected application data can then be adjusted in accordance with the length of time water had been collected prior to being measured.

Radial Uniformity. The radial application uniformity is a measure of the uniformity across the circular paths traveled by the sprinklers on the lateral. The radial uniformity across the basic irrigated circle is of primary importance. But where an end-gun or corner system is used, the uniformity between L and R should also be evaluated. Furthermore, the radial uniformity across the basic irrigated circle should be evaluated with the corner system on and off and with the lateral on the contour and in the extreme uphill positions.

Two parallel rows of collectors should be used to gather each radial line of data. The rows should be approximately 1.5 m (5 ft) to either side of the radial line. The collectors should be equally spaced and approximately 10 m (33 ft) apart in each row, and the rows should extend from 0.2 L to L. For systems with end-guns, the rows should be extended from L to R using a 3-m (10-ft) spacing between the collectors beyond L.

The irrigated area represented by each pair of collectors in a radial row is the circular band having a length equal to $2\pi r_j$ and width equal to the spacing between collectors. Since each pair represents a different irrigated area, the uniformity equations must be modified accordingly (Merriam and Keller, 1978). First the weighted average depth of catch must be determined by:

$$y_a = \frac{\Sigma 2\pi k r_j y_j}{\Sigma 2\pi k r_j} = \frac{\Sigma r_j y_j}{\Sigma r_j} \tag{14.36}$$

where

y_a = weighted average depth of catch, mm (in.)
k = constant distance between collectors, m (ft)
r_j = radial distance to collector, m (ft)
y_j = average depth of catch at r_j, mm (in.)

The radial application uniformity of the low quarter of the area requires ranking and adding the area weighted depth of catch values in ascending order. This must be done until the area represented is equal to one-fourth of the total area of the basic circle. Thus, first rank the y_j values. Then select the set of y_j values such that the sum of the corresponding r_j values equals one-fourth of the sum of all r_j values found in Eq. 14.36. Then using this set of y_j and corresponding values r_j find:

$$\mathrm{DU}_r = 100 \times \frac{\Sigma \text{ low quarter } r_j y_j}{y_a(\Sigma \text{ low quarter } r_j)} \tag{14.37}$$

The radial coefficient of uniformity is easier to determine, because it does not require ranking the catch values (Herrman and Hein, 1968). It can be found by:

$$\mathrm{CU}_r = 100\left(1.0 - \frac{\Sigma r_j \left| y_j - y_a \right|}{\Sigma r_j y_j}\right) \tag{14.38}$$

where DU_r = center pivot radial distribution uniformity based on the low quarter, %, and CU_r = center pivot radial uniformity coefficient, %.

For determining the uniformity values for the basic irrigated circle, use (y_j, r_j) values for $r_j \leq L$. For evaluating end-gun performance, use values so that $L < r_j \leq R$.

The composite of circular and radial center pivot uniformity values can be determined by:

$$(\mathrm{DU})_{cp} = \mathrm{DU}_r(\mathrm{DU}_c/100) \tag{14.39a}$$

or

$$(\mathrm{CU})_{cp} = \mathrm{CU}_r(\mathrm{CU}_c/100) \tag{14.39b}$$

Typical field uniformity values are lower than composite values. Typically CU_r values for well-designed systems with impact sprinkler range from 93 to 96%, and with spray nozzles from 91 to 95%. Depending on topography and the use of corner systems and/or discharge-control devices, CU_c values range from 90 to 98%. This gives composite CU values ranging between $82\% < (CU)_{cp} < 94\%$. On an average, for well-designed center-pivot systems, $(CU)_{cp} \approx 90\%$.

Drive Power Requirements

Most standard, electric-powered, center-pivot machines use worm gears and have a minimum cycle time of a little less than 24 hr. Such machines require 1.0 kW per drive-unit to operate. High-speed machines capable of cycling twice per day require 1.5 kW per drive unit. Machines with planetary instead of worm gearing require only 0.75 and 1.0 kW per drive unit for regular and high-speed operation, respectively. This is because planetary gearing is more efficient than worm gearing.

When an end-gun booster pump is used, additional power must be supplied to operate it. The three most typical end-gun booster pump motors used are 2, 5, or 7.5 hp, which for design purposes can be assumed to require 2.5, 5, or 7.5 kW, respectively. Thus, a 15-kW source of power must be supplied at the pivot to operate a standard 10 drive-unit center-pivot machine with a 5-hp end-gun booster pump. Where power is not available from a central source, a suitable generator must be provided at the pivot.

Machine Selection

Most systems are powered by electricity, but some have hydraulic-drive units to rotate the lateral. Hydraulic-drive machines are 10 to 20% more expensive, but they may require less maintenance, last longer, and provide more uniform water applications. This is because all drive units move continuously at the proper speed required to maintain alignment. This is done by automatically adjusting the rate of the hydraulic-power fluid flow at each drive unit.

With electric-driven machines, the drive units must cycle on and off to maintain the necessary speed and alignment. This is because during their on-cycle, all drive units move at the same speed. This on-off motion causes considerable wear on the moving parts. It also creates circular water distribution problems that smooth out during repeated lateral cycles. However, when applying chemicals through the system during only one cycle, the low uniformity may be a problem.

Ultimately the designer must select: the type, power, and speed of the drive system; the type of pipe and protective coating; the span length and lateral height; type of end-gun or corner system; the wheel and tire sizes; and the supplier. During the selection process local field experience and availability of service as well as cost should be considered.

Some considerations as to machine suitability are:

- For the application of chemicals, a drive system capable of providing a fast (12 hr) rotation speed is desirable.
- On undulating terrain, span length may need to be adjusted to keep the lateral from scraping the crop or ground.
- On unstable soils, high-flotation tires may be required.
- For steep and undulating terrain, heavy-duty drive systems are needed.
- Some irrigation water may attack galvanized pipe, in which case epoxy-coated pipe and structures are recommended.

REFERENCES

Beggs, R. G. 1981. A portable field sprinkler infiltrometer. M.S. thesis, Utah State Univ., Logan, Utah.

Chu, T. S., and D. L. Moe. 1972. Hydraulics of center-pivot systems. *ASAE Transactions* 15(5):894–896.

Herrmann, D. F., and P. R. Hein. 1968. Performance characteristics of self-propelled center pivot sprinkler irrigation systems. *ASAE Transactions* 11(1):11–15.

Hill, R. W. 1989. Personal communication, January 1989. Agricultural and Irrigation Engineering Dept., Utah State Univ.

Hill, R. W., E. L. Johns, and D. K. Frevert. 1983. *Comparison of Equations Used for Estimating Agricultural Crop Evapotranspiration with Field Research*. Denver, Colorado. Bureau of Reclamation, U.S. Department of Interior.

Kincaid, D. C., J. R. Busch, I. McCann, and M. Nabil. 1987. Evaluation of very low pressure sprinkler irrigation and reservoir tillage for efficient use of water and energy. Final report, U.S. Dept. of Energy, Contract DE-A179-86BP62961.

Merriam, J. L. and J. Keller. 1978. *Farm Irrigation System Evaluation*, 3rd ed. Logan, Utah: Agricultural and Irrigation Engineering Dept., Utah State Univ.

Reinders, F. B. and A. A. Louw. 1985. The measurement of infiltration as a design input for mechanized irrigation systems. *S.A. Irrigation* 7(1):9–21.

15

Linear-Moving System Design

In many respects linear-moving sprinkle systems are similar to center-pivots, and a set of rather parallel strategies can be used for designing them. Both irrigate while moving, and their lateral structures, moving, and guidance mechanisms are the same (see Figs. 4.10 and 4.12).

COMPARISONS WITH CENTER-PIVOTS

The major mechanical difference between linear-moving and center-pivot laterals is that the inlet end of a pivoting lateral is fixed, but the inlet end of a linear-moving lateral travels at the same speed as the distal end (see Fig. 15.1). This creates four major problems for linears that are avoided by pivots, namely:

1. Water under pressure must be available to supply the lateral along the entire length of the field (rather than at a fixed point in the center). The water can be:
   a. Pumped from an open channel, with the pumping plant attached to and moving along with the lateral (see Fig. 4.12);
   b. Transferred under pressure from a main supply pipe through a drag hose that is long enough to allow the lateral to move approximately 200 m (660 ft between hose changes, see Fig. 15.2); or
   c. Supplied under pressure through a robot mechanism that automatically attaches to, opens, and then closes special valves spaced 12 to 18 m (40 to 60 ft) apart along a buried main line (see Fig. 15.3).
2. Electric or diesel power to operate the irrigation machine must be available along the entire length of the field (rather than at a fixed point in the center). Systems powered by electricity require cumbersome, heavy-duty extension cords.
3. The lateral's inlet end must move and be guided along the full length of the edge of the field (rather than being stationary). This requires a guideline stretched along the field and a special alignment mechanism on the moving lateral to follow the line (see Fig. 15.2).
4. At the end of each forward run the lateral must be reversed and returned to the starting position to complete an irrigation cycle (whereas pivots are cycled continuously in the same direction).

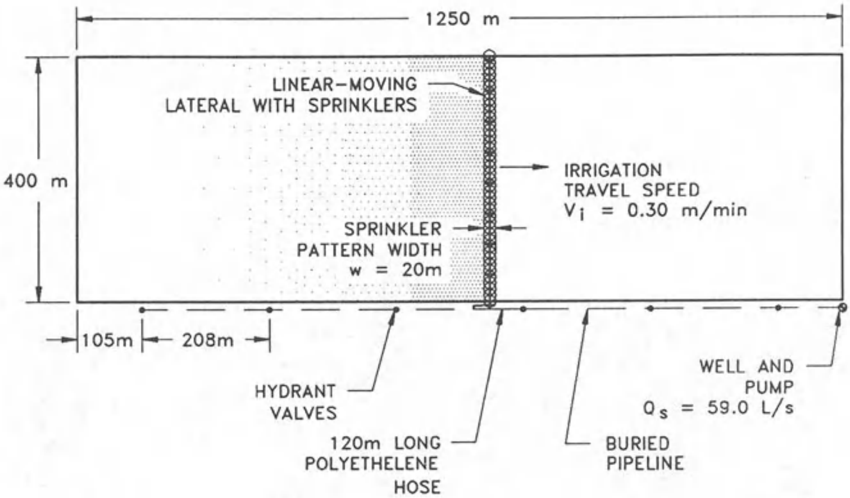


FIG. 15.1. Layout and Components of a Hose-fed, Linear-moving Irrigation System for a 50-ha Field.

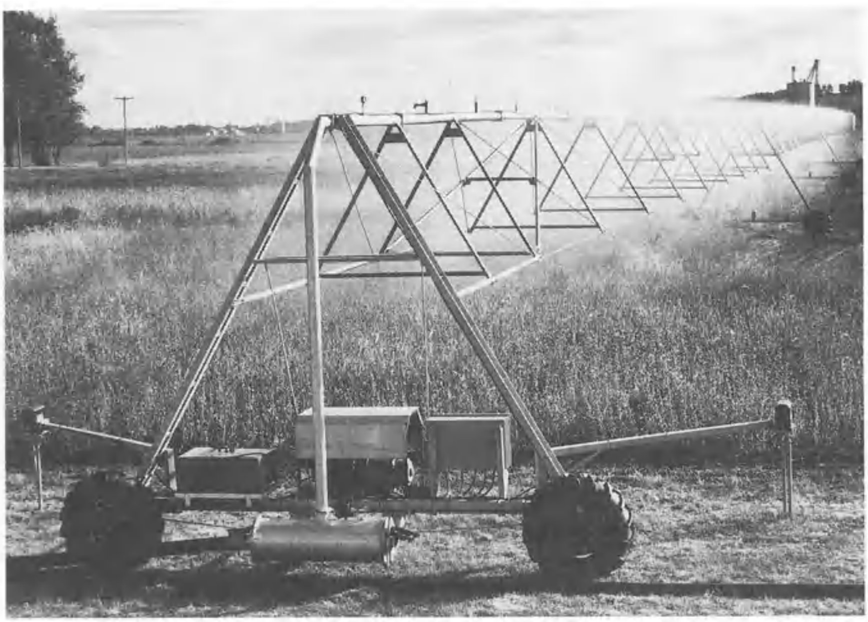


FIG. 15.2. Hose-fed, Linear-moving Sprinkle Irrigation Machine (Source: Valmont Industries Inc.).

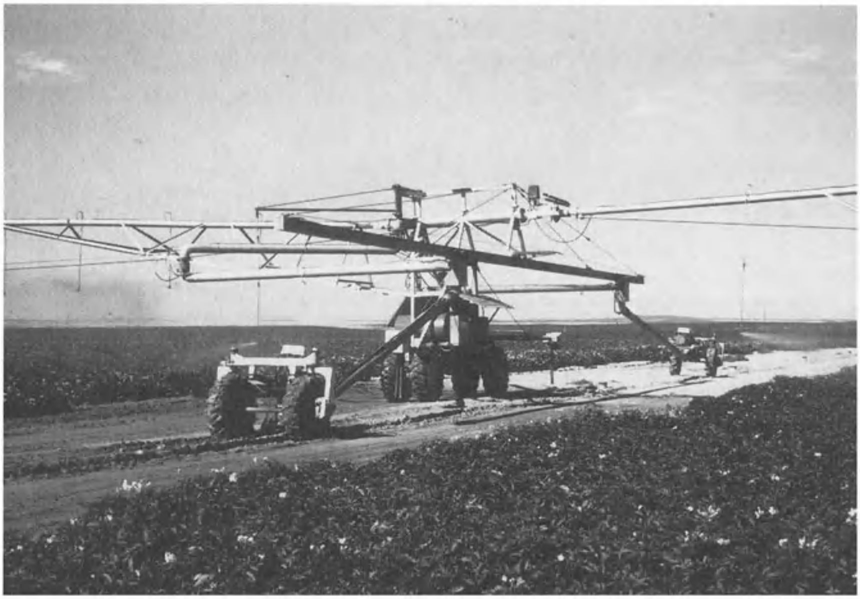


FIG. 15.3. Robot-fed, Linear-moving Sprinkle Irrigation Machine (Source: Valmont Industries Inc.).

Because of the above complications, linear-moving machines are considerably more expensive to purchase, operate, and maintain than center-pivots. Furthermore, they are restricted to operating on long, smooth to gently rolling fields with slopes of not more than 2 to 3% in the direction of travel and 1 to 1.5% along the lateral. Consequently, they are not nearly as popular as pivots. For example, the authors estimate that at the beginning of 1990 there were about 160,000 center-pivots worldwide, but only about 2000 linear-moving laterals.

Linears do have some important advantages over pivots, the most important of which is that they irrigate rectangular fields. In addition, they provide higher uniformity and lower intensity irrigation than pivots.

SYSTEM CONFIGURATION AND OPERATION

The first step in the design process is to choose the lateral layout, water-feed system, and the strategy for cycling the lateral forth and back across (or up and down) the field.

Layout and Feed System

Linear-moving laterals can be fed from one end or at the center. Center-feed systems can be 200–500 m (660 to 1650 ft) long on each side. End-feed systems can be up to 600 m (2000 ft) long.

Although the laterals can be designed to be moved laterally when empty or even in a circular fashion (like a pivot) at the ends of fields, this is seldom done. Changing from one mode of operation to another is troublesome and time-consuming; therefore, forth-and-back operation is the norm. For a linear-moving machine to be cost-effective, the field it irrigates must be long enough to more or less utilize its full capability. For most linears this will be from 1000 to 1600 m (3300 to 5300 ft) long.

Choosing the method of feeding (or delivering) water to the moving lateral is an important design decision. The three basic options for doing this, as mentioned earlier, are: canal-fed, hose-fed, and robot-fed.

Canal-fed. If a canal is used, a pumping plant must move along with the lateral. The ideal situation is for the canal to be concrete-lined and level and thus act as a reservoir with clean water along the side or the middle of the field. Floats may be installed to regulate the water level. The traveling pumping plant simply draws water from the reservoir as the lateral moves beside it.

On sloping fields, where the cost of constructing a level canal would be prohibitive, one of two approaches may be used. The canal may be divided into a few level stretches, with the water in each section controlled by gates at the inlet and outlet of that section. The other is to construct a sloping canal.

If a sloping canal is used, a traveling dam will usually be necessary to form a pool at the traveling pump's suction inlet. The dam must have an overflow because it is impractical to expect to exactly balance the canal inflow rate with the rate at which water is pumped out. Therefore, to keep the linear system operating efficiently, it is advisable to supply 5 to 10% extra water to the canal and provide a tail water-recovery (or disposal) system for it.

Level canals may or may not be lined, depending on local soil conditions and anticipated seepage losses. However, sloping canals must usually be lined to prevent excess seepage and erosion. Furthermore, lining reduces the amount of trash to contend with at the suction inlet screen. Whether lined or unlined, open-channel delivery systems lose some water to seepage and evaporation. This must be taken into account when designing the water-supply system.

Hose-fed. Polyethelyene (PE) hoses 100 to 120 m (330 to 400 ft) long with inside diameters of 121 or 152 mm (4.75 or 6.0 in.) are typically used to supply water under pressure to hose-fed linears. Like cable-drawn travelers, they drag the hose so they can travel continuously for almost twice the hose length (see Fig. 13.6). Therefore, a quick-coupling valved outlet (or hydrant) is needed only about every 200 m (660 ft) along the main line to supply water through the hose to the moving lateral (Figs. 15.1 and 15.2).

Pipe-friction losses through the main line, hose, valves, and fittings are usually quite high, so hose-fed linears require considerably more energy to operate than do canal-fed systems. They also require more labor and reset or down time, but they can be used on sloping and undulating fields where a canal would not be practical.

Each time the lateral travels the maximum distance permitted by the hose, the system is designed to automatically stop. The hose must be manually disconnected then dragged and connected to the next hydrant along the main line. The reset time associated with each such hose move is about 30 min.

Robot-fed. This method of supplying water under pressure to a linear-moving lateral is completely automatic. The main supply line must be buried and have a hydrant every 12 to 18 m (40 to 60 ft). As the lateral advances, a pair of coupling devices (see Fig. 15.3) automatically attach to the hydrant it is approaching then release the hydrant it has passed.

The advantages of robot-fed over hose-fed water-transfer systems are: they have considerably less pressure loss; and being fully automatic they require no reset labor and time. Their main advantage over canal-fed systems is that they can be used for sloping or rolling fields where a canal would be impractical.

The main disadvantage of robot-fed systems is the high initial and maintenance costs of the water-transfer mechanism. The two coupling and uncoupling devices must work flawlessly in tandem to provide a continuous water supply to the lateral. Furthermore, the hydrants must be closely spaced for the coupling devices to be manageable, and this adds further to the overall cost.

System Operations

Linear-moving machines must travel forth and back, which is twice the length of the field, to complete each irrigation cycle. This can be done with the sprinklers operating continuously or for only half of the total distance traveled each cycle. Each method has its advantage and disadvantage.

If the sprinklers are operated continuously the total operating time can be longer. This reduces system flow and application rate requirements. However, the area that was most recently watered near the ends of the field will be immediately rewatered each time the lateral reverses direction. On the other hand, if the system is returned with the water shut off (or empty) the operating time will be decreased accordingly. For a typical maximum travel speed of about 2 m (6 ft) per minute and field lengths of 1000 to 1600 m (3300 to 5300 ft), the empty return will take from 8 to 13 hr.

The continuous versus part-time watering and the possibility for backtracking over relatively dry rather than fully irrigated soil leads to four basic cycling strategies. These are:

1. To irrigate in only one direction and return empty (with the water off) as quickly as possible;
2. To irrigate while traveling at the same speed in both directions;

3. To irrigate half the field and quickly continue to the end with the water off, then on the return irrigate the other half of the field and quickly continue back with the water off; and
4. To proceed as in strategy 3, but continuing as quickly as possible with the water on (instead of off) during the last half of travel in either direction.

The purpose of strategy 3 is to eliminate returning across soil that has just been fully irrigated. Otherwise it is like strategy 1. The purpose of strategy 4 is to reduce the hazard associated with watering back across the areas at the ends that have just been fully irrigated. However, both strategies 3 and 4 require more management and labor than 1 and 2.

DESIGN STRATEGIES

The design strategies and equations needed for linear-moving laterals can either be taken directly or easily derived from those developed for center-pivot laterals. The differences are associated with either the rectangular versus circular lateral movement or the fact that end-guns are not used on linear machines. Furthermore, linears can be used only on fields that have small elevation differences across them.

In view of the above, the special equations for end-guns and to accommodate pressure head changes due to elevation differences are not needed for linears. For the most part, the only pivot equations that require modifications for use with linears are those that contain either an r_j, I_j, or other radius-related term.

Equations for Linears

A list of equations for use with linear-moving laterals is given below. Each equation that was altered is preceded by the respective center-pivot equation number in square brackets, [ ], for the reader's convenience. Those that are the same for both types of systems, as well as those that have no respective applicability for linears, are indicated accordingly. Equations 15.2a and 15.2b have no respective pivot equation number, because they are unique for linears.

The equations needed for designing linear-moving systems are:

[14.1a and b]: Same for both pivots and linears.

[14.2]:
$$Q_s = \frac{L\, L_f d'}{K\, T} \qquad (15.1)$$

where

Q_s = system capacity, L/s (gpm)
L = length of lateral, which is the width of the field, m (ft)
L_f = length of field (see Fig. 15.1), m (ft)
d' = gross daily depth of water required during peak water-use period, mm (in.)
K = conversion constant, 3600 for metric units (96.3 for English units)
T = average watering time per day, hr

The average watering time per day when irrigating in one direction only is:

$$T = \frac{24(L_f/V_i)}{L_f/V_i + L_f/V_r + RT + CT} = \frac{24(L_f/V_i)}{f'} \tag{15.2a}$$

and when irrigating in both directions:

$$T = \frac{24(L_f/V_i + L_f/V_r)}{L_f/V_i + L_f/V_r + RT + CT} \tag{15.2b}$$

where

f' = irrigation interval, min
V_i = travel speed when applying main irrigation application, m/min (ft/min)
V_r = maximum travel speed when returning empty or applying light irrigations, m/min (ft/min)
RT = total reset time per cycle for reversing matching, fueling, and moving hoses as needed, min
CT = time allowed each cycle for field-drying and contingencies, min

[14.3]:
$$I = \frac{d\,R_e}{60\,T_a} \tag{15.3}$$

[14.4a]:
$$I = \frac{L_f}{w} \cdot \frac{d'R_e}{T} \tag{15.4a}$$

[14.4b]:
$$I = \frac{L_f}{w} \cdot \frac{K\,Q_s R_e}{L\,L_f} \tag{15.4b}$$

where

I = average application rate, mm/hr (in./hr)
d = gross depth of application, mm (in.)
R_e = effective portion of applied water, decimal

T_a = application time, hr
w = width of stationary application pattern, m (ft)
K = same conversion constants as for Eq. 15.1.

[14.5, 14.6, and 14.7]: Same for both pivots and linears.

[14.8]: Same for both pivots and linears, except drop j subscripts.

[14.9]:
$$\text{LET} = I = \frac{100\ L_f\, k_f\, k_c ET}{w\ T\ DE_{pa}} \tag{15.5}$$

where LET is an application rate indix dependent on the field length L_f and other terms are as defined for Eq. 14.9.

[14.10 through 14.15]: Same for both pivots and linears with one-way watering for which $(L_f/60V_i)_x = T_{cx}$ in Eq. 14.12. However, since (unlike for pivots) having a very short irrigation interval for linears is complicated and costly, let $\text{MAD}_a = 30\%$ for fine- or medium-textured soils and 40% for coarse-textured soils in Eq. 14.10.

[14.16]:
$$d_n = \frac{K\ Q_s T_w DE_{pa} R_e}{6000\ L\ L_f} \tag{15.6}$$

where

d_n = net depth of irrigation application, mm (in.)
T_w = total watering (or irrigating) time per lateral cycle forth and back, min
DE_{pa} = sprinkler distribution efficiency for percentage area, pa, adequately irrigated, %
K = same conversion constants as for Eq. 15.1

[14.17 and 14.8]: Same for both pivots and linears with one-way watering assuming:

$T_c = L_f/60\,V_i$, which is the desired time to travel the length of the field while irrigating, hr
$(T_c)_n = L_f/60\,V_r$, which is the minimum time required to travel the length of the field, hr

[14.19]:
$$V_i = \frac{PT\ L_f}{6000(T_c)_n} = \frac{PT\ V_r}{100} \tag{15.7}$$

where PT = percentage on-time at control drive unit, %.

[14.20a]:
$$q = \frac{Q_s S_e}{L} \tag{15.8a}$$

where q = desired discharge for all outlets along lateral, L/s (gpm), and S_e = spacing between outlets, m (ft.).

[14.20b]:
$$q_g = \frac{L_f(L - L')}{L\,L_t} Q_s; \qquad \text{for } L' > 0.97\,L \tag{15.8b}$$

where q_g = discharge of part-circle sprinkler used to square-off the outer edges of the application profile, L/s (gpm), and L' = actual length of lateral pipe, m (ft).

[14.21 through 14.24]: Not applicable for linear-moving laterals.

[14.25a]: Same for both linear-moving laterals and laterals that are not moving while watering, which is:

$$h_f = J\,F\,\frac{L}{100} \tag{8.8a}$$

[14.25b]: Same for both pivots and linears

[14.26a and b]:
$$h_{f_j} = h_f\left[1 - \left(\frac{L - l_j}{L}\right)^{2.852}\right] \tag{15.9}$$

where

h_{f_j} = pipe-friction loss from the lateral elbow to l_j, m (ft)
h_f = total pipe friction loss computed by Eq. 8.8a, m (ft)
l_j = linear distance from lateral inlet (at the elbow) to the outlet (or point) under study, m (ft)

[14.27a and 14.27b]: Same for both pivots and linears, letting subscript j designate the distance, l_j, from the lateral inlet elbow to the outlet under study.

[14.27c]: The average pressure head at any outlet along linear moving laterals is:

$$\begin{aligned}(H_j)_{\text{av}} &= (H_n + h_f + \Delta El_x) - h_{f_j} \\ &= H_l - h_{f_j} - (\Delta El_j)_{av}\end{aligned} \tag{15.10}$$

where

$(H_j)_{av}$ = average pressure head at any linear distance, l_j, from the lateral elbow, m (ft)
H_n = desired minimum pressure head at distal end of lateral, m (ft)
ΔEl_x = maximum elevation difference between the inlet and distal ends of the lateral, m (ft)
$(\Delta El_j)_{av}$ = average ground elevation difference between the lateral inlet and any linear distance, l_j, from it, m (ft)
H_l = lateral inlet pressure head required at the moving elbow on top of the control tower, m (ft)

For canal-fed linears H_l is nearly constant, because elevation variations along the canal are very small. However, for hose-and robot-fed linears, pressure head differences due to both pipe friction and elevation along the main line should be taken into account, and H_l should be the average lateral inlet pressure head in Eq. 15.10.

[14.28 through 14.30]: Same for both pivots and linears. However, linears can be operated only on fields with minor elevation differences, and they do not have corner sections. Therefore, sprinkler flow-control devices are not required for canal-fed systems. For hose- and robot-fed systems, pressure variations between the main line outlets may be large enough so that flow control is required at the lateral inlet or for each lateral outlet.

[14.31 through 14.35]: Not applicable to linears.

[14.36 through 14.38]: Replace with Eqs. 6.1 and 6.2, because the catch in each container is representative of an equal area of land.

[14.39]
$$CU_{lm} = CU_l CU_t / 100 \tag{15.11}$$

where

CU_{lm} = composite coefficient of uniformity for linear-moving lateral, %
CU_l = linear uniformity coefficient along linear-moving lateral across the direction of travel, %
CU_t = travel direction uniformity coefficient under the midsection of the linear-moving lateral along the direction of travel, %

System Design

The procedures for designing linear-moving systems are about the same as those presented for center-pivot systems in Chapter 14. The traveling part of a linear system is as easy to design as a pivot without a corner system on a nearly level field. This eliminates the design complexities associated with on-off corner system operation and large elevation differences.

In the section above only 10 of the 39 equations presented for pivots in Chapter 14 needed to be modified (and 12 are not applicable) for linears. Furthermore, Eqs. 15.2a and 15.2b, related to the average watering time, are the only equations that are completely unique for linears. This is because of the unavoidable loss of potential watering time to reset and recycle linears. This may be up to 25% of the total time in the irrigation interval.

The comments given along with the equations in the following sample calculations are presented to supplement the related text material for pivots.

Sample Calculation 15.1. *Determining the system capacity and application rate for a hose-fed, linear-moving system.*

GIVEN: A 50-A, rectangular corn field 400 m wide and 1250 m long, as shown in Fig. 15.1, with a hose-fed linear fitted with low-pressure impact sprinklers. The following site conditions, desired irrigation performance, and nozzling characteristics as given for or computed in Sample Calculation 14.1:

Peak water requirement, $U_d = 7.0$ mm/day;
Desired percentage adequacy, $pa = 80\%$;
Sprinkler pattern width, $w = 20$ m;
Effective portion of discharged water, $R_e = 0.94$; and
The leaching requirement, $LR < 0.1$.

A reset time of 20 min for each valve change and to reverse the lateral at each end.

The anticipated uniformity of application, $CU_{lm} = 92\%$.
An anticipated irrigation interval, $2.5 < f < 5$ day.
The maximum lateral travel speed, $V_r = 2.0$ m/min.
A valve spacing of up to 210 m along the main line.
Irrigation while traveling in only one direction is preferred.

FIND: The required system capacity and average application rate.

CALCULATIONS: The system capacity and application both depend on the average operating time per day, T. The T is a function of the irrigation interval, f', and the travel speed when applying water, V_t, which is not known yet. Therefore, f' must be chosen and T estimated.

Using Eq. 14.10 with $MAD_a = 30\%$ (instead of the 20% used in the pivot system in Sample Calculation 14.6) gives a maximum interval of:

$$f_x = \frac{0.24 \times 30 \times 167 \times 0.6}{7.5} = 96 \text{ hr}$$

Thus, the maximum interval could be 4 days, but an interval of $f' = 3.5$ days will be used. This is because it gives exactly two irrigation cycles per week and will advance the cycle one-half day each time.

The rationale for estimating T is as follows: with a valve spacing of 208 m the total reset time, $RT = 8 \times 20 = 160$ min.; the return time, $L_f / V_r = 1250/2 = 625$ m; assuming an irrigation interval, $f' = 3.5$ days $= 5040$ min; and $CT = 0$; then by inference from Eq. 15.2a:

$$T = 24(5040 - 625 - 160)/5040 \approx 20 \text{ hr}$$

To determine the system capacity, Q_s, the gross daily application depth, d', must first be computed. Entering Eq. 14.1a with $k_f = 1.05$ (from Table 14.1 for $f \approx 3$ days), $DE_{80} = 92\%$ (from Table 6.2 for the given data), $U_d = 7.0$ mm/day, $R_e = 0.94$, and $O_e = 1.0$ gives:

$$d' = \frac{100 \times 1.05 \times 7.0}{92 \times 0.94 \times 1.0} = 8.5 \text{ mm}$$

The required system capacity can now be determined by:

$$Q_s = \frac{L\, L_f\, d'}{K\, T} \tag{15.1}$$

$$= \frac{400 \times 1250 \times 8.5}{3600 \times 20} = 59.0 \text{ L/s}$$

And the average application rate can be determined by:

$$I = \frac{L_f}{w} \cdot \frac{d'\, R_e}{T} \tag{15.4a}$$

$$= \frac{1250}{20} \cdot \frac{8.5 \times 0.94}{20} = 25.0 \text{ mm/hr}$$

Sample Calculation 15.2. *Testing the suitability, based on infiltration data, of a sprinkler configuration (or application package) for a linear-moving system.*

GIVEN: The design information from Sample Calculation 15.1 and the related findings plus the sprinkler infiltrometer data shown in Fig. 14.8.

The available surface storage, $SS = 2.84$ mm found in Sample Calculation 14.4.

FIND: The slowest travel speed that will not cause runoff and the travel speed that satisfies the design assumptions. Then verify the results and determine the maximum depth of water the system could apply each cycle under emergency conditions.

CALCULATIONS: From Fig. 14.8, the regression coefficients for the infiltrated depth, d_i, under constant application rate sprinkling are: $k_p = 2.924$; and $p = 0.52$. For $I = 25.0$ mm/hr the application time, T_a that will fully utilize $SS = 2.84$ mm can be found by trial and error using:

$$SS = \frac{I\,T_a}{60} - k_p(T_a)^p \qquad (14.8)$$

Try different T_a values to find that when $T_a = 71$ min:

$$SS \approx \frac{25.0 \times 71}{60} - 2.924 \times (71)^{0.52} = 2.75 \text{ mm}$$

For the pattern width, $w = 20$ m, to pass in 71 min, the minimum (slowest) travel speed when irrigating must be:

$$(V_i)_n = \frac{20}{71} = 0.28 \text{ m/min}$$

This is the minimum allowable travel speed, and any $V_i \geq 0.28$ m/min will not cause runoff.

The system capacity is sufficient to meet peak consumptive-use requirements with an average daily irrigating time of $T = 20$ hr. Therefore, if the V_i that gives $T = 20$ hr (by Eq. 15.2a) is ≥ 0.28 m/min, the application package with $w = 20$ m is acceptable.

To check for acceptability, first determine the exact contingency time, CT, that would give $T = 20$ hr for the $f' = 3.5$ days $= 5040$ min assumed in Sample Calculation 15.1. This can be done by rearranging the parameters in Eq. 15.2a to obtain:

$$\begin{aligned} CT &= f - L_f/V_r - RT - f'(T/24) \qquad (15.2a') \\ &= 5040 - 625 - 160 - 5040 \times 20/24 \\ &= 55 \text{ min} \end{aligned}$$

Then determine the exact travel speed that gives the above parameters by rearranging Eq. 15.2a again to obtain:

$$\begin{aligned} V_i &= \frac{L_f[(24/T) - 1]}{(L_f/V_r + RT + CT)} \qquad (15.2a'') \\ &= \frac{1250\,[(24/20) - 1]}{(625 + 160 + 55)} = 0.298 \text{ m/min} \end{aligned}$$

As this is greater than $(V_i)_n = 0.28$ m/min, the design with $w = 20$ m is acceptable.

The design can be checked in two ways. First by Eq. 15.2a to affirm that $T = 20$ hr and $f = 3.5$ days. Then by Eq. 15.6, which for a watering time per lateral cycle of:

$$T_w = 1250/0.298 = 4195 \text{ min}$$

gives an *applied* net depth of irrigation of:

$$d_n = \frac{3600 \times 59 \times 4195 \times 92 \times 0.94}{6000 \times 400 \times 1250} = 25.7 \text{ mm}$$

With $f' = 3.5$ days, $k_f = 1.05$, and $U_d = 7.0$ m/day the *required* net depth of application per irrigation is also:

$$d_n = 3.5 \times 1.05 \times 7.0 = 25.7 \text{ mm}$$

Under emergency conditions the system could be set to irrigate during the return part of the cycle, and the contingency time could be eliminated. This would increase the d_n to approximately 29.6 mm per cycle. However, it would increase f' by the time required to reset the hose four times during the return run minus the time previously allowed for contingencies, *CT*. Thus, f' would be increased by:

$$4 \times 20 - 55 = 25 \text{ min}$$

Necessary Pattern Width

Usually an assumed sprinkler pattern width, w, and average daily operating time, T, will not fit the site conditions as well as they did in Sample Calculation 15.2. Therefore, it is easier and quicker to directly determine the w necessary to avoid runoff for the field length and operating schedule desired.

As in Sample Calculation 15.1, an irrigation interval should first be selected that satisfies Eq. 14.10. Then by manipulating Eq. 15.2a as before, select a suitable value for T. The corresponding travel speed when applying the main irrigation application, V_i, can then be determined using this value of T in a rearranged version of Eq. 15.2a, as in Sample Calculation 15.2.

The minimum pattern width, w, that will not cause runoff with the above V_i can be determined directly if time-to-ponding data from a sprinkler infiltrometer test are available. To solve for w, directly replace I in the first term of Eq. 14.8 with Eq. 15.3 after combining it with Eq. 14.1a multiplied by f' in days. Then, substitute w/V_i for T_a in the second term of Eq. 14.8, and rearrange the resulting equation to obtain:

$$(w)_n = V_i \left[\frac{100\, k_f U_d f'}{k_p DE_{pa}} - \frac{SS}{k_p} \right]^{1/p} \qquad (15.12)$$

where

$(w)_n$ = minimum pattern width necessary to avoid runoff from a linear-moving lateral, m (ft)

V_t = travel speed when applying main irrigation application, m/min (ft/min)
k_f = frequency factor to adjust standard crop water-use values for high-frequency irrigation taken from Table 14.1, decimal
U_d = average daily crop water-use rate during peak use month, mm (in.)
f' = irrigation interval or frequency, days
SS = surface storage, mm (in.)
p = time-to-ponding exponent dependent on soil and water characteristics at the time of the test
k_p = time-to-ponding coefficient dependent on soil and water characteristics at the time of the test and the measurement units used
DE_{pa} = sprinkler distribution efficiency based on adequately irrigating a given percentage, pa, of the field area, %

The minimum pattern width that would not cause runoff for the conditions and data in Sample Calculation 15.1 and 15.2 can be determined by Eq. 15.12 as:

$$(w)_n = 0.30\left[\frac{100 \times 1.05 \times 7.0 \times 3.5}{2.924 \times 92} - \frac{2.84}{2.924}\right]^{1/0.52} = 18.8 \text{ m}$$

This is a little less than the $w = 20$ m that was evaluated and found acceptable in Sample Calculation 15.2. This is as expected, because the ratio between the $V_t = 0.298$ m/min and $(V_t)_n = 0.28$ m/min found earlier is similar, i.e.,

$$\frac{20.0}{18.8} \approx \frac{0.298}{0.28}$$

The value of w required for the operating conditions selected may be larger than desired or practical. If this is true, then one or more of the following would be required: f' could be decreased; T could be increased by watering continuously in both directions; or the length of the field irrigated, L_f, could be decreased.

Equation 15.12 could be modified for use with center-pivot laterals by replacing V_t with the travel speed, V_j, of any outlet at radial distance r_j along the lateral. Values for V_j can be computed for the desired cycle time by:

$$V_j = (2\pi r_j)/(\text{minutes per cycle})$$

Knowing the minimum pattern width $(w_j)_n$ required at any radial distance r_j would be useful when designing the nozzling package. For example, when designing to use spray nozzles, the need for or length of boom could be determined for each outlet.

Other Design Procedures

The remaining design procedures applicable for linears are similar to their respective counterparts for pivots. The strategies used for designing the nozzling package are similar in that the nozzle sizes can be selected to give the desired discharge with the pressure available. However, because each outlet along a linear moving lateral travels the same distance, the discharge from each of them should be the same. Thus, the lateral hydraulics are the same as for set sprinkle systems, which is covered in Chapter 9.

Linears are generally used in fields with little elevation difference along the lateral. Therefore, uniform discharge can easily be maintained along the lateral by compensating for the pressure loss due to pipe friction. This is done by using progressively larger nozzles toward the outer end of the lateral.

The life-cycle economic pipe-sizing techniques presented for center-pivot laterals are even more important for linear-moving laterals. Because discharge is uniform along the linear-moving laterals, rather than skewed toward the outer end as with a pivoting lateral, there is greater opportunity to use more smaller diameter pipe.

16

Multipurpose and Special Uses

Both sprinkle and trickle irrigation systems can be used in a multipurpose way to apply agricultural chemicals along with the irrigation water. The general term for this is *chemigation*. Various types of sprinkle irrigation equipment are also adaptable to a variety of special uses in addition to ordinary irrigation to control soil moisture.

Chemigation involves injecting a water soluble-fertilizer, herbicide, insecticide, fungicide, or nematicide into the irrigation system. When fertilizer is injected it is called *fertigation*, which is an important multipurpose function of all types of sprinkle and trickle systems. The other forms of chemigation are called *herbigation*, *insectigation*, *fungigation*, and *nemigation*. Fixed and continuously moving sprinkler laterals, especially center-pivots, have been successfully used for all of them. However, trickle and periodic-move sprinkle systems are not as well adapted to these forms of chemigation.

Because of fertigation's importance, the design and management requirements for it will be presented herein. These requirements are well-documented for all types of sprinkle and trickle systems and generally transferable. Some of the other forms of chemigation are also quite well-developed, but they are more site- and system-specific, thus beyond the scope of this text. However, before they are used extensively, precise details of dosages, effectiveness, economics, and safety should be known for the prevailing local conditions and crops.

Frost protection, bloom delay, microclimate control, and disposing of waste waters are the most important special-use functions of sprinkle systems. Some additional special uses of sprinkle equipment are: providing farm fire protection; providing cooling and dust control for feed lots and poultry buildings; providing moisture for earth-fill construction; and curing log piles.

FERTIGATION

Fertigation, which is the dissolving of soluble fertilizers in water and applying the solution through a sprinkle or trickle system, is economical, easy, and effective. A minimum of equipment is required. Once the apparatus for adding the fertilizer to the irrigation water is set up, the crop being irrigated can be fertilized with less effort than is required for mechanical application.

In arid areas fertigation is necessary to supply sufficient fertility, especially nitrogen, for fields irrigated with drip or subsurface-type trickle systems. This is because dry fertilizer broadcast over the soil surface will not be moved into the root zone by the irrigation water. Slow-release nitrogen and other fertilizers can be incorporated into the soil at planting time, but they should be incorporated only into areas that will be thoroughly wetted by rainfall or irrigation. This may be difficult to do in arid areas for fields irrigated by trickle systems because of the small and irregular volume of soil wetted.

There are several advantages in using sprinkle or trickle irrigation to distribute fertilizers. First, both irrigation and fertilization can be accomplished with only slightly more labor than is required for irrigation alone. This is particularly important in arid and semiarid areas, where the applications of irrigation water and fertilizers can, in most cases, be scheduled to coincide with one another. Secondly, close control can usually be maintained over the depth of fertilizer placement, as well as over the lateral distribution.

Penetration of the fertilizer into the soil can be regulated by the time of application in relation to the total irrigation period. The uniformity of fertilizer distribution, however, can be only as good as the uniformity of water distribution. If the irrigation system has been properly designed and is properly operated, both fertilizer and water distribution will be acceptable.

Fertilizer Materials

Many dry, liquid, and liquid-suspension fertilizer materials are suitable for application through trickle and sprinkle systems. The main criteria used in selecting a fertilizer material are the solubility, convenience, and cost of the desired nutrients.

Clear liquid fertilizers contain nutrients in solution. Thus, they are very convenient to handle with pumps and gravity flow from bulk storage tanks for injection into the irrigation water. Liquid fertilizers may contain a single nutrient or combinations of nitrogen (N), phosphate (P), and potash (K).

There is a wide variety of soluble dry fertilizers containing N, P, and K singly or in combination for dissolution in the irrigation water. Dry fertilizer products may be dissolved by mixing with water in a separate open tank in the approximate ratio of 1 kg of fertilizer to 8 L of water (1 lb per gal) and then pumped into an irrigation pipeline. They may also be placed in a pressurized container through which a portion of the flow in the main pipeline is passed. In the latter instance, the flow of water from the bypassed stream continually dissolves the solid fertilizer until it has all been applied.

Unfortunately, certain chemical problems are associated with injection of various fertilizers into the irrigation water as discussed below. The numerical analysis in parentheses following the different fertilizers represents the percentage by weight of nitrogen, phosphorus, and potassium (N-P-K).

Nitrogen. Nitrogen injection is relatively problem-free. Anhydrous ammonia (82-0-0) and aqua-ammonia (24-0-0) can be injected directly into the irrigation water. However, with sprinkle irrigation some of the nutrients of the fertilizer may be lost, because gaseous ammonia is likely to volatilize.

Another problem incidental to ammonia fertigation is the increase of hydroxide ion concentration in the water. Ammonia injection causes a rise in pH and potential precipitation of both soluble calcium and magnesium. These precipitates may coat the inside of pipes and plug emitters. This kind of precipitation problem can be prevented by injecting a commercial water softener ahead of the ammonia gas. This eliminates the problem by complexing the calcium and magnesium, but it adds considerably to the cost of fertilizing.

Most nitrogen salts and urea dissolve readily in water. However, the nitrogen fertilizers mentioned below in connection with phosphorus fertigation should not be considered in this context because of the indicated interactions involving phosphorus in water and soil.

Ammonium sulfate (21-0-0) and ammonium nitrate (34-0-0) are very widely used fertilizers. In ammonium sulfate, all the nitrogen is in the ammonium form, but in ammonium nitrate about 26% by weight of the fertilizer is ammonium nitrogen, and 8% is nitrate nitrogen. Urea (44-0-0), a very soluble nitrogen fertilizer, does not react with water to form ions and therefore is a neutral molecule. All these nitrogenous materials may be applied satisfactorily by fertigation with no adverse side effects to either the water or to the irrigation system.

Both urea and nitrate-nitrogen tend to persist in the soil in solution and to drift in whichever direction the soil moisture moves. This means that these materials are highly susceptible to loss by leaching if excessive water is applied.

The chemical nature of ammonium-nitrogen ions makes the behavior of this material quite different from that of nitrate-nitrogen. They are cations and enter into exchange reactions with soils. Because cation-exchange reactions are very rapid, ammonium in irrigation water is immobilized almost instantly upon contact with soil. Consequently, most of the ammonium remains on or near the soil surface.

Ammonia or ammonium applied in water readily converts to exchangeable ammonium and simultaneously generates an equivalent number of cations in solution. In semiarid and arid regions, soils are naturally neutral to alkaline in reaction (pH 7.0 to 8.2), depending on how much free lime or calcium carbonate is present. In these kinds of soils, where exchangeable ammonium exists at the soil surface, probability of volatilization is high. This mechanism of ammonia loss is very sensitive to temperature and moisture conditions. Water vaporizes very rapidly from soil after an irrigation, and while the soil is drying ammonium is especially susceptible to gaseous loss. However, frequent applications of trickle irrigation keep the soil surface moist, which reduces gaseous losses of ammonia.

Phosphorus. Phosphorus fertigation is difficult. Treble-superphosphate (0-45-0) is only moderately water-soluble; therefore, it cannot be considered for fertigation. It is a relatively inexpensive source of phosphorus, but it must be incorporated into the soil to be effective.

Several kinds of ammonium phosphates are available on the market and are commonly used for both nitrogen and phosphorus in fertilizer. These include ammonium phosphate sulfate (16-20-0), monoammonium phosphate (11-48-0), and diammonium phosphate (16-46-0). As all these forms of phosphorus are very soluble in water, they may be adaptable to fertigation. Phosphoric acid, a very soluble form of phosphorus, has the additional advantage of lowering the pH of the water.

The quality of the irrigation water must be considered before deciding to inject phosphorus fertilizers into the system. If the water contains appreciable amounts of calcium, any form of phosphorus will precipitate as dicalcium phosphate in the pipeline and emitters. This will eventually restrict the flow of water and plug emitters.

Phosphoric acid is the best candidate for fertigation with inorganic phosphate. Precipitation can be prevented by careful adjustment of the pH of irrigation water with low levels of calcium and magnesium. The potential for clogging can be overcome by immediately following the injection of phosphoric acid with an injection of either sulfuric or hydrochloric acid. This should be done to maintain an acid pH for enough time to completely purge the system.

Organic phosphate compounds, such as glycerophosphoric acid, are suitable for injection through irrigation systems without fear of precipitation. The organic compounds are comparable to urea in their behavior in soils. However, they are relatively expensive when compared with even the soluble forms of inorganic phosphorus. These, in turn, are more expensive than triple-superphosphate.

Another important fact about phosphorus is its immobility in soil. Soluble phosphorus transforms to insoluble dicalcium phosphate almost the moment it comes into contact with calcium in the soil. Therefore, most of the fertilizer phosphate applied in irrigation water is deposited at the soil surface where it is unavailable to the drop. Subsequent crops will benefit because the next plowing will mix the fertilizer throughout the plowed layer.

This problem is most likely to occur when sprinkle or spray-type trickle irrigation is used. Under drip irrigation the phosphate application is concentrated around the emission points. The larger equivalent application rate over a small area saturates the absorption and precipitaton sites in the soil. This allows the phosphate to spread away from the point of application. This spreading of phosphate from the emission point is usually extensive enough to give satisfactory placement of the nutrient in a crop's root zone.

Potassium. Potassium is easily applied by fertigation. Potassium oxide, the most common form, is so soluble that the fertilizer moves freely into the soil.

However, potassium molecules become exchanged on the soil complex and are not readily leached away.

Trace Elements. Secondary and micronutrients, such as magnesium, zinc, boron, iron, and copper, can be applied through sprinkle or trickle systems. However, they are needed in very small quantities, they may react with salts in the water, and some are also toxic when too much is applied. Therefore, careful water, soil, and foliar analysis and interpretation are necessary to select suitable compounds and dosages. It is best to apply trace elements by conventional methods where the complete details for injecting trace elements have not been field-checked.

Summary. Materials in common use and suitable for fertigation include:

Urea-ammonium nitrate solutions
Ammonium nitrate
Ammonium sulfate
Urea (potential NH_3 loss)
Calcium nitrate
Potassium nitrate
Liquid ammonium phosphates (often precipitate in system)
Some dry ammonium phosphates (often precipitate in system)
Phosphoric acid (may precipitate in system)
Glycerophosphoric acid (expensive)
Potassium chloride (may be hard to dissolve)
Potassium oxide

Trace elements (secondary and micronutrients) that can be applied through irrigation systems include:

Magnesium sulfate
Zinc sulfate and zinc chelates
Manganese sulfate and manganese chelates
Copper sulfate and copper chelates
Iron sulfate and iron chelates
Solubor (boron)
Molybdenum

Materials that should not be applied through sprinkle and trickle irrigation systems include:

Aqua ammonium (excessive N loss and calcium precipitation with hard water)
Anhydrous ammonium (excessive N loss and will precipitate with hard water)
Single superphosphate (material will not dissolve)
Concentrated or trebel superphosphate (material will not dissolve)

Some dry ammonium phosphates (will not dissolve)
Potassium sulfate (hard to dissolve)
Magnesium sulfate (hard to dissolve)
Almost all N-P-K dry fertilizers (contain superphosphate, which will not dissolve)
Liming materials (will not dissolve)
Elemental sulfur (will not dissolve)
Ammonium polyphosphate (will precipitate with hard water)

Injecting Fertilizers

For periodic-move and fixed sprinkle and trickle systems, fertilizer should be applied by the batch method. The fertilizer required for a given area is put into a tank and the batch used in a single set or it is metered into the systems, and the solution is injected into the irrigation water. High concentrations of solution must be avoided because of corrosion problems, but exact proportions are not important with the batch method. The sprinklers and emitters in operation simultaneously cover a specific area. Thus, the quantity put into the tank (or metered into the system) is the quantity that will be applied to that area.

The common procedure followed in applying fertilizer by the batch method consists of three periods. First, the system should be operated normally to wet the soil and foliage if it is a sprinkle system. Then the fertilizer should be injected into the system for a period of an hour or longer. A long application reduces the possibility of poor distribution due to the time required for the solution to travel throughout the pipe network. Also, the solution passing through the system will be more diluted, which lessens the possibility of foliage burn when sprinkling and corrosion damage to the system.

The last period should be long enough to completely rinse the system with clear water to prevent excess corrosion. With sprinkle systems this also rinses the plants, which may also be important for some crops. Thorough rinsing usually requires about an hour. During the rinsing, the fertilizer is also moved farther into the crop-root zone, which improves its efficiency.

For continuously moving sprinkle systems, such as center-pivots, fertilizer must be applied by the proportional method. After applying fertilizer, the system should be operated with clear water long enough to completely rinse it, to help prevent corrosion. With the proportional method, the rate at which the fertilizer is injected is important, because this determines the amount of fertilizer applied per unit area.

Injection Equipment

Many commercial fertilizers and soil amendments are corrosive to metals and are apt to be toxic to plant leaves. The approximate order of metal susceptibility to corrosion is as follows: galvanized steel; phosphobronze; yellow brass; aluminum; and stainless steel. There are several grades of stainless steel, of which the best are practically immune to corrosion. Protection is afforded by: diluting the fertilizer; minimizing the period of contact; and thorough flushing of the system.

When corrosive agents are to be added continuously, system components should be selected with caution. Metal valves and fittings are more subject to chemical corrosion than plastic. To avoid possible problems where acid injections are required, plastic control valves should be used, and bare metal parts, other than stainless steel, should be avoided.

Injection. Suction of chemicals through the intake side of the pump is a simple method for injection. However, it is *not recommended, because contamination of the water supply must be prevented*, and this cannot be assumed with this type of setup (except when pumping from an isolated sump). Furthermore, corrosive materials may cause excessive degradation to pump parts, and it is difficult to monitor the rate of chemical input accurately. The preferred methods of injecting fertilizers and chemicals into the system are by pumping or by differential pressures.

Pumping is the most versatile method for injecting chemicals into sprinkle and trickle irrigation systems. Positive displacement piston, gear, or paddle pumps (see Fig. 16.1) can be designed and calibrated to give accurate low or high rates of injection. The pump draws the chemical solution from an open tank and injects it by positive displacement (or pressure) into the irrigation system.

If the water supply system fails, it is possible for the flow to reverse. Water containing chemicals must be prevented from flowing back into the supply. This can be done by installing an air inlet and check valve in the irrigation main line upstream from where the chemical is injected (see Fig. 16.1). Another check

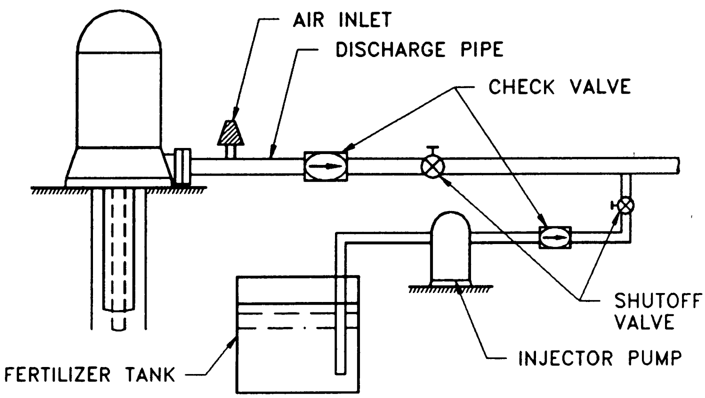


FIG. 16.1. A Method For Adding Fertilizers to an Electrically Driven Turbine Pump System Using an Injector Pump.

valve should be installed in the fertilizer pipeline to prevent the irrigation water from overflowing the fertilizer tank if the injector pump fails. A final safety precaution is to have the injector and irrigation pumps interconnected to prevent the injection of fertilizer if the irrigation pump fails.

The pressurized water from the irrigation line can be used to drive a chemical injection pump by means of diaphragms or pistons that have a larger surface area than the injection piston. Thus, the pump injects chemical at a pressure higher than that of the water that drives it. The small amount of water that drives the pump (two to three times the volume of fertilizer injected) is expelled.

On engine-driven main pumping plants a belt-and-pulley arrangement can be used to drive the chemical injector pump. On electric installations a fractional horsepower electric motor is needed. Both engine-driven and electrically driven pumps are usually less expensive than water-driven pumps and have fewer moving parts to maintain.

Automatic volumetric shutoff valves are available for water-driven pumps, and automatic time controllers are available for electrically driven pumps. Injection can be stopped by letting the chemical tank run dry, but this may damage the injector pump unless it is also shut off.

Differential pressure can be used to inject chemicals into the irrigation water. In a typical differential pressure system, the chemical tank is under the same pressure as the irrigation main line. A Venturi pipe section can be used to develop a significant pressure differential between two points along the main line with little loss of energy. Gate valves or pressure regulators can be used to obtain a sufficient head differential, about 1.5 m (5 ft), to inject chemicals, but they are less efficient.

The chemicals to be injected must be placed in a pressure tank whose inlet is connected to the main line pipe and whose outlet is connected at the throat of the Venturi tube (see Fig. 16.2). The difference in pressure causes water to

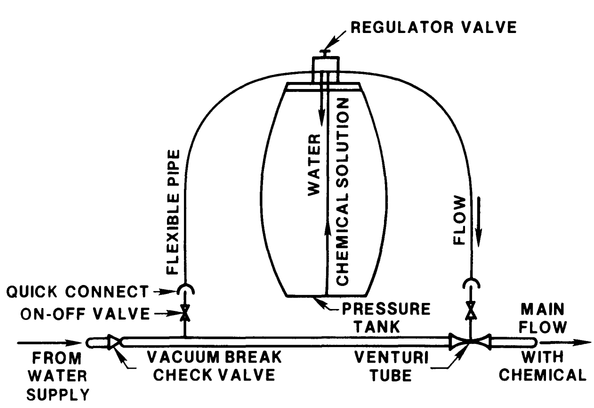


FIG. 16.2. Pressure Differential Chemical Injection System (Source: Karmeli and Keller, 1975 (Fig. 8.1)).

flow through the tank. It is important to have accurate metering valves to control the rate and quantity of fertilizer entering the system. This can be accomplished by using precision valves and differential pressure regulators in the fertilizer line.

Differential pressure injection systems have no moving parts and require no external source of power. Furthermore, they are less expensive than pump injectors for small systems. Their main disadvantage is that large, corrosion-resistant, high-pressure tanks are expensive. Therefore, small tanks are usually used at the expense of added labor required for more frequent servicing.

Other Considerations. Some fertilizers and other chemicals require time to react and mix with water and must be agitated for proper mixing. A mixing tank system allows small volumes of chemicals to be added to the system over long periods without loss. The mixing system also permits either simultaneous or sequential addition of other chemicals.

The capacity of the fertilizer tank is an important consideration. Large, low-cost tanks are practical where injection pumps are used. A large tank provides a good place to store fertilizer for periods of short supply and reduces the labor associated with frequent filling. If a large tank is being used, an automatic shutoff valve is a convenient way to control the amount of fertilizer injected. However, the injection pump must also be shut off to prevent damaging it.

Clogging of emitters is a serious problem for trickle irrigation systems. To keep clogging hazards at a minimum, it is important that the chemicals be injected upstream of the filters at the control head (see Fig. 2.6).

Injection Rate. The rate at which any chemical is to be injected into the irrigation water should be calculated carefully. It depends on the concentration of the liquid fertilizer and the desired quantity of nutrients to be applied during the irrigation. It can be computed by:

$$q_c = \frac{F_r A}{c' t_r T_a} \tag{16.1}$$

where

q_c = rate of injection of liquid fertilizer solution into the system, L/hr (gph)
F_r = fertilizer application rate (quantity of nutrients to be applied) per irrigation cycle, kg/ha (lb/A)
A = area irrigated in T_a, ha (A)
T_a = irrigation application or set time, hr
c' = concentration of actual nutrients in the liquid fertilizer, kg/L (lb/gal)
t_r = ratio between fertilizing time and irrigation application time

For periodic-move and fixed sprinkle and trickle systems, let the area, A, equal the area covered per set, and let t_r equal some convenient portion of the set time, T_a. For continuous-move sprinkle systems let A equal the area covered in a setup or in the case of center-pivots in the entire field. Then let $t_r = 1.0$ and T_a equal the time required to complete a setup, which for a center-pivot is the cycle time, T_c.

Sample Calculation 16.1. *Estimating fertilizer injection rates.*

GIVEN: A side-roll system with four sprinkler laterals that are 400 m long and are moved 18.2 m each set; and a center-pivot lateral that is 400 m long. Urea-ammonium nitrate, a liquid fertilizer with 32% nitrogen and weighing 1.32 kg/L, is available for fertigation.

FIND: The injection rates to apply 40 kg/ha of elemental nitrogen with each system.

CALCULATIONS: The concentration of elemental nitrogen, N, in the liquid fertilizer is

$$c' = 1.32 \times 32/100 = 0.42 \text{ kg/L}$$

For the side-roll system the area irrigated by the four laterals during each set is:

$$A = \frac{4 \times 400 \times 18.2}{10{,}000} = 2.91 \text{ ha}$$

Assume an irrigation set time, $T_a = 11$ hr and a ratio between the fertilizing time and T_a of $t_r = 0.5$, to provide plenty of time for flushing. Then by Eq. 16.1 the injection rate should be:

$$q_c = \frac{40 \times 2.91}{0.42 \times 0.5 \times 11} = 50 \text{ L/hr}$$

For the center-pivot system the irrigated area $A = 50.3$ ha, and for a cycle time $T_c = T_a = 24$ hr, by Eq. 16.1:

$$q_c = \frac{40 \times 50.3}{0.42 \times 1.0 \times 24} = 200 \text{ L/hr}$$

PREVENTING FROST DAMAGE

Portable and fixed lateral sprinkle irrigation systems can be used to protect crops from frost. However, the equipment must be specifically set up for frost control.

For adequate protection it is necessary to have sufficient capacity so the entire area being protected can be simultaneously watered.

There are two approaches to protecting against frost. One is to try to protect the leaves, flowers, or fruit from freezing when temperatures fall. The other is to use evaporative cooling to delay early bud formation on fruit trees until after the last expected frost. The first is called *frost protection* and the latter *bloom delay*.

Frost Protection

Both overhead and under-tree sprinkling are used for frost protection. Overhead (over the crop) systems are the most versatile and can protect some crops down to temperatures as low as 7°C (20°F). However, the crop must be able to support the ice loads. Some crops that are able to do this are: potatoes and other low-growing vegetable plants; young flower plants; berry bushes and vineyards; and apple, cherry, pear, prune, and plum trees.

The liquids in the plant parts being protected have a higher freezing point than water due to the salts and sugars in them. Buds, blossoms, leaves, or young fruit, for the crops of greatest interest for frost protection, can survive wet bulb temperatures ranging from roughly -1 to -3°C (30 to 26.5°F). The actual lethal (killing) temperature depends on the crop and stage of development.

The protective effect of overhead sprinkling is mainly from the release of 80 kcal/L (1200 BTU/gal) of latent heat as water freezes. The freezing water encases the plant parts being protected in ice (see Fig. 16.3). This keeps their temperature at 0°C (32°F), the freezing point of water, which is higher than the lethal temperature.

The plant parts being protected will remain at the freezing point of water as long as ice continues to form around them. However, if the water-application rate is insufficient or the interval between wettings is too long, the protected parts will rapidly approach the wet bulb temperature. This happens in a matters of seconds when there is no liquid water to freeze. Therefore, the cycle or sprinkler rotation time should be a minute or less. Furthermore, the water drops should be small to provide good microcoverage and the CU $>$ 80% to give reasonable macrocoverage.

A small amount of heat, 1 kCal/L per °C (8.3 BTU/gal per °F) of temperature drop, comes from the water as it cools. However, more heat may be lost due to evaporation. To offset this, approximately 7.5 L of water must be frozen for each liter that evaporates. Therefore, if too little water is applied or the system fails or is turned off too soon, damage from freezing may be worse than with no sprinkling.

The main design consideration for overhead frost protection by sprinkling is the recommended application rate, I_f, for different environmental situations, crops, and crop growth stages. The required rate can be easily converted to

FIG. 16.3. Ice Encasement Resulting from Overhead Sprinkling.

system capacity, because the area being protected by overhead sprinkling must be watered continuously.

The literature on frost protection is sketchy and inconsistent in terms of I_f and operating procedures. The inconsistencies result because I_f depends on many factors. These include: the type of frost (radiant or advective); the lowest expected air temperature and relative humidity and associated wind speed; the uniformity of application, drop size, and turning speed of the sprinklers; and the lethal temperature of the plant parts to be protected.

Air movement due to wind rapidly dissipates the latent heat released as water freezes. Therefore, the application rate must be increased accordingly. This is necessary to maintain an almost continuous wet surface over the ice encasing the plant parts being protected.

On clear nights plants radiate energy to the cold sky. On calm, clear nights the plant temperature may be 1.5 to 2°C (2.7 to 3.6°F) colder than the air temperature even when the relative humidity is near 100%. With a cloud cover or light wind the plant temperature would be about the same as the air temperature when the humidity is near 100%.

Buds, flowers, and leaves act as relatively wet surfaces and tend to approach the wet bulb temperature if there is sufficient air movement. Furthermore, when sprinkling first begins they are wet surfaces and tend to drop to the wet bulb

Table 16.1 Wet bulb temperatures for different air temperatures and relative humidities

Air temp, °C	Relative humidity, %								
	90	80	70	60	50	40	30	20	10
	Wet bulb temperature, °C[1]								
−4	−4.4	−4.9	−5.3	−5.8	−6.3	−6.7	−7.2	−7.6	8.1
−3	−3.4	−3.9	−4.4	−4.9	−5.4	−5.9	−6.4	−6.9	−7.4
−2	−2.5	−3.0	−3.5	−4.0	−4.5	−5.0	−5.5	−6.1	−6.6
−1	−1.5	−2.0	−2.5	−3.1	−3.6	−4.2	−4.8	−5.3	−5.9
0	−0.5	−1.1	−1.6	−2.2	−2.8	−3.4	−4.0	−4.5	−5.2
1	0.4	−0.2	−0.8	−1.4	−2.0	−2.6	−3.2	−3.8	−4.5
2	1.4	0.8	0.2	−0.6	−1.2	−1.8	−2.4	−3.1	−3.8
3	2.4	1.7	1.0	0.3	−0.4	−1.0	−1.7	−2.4	−3.1
4	3.4	2.7	1.9	1.2	0.5	−0.2	−0.9	−1.7	−2.4
5	4.4	3.6	2.9	2.2	1.3	0.6	−0.2	−1.0	−1.7
10	9.2	8.3	7.4	6.5	5.5	4.6	3.6	2.5	1.5
15	14.0	13.0	11.9	10.9	9.7	8.5	7.3	6.0	4.6
20	18.9	17.7	16.5	15.1	13.8	12.4	10.8	9.3	7.6
25	23.3	22.4	21.0	19.5	18.0	16.3	14.5	12.5	10.5
30	28.6	27.1	25.5	23.8	22.0	20.2	18.0	15.7	13.1
35	33.5	31.8	30.2	28.3	26.3	24.0	21.5	19.0	16.0
40	38.3	36.5	34.7	32.7	30.3	28.0	25.8	22.0	18.5

[1]For barometnc pressure of 7.25 cm Hg. °F = 32 + 1.8°C

temperature as the ice first begins to form. Table 16.1 gives the wet bulb temperatures as a function of the air temperature and relative humidity.

In humid regions during frosty nights in the early and late parts of the growing season, the relative humidity is usually between 90 and 100%. In arid regions it is usually above 60%. However, as the air begins to warm in the early morning, the humidity drops. Consequently the rise in wet bulb temperature is much slower.

Where frost-protection systems are supplied from small reservoirs it is necessary to know the volume of water required. This in turn is dependent on the system capacity and how long the system must be operated during the entire length of anticipated cold spells. Usually, the system will need to be operated only 5 to 10 hr per day, but this may be required for several days in sequence.

Overhead System Design. An application rate of 2.5 mm/hr (0.1 in./hr) is about the lowest that will give a CU > 80%, even in winds of only 0–6 km/hr (0–4 mph). This requires a system discharge of roughly 7 L/s per hectar (45 gpm per acre) of frost-protected area. The highest practical application rate is roughly 7.5 mm/hr (0.3 in./hr) because of ice loading, infiltration, or system capacity problems.

The recommended application rate for different temperatures and wind conditions for $t < t'$ and $WS > 1.6$ km/hr (1 mph) is roughly:

$$I_f = K_1\,(t' - t) \cdot (K_2\,WS + 1)/2 \tag{16.2}$$

For $WS \leq 1.6$ km/hr (1 mph), let $WS = 1.6$ km/hr (1 mph); and if $I_f \leq 2.5$ mm/hr (0.1 in./hr), let $I_f = 2.5$ mm/hr (0.1 in./hr).
where:

I_f = recommended overhead sprinkle application rate for frost protection, mm/hr (in./hr)
K_1 = conversion constant, which is 0.9 for metric units (0.02 for English units)
t' = lethal (killing) temperature for plant parts being protected, °C (°F)
t = lowest dry leaf or wet bulb temperature expected, °C (°F)
K_2 = conversion constant, which is 0.62 for metric units (1 for English units)
WS = highest wind speed expected during period of low temperature, km/hr (mph)

Equation 16.2 was developed in an effort to encompass the I_f data available assuming $t' = -2$°C (28.4°F) and high relative humidity (wet and dry bulb temperatures are about equal). It fits the data presented by Berber and Martsolf (1966), which are based on the theory presented by Berber and Harrison (1964). Unfortunately, there is not a good source of lethal temperature data; however, setting $t' = -2$°C (28.4°F) is satisfactory for many sensitive crops. The lower value of $I_f = 2.5$ mm/hr (0.1 in./hr) is used to make certain the plant parts do not become cooler by evaporation than when dry. It is also about the lowest application rate suitable for good uniformity.

Overhead System Operation. Overhead frost-protection systems must be started when the wet bulb temperature plus 1.5 to 2°C (2.7 to 3.6°F) equals t'. This is necessary because radiation to the sky can depress the temperature of the plant parts to be protected. Also when sprinkling begins before ice forms, the temperature of plant parts can be further depressed by evaporation.

The system should be operated until $t > t'$ and the ice encasing the plant parts is continuously melting. This is very important, because if sprinkling is stopped too soon and $t < t'$, severe frost damage may result.

Weather forecasts should be used to predict when system operation may be necessary. However, accurate thermometers are necessary at the exact site to be protected. The thermometer should be at plant level and sheltered by a radiation screen. Temperature alarm systems are also recommended to alert the system operator.

Sample Calculation 16.2 *Overhead sprinkling frost-control system design.*

GIVEN: An apple orchard in which the lethal temperature for the blossoms, $t' = 27°F$. Radiant frosts with the air temperature dropping to 26°F and the relative humidity at 80% are common. A freezing period usually lasts for 6 to 8 hr and they sometimes occur for three nights in a row with winds of 0–3 mph.

FIND: The overhead sprinkling application rate required to protect the blossoms and young fruit from frost damage, the system capacity for a 40-A orchard, and the size of reservoir required to supply the system.

CALCULATIONS: From Table 16.1 the wet bulb temperature for 26°F = −4°C and 80% humidity would be −4.9°C = 23.2°F. From Eq. 16.2 the required sprinkle application rate with $WS = 3$ mph and $t' = 27°F$ is:

$$\begin{aligned} I_f &= 0.02(27 - 23.2)(1 \times 3 + 1)/2 \\ &= 0.15 \text{ in./hr} \end{aligned}$$

To protect the 40-A orchard, the required system capacity (see Eq. 5.4) should be:

$$Q = 453(0.15 \times 40) = 2718 \text{ gpm}$$

The storage reservoir to provide for 8 hr of sprinkling three nights in a row would be:

$$\begin{aligned} \text{Volume} &= 40 \times 43560(0.15 \times 24)/12 \\ &= 523{,}000 \text{ ft}^3 \end{aligned}$$

This would require a reservoir with an average depth of 12 ft and a surface area of 1 A.

Under-tree Sprinkling. Such trees as almond, peach, and apricot cannot handle an ice load. Therefore, over-tree sprinkling to protect them from frost damage is not advisable. However, under-tree sprinkling can be used to protect them and other tree crops against minor radiant frosts of −2.5°C (27.5°F), but it is satisfactory only in arid areas where the soil surface is dry and the humidity is low.

Under-tree sprinkling saturates the air and aids in releasing heat from the soil. The higher humidity increases the wet bulb temperature and reduces the cooling effects of evaporation from plant surfaces. Heat is also released as the irrigation water cools and freezes.

The advantage of under-tree sprinkling is that the system can be cycled every few minutes so only half the system is operating simultaneously. Thus, system capacities need be only about 4 L/s per hectare (25 gpm per acre).

Bloom Delay

In the fall, deciduous trees, vines, and bushes lose their leaves and enter a condition known as winter rest. Plants are normally incapable of growth during this period, and fruit buds do not develop until the rest period has been completed. After rest is completed, changes occur in the buds that will eventually cause blossoming and leafing of the trees.

The rate of bud development depends upon the air temperature around the buds after the completion of rest. Bud development accelerates and the trees blossom early if the early spring temperatures are above normal. If early bud development is followed by a sudden cold spell, the potential for frost damage becomes serious.

Overhead sprinkling can be used to cool the buds before they develop and keep them dormant until after the major danger of frost damage is past. The cooling is caused by evaporation. Therefore, overhead sprinkling for bud delay is not effective during periods of high humidity. The wet bulb temperatures are not enough lower than the air temperature to be very effective (see Table 16.1).

Bud development proceeds after rest whenever the air temperature exceeds 5°C (40°F). The rate of development is equated to the accumulated growing degree (centigrade)-hours. The growing degree-hours accumulated each hour are equal to the average temperature above 5°C. The relationship is linear between 5 and 20°C; then the rate of accumulation stays at 15 degree-hours for each hour above 20°C (68°F).

As the buds continue to develop, their susceptibility to damage from freezing temperatures increases (see Fig. 16.4). Each fruit cultivar has different chill unit requirements to complete rest. It also has its own growing degree-hour accumulation versus bud development relationship. Mathematical models available[1] can provide weekly printouts that predict when rest is completed and the various stages of blossom development (Griffin, 1976 and Griffin and Richardson, 1979.)

This information can be used for both frost protection and bloom delay. Figure 16.4 shows the lethal temperatures, t' at which 50% of the buds or blossoms can be expected to freeze for the various stages of development of Red Delicious apples.

Maximum blossom delay has been achieved when sprinkling is begun just after rest is completed and whenever the air temperature exceeds 7°C (45°F). The system can be cycled with only half the sprinklers operating at a given time. However, a very short cycle of 2 min on and 2 min off should be used.

[1]Computer programs developed by R. E. Griffin and E. A. Richardson (1976) are available from the Utah Agricultural Experiment Station, Utah State University, Logan, Utah.

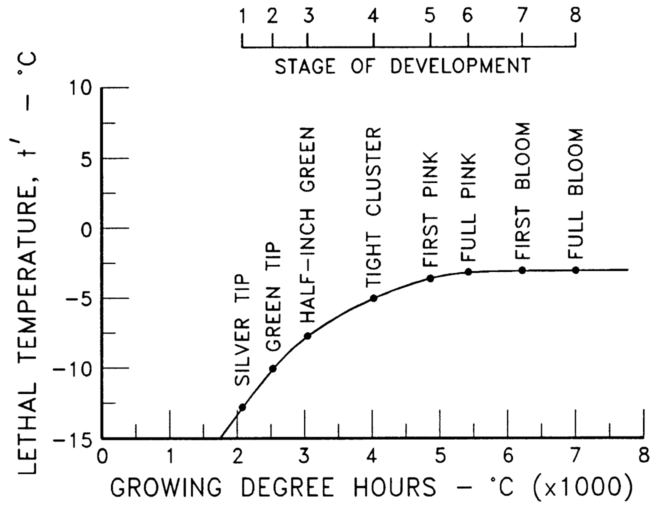


FIG. 16.4. Phenoclimatology of Red Delicious Apples and Lethal Temperatures. (Source: Griffin, 1976.)

Ordinary impact sprinklers nozzled and spaced to give an application rate of roughly 3.0 mm/hr (0.12 in./hr) with a CU > 80% have given near maximum bloom delay. The maximum delay possible is achieved by keeping the buds wet and at the wet bulb temperature whenever the air temperature exceeds 5°C (40°F). Cycling, while using half as much water, has been found to decrease the amount of delay by only 20% (Griffin and Richardson, 1979).

The amount of evaporative cooling that takes place on bare limbs depends upon: the temperature of the tree buds; the difference in vapor pressure between the bud surfaces and the air; and the rate at which evaporated water is removed from the boundary layer by diffusion or by wind currents. Maximum cooling with the least water is achieved by periodically wetting the buds and allowing them to almost dry before rewetting.

In the early spring, less water is required to provide adequate cooling and protection. A savings in water can be realized if the "on" portion of the watering cycle is short in the early spring and is increased as daytime temperatures rise.

Sprinkling for blossom delay can be combined with frost protection by overhead sprinkling. The former can be used in the early spring and the latter in late spring. This gives maximum protection and may conserve water and reduce the maximum system flow rate required for overhead sprinkling alone.

MICROCLIMATE CONTROL

Crop or soil cooling can be provided by sprinkle irrigation. Soil cooling can usually be accomplished by applications once or twice every one or two days.

Therefore, ordinary fixed systems with or without automatic controls and center-pivot systems with high-speed drives are suitable for soil cooling. The additional system capacity required for vegetable and fruit crops is roughly 1.15 times greater than for conventional periodic-move systems in a similar environment. For daily or bidaily irrigation of field and row crops, it is about 1.1 times greater (see Table 14.1).

Foliar Cooling

Foliar cooling requires two to six short applications of water every hour, which is practical only with automated fixed sprinkler systems. The small amounts of water intermittently applied cool the air and plant, raise humidity, and in theory improve the produce quality and yield. When water is applied on the plant surfaces, the plant is cooled and the excessive transpiration demand reduced. Therefore, a plant that would wilt on a hot afternoon can continue to function normally.

Each crop has an upper transpiration demand above which it can no longer function efficiently. For the cool-season crops this occurs at temperatures in the neighborhood of 28°C (82°F). For many warm-season crops it is about 32°C (90°F). These are ambient air temperatures in the irrigated fields, which may be 5°C (9°F) cooler than surrounding unirrigated areas.

Fixed sprinkler systems used for foliar cooling require high-quality water and up to double the capacity of ordinary high-frequency systems. In simple terms, foliar cooling systems are sequenced so the leaves are kept wet. Water is applied until the leaf surfaces are saturated, withheld until they are nearly dry, then reapplied.

This generally requires having from one-sixth to one-fourth of the system operating simultaneously and cycling through the system once every 10 to 40 min depending on weather conditions. For example, each lateral might be operated for 6 min every 30 min, so that one-fifth of the area is being sprinkled at any one time.

Foliar cooling systems must have sufficient capacity to satisfy the excess transpiration demand on a minute-by-minute basis throughout the peak-water-use-rate hours during the peak-use-rate days. To accomplish this, the system capacity must be 1.5 to 2.5 times greater than for a conventional periodic-move system in a similar environment.

Required average application rates over the total cycle time typically range from 0.5 to 1.0 mm/hr (0.02 to 0.04 in./hr). For example, a system that applies 5.0 mm/hr (0.2 in./hr) for one-fifth of the time would give an average rate of 1.0 mm/hr (0.04 in./hr).

On low-growing crops and vines a 4-min application at 3 mm/hr (0.12 in./hr) every 16 min has usually been found adequate. This plus the plant's transpiration can reduce plant temperatures by about 10°C (18°F) when the humidity is 40% and the ambient air temperature is 35°C (95°F) in uncropped areas (see Table 16.1). On larger trees, a 6-min application at 5 mm/hr (0.2 in./hr) every 30 to 36 min has been found satisfactory.

The ideal threshold crop canopy temperature for starting a system depends on the crop. For cool-season crops, such as carrots and potatoes, it is about 28°C (82°F), and for warm-season crops, such as egg plant and peppers, it is about 32°C (90°F). For optimum effect the system should be continuously cycled during the period of high temperatures.

DISPOSING OF WASTE WATER

Land application of waste waters by sprinkle irrigation can be a cost-effective alternative to conventional waste water treatment. Waste waters are divided into municipal, industrial, and agricultural categories. Municipal sewage requires extensive treatment before it can be safely discharged into the natural drainage system. Industrial waste waters may also require extensive treatment. It can range from simple screening to primary and secondary treatment for removing oils, greases, metals, harmful chemicals, and for pH adjustment and chlorination.

Agricultural waste waters include effluents from animal-production systems and food-processing plants. For land application through sprinkle irrigation, most animal wastes must undergo some treatment, such as removal of large fibrous solids. Agricultural plant waste waters will generally require more extensive pretreatment, such as removal of solids, greases, and oils and pH adjustments.

Design Considerations

A major concern of land application of waste waters is the potential for destroying or rendering ineffective the disposal site, or polluting ground and surface water in neighboring areas. Oils, greases, and heavy metals can harm the soil and the vegetative cover. Furthermore, solids can build up a mat on the surface that will kill the vegetative cover. Therefore, the composition of the effluent, vegetative cover, soil type, and frequency of application should be considered when designing a sprinkle system to dispose of waste waters.

Deep sandy or loamy soils are most suitable for land application of wastes. The soils must be well-drained; therefore, some sites may require subsurface drainage. For best results the application rate should be less than 75% of the infiltration rate of the soil.

Woodlands and grass areas make good disposal sites for waste waters. The soil surface is usually stable, and the surface cover is effective for digesting organic matter. Cropped areas, such as corn fields, can also be used as disposal sites, but effluent can be applied on them only during selected times of the year.

In general waste water should be applied only on disposal sites with live and preferably actively growing plants. The plants are necessary to maintain the soil's tilth and infiltration capacity and to remove nutrients.

Rates of nitrogen that can be removed by growing and harvesting plants vary, depending on the crop and length of the growing season. The variation ranges from roughly 225 kg/ha (200 lb/A) for a single crop of corn to 800 kg/ha (700 lb/A) per year for coastal Bermuda grass where there is a 12-month growing season. Contrary to general opinion, disposal sites cannot be loaded with excess nutrients and retain or "fix" them. Nitrogen in waste water applications that exceeds plant use can result in leaching of nitrates and pollution of the groundwater.

The total application can be as much as 100 mm (4 in.) per week during summer months, but there should be a rest period between applications. In temperate regions during the winter months, sprinkling may be continued when ice buildup is not occurring, but the weekly applications must be reduced by as much as 75%. Sprinkling should be terminated during periods when the temperatures remain below freezing and ice continues to build up, because it will kill the crop.

During windy weather small droplets from impact sprinklers, especially large ones, will drift a considerable distance. For this reason a large buffer zone of 50 to 100 m (150 to 300 ft) is usually required around waste disposal sites. When designing sprinkle waste disposal installations, local safety, health, and environmental regulations should be strictly followed.

The design of a system for land application of waste water is similar to the design of an ordinary sprinkle irrigation system. The rules of good design must be followed, keeping in mind that the effluent is not plain water, but a mixture of water and both dissolved and suspended wastes. Furthermore, waste waters that contain abrasive or corrosive materials will shorten the life of the system. Therefore, special equipment may be required.

Equipment Considerations

Either portable aluminum or buried plastic or corrosion-resistant pipe may be used for main and lateral lines of periodic-move or fixed sprinkler systems. Single-nozzle sprinklers should be used to reduce nozzle clogging. If systems are designed to operate during freezing temperatures, sprinklers that will operate under these conditions should be selected.

Automation is recommended for fixed systems to reduce labor requirements and simplify management of the disposal operation. Automatic valves can be operated by air, water, or electricity. However, the most desirable are hydraulic valves actuated with water from a clean source or air. The solids in typical waste waters tend to clog ordinary electric solenoid valves.

If the site is in a freezing climate, drain valves should be installed to facilitate draining the pipe system. The best protection against pipe damage to freezing is an air purge system that can be used to clear the pipe of water.

Where the system is operated only part of the time, and the waste water is corrosive or has a high solids content, the system should be flushed with fresh water after each use. Effluents left in the pipes may become septic and create a nuisance. Also suspended solids will settle and accumulate at low points in the lines and may cause severe clogging.

Center-pivots and traveling sprinklers can be effectively used for land application of waste waters, but should not be operated during freezing temperatures. They both produce fairly high application rates and because of the large amounts of water applied, traction problems often occur. To reduce such problems the wheel tracks of pivots or traveler tow paths can be compacted and formed (or even graveled) to facilitate drainage.

REFERENCES

Berber, J. F., and D. S. Harrison. 1964. Sprinkler irrigation for cold protection of citrus. *ASAE Transactions* 7(4):464–468.

Berber, J. F., and J. D. Martsolf. 1966. Protecting citrus from cold damage. *Florida Agricultural Extension Circular* 287. Univ. of Florida, Gainsville.

Griffin, R. E. 1976. Micro-climate control of deciduous fruit production with over-head sprinklers. In *Reducing Fruit Losses Caused by Low Spring Temperatures*. ed. R. E. Griffin, Final report of Utah Agricultural Experiment Station to Four Corners Regional Commission (Document No. 10550101), Appendix F. Logan, Utah: Utah State Univ.

Griffin, R. E., and E. A. Richardson. 1979. Sprinklers for micro climate cooling of bud development. In *Modifications of the Aerial Environment of Plants*, eds. B. J. Barfield and J. Gerber, pp. 441–455. St. Joseph, Michigan: American Society of Agricultural Engineers.

Karmeli, D., and J. Keller. 1975. *Trickle Irrigation Design*. Glendora, California: Rain Bird Sprinkler Manufacturing Corp.

RECOMMENDED READING

Chesness, J., L. Harper, and T. Howell. 1979. Sprinkling for heat stress reduction. In *Modifications of the Aerial Environment of Plants*, eds. B. J. Barfield and J. Gerber, pp. 338-393. St. Joseph, Michigan: American Society of Agricultural Engineers.

Landass, J. N., and K. Witte. 1967. Irrigation for frost protection. In *Irrigation of Agricultural Lands*, eds. R. M. Hagan, H. R. Haise, and T. W. Edminster, pp. 1037–1057. Madison, Wisconsin: American Society of Agronomy.

Rolston, D. E., R. J. Miller, and H. Schulbach. 1986. Management principles: Fertilization. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama and D. A. Bucks, pp. 317–344. Amsterdam: Elsevier.

Viets, F. G., Jr., R. P. Humbert, and C. E. Nelson. 1967. Fertilizers in relation to irrigation practice. In *Irrigation of Agricultural Lands*, eds. R. M. Hagan, H. R. Haise, and T. W. Edminster, pp. 1009–1023. Madison, Wisconsin: American Society of Agronomy.

III

TRICKLE IRRIGATION

17

Types and Components of Trickle Systems

A trickle irrigation system discharges water close to each plant. Travel over the soil surface or through the air is of limited importance for distributing the water. The application uniformity basically depends on the uniformity of discharge from the emission devices (emitters). Thus, the design strategy for trickle irrigation systems focuses on achieving the desired emission uniformity.

Trickle is an irrigation method that includes surface or subsurface drip, spray, bubbles, and hose-basin application techniques. The word *trickle* is used herein, because it is a crisp word like *sprinkle* and adequately descriptive. However, it cannot be directly translated into many other languages, for example, French and Spanish, and that is a problem. Because of this, *drip, microirrigation*, and *localized irrigation* are often used, instead of trickle, as the name of this irrigation method. But these alternative words or expressions have their own limitations.

Components that are usually required for a trickle irrigation system include the pumping station, control head, main and submain lines, lateral lines, emitters, valves, fittings, and other necessary appurtenances (see Fig. 2.6).

The types of components suitable for a given system depend to a certain degree upon the type of system being considered. For example, the system operating pressure and filtration capacity will be, in part, determined by whether the water is applied by a surface or subsurface drip, bubbler, or spray system. In this chapter some of the components characteristic of each of these types of systems will be described.

TYPES OF WATER APPLICATORS

Water applicators are the small dispensing devices used to control the discharge of water in trickle irrigation systems.

Emitter

This is an applicator used in drip, subsurface, or bubbler irrigation. Emitters are designed to dissipate pressure and to discharge a small uniform flow or trickle of water at a constant rate that does not vary significantly because of minor differences in pressure. Ideally emitters should have a relatively large flow cross section or some means for flushing, to reduce clogging problems. Furthermore, emitters should be inexpensive and compact.

Different types of emitters are often classified according to the mechanism each uses to dissipate pressure. For example, *long-path emitters* use a long, capillary-sized tube or channel to dissipate pressure, but *orifice emitters* rely on individual or a series of orifices, and *vortex emitters* employ a vortex effect. *Flushing emitters* are designed to have a flushing flow of water to clear the discharge opening every time the system is turned on. *Continuous-flushing emitters* permit continuous passage of large solid particles while operating at a trickle or drip flow; this reduces requirements for filter fineness. *Compensating emitters* discharge water at a constant rate over a wide range of lateral line pressures. *Multioutlet emitters* supply water to two or more points through small-diameter auxiliary tubing.

Emission Point

This is a point on or beneath the ground surface where water is discharged from an emitter. Trickle irrigation with water discharged from emission points that are rather widely spaced, usually 1 m (3.3 ft) or more, is called *point-source* application. When water is discharged from more closely spaced outlets, it is called *line-source* application.

Line-Source Tubing

There are three types of line-source tubing. *Single-chamber tubing* is a small-diameter (less than 25-m (1-in.)) hose that has orifices punched or more complex emitters fabricated or inserted at intervals of 0.6 m (2 ft) or less. *Double-chamber tubing* is a small-diameter (less than 25-m (1-in.)) hose that has both a main and an auxiliary bore separated by a single wall. Widely spaced inner orifices are punched in the separator wall between the main and auxiliary bores; for each inner orifice, three to six exit orifices are punched at intervals of 0.15 to 0.60 m (0.5 to 2 ft) in the outer wall of the auxiliary bore. *Porous-wall tubing* is small-diameter (less than 25-m (1-in.)) hose that has a uniformly porous wall. The pores are of capillary sizes and ooze water when under pressure.

Sprayers

These are small applicators (also called aerosol emitters, foggers, spitters, misters, microsprayers, or miniature sprinklers) used in spray irrigation. They are designed to dissipate pressure and discharge a small uniform spray of water to cover an area of 1 to 10 m^2 (10 to 100 ft^2). Ideally sprayers should apply a relatively uniform depth of water to the area wetted and should have a single large-flow cross section and a low water trajectory angle.

FLOW FROM WATER APPLICATIONS

Detailed discussion of the flow characteristics, the emitter selection, and the expected uniformity of application under trickle irrigation is presented in Chapter 20. In most of the following definitions and terms, *water applicator* or *sprayer* can be substituted for the word *emitter* because sprayers and other water applicators are types of emitters.

Emitter Discharge Exponent

The flow or discharge from most trickle irrigation emitters or sprayers with fixed or flexible cross sections can be expressed by:

$$q = K_d H^x \tag{17.1}$$

in which

q = emitter flow rate or discharge, L/hr (gph)
K_d = discharge coefficient, which is a constant of proportionality that characterizes each emitter
H = working pressure head at the emitter or sprayer, m (ft)
x = emitter discharge exponent

The value of x characterizes the flow regime and discharge versus pressure relationship of the emitter. The lower the value of x, the less discharge will be affected by variations in pressure (see Fig. 17.1). Noncompensating simple *orifice* and *nozzle emitters* and *sprayers* are typically fully turbulent and x = 0.5. For *fully compensating emitters*, x = 0.0. The exponent of *long-path emitters* is usually between 0.7 and 0.8. For *vortex sprayers or emitters*, x is about 0.4. The exponent of *tortuous path emitters* usually falls between 0.5 and 0.7.

Emitter Flow and System Relations

The following relations characterize the interaction between pressure variations due to friction loss and elevation differentials and emitter discharge. Usually, trickle irrigation systems are provided with some means for regulating pressure, flow, or both. The area served downstream from each pressure or flow regulation point is called a *subunit* (except where compensating emitters are used).

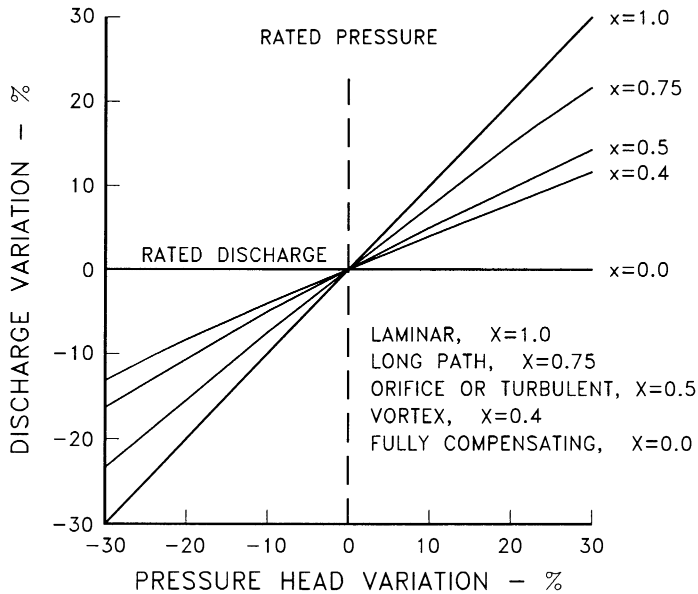


FIG. 17.1. Discharge Variation Resulting from Pressure Changes for Emitter Having Different Discharge Exponents (Adapted from: Karmeli and Keller, 1975 (Fig. 2.7)).

The *average emitter discharge*, q_a, is the flow rate for a system, or subunit, divided by the number of emitters in operation. The *average emitter pressure head*, H_a, is the average pressure head that will produce the average emitter flow rate, q_a.

Emission Uniformity

Emission uniformity, EU, is a measure of the uniformity of emissions from all the emission points within an entire trickle irrigation system. For *field tests*:

$$\mathrm{EU}' = 100\, q_n'/q_a \tag{17.2}$$

where

EU' = field test emission uniformity, %
q_n' = average rate of discharge of the lowest one-fourth of the field data emitter discharge readings, L/hr (gph)
q_a = average discharge rate of all the emitters checked in the field, L/hr (gph)

For *design estimates*, the design EU is based on the anticipated variations of pressure within the system and consequent variations of emitter flow from Eq.

17.1. It is also dependent on the manufacturing variation between emitters and the number of emitters per plant.

The *allowable variation in pressure head*, ΔH_s, is the variation of pressure head between emitters in a subunit that will give the design EU in the subunit. The subunit may be the manifold and attached laterals, a group of laterals, or a single lateral, depending on where the pressure is regulated.

The *manufacturer's coefficient of variation* of emitters, v, is a term used to designate the anticipated variations in discharge of a sample of new emitters (or sprayers) when operated at any given pressure head. The value of v is calculated by dividing the standard deviation by the mean discharge from a representative sample of 50 or more emitters operated at the same pressure head.

SYSTEM LAYOUT AND PIPE NETWORK

The pipe network should be designed to deliver water to the emitters at the appropriate pressure. It should be designed to optimize cost-effectiveness for the particular application. In designing pipe networks for trickle irrigation systems, the life-cycle costing techniques presented in Chapter 8 should be used. This should be done to achieve the minimum total costs of operation, maintenance, and capital at the desired rate of return over the presumed economic life of the system.

Emitters require a supply of clean water. Once water has left the filter, recontamination must be avoided. The components of the pipe network must be noncorrosive and nonscaling and otherwise highly reliable against failure. The most widely used pipe materials are polybutylene, polyethylene, or PVC for laterals; and PVC or polyethylene for manifolds and main lines; however, other materials now being used for main lines include filament-wound epoxy pipe, epoxy-coated steel pipe, and asbestos-cement.

Basic Components

Figure 17.2 shows a basic system layout for a level field with the water supply in the center. The *control head* includes the pump station, filtering equipment, fertilizer and chemical injection equipment, controllers, main pressure regulators, valves, and water-measuring devices. The *main lines* transfer water from the source to the manifolds. Usually control valves are used at the main line-to-manifold connections. The *manifolds*, in turn, supply water to the laterals that branch from it on one or both sides (see Figs. 2.6 and 17.2). Both submains and the manifolds may be on the surface, but are usually buried.

Sometimes *headers*, which are connected to and run parallel with the manifolds, are used to serve several laterals (see Fig. 17.3). *Lateral lines* are usually 9.5- to 25-m ($\frac{3}{8}$- to 1-in.) diameter polyethylene or PVC hose or tubing. Laterals supply water to the emitters or directly to the soil in the case of porous, single-,

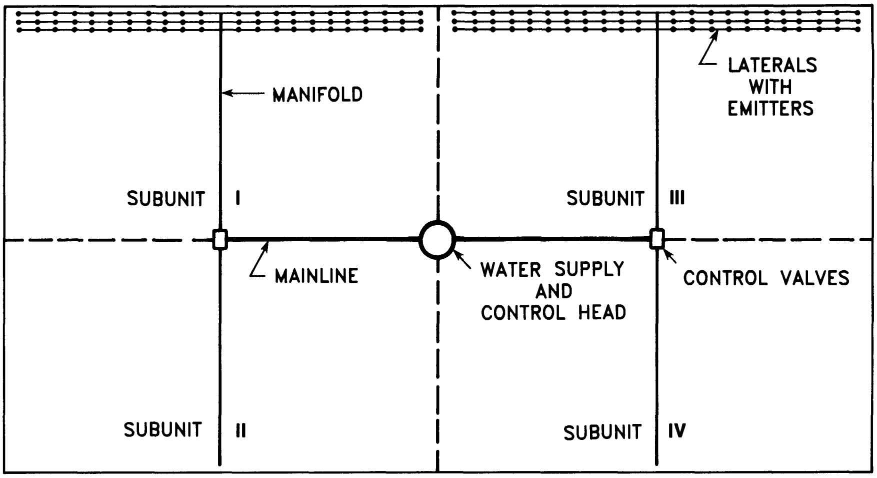


FIG. 17.2. Typical Two Station Split Flow Layout for Trickle Irrigation System with Subunit I and III or II and IV Operating Simultaneously.

or dual-chamber line-source tubing. *Pressure or flow regulators* (manual or automatic), *control valves* (electric, hydraulic, or manual), and *secondary filters* are often located at the inlets to the manifolds, headers, or lateral lines.

Control Head

The most important components of a main control head for a trickle irrigation system (see Fig. 2.6) are the chemical injector, controller, and filter necessary for continuous operation.

Injectors are used to put fertilizer, systemic insecticides, algaecides, acids, and other liquid materials into the irrigation water. Piston-type injectors or Venturi injectors that create a pressure drop across an orifice to siphon the chemical solutions from a tank are most commonly used (see Figs. 16.1 and 16.2).

Automatic controllers provide a signal to actuate the main pump, the automatic manifold valves, or both. The actuating signal may either be time- or volume-based or may be controlled by a soil moisture sensor placed in the plant root zone.

Filters that remove debris that might clog or otherwise foul the emitters or sprayers are essential on most systems. *Screen filters* are the simplest and provide the least expensive and most efficient means for filtering water where they are suitable. *Gravel and graded-sand filters* are cylindrical tanks that have fine gravel and sand of selected sizes placed inside. They are used principally for filtering out heavy loads of very fine sand and organic matter. *Vortex sand separators* depend on centrifugal force to remove and eject high-density particles from the water. Vortex devices do not remove organic materials, but they are efficient devices for ejecting large quantities of very fine sand or larger inorganic solids prior to final filtration.

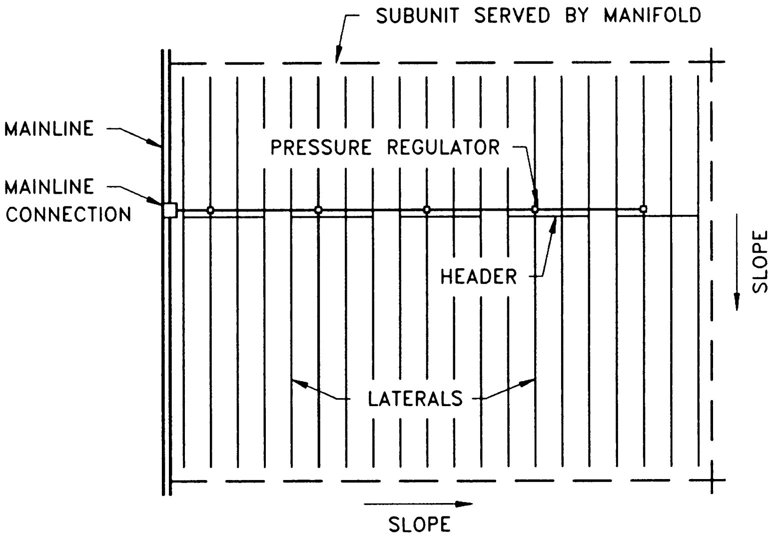


FIG. 17.3. Subunit with the Manifold Positioned Uphill from Center and Pressure Regulated Headers.

Main Pipelines

Normally, pressure-control or adjustment points are provided at manifold inlets. A manifold, with its attached laterals is the basic system subunit. Upstream from pressure-control points the allowed pressure variation for the subunit does not affect pipe-size selection. Therefore, the selection of pipe sizes for the main water supply lines should be based primarily on the economic tradeoff between power costs and costs for pipe and installation (see Chapter 8).

Because normal operating pressures are low, the lowest pressure class available for each pipe size is often suitable for trickle irrigation systems. As with other irrigation pipelines, the velocity of flow, and the number, sizes, and locations of air-, vacuum-, and pressure-relief valves, must be considered and incorporated into the system. Furthermore, some means of flushing and draining the pipelines should also be provided.

The water should be divided as much as is practical among all the mains. By splitting the flow smaller, lower cost pipe can be used. For example, for the system shown in Fig. 17.2, it is best to operate blocks I and III or blocks II and IV simultaneously.

Manifolds and Headers

A manifold (with or without headers) is the portion of the pipe network between the main line and the laterals. It is usually buried but can also be on the surface. The allowable pressure loss for a manifold, header, or both depends on the topography, lateral losses, and total allowable pressure variation for the emitters chosen. Once these limits have been established, standard calculations for hydraulic pipelines that have multiple outlets may be used (see Chapters 8 and 9).

On flat terrain, the manifold-to-lateral connections are usually located in the center of the laterals. Where slope is appreciable, the downhill elevation gain can be balanced by reducing the lateral diameter or by moving the connection point uphill to increase the number of downhill emitters served (see Fig. 17.3). Typically, the manifold is moved uphill to balance the elevation effects.

Frequently the main line-to-manifold connection is the point where in-field pressure is regulated, and where automated control valves may be installed (see Fig. 2.6). However, sometimes the slope is so steep that more pressure-regulating points are required. Such a steep slope may require a pressure- or flow-regulating point at the inlet to each header or at each lateral. In Fig. 17.3 each header serves four laterals with one uphill, one at the pressure-regulated header inlet, and two downhill to balance the elevation effects.

Laterals

Point-source emitters are systematically spaced along the lateral and are connected to the lateral by various means (see Fig. 17.4). Types of lateral lines that themselves function as emitters include single- or dual-chamber line-source tubing as described earlier. *Tapered laterals* have two or more pipe sizes beginning with the larger pipe at the inlet end where flows are largest. They take advantage of the progressively decreasing flow by using smaller pipe sizes where flow rates permit this reduction (see Chapter 9).

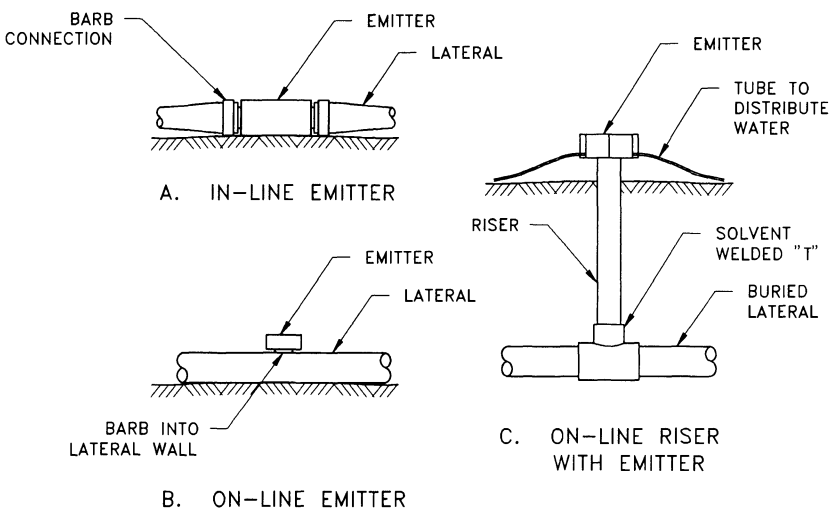


FIG. 17.4. Typical Connections of Emitters to Laterals (Adapted from: Karmeli and Keller, 1975 (Fig. 1.2)).

Emission Point Layouts

For wide-spaced tree crops *double laterals* or two laterals per row are sometimes used to provide more emission points per tree. Other methods of providing more emission points per tree are: to *zigzag, snake, or loop the lateral* around or between the trees; to use short *pigtail lines* looped around each tree; or to use multioutlet emitters with *small-diameter tubing* to distribute the water.

Figure 17.5 shows the various emission point layouts for widespread tree crops. Definitions of the terms used are:

P_d = percent shaded, which is the average horizontal area shaded by the crop canopy as a percentage of the total crop area, %
P_w = percent wetted, which is the average horizontal area wetted in the top part of the crop root zone as a percentage of the total crop area, %
S_e = emitter spacing, which is the spacing between emitters or emission points along a line, m (ft)
S_l = lateral spacing, m (ft)
S_p = plant spacing in the row, m (ft)
S_r = row spacing, m (ft)
w = width of the wetted strip, m (ft)

PRESSURE CONTROL AND REGULATION OF FLOW

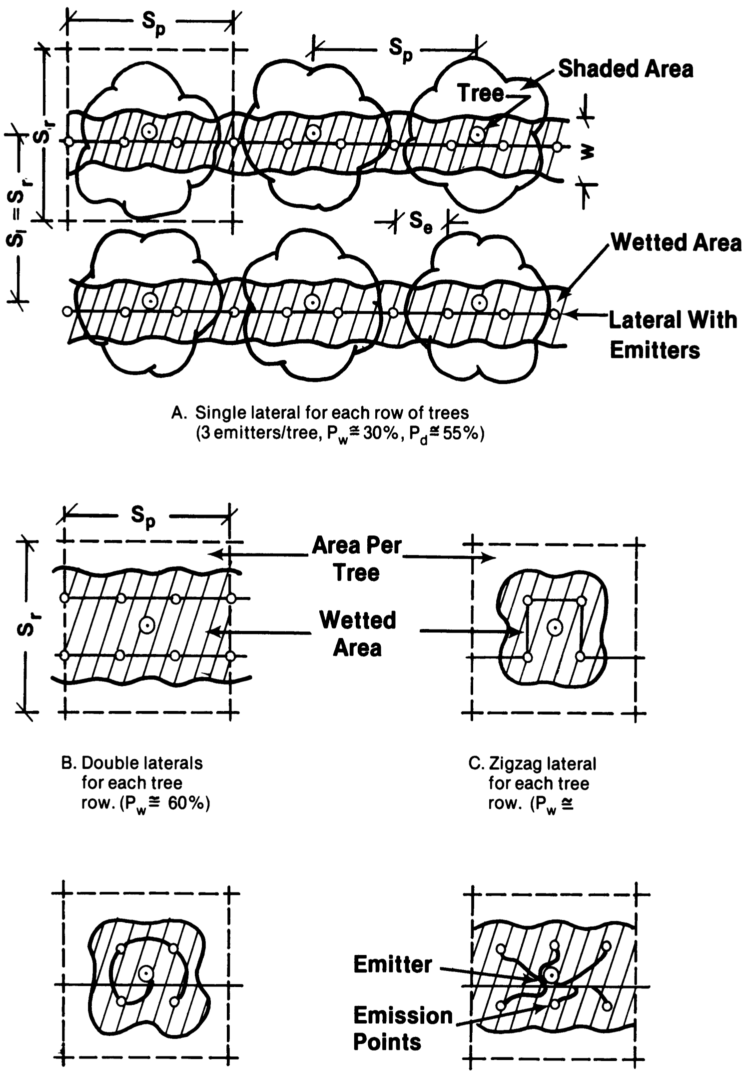
System controls should either be volumetric or should incorporate volumetric monitoring with time-sequencing. This is important because it is impossible to precisely control emitter discharges because of temperature and pressure fluctuations and aging, plugging, and slow clogging of the emitters.

Flow Regulation

The flow rate in most systems is controlled by adjusting the pressure at the manifold inlets. This is done by either automatic or manual valves that are set to balance flow rates between the subunits. Another method for controlling flow is to use pressure or flow regulators at the inlet to each lateral or header (see Fig. 17.3). The inexpensive valves used for this purpose are similar to those used along center-pivot laterals (see Fig. 14.14). They are usually preset (and often nonadjustable) for a given pressure or flow rate.

An inexpensive way to control the flow to each lateral is to use *jumper tubes* of various diameters and lengths to connect each lateral to the manifold. The lengths of the tubes can be cut to provide the pressure loss required to produce uniform lateral inlet pressures. In effect, the jumper tubes serve as fixed precision fluid resistors, and the degree of pressure uniformity that can be achieved with them is limited only by practical design and installation considerations.

Regulation of pressure or flow at each lateral eliminates the reduction in EU



A. Single lateral for each row of trees
(3 emitters/tree, $P_w \cong 30\%$, $P_d \cong 55\%$)

B. Double laterals
for each tree
row. ($P_w \cong$ 60%)

C. Zigzag lateral
for each tree
row. ($P_w \cong$

D. Pigtail with
4 emitters per
tree. ($P_w \cong$ 40%)

E. Multiexit 6 outlet emitter
with distribution tubing.
($P_w \cong$ 60%)

FIG. 17.5. Various Emission Point Layouts for a Wide-spaced Tree Crop.

resulting from friction head loss and differences in elevation along the manifold. However, the variation between valves does reduce the system emission uniformity to $(1.0 - 1.27\, v')$ of the design EU in which v' is the coefficient of variation between lateral flow rates (see Chapters 14 and 20). The small inexpensive valves normally used for regulating pressure and flow are often unstable, so periodic checking of pressures or flow rates is important.

Fully compensating emitters (i.e., $x = 0.0$) have built-in flow control and require no other system flow or pressure regulation. Unfortunately, the discharge from compensating emitters often decreases with time. Therefore, a representative sample of emitters should be tagged in the field and checked periodically for variations in discharge. Fully compensating emitters eliminate the reduction in EU resulting from variations in pressure head throughout the pipe networks. However, the manufacturer's coefficient of variation, v, of fully compensating emitters is usually considerably larger than that of other emitters. Therefore, the use of compensating emitters may not increase EU′ except on fields with large elevation differences.

Control Valves

The principal valves used for the control, management, and operation of trickle irrigation systems include the valves located at the control head, manifold valves, riser (or header) valves, and flush-out valves.

Manifold valves are located where manifolds are connected to the main line pipe distribution system. They may be operated automatically or manually and typical K_r values range from 3 to 8 for use in Eq. 11.1 to determine valve friction head losses. *Automatic valves* are either operated on a time schedule or are actuated after a predetermined volume of water has been discharged. The automatic valves can be actuated either from a central controller or by a self-contained mechanism at each valve. *Semiautomatic volumetric (or clock) valves* must be turned on manually but can close automatically. *Sequential operation* can be achieved with volumetric valves that are interconnected by hydraulic control lines.

Lateral (or header) valves are located on the manifold risers (outlet devices installed on the manifold) that serve each lateral (or header). These small, hand-operated, on-off valves provide a means for shutting off flow in the lateral when repair or maintenance work is necessary.

Flush-out valves should be placed at the ends of main lines, manifolds, and laterals, so all portions of the pipe network can be periodically flushed. This is essential for removing debris from the system after construction or repairing broken pipelines. Flush-out valves are also useful for removing fine particles that settle out near the ends of lines during normal operation.

OPERATING CONTROLS

Even without automation, trickle irrigation systems require relatively little labor. The main activities of the irrigator are to schedule water and chemical applications and to see that filters are kept clean and the emitters are not plugged.

Most trickle irrigation systems are designed to have two, three, or four sets (or stations). A two-set system is operated with one-half of the emitters operating and one-half off, a three-set with one-third on and two-thirds off, and a four-set with one-fourth on and three-fourths off. The different levels of automation for cycling the sets on and off can be characterized by the methods used for opening and closing the valves as follows. (Similar methods can be used to automate the various types of fixed sprinkler systems described in Chapter 4.)

Partial Automation

Volume control is ideal for trickle irrigation. The simplest means of achieving volume control along with some automation is to use volumetric valves. The valves can be installed at the head of each subunit, or a single valve can be used at the control head along with ordinary valves that control each subunit. Such valves require manually opening and setting the desired volume of discharge, but they close automatically after the preset volume of water has passed. However, unless they are interconnected by hydraulic control lines, the use of volumetric valves does not dictate a special operating sequence.

Partial automation can also be achieved in a similar manner with mechanically or electronically timed valves. Mechanically timed valves must be opened manually and the estimated time required to apply the desired volume of water must be set, but they close automatically after the preset time has elapsed.

Volumetric (or mechanically timed) valves can be interconnected by hydraulic (or electric) control lines to operate in sequence so that, as each valve closes, the next valve opens. However, the valves must all be reset, and the first valve must be activated manually to repeat the cycle again.

Electronically timed valves, which are powered by small batteries, come closer to providing full automation because they can be designed to store (remember) a preset length of operating time. Furthermore, they are easily activated manually by a switch or remotely through electric control wires.

Full Automation

Full automation can be accomplished in several ways. It can be done with accurate *electronically timed clock valves* installed at the inlet to each subunit. Such valves are powered by small batteries and can be set to turn on or off at any specific time during a one- or two-week period. Thus, a set of individual valves can be set to sequence in any desired order.

Full automation is most often done with a central controller operated on a time or volume basis or by various types of sensors. *Centrally controlled* full automation is achieved by using a time- or volume-based control system that operates either hydraulic, electric, or electronically activated valves. Hydraulic valves must be linked to the controller by hydraulic tubes and electric valves must be linked to it with wires. However, electronically activated valves can either be linked to the controller with wires or each fitted with a radio receiving unit and activated by signals transmitted from the central controller.

Controllers are available that can be set to actuate the irrigation cycle in practically any desirable manner. Both the operating time of each valve and the quantity of water discharged through each valve can be easily changed at the control panel. Furthermore, the cycle of each irrigation can be started by a sensor connected to an evaporation pan or to weather instruments or buried in the soil.

REFERENCES

Karmeli, D., and J. Keller. 1975. *Trickle Irrigation Design*. Glendora, California: Rain Bird Sprinkler Manufacturing Corp.

RECOMMENDED READING

Anon. 1973. Trickle Irrigation. Food and Agriculture Organization of the United Nations Irrigation and Drainage Paper 29.

Benami, A., and A. Ofen. 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).

18

Clogging and Filtration

Clogging of emitters is the most difficult problem encountered in the operation of trickle irrigation systems. It is difficult to detect and expensive to clean or replace clogged emitters. Emitters can be clogged by particles in the water supply or by precipitates or bacterial slimes resulting from dissolved calcium or other salts in the water supply. Filtering and keeping contaminants out of the system are the main defense against clogging caused by mineral and organic particles. Periodic or continuous chemical injections are necessary to dissolve mineral precipitates or prevent the growth of slimes.

Clogging causes poor distribution of water along the laterals. This may damage a crop severely if emitters are clogged for a long time before they are cleaned or repaired. Table 18.1 shows a proposed, irrigation-water-quality-classification scheme for predicting potential emitter clogging (Bucks et al., 1979). The development of the classification relied upon various sources, which included laboratory and field experiments at various locations with different water quality.

FILTRATION

Normally, the main bank of filters and chemical injection equipment is located at the pumping plant. In addition, small screens should be installed at the inlet to each lateral hose or header as a safety precaution. These auxiliary screens stop any debris that enters the pipe network when the main filters are cleaned or breaks in the pipe are repaired. The main filters commonly used in trickle irrigation are screen (or net) and graded gravel-sand media filters. In addition, vortex or other sand separators and settling ponds can be used to remove heavy sand loads.

Suspended solids that can plug the small passages in emitters have both organic and inorganic components. Algae, diatoms, larvae, fish, snails, seeds and other parts of plants, and bacteria are the major organic solids. The main inorganic solids are sand and soil particles.

Because a consistent water quality is so important, the filtration and treatment of the water supply must be planned to handle the worst expected condition. Open water, such as is supplied from lakes, ponds, rivers, streams, and canals, can vary greatly in quality and often contains large amounts of organic matter

Table 18.1. Plugging potential of irrigation water used in trickle systems[1]

Type of problem	Little	Some	Severe
Physical[2]			
Suspended solids (maximum ppm)	<50	50–100	>100
Chemical[2]			
pH	<7.0	7.0–8.0	>8.0
Dissolved solids (maximum ppm)	<500	500–2000	>2000
Manganese (maximum ppm)	<0.1	0.1–1.5	>1.5
Iron (maximum ppm)	<0.1	0.1–1.5	>1.5
Hydrogen sulfide (maximum ppm)	<0.5	0.5–2.0	>2.0
Biological[3]			
Bacteria populations (maximum number/mL)	<10,000	10,000–50,000	>50,000

[1]From Bucks et al. (1979).
[2]Maximum measured concentration from a representative number of water samples using standard procedures for analysis.
[3]Maximum number of bacteria per milliliter can be obtained from portable field samplers and laboratory analysis. Bacterial populations do reflect increased algae and microbial nutrients.

and silt. Rain and wind sweep debris into open water, while moving streams carry a suspended load of sand and soil. Warm weather, light, and slow-moving or still water provide conditions that promote rapid growth of algae.

Where open waters are used, a complex filtration system is usually required. The system may consist of a prefilter, such as a settling basin or vortex separator, followed by a sand media filter and then by a screen filter (see Fig. 18.1). Sometimes chemical coagulants are also required to control silt, clay, or suspended colloids, and chlorine is required to control algae. The necessary filtration system depends on the kind and amount of contaminants present and the quality of water required for satisfactory operation of the emitters selected.

Water from wells usually has consistently good quality, but can contain sand. Therefore, a single-screen filter is recommended at the well discharge. Some well water may also be chemically unstable and produce chemical precipitates in the pipes and emitters or provide nutrients for bacterial slimes.

Particle Size

Adequate filtration requires the processing of all water that enters the system. The size of particle that can be tolerated depends on emitter construction and should either be indicated by the manufacturer or derived from experience. Most manufacturers recommend removing particles larger than 0.075 mm or 0.15 mm (0.003 or 0.006 in.), but some allow particles as large as 0.6 mm (0.024 in.).

Obviously, particulate matter larger than the orifice must be removed. Typically the recommendation is to remove all particles larger than one-tenth the diameter of the orifice or flow passages of the emitter. This is necessary because several particles may become grouped together and bridge the passageway.

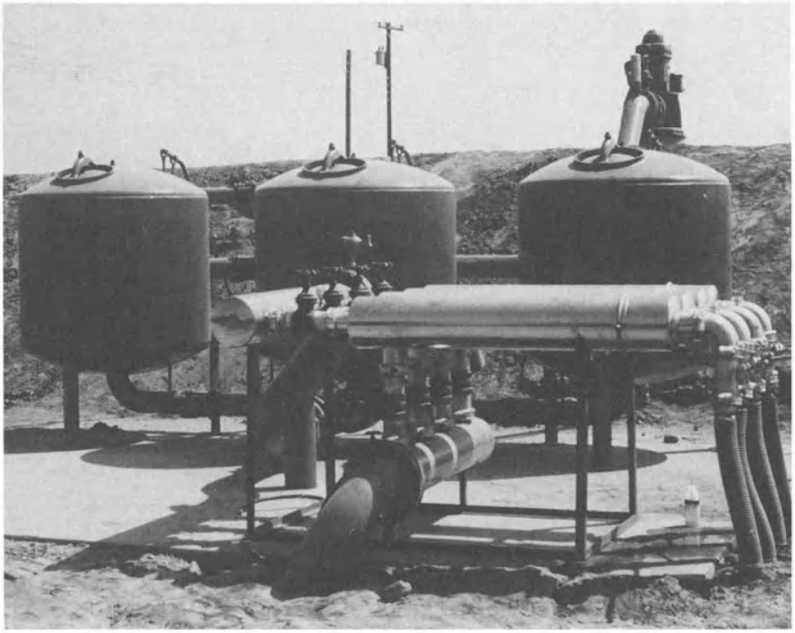


FIG. 18.1. Typical Bank of Sand Filters Followed by Screen Filters for a Trickle Irrigation System.

Inorganic particles, such as fine and very fine sands, are likely to settle out and deposit in zones where flow is slow. Such zones occur near the ends of laterals or at low points when the system is turned off and the laterals slowly drain. Fine inorganic particles can also settle along the walls of laminar flow emitters where even during operation the flow rate is near zero. The clogging that results may not be rapid, but it is inevitable. Therefore, it may be necessary to use a 200-mesh screen that has a 0.075-mm (0.003-in.) hole size even when the emitter flow passageways are 0.04 in. in cross section.

Inorganic particles are usually classified according to size by passing them through a standard series of screens. Screens are classified by the number of openings per inch, with a standard wire size for each screen size. In addition to the average hole size, it is important to define two related terms: the absolute filtration size, which is the largest passage size in the filter; and the average filtration size, which is the average passage size. Some manufacturers use devices other than screens and refer to a pore filtration size, usually expressed in microns (1000 microns = 1 mm = 0.039 in.).

The standard classification of soil particulate size relates to corresponding screen mesh numbers, as given in Table 18.2. Note that a 200-mesh screen, which has the smallest hole size commonly used in trickle irrigation filters, can filter out only a portion of the very fine sand.

Table 18.2. Classification of soils by particle size, with corresponding screen mesh numbers

Soil classification	Particle size mm	microns	in.	Screen mesh number
Very coarse sand	1.00-2.00	1000-2000	0.0393-0.0786	18-10
Coarse sand	0.50-1.00	500-1000	0.0197-0.0393	35-18
Medium sand	0.25-0.50	250-500	0.0098-0.0197	60-35
Fine sand	0.10-0.25	100-250	0.0039-0.0098	160-60
Very find sand	0.05-0.10	50-100	0.0020-0.0039	270-160
Silt	0.002-0.05	2-50	0.00008-0.0020	400-270[1]
Clay	<0.002	<2	<0.00008	—

[1]400 mesh wire screen has the smallest opening, i.e., approximately 0.03 mm (0.0012 in.).

As mentioned earlier, bridging of particles creates a need to screen out particles too small to cause blockage individually. Organic particles have about the same density as water and frequently cause bridging. Particle bridging can also be accelerated by adhesion where particles are attracted to each other. This increases the chance of several particles arriving at an orifice at the same time.

Settling Basins

Settling basins, ponds, or reservoirs can remove large volumes of sand and silt. Basins should be constructed so water entering the basin takes at least one-quarter hour to travel to the system intake. In this length of time most inorganic particles larger than 80 microns (about 200-mesh) will settle out. A basin 1.2 m deep by 3.3 m wide by 13.7 m long (4 by 10 by 45 ft) is required to provide a one-quarter hour retention time for a 57-L/s (900-gpm) stream.

Settling basins should be relatively long and narrow to eliminate short circuit currents that reduce effective retention time. The sides and bottom of the basin should be lined to discourage vegetative growth, and there should be a good algae control program.

Vortex Sand Separators

Modern vortex (centrifugal) sand separators can remove as much as 98% of the sand particles that would be contained by a 200-mesh screen. They depend on centrifugal force to remove and eject high-density particles from the water and consequently cannot remove organic materials. They are efficient devices for ejecting large quantities of very fine sand from wells, streams, rivers, or canals. However, vortex separators must be supplemented by screen filters downstream to catch contaminants that pass through.

Sand Media Filters

Graded-sand media filters consist of fine gravel and sand of selected sizes placed inside a cylindrical tank (see tanks in background of Fig. 18.1). As water passes through the tank, the gravel and sand perform the filtration. Media filters are used primarily for filtering out heavy loads of very fine sand and organic material. They are often constructed so they can be automatically backwashed as needed. Recommended practice is to use a screen filter downstream from the media filter to catch particles that escape during backwashing.

Sand media filters are most effective in filtering organic material, because they can collect contaminants through the depth of the sand bed and accumulate large quantities of algae before backwashing is necessary. Also, long, narrow particles, such as some algae or diatoms, are more apt to be caught in the multilayered sand bed than on the surface of a screen.

Factors that affect filter characteristics and performance are water quality, types and sizes of sand media, flow rate, and allowable pressure drop. A sand media filter can handle larger loads of contaminants than a screen of comparable fineness. It can do it with less frequent backflushing and a smaller pressure drop. However, sand media filters are considerably more expensive. They are generally used only when a screen filter would require very frequent cleaning and attention or to remove particles smaller than 0.075 mm (0.003 in.).

The sand media used in most trickle irrigation filters is designated by numbers. Numbers 8 and 11 are crushed granite, and numbers 16, 20, and 30 are silica sands. The mean granule size in microns for each media number is approximately 1900, 1000, 825, 550, and 340 for numbers 8, 11, 16, 20, and 30, respectively.

At a flow velocity of 17 L/s per m^2 (25 gpm per ft^2) of bed, the numbers 8 and 11 crushed granite remove most particles larger than one-twelfth of the mean granule size or approximately 160 and 80 microns, respectively. The silica sand numbers 16, 20, and 30 remove particles approximately one-fifteenth the mean granule size or approximately 60, 40, and 20 microns, respectively.

Typically, the *initial pressure drop* across the number 8, 10, and 16 media is between 14 and 21 kPa (2 and 3 psi). For the number 20 and 30 media it is approximately 34 kPa (5 psi). The *rate of pressure-drop increase* is usually linear with time for a given quality of water and flow rate. Assuming 1.0 unit of pressure drop per unit of time for a number 11 media, the units of pressure drop per unit is time across the other media would be: 0.2 unit for number 8 media; 2 units for number 16 media; 8 units for number 20 media; and 15 units for number 30 media. For example, if it takes 24 hr for the pressure drop to increase by 34 kPa (5 psi) across a number 11 media, it would take only about 3 hr for the same increase across a number 20 media.

The *maximum recommended pressure drop* across a sand filter is generally about 70 kPa (10 psi). Backflushing must be frequent enough to hold the pressure drop within the prescribed design limits. Where frequent backflushing is required, automatic backflushing is recommended. It can be actuated by a timer or by sensing the pressure differential across the media.

Backflushing flow rates vary with the size of the media and the construction of the filter. Typical recommended backflushing flow rates vary from 7 to 10 L/s per m^2 (10 to 15 gpm per ft^2) of filter bed for numbers 30 and 20 media and between 14 and 17 L/s per m^2 (20 and 25 gpm per ft^2) of bed for numbers 16 and 11 media.

Flow rate across the media is an important consideration in designing filters. For a given quality of water and filter media, the size of particles passing through increases as the rate of flow increases. Figure 18.2 shows the effect of flow rate on the size of particles passing through different-sized medias.

High-rate filter technology is based on a nominal value of roughly 14 L/s per m^2 (20 gpm per ft^2) of bed. This value has been established relative to a given bed composition and filter use. However, flow rates as high as 20 L/s per m^2 (30 gpm per ft^2) may be allowable for trickle irrigation.

Screen-Mesh Filters

Where they are suitable, screen-mesh filters provide a simple and efficient means for filtering water. Hole size and the total amount of open area determine the efficiency and operational limits of screen filters. Screen filters efficiently remove very fine sand or small amounts of algae. Even moderate amounts of algae

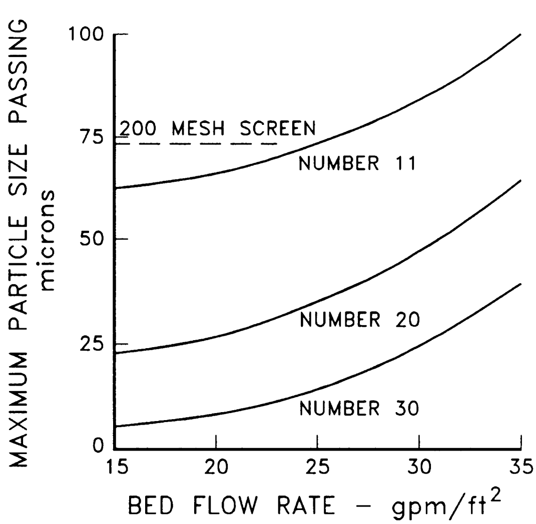


FIG. 18.2. Effect of Flow Rate on Maximum Particle Size Passing Through Typical Free Flow Sand Filters with Different Media.

can quickly block single screens unless they are specifically designed to accommodate an organic contaminant.

Screen-mesh filters are of several kinds: simple manually cleaned with stainless steel, nylon, or polyester mesh: backflushing; blow-down; and gravity flow. *Stainless steel mesh* offers relative strength. *Nylon mesh* in some blow-down filters has the advantage of fluttering during a flushing cycle; this action helps to dislodge the collected material. *Blow-down filters* are configured so that without disassembling the filter a high velocity jet of water can be run over the screen to sweep away the collected contaminants. The flow of water through a *backflushing filter* can be reversed to remove the collected particles. *Gravity-flow filters* function by running the water onto and through a large screen before pumping it into the pipe delivery network. Some gravity-flow filters are provided with jets under the screen that help lift the contaminants and move them to one side and away.

The *duration of operation* of the filter is the period between cleanings. The need for cleaning is determined by drop of pressure across the filter. It is customary to clean screen filters whenever the pressure drop has increased by 20 to 35 kPa (3 to 5 psi) or at predetermined intervals. The *common cleaning systems for screen mesh filters* are: simple manual cleaning by pulling out the filter basket and washing it; and manually actuated or automatically actuated blow-down or backflushing filters.

Simple manual cleaning is satisfactory when cleaning is required only once or twice a week. Where daily cleaning is required, manually actuated blow-down or backflushing filters should be used. Where cleaning is required more than once a day, automatic cleaning systems should be used.

Regardless of the cleaning system used, extreme care should be taken to prevent contaminants from bypassing the filter during cleaning. Backflushing with filtered water is recommended. Also, downstream safety devices, such as small filters or hose washer screens at each lateral connection, provide additional safety. Extreme caution in keeping large particles out of the system is necessary. This is especially important in view of the potential for accidents, such as breaks in the main line. A few handsful of sand or organic particles can ruin a system.

The head loss in a clean mesh filter normally ranges between 14 and 35 kPa (2 and 5 psi). The loss depends on the valving, filter size, percentage of open area in the screen (sum of the holes), and discharge. The anticipated head loss between the inlet and outlet of the filter system just before cleaning should be used when computing the required system inlet pressure. The head loss through a mesh filter will normally range between 35 to 70 kPa (5 and 10 psi).

A mesh filter with a high discharge in relation to the screen area may require frequent cleaning and have a short life. The factors that should be considered when selecting screen filters are: water quality, system discharge; filtration area and percentage of open area per filter; desired cleaning cycle; and allowable pressure drop.

The *maximum recommended flow rate* through a fine screen should be less than 135 L/s per m^2 (200 gpm per ft^2) of screen open area. The wire or nylon mesh obstructs much of the open area. For example, a standard 200-mesh, stainless steel screen has only 58% open area. An equivalent nylon mesh with the same-sized openings has only 24% open area. Therefore, it is important to consider the percentage of open area when sizing a filter for a given system discharge.

PRECIPITATES AND ORGANIC DEPOSITS

Dissolved solids are a problem when they precipitate as a solid mineral or are a source of nutrients for algae and bacterial slimes. Slow clogging and eventual plugging of emitters by precipitates and organic deposits are problems that cannot be solved by filtration.

Precipitates

Precipitates are caused by dissolved minerals coming out of solution because of a change in pH or temperature. Precipitates form inside the pipe and emitters and can cause slow and ultimately full clogging. Precipitates are not the same as mineral deposits resulting from evaporation. Such deposits basically build up on the outside of emitters and usually are not a problem except at emitter outlets. However, this can cause plugging if the outlets are not protected from evaporation.

Calcium and iron precipitates are a potential problem with most well waters. An analysis can indicate whether the bicarbonate or iron concentration is high enough to cause precipitation. General observations show that a bicarbonate concentration greater than 2.0 meq/L coupled with a pH greater than 7.5 is likely to produce troublesome *calcium* precipitates.

One way to deal with bicarbonates is to inject acid into the irrigation pipe network. The least expensive acid should be chosen and used at a concentration sufficient to offset the excess bicarbonates. The amount of acid needed and the optimum pH are a function of water quality, equipment composition, temperature effects, and the acid. Maintaining a pH of 5.5 to 7.0 has effectively eliminated precipitates when the irrigation water contained excessive amounts of bicarbonates.

Typical flow rates of acid injection required to maintain a sufficiently low pH range from roughly 0.02 to 0.2% of system capacity. To reduce costs, slug treatment at the end of each irrigation or on a periodic basis is sometimes effective. Where concentrations of bicarbonate are high, it is more practical to aerate the water and hold it in a reservoir until it reaches chemical equilibrium and the precipitate settles out.

As little as 0.3 ppm of *iron* can cause troublesome precipitation in a trickle irrigation system. Iron is present in water in the soluble (ferrous) form. In the presence of oxygen, it oxidizes to the insoluble ferric form, which causes a reddish brown precipitate.

To prevent precipitation in the pipe network, iron can be deliberately precipitated and filtered out beforehand. A chemical feeder can be set to provide a measured volume of chlorine solution sufficient to effectively oxidize the iron and other organic compounds present and to allow a residual 1.0 ppm chlorine. Sodium hyperchlorite is preferred over calcium hyperchlorite where groundwater supplies are somewhat "hard" because calcium hyperchlorite tends to precipitate the calcium.

Chelating the iron with a phosphate chelating agent two to five times the concentration of the ion molecules should prevent precipitation of iron. However, where iron concentrations as high as 10 ppm are encountered, aeration by a mechanical aerator and settling in a reservoir may be the most practical method of control. Injecting air into the water supply by mechanical means followed by filtration is another effective method for removing iron.

Organic Growths and Deposits

Algae and slime created by bacteria in the water can cause severe clogging. *Algae* are a family of plant organisms that grow by converting light energy and the nutrients presents in their environment into food. Algae are commonly found in almost all supplies of surface water. Because small particles of algae can pass through filters, it is important to keep them from growing inside the system. This is accomplished by using black emitters and black pipe above ground, because most algae need sunlight to grow. In darkness, bacteria tend to break down the algae particles. The residue can then leave the system through the emitters along with suspended silt and clay.

Slime is a generic term for long-filament microorganisms produced primarily by bacteria. The possibility of proliferation of slime-producing bacteria in the system is of equal or greater importance than precipitates or particle size, adhesion, and briding. The slime acts as a "glue" creating gelatinous agglomerations from the particulate matter that can easily plug emitters. These microorganisms do not produce their own food and do not require sunlight for growth. The more common are airborne; therefore, systems with open water supplies are most susceptible to slime growths.

Iron bacteria produce a reddish brown slime that appears in or on the pipe and emitters. These organisms grow in water and feed on metallic iron dissolved in the water, as well as iron that is in pumps, pipes, and tanks. Some bacteria can produce sufficient slime to plug an emitter with iron concentrations as low as 0.3 ppm when the pH of the water supply ranges between 4.0 and 8.5.

The methods used to clear drip systems of iron bacteria employ oxidation or reduction reactions or both. Normally the irrigation system is superchlorinated to achieve a chlorine concentration of at least 10 ppm in the pipe system. This is done until the organic material is oxidized and is discharged from the system. Continuous chlorine injection appears to be the best method of combating iron bacteria.

Injection of Acids and Chloride

Both algae and slime can be inexpensively controlled by chlorination. Algae and slime can usually be eliminated in trickle systems by maintaining a continuous concentration of 1.0 ppm residual chlorine at the ends of laterals. It can also be done by injecting sufficient chlorine to bring the concentration to between 10 and 20 ppm during the last 20 min of the irrigation cycle. Typical recommended chlorine dosages for different organic growth and precipitation problems are:

- For *algae* use 0.5 to 1.0 ppm continuously or 20 ppm for 20 min at the end of each irrigation cycle;
- For *hydrogen sulfide* use 3.6 to 8.4 times the hydrogen sulfide content;
- For *iron bacteria* use 1.0 ppm over the number of ppm of iron present (this can vary depending on the number of bacteria to be controlled);
- For *iron precipitation* use 0.64 times the Fe^{2+} content to maintain 1.0 ppm free residual chloride at the ends of laterals;
- For *manganese precipitation* use 1.3 times the Mn content; and
- For *slimes* maintain 1.0 ppm free residual chlorine at the ends of laterals.

Efficiency of chlorine injection is related to the pH of the water to be treated. More chlorine is required at a high pH. In severe cases for algae and slime treatment, a detention destruction facility is necessary. This consists of a pond or concrete tank to retain the irrigation water long enough for sufficient oxidation and destruction of the chlorinated algae-slime mixture.

Figures 16.1 and 16.2 show common methods used for injecting chemical solutions into irrigation systems. Liquid chlorinators that use positive displacement pumps are usually preferred over gas chlorinators. This is because they can also be used to inject other chemicals and they are much less expensive and hazardous.

The rate of injection of liquid chlorine or acid depends on the system flow rate and can be calculated by:

$$q_c = K\frac{uQ_s}{c'} \qquad (18.1)$$

where

q_c = rate of injection of the chemical into the system, L/hr (gph)
K = conversion constant, 3.60×10^{-3} for metric units (5.01×10^{-4} for English units)
u = desired dosage in irrigation water, ppm
Q_s = irrigation system discharge capacity, L/s (gpm)
c' = concentration of the desired component in liquid chemical concentrate, kg/L (lb/gal)

SYSTEM FLUSHING AND MAINTENANCE

Flushing is an important part of the system start-up and operating maintenance program. After construction or repairs, the system should be carefully flushed, beginning with the main line and proceeding to the submains, manifolds, and laterals. Therefore, valves should be provided at the ends of submains and manifolds, and provisions should be made for flushing each lateral. Furthermore, the end sections of manifolds should have large enough diameters to facilitate flushing.

The main lines and then the submains should be flushed one at a time with the manifold or riser valves turned off. Closing all valves but the one on the submain being flushed allows large flows to wash out coarse debris. Next, each manifold should be flushed with all the lateral riser valves turned off. Finally, the lateral hoses should be connected and flushed for about an hour on each operating station.

Periodic Flushing

Good maintenance requires keeping filters in perfect operating condition and that emitter discharge be uniform and sufficient to meet crop water requirements. Therefore, the main filters must be cleaned periodically, and the secondary filters at inlets to manifolds and laterals must be routinely checked.

Fine inorganic particles usually settle out at the ends of manifolds and laterals where flow velocities are slow. Because high concentrations of fine contaminants are likely to clog emitters, periodic flushing is recommended. Annual flushing is often sufficient, but some combinations of water and emitters require almost daily flushing to prevent clogging. Where frequent flushing is required, semiautomatic or automatic flushing valves at the ends of lateral are recommended.

A velocity of 0.3 m/s (1.0 ft/s) is necessary to adequately flush fine particles from lateral tubing. For 13-m. ($\frac{1}{2}$-in.) hose, this converts to approximately 3.8 L/min (1.0 gpm). This must be taken into account where semiautomatic or automatic flushing valves are to be utilized.

Discharge Checks

Systematic field checks are required to spot malfunctioning emitters. Discharge from any emitter may be affected by partial or total blockage or by physical deterioration of emitter parts. Slow clogging that causes partial blockage can result from an agglomeration of sediments, precipitates, organic deposits, or mixtures of these. Physical deterioration is a concern in pressure-compensating emitters. The flow passage may slowly close as the compensating portion distorts with time. Mechanical malfunction can also be a problem in flushing emitters.

Discharge from emitters should be checked periodically for uniformity. A good practice is to determine the field emission uniformity, EU′, at least once in each irrigation season. In addition, the average discharge of all the emitters, q_a, should be computed to evaluate overall system performance. This can be done from the system flow-rate data used for scheduling irrigations.

Emitters should be cleaned, replaced, or repaired whenever EU′ has degenerated more than 10% or $q_a\,(\mathrm{EU}'/100)$ is insufficient to satisfy plant water requirements. (The value $q_a\,(\mathrm{EU}'/100)$ is the average emitter discharge to the least watered one-fourth of the trees or other plants.)

Cleaning Emitters

The cleaning required depends on the type and size of emitters and the type of clogging. Some emitters can be disassembled and cleaned manually. Others can be manipulated and flushed to eject loose deposits. Carbonate deposits can be removed using solutions of 0.5 to 1.0% HCl acid injected at manifold or lateral inlets to give a contact time of 5 to 15 min in the emitters. For iron precipitates, sulfuric acid should be used. Acid treatment may not be 100% effective or practical and is ineffective for completely clogged emitters.

Air pressure of 490 to 980 kPa (70 to 140 psi) applied at lateral inlets has successfully ejected jellylike slime deposits from long-tube emitters. However, the emitters and connections to the lateral hose must be very strong to withstand the pressure. Furthermore, compressed air is not effective for many types of emitters and clogging materials. The use of high water pressure has only limited possibilities for cleaning emitters. It is virtually impossible to get sufficient pressure to the emitters at the ends of the laterals.

The best practice to keep a trickle system operating efficiently is to do everything practical to prevent clogging of pipes and emitters. The first step is to select the type of emitter and filtration system best adapted for use with the available water supply. Then a good preventive maintenance program requires that the irrigator do the following:

- Carefully flush the entire pipe network after construction or repairs.
- Treat the water chemically, if necessary;

- Filter the water sufficiently to assure uninterrupted operation of the emitters being used;
- Maintain the filter system so the emitters receive either 100% filtered water or no water at all; and
- Flush the system periodically.

REFERENCES

Bucks, D. A., F. S. Nakayama, and R. G. Gilbert. 1979. Trickle irrigation water quality and preventative maintenance. *Agricultural Water Management* 2:149–162.

RECOMMENDED READING

Gilbert, R. G., and H. W. Ford. 1986. Operating principles: Emitter clogging. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama and D. A. Bucks, pp. 142–163. Amsterdam: Elsevier.

Nakayama, F. S. 1982. Water analysis and treatment techniques to control emitter plugging. *Water, Energy and Economic Alternatives*, pp. 97–112. (1982 Annual Technical Conference Proceedings, Feb. 21–24, 1982, Portland, Oregon). Arlington, Virginia: The Irrigation Association.

Nakayama, F. S. 1986. Operational principles: Water treatment. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama and D. A. Bucks, pp. 164–216. Amsterdam: Elsevier.

19

Trickle Irrigation Planning Factors

Trickle irrigation systems are usually designed and managed to deliver frequent light applications of water and to wet only a portion of the soil surface. Therefore, the procedures used for other methods must be adjusted to compute water- and salinity-control requirements, irrigation depth, and frequency (Keller and Karmeli, 1974; Karmeli and Keller, 1975).

SOIL WETTING

Trickle irrigation systems normally wet only a portion of the horizontal, cross-sectional area of soil, as depicted in Fig. 2.7. The percentage wetted area, P_w, compared with the entire cropped area, depends on the volume and rate of discharge at each emission point, spacing of emission points, and type of soil being irrigated. The area wetted at each emission point is usually quite small at the soil surface and expands somewhat with depth to form an inverted bulb-shaped cross section. P_w is determined from an estimate of the average area wetted at a depth of 150 to 300 mm (6 to 12 in.) beneath the emitters divided by the total cropped area served.

Ideal Percentage Wetted Area

No single ''right'' or proper minimum value for P_w has been established. Nevertheless, systems having high P_w provide more stored water (a valuable protection in the event of system failure). Consequently, they should be easier to schedule and bring more of the soil system into action for storage and supply of nutrients. A reasonable objective of design for widely spaced crops, such as vines, bushes, and trees, is to wet at least one-third and as much as two-thirds of the potential horizontal cross-sectional area of the root system, i.e., $33\% < P_w < 67\%$. In regions that receive considerable supplemental rainfall, values lower than one-third are acceptable for medium- and heavy-textured soils.

For widely spaced crops, P_w should be held below 67% to keep the strips between rows relatively dry for cultural practices. Low P_w values also reduce loss of water due to evaporation even where cover crops are used. Furthermore, it is less costly to have a low P_w, for more emitters and tubing are required to obtain a larger coverage. However, in closely spaced crops with rows and emitter laterals spaced less than 1.8 m (6.0 ft) apart, P_w often approaches 100%.

If precipitation is sufficient to wet a meter (few feet) deep, plant roots tend to explore beyond the trickle-irrigated moisture profile. This root activity is important, because it may account for a significant amount of water and nutrient uptake. There is little evidence that root anchorage is a problem under trickle irrigation where $P_w \geq 33\%$. However, in high-wind areas, where root anchorage is a problem, extension of root development resulting from natural precipitation would be helpful.

Figure 19.1 shows the type of relationship that may exist between potential production and P_w even when plant water requirements are fully met. Although the data for specific curves are insufficient, from current experience the following assumptions are logical. P_w versus potential yield curves must start near the origin where there is little or no rainfall. Significant production is achieved when only a relatively small portion of the soil volume receives water. Maximum production will be achieved with considerably less than full wetting. And for a given value of P_w, different crop-soil-climate systems may show significant variations.

For some crops potential yields under trickle irrigation are higher than yields being obtained by other irrigation practices. Therefore, trickle systems that seem

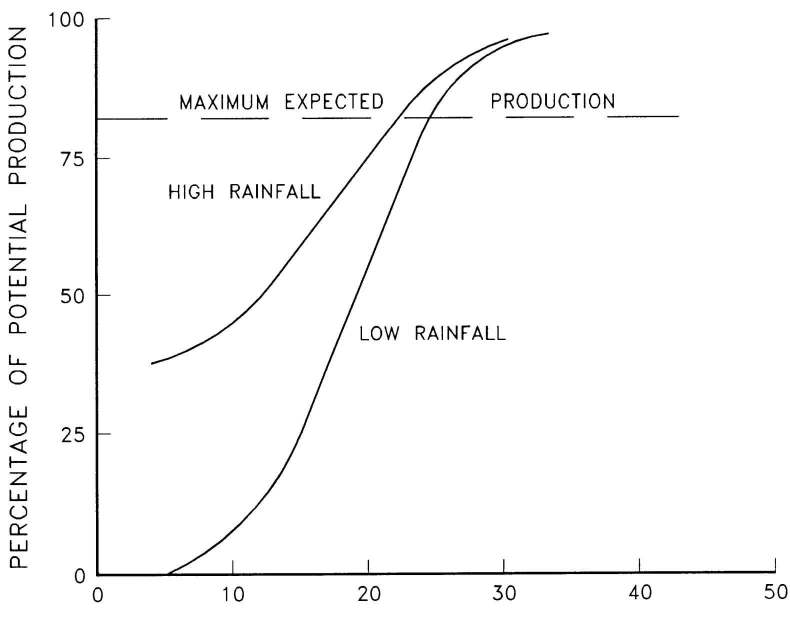


FIG. 19.1. Relative Production as a Percentage of Potential Production for Different P_w Values (Adapted from: Keller and Karmeli, 1974 (Fig. 1)).

adequate may in fact be underdesigned. This is demonstrated in Fig. 19.1 by the dotted line. For example, a system having $P_w = 25\%$ may appear to be performing as well as might be expected from current experience with other irrigation systems. However, increasing P_w to more than 33% may increase production by another 20%.

Wetted Area

The area wetted by each emitter, A_w, along a horizontal plane about 30 cm (12 in.) below the soil surface depends on the rate and volume of emitter discharge. It also depends on the texture, structure, slope, and horizontal layering of the soil.

A number of models for estimating the dimensions of the wetted soil under a point-source emitter have been developed. In one interesting model (Schwartzmass and Zur, 1985), the wetted soil volume is assumed to depend on the hydraulic conductivity of the soil, the emitter discharge, and on the total amount of water in the soil. From this model the following general empirical equations were developed to estimate the wetted depth and width:

$$z' = K_1 (V_w)^{0.63} \left(\frac{C_s}{q} \right)^{0.45} \tag{19.1}$$

and

$$w = K_2 (V_w)^{0.22} \left(\frac{C_s}{q} \right)^{-0.17} \tag{19.2a}$$

Equations 19.1 and 19.2a can be combined to yield the relationship between w and z' directly as:

$$w = K_3 (z')^{0.35} (q)^{0.33} (C_s)^{-0.33} \tag{19.2b}$$

where

z' = vertical distance to wetting front, m (ft)
w = wetted width or diameter of water pattern, m (ft)
K_1 = empirical coefficient, 29.2 for metric units (71.3 for English units)
V_w = volume of water applied, L (gal)
C_s = saturated hydraulic conductivity of the soil, m/s (ft/s)
q = point-source emitter discharge, L/hr (gph)
K_2 = empirical coefficient, 0.031 for metric units (0.206 for English units)
K_3 = empirical coefficient, 0.0094 for metric units (0.047 for English units)

It is assumed that these equations will give valid results under a wide range of conditions. They were derived using three-dimensional cylindrical flow geometry and verified results from a plane flow model. The equations should be useful for gaining insights into the sensitivity of the wetting geometry to variations in C_s, q, and V_w.

Sample Calculation 19.1 *Estimating the sensitivity of changes in C_s, q, V_w, and root depth on the width and depth of wetting from a point-source emitter.*

GIVEN: A point-source emitter with $q = 4\ L/\text{hr}$ operating in an orchard with a sandy loam soil where the root depth is 1.5 m and the saturated hydraulic conductivity, $C_s = 7 \times 10^{-6}\ \text{m/s} = 25\ \text{mm/hr}$.

FIND: Find the width, w, of the wetted soil bulb and the sensitivity of w to changes in C_s and of w and z' to changes in q.

CALCULATIONS: By Eq. 19.2b the w produced by a point-source emitter irrigating to a depth of $z' = 1.5$ m is:

$$\begin{aligned} w &= 0.0094(.15)^{0.35}(4)^{0.33}(7 \times 10^{-6})^{-0.33} \\ &= 0.86\ \text{m} \end{aligned}$$

If the soil were a medium-textured soil with a C_s only one-fourth as high, then:

$$w = 0.86(0.25)^{-0.33} = 1.36\ \text{m}$$

By Eq. 19.1 the expected effect on z' of doubling q for a given C_s and V_w would be:

$$\begin{aligned} (z')_2 &= (q_1/q_2)^{0.45}(z')_1 \\ &= (1/2)^{0.45}(z')_1 = 0.73(z')_1 \end{aligned}$$

and by Eq. 19.2a the effect on w would be:

$$(w)_2 = (1/2)^{-0.17}(w)_1 = 1.13\ (w)_1$$

Because soils vary so greatly, universal mathematical relationships like Eqs. 19.1 and 19.2 for estimating A_w are not too promising. Furthermore, to determine the necessary saturated hydraulic conductivity or other necessary data for the equations is more difficult than doing sample field tests.

Simple field tests are the most reliable way to determine A_w for trickle irrigation design. Equations 19.1 and 19.2 can then be used as guides for extending the field data. Field tests are simply operating drip emitters at a few representative sites and then measuring the resulting wetting patterns. Where a trickle system in the vicinity of the field under consideration is irrigating a similar soil, wetting patterns could be checked on it instead. In either case, discharge rate and volume of water applied should simulate the anticipated design values.

The only items of equipment needed for a field test are:

1. An 80- to 120 L (20- to 30-gal) plastic container;
2. A stand 1.2 m (4 ft) high for the container;
3. A piece of 6- or 9-m ($\frac{1}{4}$- or $\frac{3}{8}$-in.) diameter tubing about 3.0 m (10 ft) long;
4. An emitter that has a discharge rate about equal to the design q_a when operating with the container on the stand and about half full. (A turbulent flow, $x = 0.5$ emitter is better than a capillary tube, because its flow rates is less affected by changes in water level as the container drains);
5. A 100-m graduated cylinder;
6. A watch with a second hand; and
7. A shovel and soil probe or auger.

If the water level in the test ranges from 2.1 to 1.4 m (7 to 4.5 ft) above the emitter outlet, the flow rate from an orifice-type emitter rated to discharge 10 L/hr (2.5 gph) at 105 kPa (15 psi) will range from 4.2 to 3.4 L/hr (1.1 to 0.9 gph). For the same variation in water level, the flow rate from a capillary tube ranges from 4.5 to 3.0 L/hr (1.2 to 0.8 gph), which would still be acceptable. Before tests are run, the emitter should be carefully attached to the tubing and its discharge rate calibrated.

A volume of water equal to the estimated daily discharge requirement per emitter should be applied through the test emitter. This is best done by putting the desired volume in the tank and letting it run dry. If the soil is very dry, identical applications should be made on two or three successive days before checking the wetting pattern. The wetting pattern can be observed best on the vertical wall of a trench dug through the emission point location down to the bottom of the wetted zone. However, a trench dug about 30 to 45 cm (12 to 18 in.) deep to observe the wetted diameter, followed by probing to determine the depth of wetting, may be sufficient.

Figure 19.2 (Roth, 1974) shows wetting profiles from applying equal 45-L (12-gal) volumes of water with drip emitter discharge rates of 4, 8, and 16 L/hr (1, 2, and 4 gph). The soil was a deep medium- to fine-textured desert sand with a uniformly textured profile and was dry prior to the wetting pattern experiments.

It is interesting to note that the vertical and horizontal wetting patterns are similar for the three discharge rates. This does not agree with the results predicted by Eqs. 19.1 and 19.2. The 4-L/hr (1-gph) emitter produced a slightly larger wetted area than the emitters with higher discharge rates; this is unusual. The 16-L/hr (4-gph) emitters did not cause ponding, and the 4-L/hr (1-gph) emitter provided more time for horizontal water movement.

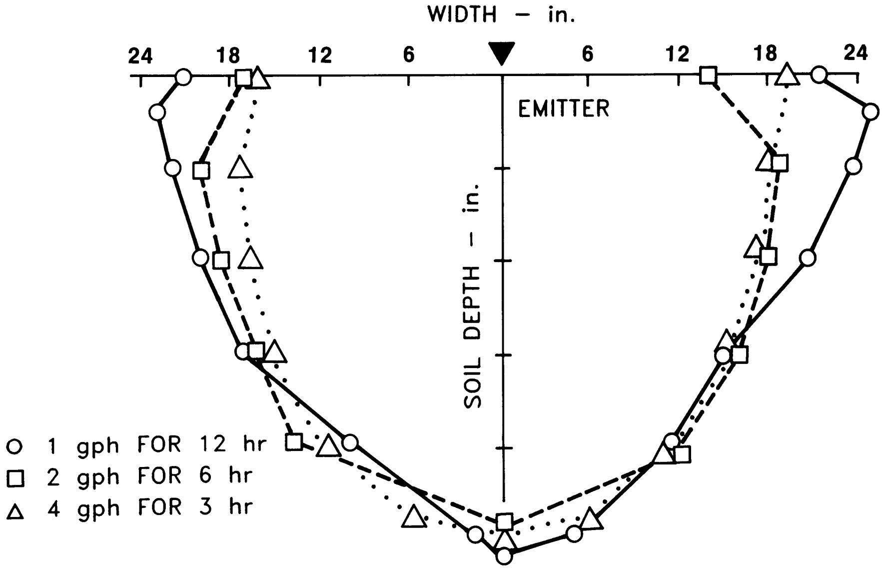


FIG. 19.2. Wetting Profiles for Equal Volumes (12 gal) of Water Applied at Three Different Application Rates on a Dry Sandy Soil (Data from: Roth, 1974)).

For repeated wettings, as would occur in an irrigated field, the A_w probably would be larger for the emitters with higher discharge rates. But, the field test results demonstrate the lack of reliability of Eqs. 19.1 and 19.2, which is also typical of other equations used for predicting w and z'.

Figure 19.3 shows the relationship between maximum horizontal and vertical movement for different depths of water applied to the uniform sandy soil. The data points represent different emitter discharge rates. They further demonstrate that the rate of application affected the wetting pattern very little. The volume of soil wetted was a direct function of the amount of water applied and relatively independent of the application rate on this uniform soil. Figure 19.3 also demonstrates that excessive applications could easily extend the vertical movement below the root zone. Light frequent (daily) applications tend to minimize losses from deep percolation, but they produce a smaller A_w.

Estimating Area Wetted

Table 19.1 gives estimates of the areas wetted, A_w, by a standard 4-L/hr (1-gph) emitter for different soil conditions and depths. The area of soil surface wetted by a drip emitter usually is less than half as large as A_w measured at a depth of 15 to 30 cm (6 to 12 in.) unless the rate of application is high enough to cause surface ponding. The standard of 4 L/hr (1 gph) approximates the most common average emitter discharge rate, q_a.

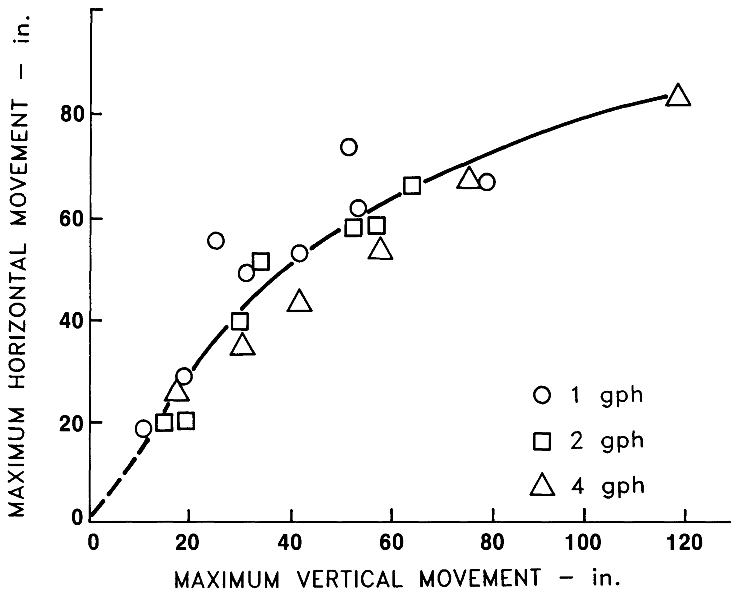


FIG. 19.3. Relations Between Vertical and Horizontal Water Movements for Different Depths of Application and Emitter Discharge Rates to the Dry Sandy Soil (Data from: Roth, 1974)).

Table 19.1. Estimated areas wetted, A_w, by a 4-L/hr, (1-gph) drip emitter operating under various field conditions

Soil or root depth and soil texture[1]	Degree of soil stratification[2] and equivalent wetted soil area,[3] m × m (ft × ft)		
	Homogeneous	Stratified	Layered[4]
Depth 0.75 m (2.5 ft)			
Coarse	0.4 × 0.5	0.6 × 0.8	0.9 × 1.1
	(1.2 × 1.5)	(2.0 × 2.5)	(2.8 × 3.5)
Medium	0.7 × 0.9	1.0 × 1.2	1.2 × 1.5
	(2.4 × 3.0)	(3.2 × 4.0)	(4.0 × 5.0)
Fine	0.9 × 1.1	1.2 × 1.5	1.5 × 1.8
	(2.8 × 3.5)	(4.0 × 5.0)	(4.8 × 6.0)
Depth 1.5 m (5.0 ft)			
Coarse	0.6 × 0.8	1.1 × 1.4	1.4 × 1.8
	(2.0 × 2.5)	(3.6 × 4.5)	(4.8 × 6.0)
Medium	1.0 × 1.2	1.7 × 2.1	2.2 × 2.7
	(3.2 × 4.0)	(5.6 × 7.0)	(7.2 × 9.0)
Fine	1.2 × 1.5	1.6 × 2.0	2.0 × 2.4
	(4.0 × 5.0)	(5.2 × 6.5)	(6.4 × 8.0)

[1] *Coarse* includes coarse to medium sands; *medium* includes loamy sands to loams; *fine* includes sandy clay to loam to clays (if clays are cracked, treat like coarse to medium soils).

[2] Almost all soils are stratified or layered. *Stratified* refers to relatively uniform texture, but having some particle orientation or some compaction layering that gives higher horizontal than vertical permeability. *Layered* refers to changes in texture with depth, as well as particle orientation and moderate compaction.

[3] The equivalent wetted rectangular area dimensions, S_e' and w, are 0.8 times the wetted diameter and the wetted diameter, respectively.

[4] For soils that have extreme layering and compaction that causes extensive stratification, the S_e' and w may be as much as twice as large.

The A_w values in Table 19.1 are given for various soil textures, depths, and degrees of stratification. They are based on daily or every-other-day irrigations that apply volumes of water sufficient to slightly exceed the crop's water-use rate. The area wetted is given as a rectangle. The long dimension, w, is equal to the maximum expected diameter of the soil bulb wetted by an emitter. The short dimension, S_e', is 80% of the maximum expected wetted diameter. This is the emitter spacing that would give a reasonably uniform and continuous wetted strip. Multiplying the two values together gives approximately the same area as the circular wetted area, i.e., $1.0 \times 0.8 \approx 1.0\ \pi/4$.

Almost all soils are either stratified or layered to some extent. However, assuming stratification or layering and that A_w will be large (without making the field tests described earlier) is risky. If horizontal stratification, layering, and compaction are extreme, the A_w may be twice as large as the values given for a layered soil in Table 19.1. But this must be determined by actual field checks.

Table 19.1 should be used only for estimating purposes. Values for wetted areas greater than those given for stratified conditions should be used with caution until they have been verified by field tests.

On sloping fields the wetted pattern may be distorted in the downslope direction. On steep fields this distortion can be extreme; as much as 90% of the pattern may be on the downslope side. Emitters should be positioned with this distortion in mind, but the actual A_w will be similar to that on flat ground.

In general, A_w increases with the number of emitters per plant, application time, and average emitter flow rate, q_a. Therefore, decreasing the frequency of irrigation usually produces a larger wetted area. However, low-frequency systems are more expensive to install than single-station systems designed for almost continuous operation. Increasing q_a increases A_w on low-infiltration-rate soils where a surface pool forms or where there are low-permeability restrictive layers in the profile.

Single- and dual-chamber line-source tubing usually has outlets spaced at 15-, 20-, 30-, 45-, or 60-cm (6-, 8-, 12-, 15-, 18-, or 24-in.) intervals and is generally used on relatively shallow-rooted crops. The typical range of available flow rates per outlet is between 0.5 and 2.0 L/hr (0.13 and 0.5 gph). When the spacing between outlets is S_e' or closer, the average wetted width approaches the w values given in Table 19.1.

Spray emitters wet a larger surface area of soil than drip emitters. They are often used on coarse-textured, homogeneous soils where wetting a sufficiently large area would require a large number of drip emitters. Figure 19.4 shows a comparison between patterns and areas wetted under drip and spray emitters. Water moves laterally out from the wetted surface area under a spray emitter. A reasonable estimate of the lateral subsurface water movement is to use one-half of the S_e' values for a homogeneous soil shown in Table 19.1. A homogeneous soil should be assumed because the application rate around the fringes of the surface area wetted is normally very low. Therefore, the lateral movement will not be very great.

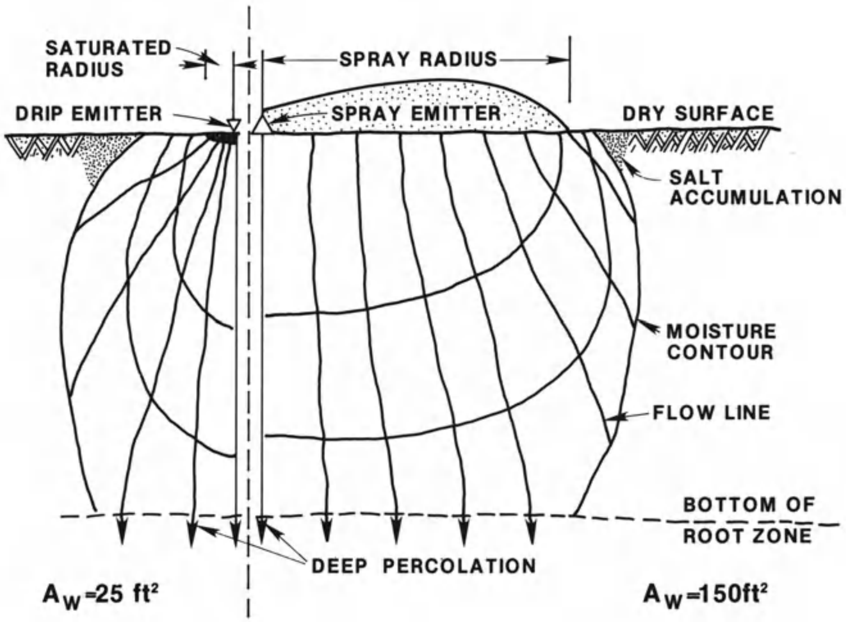


FIG. 19.4. Comparison of Idealized Wetting Profiles in a Homogeneous Fine Sandy Soil under a Drip and a Spray Emitter.

Computing Percentage Wetted Area

The percentage area wetted, P_w, is the average horizontal area wetted in the top 15 to 30 cm (6 to 12 in.) of the crop root zone as a percentage of the total crop area. Additional terms and information to describe the emitter layout and wetting patterns depicted in Fig. 17.5 are:

Emitter spacing, S_e, is the spacing between emitters or emission points along a lateral line, m (ft);
Optimal emitter spacing, S_e', is the drip emitter spacing, which is 80% of the wetted diameter estimated from field tests or from Table 19.1, m (ft);
Wetted width, w, is the width of the strip that would be wetted by a row of emitters spaced at S_e' (or closer) along a single lateral line. The w is also equal to the diameter of the circular area wetted by a single emitter. It can be estimated from field tests or from Table 19.1, m (ft);
Lateral spacing, S_l, is the spacing between trickle irrigation laterals, m (ft);
Plant spacing, S_p, is the distance between plants in the row, m (ft);
Row spacing, S_r, is the distance between plant rows, m (ft); and
Number of emission points per plant, N_p.

For *straight single-lateral systems* with $S_e \le S'_e$, the percentage wetted can be computed as:

$$P_w = \frac{N_p S_e w}{S_p \times S_r} 100 \tag{19.3}$$

where P_w = percentage of soil area wetted along a horizontal plane 30 cm (12 in.) below the soil surface, %, N_p = number of emitters per tree, and $S_p \times S_r$ = tree spacing, m × m (ft × ft). If $S_e > S'_e$, then S_e in Eq. 19.3 must be replaced by S'_e.

For *double-lateral systems* (see Fig. 17.5), the two laterals should be placed S'_e apart. This is done to maximize the wetted area without leaving extensive dry areas between the lines. For *zigzag, pigtail, and multiexit layouts*, the emitters or emission points should be spaced to maximize the wetted area per outlet. To do this without leaving extensive dry spots within the wetted area, the emission point should be placed S'_e apart in each direction, as shown in Fig. 17.5. The estimated P_w for the optimum spacing is:

$$P_w = \frac{N_p S'_e (S'_e + w)/2}{S_p \times S_r} 100 \tag{19.4}$$

If the layout is not designed for maximum wetting, and $S_e < S'_e$, then S'_e in Eq. 19.4 should be replaced by the actual S_e values used.

For *spray emitters* the surface area wetted should first be estimated. This can be done easily by observing a few sprayers operating at the design pressure and flow rate. To quickly check the wetting pattern shape, size, and uniformity, let the test sprayers wet a smooth, dry blacktop surface. As the water darkens the blacktop, the uniformity of coverage and area wetted can easily be observed.

To estimate A_w for *spray emitters* add one-half the S'_e value taken from Table 19.1 for a homogeneous soil to the perimeter of the wetted surface area. Thus, P_w is approximately equal to:

$$P_w = \frac{N_p [A_s + (S'_e \times PS)/2]}{S_p \times S_r} \times 100 \tag{19.5}$$

where A_s = soil surface area directly wetted by the sprayer, m^2 (ft^2), and PS = the perimeter of the area directly wetted by the sprayers, m (ft).

Sample Calculation 19.2. *Computing and comparing percentage wetted areas for different emitter configurations.*

GIVEN: An orchard with a tree spacing of S_p = 3.0 m and S_r = 5.0 m planted on a deep, medium-textured homogeneous soil. Three emitter configurations are being considered:

1. A single row of 4-L/hr emitters;
2. A zigzag layout like in Fig. 17.5 C with four 4-L/hr emitters per tree; and
3. A small sprayer that directly wets a surface area with a radius of 1.0 m.

FIND: The percentage area wetted by each emitter configuration, and recommend which to select.

CALCULATIONS: From Table 19.1, for the deep-rooted crop on a medium textured homogeneous soil, $S'_e = 1.0$ m and $w = 1.2$ m. Therefore, the spacing between emitters should be $S_e = 1.0$ m, which would give N_p = three emitters per tree. By Eq. 19.3 the percentage area wetted by a single row of emitters for each tree row would be:

$$P_w = \frac{3 \times 1.0 \times 1.2}{3.0 \times 5.0} \times 100 = 24\%$$

For the zigzag configuration $N_p = 4$ and by Eq. 19.4:

$$P_w = \frac{4 \times 1.0(1.0 + 1.2)/2}{3.0 \times 5.0} \times 100 = 29\%$$

A spray emitter that wets a circular area with a 1.0-m radius would directly wet a surface area of $A_s = 3.14$ m with a perimeter of $PS = 6.28$ m. Therefore by Eq. 19.5:

$$P_w = \frac{1[3.14 + (1.0 \times 6.28)/2]}{3.0 \times 5.0} \times 100 = 42\%$$

The spray emitter is the only one of the three configurations considered that will meet the criteria of $P_w > 33\%$. Therefore, it should be selected for the system. Either a double-lateral system or multiexit 6-outlet emitters (Figs. 17.5 B and E) would also give roughly the same P_w as the spray emitter.

SALINITY CONTROL

All irrigation water contains some dissolved salts that are concentrated and pushed toward the fringes of the wetted soil mass during the irrigation season. By applying more water than the plants consume, most of the salts can be pushed or leached below the root zone. But it is impossible to avoid having some areas with salt accumulation.

The most critical zones of accumulation are along the fringes of the wetted surface (see Fig. 19.4). A light rain can leach these accumulated salts down into the zone of extensive root activity and thereby severely injure the plants. To minimize this hazard the trickle system should be operated during and after rainy periods to wash the salts down and out of the soil profile.

Salts may accumulate where rainfall is less than 150 to 250 mm (6 to 10 in.) per year. Therefore, supplemental applications of sprinkle or surface irrigation may be necessary to prevent critical levels of salt buildup. This is especially important where irrigation water is saline. It is also helpful for annual crops where a new planting may be made in the salty fringe areas of the previous year's wetted patterns.

Crop Tolerance and Yield

Trickle irrigation affords a convenient and efficient method of frequent irrigation that does not wet the plant leaves. Applying frequent light irrigation holds the salt concentration in the soil water to a minimum. Daily applications and sufficient leaching keep the salt concentrations in the soil water at almost the same level as in the irrigation water. This is because there is little drying between irrigations, and the salts in the soil remain well diluted. However, when irrigations are infrequent, the salts become more concentrated as the soil dries.

Crop yields should equal or slightly exceed those produced under other methods of irrigation with good-quality irrigation water. However, when water is of poor quality, yields under trickle irrigation are usually considerably higher than under other methods, but not as high as from good-quality water. This yield advantage results because the salts remain diluted by the continuous high soil moisture resulting from frequent replenishment of the water lost by evapotranspiration. Frequent sprinkle irrigation applications might give similar results, but leaf burn would be a problem for many crops if the water were saline.

Knowing the *electrical conductivity of the irrigation water*, EC_w, and the *electrical conductivity of the saturated soil extract*, EC_e is necessary for salinity management. A useful parameter for estimating leaching requirements under trickle irrigation is the *maximum* EC_e. This is the theoretical level of salinity that would reduce yield to zero; i.e., if the entire root zone were at max EC_e, the plants would not extract water and growth would stop. Table 19.2 gives max EC_e values for various crops.[1] These values were extrapolated from test data that gave 0, 10, 25, and 50% reductions in yield.

The *minimum* EC_e is a useful parameter for estimating the effect of trickle irrigation, if $EC_w \leq \min EC_e$, there will be essentially no reduction in yield.

Relative Yield. The relative yield of various crops resulting from high-frequency trickle irrigation with water of various qualities and $LR_t \geq 0.1$ can be estimated using the following criteria. If the quality of the irrigation water is relatively high:

$$Y_r = 1.0; \qquad \text{for } EC_w \leq \min EC_e \tag{19.6a}$$

[1]Table 19.2 and Eqs. 19.6, and 19.7, were adopted from Ayers and Westcott (1985).

Table 19.2. Minimum and maximum values of EC_e for various crops

Crop	EC_e, dS/m Min[1]	EC_e, dS/m Max[2]	Crop	EC_e, dS/m Min[1]	EC_e, dS/m Max[2]
Field crops					
Cotton	7.7	27	Corn	1.7	10
Sugar beets	7.0	24	Flax	1.7	10
Sorghum	6.8	13	Broadbeans	1.5	12
Soybean	5.0	10	Cowpeas	1.3	8.5
Sugarcane	1.7	19	Beans	1.0	6.5
Fruit and nut crops					
Date palm	4.0	32	Apricot	1.6	6
Fig, olive	2.7	14	Grape	1.5	12
Pomegranate	2.7	14	Almond	1.5	7
Grapefruit	1.8	8	Plum	1.5	7
Orange	1.7	8	Blackberry	1.5	6
Lemon	1.7	8	Boysenberry	1.5	6
Apple, pear	1.7	8	Avocado	1.3	6
Walnut	1.7	8	Raspberry	1.0	5.5
Peach	1.7	6.5	Strawberry	1.0	4
Vegetable crops					
Zucchini squash	4.7	15	Sweet corn	1.7	10
Beets	4.0	15	Sweet potato	1.5	10.5
Broccoli	2.8	13.5	Pepper	1.5	8.5
Tomato	2.5	12.5	Lettuce	1.3	8
Cucumber	2.5	10	Radish	1.2	9
Cantaloupe	2.2	16	Onion	1.2	7.5
Spinach	2.0	15	Carrot	1.0	8
Cabbage	1.8	12	Beans	1.0	6.5
Potato	1.7	10	Turnip	0.9	12

Adapted from Ayers and Wescott (1985).
[1]Minimum EC_e does not reduce yield.
[2]Maximum EC_e reduces yield to zero.

If the quality of the irrigation water is such that EC_w is between the min EC_e and (max EC_e + min EC_e)/2, then:

$$Y_r \simeq \frac{\max EC_e - EC_w}{\max EC_e - \min EC_e} \tag{19.6b}$$

where

Y_r = relative yield, which is the ratio of the estimated reduced yield with saline water to full potential under trickle irrigation, decimal

EC_w = electrical conductivity of the irrigation water, dS/m (mmhos/cm)
min EC_e = electrical conductivity of the saturated soil extract that will not decrease crop yield, dS/m (mmhos/cm)
max EC_e = electrical conductivity of the saturated soil extract that will reduce yield to zero, dS/m (mmhos/cm)

Leaching Requirement (LR_t)

In arid regions where salinity is of major importance, most natural precipitation is accounted for in beneficial and nonbeneficial consumptive use and runoff. There is usually very little drainage water (deep percolation) produced due to excess natural precipitation, D_p, that can help satisfy leaching requirements. Furthermore, only a portion of the soil area is wetted and needs leaching under trickle irrigation. Thus, the effective excess precipitation is reduced to $(P_w/100)D_p$ and can often be neglected.

The leaching requirement is the ratio of the net depth of leaching water to the net depth of irrigation water that must be applied for consumptive use and leaching. Calculating the leaching requirement for trickled irrigation is greatly simplified by neglecting $(P_w/100)D_p$:

$$LR_t = \frac{L_n}{(d_n + L_n)} = \frac{L_N}{(D_n + L_N)} = \frac{EC_w}{EC_{dw}} \tag{19.7}$$

where

LR_t = leaching requirement ratio under trickle irrigation
d_n = net depth of application per irrigation to meet consumptive use requirements, mm (in.)
D_n = net annual or seasonal irrigation depth to meet consumptive use requirements, mm (in.)
L_n = net leaching requirement for each irrigation, mm (in.)
L_N = net annual leaching requirement, mm (in.)
EC_w = electrical conductivity of the irrigation water, dS/m (mmhos/cm)
EC_{dw} = electrical conductivity of the drainage (deep percolation) water, dS/m (mmhos/cm)

Equation 19.7 is based on a steady-state salt balance. It is important to understand the meaning of the number calculated for LR_t. It represents the minimum amount of water (in terms of a fraction of the applied water) that must pass through the root zone to prevent salt buildup. The actual L_n or L_N, however, is that amount of leaching water necessary to prevent salts from building up in the root zone. This must be determined by monitoring soil salinity, which is then related to field water management.

Salts that accumulate below the emitters can be almost continuously flushed down by properly applied daily or alternate-day irrigations. When $LR_t > 0.1$, daily or alternate-day irrigations should be used that include enough excess water to keep the salts from concentrating in the plant root zone.

To obtain the yield expectation predicted by Eq. 19.6 from high-frequency (daily or alternate-day) trickle irrigation, EC_{dw} can equal $2(\max EC_e)$. Substituting into Eq. 19.7 gives:

$$LR_t = \frac{EC_w}{2(\max EC_e)} \tag{19.8}$$

where $\max EC_e$ can be determined from Table 19.2, and $EC_w \leq (\max EC_e + \min EC_e)/2$.

Once d_n or D_n is determined, the total net depth of water to satisfy both consumptive use and leaching requirements can be computed by $d_n/(1.0 - LR_t)$ or $D_n/(1.0 - LR_t)$. Where $(P_w/100)D_p$ is appreciable, the net seasonal water requirement for leaching can be reduced accordingly.

Inefficiencies in water application often result in sufficient extra water to accomplish the necessary leaching. This is especially true where leaching requirements are low, as they are when water quality is good, where soils are shallow and sandy, or where the irrigation is in a semihumid or humid area.

The calculated LR_t should be adequate to control salts unless they are already present in excess of the crop's tolerance. If they are, reclamation is required. This should be done with heavy leaching using sprinkle or surface irrigation techniques. Furthermore, special reclamation practices may be required where sodium or other salts have caused soil structural or toxicity problems.

Sample Calculation 19.3. *Estimating yield reduction and leaching requirements.*

GIVEN: A mature trickle-irrigated apricot orchard in a low desert area where:

Irrigation water quality, $EC_W = 3.0$ ds/m, and
Net annual consumptive use deficit, $D_n = 762$ mm.

FIND: The net annual leaching requirement and the effect on yield due to salinity in the irrigation water.

CALCULATIONS: From Table 19.2 the min and max EC_e values are 1.6 and 6 ds/m, respectively. By Eq. 19.8 the leaching requirement is:

$$LR_t = \frac{3.0}{2(6)} = 0.25$$

and by rearranging Eq. 19.7, the net annual leaching requirement is:

$$L_N = \frac{LR_t D_n}{1 - LR_t} = \frac{0.25 \times 762}{1 - 0.25} = 254 \text{ mm}$$

The relative compared to full potential yield resulting from the salinity in the irrigation water can be estimated by Eq. 19.6b as follows:

$$Y_r \simeq \frac{6.0 - 3.0}{6.0 - 1.6} = 0.68$$

NET WATER REQUIREMENTS

The plant canopies of young and wide-spaced crops shade only a portion of the soil surface area and intercept only a portion of the incoming radiation. Conventional estimates of water requirements of young crops assume part of the applied water will be lost to nonbeneficial consumptive use. This loss is through evaporation from the wetted soil surface or through transpiration from undesirable vegetation.

Daily Use Rate

Trickle irrigation reduces evaporation losses to a minimum, so transpiration by the crop accounts for practically all of the water consumed. Therefore, estimates of consumptive use that assume wetting the entire field surface should be modified for trickle irrigation.

The transpiration rate under trickle irrigation is a function of the conventionally computed consumptive use rate and the extent of the plant canopy (see Sharples et al., 1985). A simple equation for estimating the average peak daily transpiration rate is:

$$T_d = U_d\left[0.1(P_d)^{0.5}\right] \tag{19.9}$$

where T_d = average daily transpiration rate during the peak-use month for a crop under trickle irrigation, mm/day (in./day), U_d = conventionally estimated average daily consumptive use rate during the peak-use month for the mature crop with a full canopy, mm/day (in./day), and P_d = percentage of soil surface area shaded by crop canopies at midday (solar noon), %.

Equation 19.9 is based on the observation that even when the plant canopy is very small and P_d is 1% or greater, the minimum $T_d > (0.1\,U_d)$. This is because there is an oasis effect, and some additional vegetation usually grows in the area wetted by the emitters. Furthermore, as the canopies of the plants increase toward full coverage, T_d approaches U_d, and at full coverage when P_d = 100%, $T_d = U_d$.

The P_d can be estimated by inspection. Simply mark off the area allocated to a tree or plant and observe the percentage of the area that is directly under the plant canopy. (Part A of Fig. 17.5 depicts a tree canopy with a $P_d \approx 55\%$.) A mature orchard usually has a maximum $P_d \approx 100\,\pi/4 \approx 80\%$.

Seasonal Water Use

The seasonal transpiration can be computed by replacing U_d with the total estimated seasonal consumptive use, in Eq. 19.9 to obtain:

$$T_s = U\left[0.1\left(P_d\right)^{0.5}\right] \tag{19.10}$$

where T_s = seasonal transpiration under trickle irrigation, mm (in.), and U = conventionally estimated seasonal consumptive use for the mature crop with a full canopy, mm (in.).

Seasonal Water Deficit

The *net seasonal irrigation depth*, D_n, needed to meet seasonal transpiration requirements is an important design parameter for estimating annual irrigation requirements. The D_n is reduced by the residual stored soil water from off-season precipitation plus effective rain during the growing season. Because natural precipitation wets all the area, it must be subtracted from the conventionally estimated consumptive use, U, rather than the seasonal transpiration, T_s. Adjusting Eq. 19.10 accordingly gives:

$$D_n = \left(U - R_n - M_s\right)\left[0.1\left(P_d\right)^{0.5}\right] \tag{19.11}$$

where R_n = effective rain during the growing season, mm (in.), and M_s = residual stored soil moisture from off-season precipitation mm (in.).

Net Depth per Irrigation

Normally, trickle irrigation wets only part of the soil area. Therefore, the equations for determining the desirable depth or volume of application per irrigation cycle and the maximum irrigation interval must be adjusted accordingly. The soil moisture deficit at which irrigation should be started depends on the soil, the crop, and water-yield-economic factors.

The *maximum net depth per irrigation*, d_x, is the depth of water that will replace the soil moisture deficit when it is equal to MAD (see Table 3.4). The d_x is computed as a depth over the whole crop area not just the wetted area; however, the percentage area wetted, P_w, must be taken into account. Thus, for trickle irrigation Eq. 3.1 must be modified to:

$$d_x = \frac{\text{MAD}}{100} \frac{P_w}{100} W_a Z \tag{19.12}$$

where

d_x = maximum net depth of water to be applied per irrigation, mm (in.)
MAD = management allowed deficit, %
W_a = available water-holding capacity of the soil, mm/m (in./ft)
Z = plant root depth, m (ft)

The *net depth to be applied per irrigation*, d_n, to meet consumptive use requirements can be computed by modifying Eq. 3.2 to:

$$d_n = T_d f' \tag{19.13a}$$

and

$$f_x = \frac{d_x}{T_d} \tag{19.13b}$$

where

d_n = net depth of water to be applied per irrigation to meet consumptive use requirements, mm (in.)
f' = irrigation interval or frequency, days
f_x = maximum irrigation interval, days
T_d = average daily transpiration during peak-use period, mm (in.)

For design purposes, the T_d for the mature crop (maximum expected P_d) should be used for sizing the pipe network. Furthermore, assuming $f' = 1$ day, so that $d_n = T_d$, simplifies the design process. The actual irrigation frequency to be used is a management decision. But f' should be chosen so that $d_n \leq d_x$ computed by Eq. 19.12.

GROSS IRRIGATION REQUIREMENTS

Gross irrigation depth and volume requirements for trickle systems are based on net requirements and efficiencies.

Efficiencies

The *seasonal irrigation efficiency*, E_s, is primarily a function of application uniformity, but it also depends on: minor *losses* due to runoff, leaks, filter and line flushing, and drainage; *unavoidable losses* to deep percolation due to the soil wetting pattern and untimely rainfall; and *avoidable losses* resulting from poor scheduling. E_s represents the percentage of the gross water applied that is beneficially utilized to meet crop consumptive use and leaching requirements. It is most useful when a specified adequacy of irrigation is implied.

The *application efficiency of the low quarter*, E_q, is a useful concept for dealing with E_s. The concept and measuring of E_q was developed in Chapter 6 for sprinkle irrigation. For trickle irrigation it is computed from the average low quarter volume of irrigation water per unit area infiltrated and stored in the root zone or required for leaching. This is then divided by the gross volume of system discharge per unit area.

The average low quarter volume applied is the average of the lowest one-fourth of the measured or estimated emitter discharge values where each emitter supplies an equal area of the field (see Eq. 17.2). Therefore, E_q is equal to the field EU′ when this low quarter volume is equal to or less than the soil moisture deficit plus leaching requirements and minor losses are negligible. Furthermore, $E_s = E_q$ where potential average seasonal losses due to unavoidable deep percolation and irrigation scheduling problems are less than leaching requirements.

The *peak-use-period transmission ratio*, T_r, is the depth of irrigation water transmitted to exactly satisfy T_d divided by the depth of water actually transpired, T_d. It represents the extra water that must be applied even during the peak-use period to offset unavoidable percolation beyond the root zone. This deep percolation is due to excess vertical movement of water below the active root zone. It is unavoidable in porous and shallow soils when sufficient lateral wetting is achieved. Peak use period T_r values for design and efficient scheduling of irrigation systems are presented in Table 19.3.

Systems should be designed to have only negligible minor losses, and irrigation scheduling should be carefully planned to eliminate avoidable losses. The unavoidable seasonal deep percolation is represented by the seasonal transmission ratio, T_R values given in Table 19.4 for different site conditions.

The concept of T_R is similar to that of T_r. But it represents the minimum

Table 19.3. Peak period transmission ratios, T_r, for different soil textures and rooting depths[1]

	Soil texture			
Crops' root depth	Very coarse	Coarse	Medium	Fine
Shallow				
<0.8 m (2.5 ft)	1.10	1.10	1.05	1.00
Medium				
0.8 to 1.5 m (2.5 to 5 ft)	1.10	1.05	1.00	1.00
Deep				
>1.5 m (5 ft)	1.05	1.00	1.00	1.00

[1]Peak-period transmission ratios, T_r, are for drip emitters. For spray emitters add 0.05 to T_r in humid climates and 0.10 in arid climates to allow for the extra evaporation.

Table 19.4. Seasonal transmission ratios, T_R, for arid and humid regions with different soil textures and rooting depths[1]

Climate zone and root depth	Soil texture Very coarse	Coarse	Medium	Fine
Arid				
<0.8 m (2.5 ft)	1.15	1.10	1.05	1.05
0.8 to 1.5 m (2.5 to 5.0 ft)	1.10	1.10	1.05	1.05
>1.5 m (5.0 ft)	1.05	1.05	1.00	1.00
Humid				
<0.8 m (2.5 ft)	1.35	1.25	1.15	1.10
0.8 to 1.5 m (2.5 to 5.0 ft)	1.25	1.20	1.10	1.05
>1.5 m (5.0 ft)	1.20	1.10	1.05	1.00

[1]Seasonal transmission ratios, T_R are for drip emitters. For spray emitters add 0.05 to T_R in humid climates and 0.10 in arid climates to allow for the extra evaporation.

excess water that must be applied to offset the unavoidable deep percolation on a seasonal basis. This can be due to untimely rains, leakage from the soil profile while sufficient horizontal water movement is being obtained, or both. The T_R values given in Table 19.4 assume good system design and scheduling. The higher T_R values shown for humid areas are included to account for the scheduling difficulties due to rainfall.

Both potential unavoidable and avoidable losses can be used to offset leaching requirements. In the following equations, avoidable scheduling losses are assumed to be zero. Therefore, when unavoidable, seasonal deep percolation is less than or equal to the leaching requirement, i.e., $T_R \leq 1.0/(1.0 - LR_t)$, then:

$$E_s = \text{EU} \tag{19.14a}$$

When unavoidable deep percolation is greater than the leaching requirement, then the excess is an unavoidable loss. Therefore, when $T_R > 1.0/(1.0 - LR_t)$, the seasonal irrigation efficiency can be approximated by:

$$E_s \approx \frac{\text{EU}}{T_R(1.0 - LR_t)} \tag{19.14b}$$

where E_s = seasonal irrigation efficiency, %, and T_R = seasonal transmission ratio. The EU can be based on either a field evaluation (see Eq. 17.2) or an estimated value for design purposes, as discussed in Chapter 20 (see Eq. 20.13.)

Gross Depths and Volumes

The *gross depth per irrigation*, *d*, should include sufficient water to allow for unavoidable deep percolation. However, unavoidable deep percolation that satisfies leaching requirements is not considered a loss. To minimize avoidable losses, systems should be well designed, accurately scheduled, and carefully maintained. To compute d, Eq. 5.3 must be modified for trickle irrigation to account for the peak-use-period transmission ratio, T_r. Also EU can be used in place of E_a. Where $LR_t \leq 0.1$ or the unavoidable deep percolation is greater than the adjusted leaching water required, $T_r \geq 0.9/(1.0 - LR_t)$:

$$d = \frac{d_n T_r}{\mathrm{EU}/100} \tag{19.15a}$$

or

$$d' = \frac{T_d T_r}{\mathrm{EU}/100} \tag{19.15b}$$

and where $LR_t > 0.1$ or $T_r < 0.9/(1.0 - LR_t)$:

$$d = \frac{100\, d_n}{\mathrm{EU}(1.0 - LR_t)} \tag{19.15c}$$

or

$$d' = \frac{100\, T_d}{\mathrm{EU}(1.0 - LR_t)} \tag{19.15d}$$

where

d = gross depth of application per irrigation, mm (in.)
d' = maximum gross daily irrigation requirement, mm (in.)
T_r = peak-use-period transmission ratio
EU = emission uniformity, %
LR_t = leaching requirement under trickle irrigation

The EU in all versions of Eq. 19.15 can be the field test emission uniformity, EU′, as defined by Eq. 17.2 for existing systems. It can also be the design emission uniformity, EU, for systems in the planning stages, which was mentioned previously and is fully presented in Chapter 20.

The *gross volume of water required per plant per day*, G, is a useful design parameter for selecting emitter discharge rates:

$$G = K \frac{d}{f'} S_p S_r \tag{19.16a}$$

or

$$G = K\, d' S_p S_r \tag{19.16b}$$

where G = gross volume of water required per plant or unit length of row per day, L/day (gal/day), and K = conversion constant, which is 1.0 for metric units (0.623 for English units).

The *gross seasonal depth*, D_g, of irrigation water required is:

$$D_g = \frac{100\, D_n}{E_s(1.0 - LR_t)} \tag{19.17}$$

where D_g = gross seasonal depth of irrigation required to satisfy uniformity, leaching, and unavoidable losses, mm (in.), and E_s = seasonal irrigation efficiency (calculated by Eq. 19.14a or 19.14b), %. Combining Eqs. 19.14b and 19.17 for the condition when: $T_R > 1.0/(1.0 - LR_t)$ gives:

$$D_g = \frac{D_n T_R}{\mathrm{EU}/100} \tag{19.18a}$$

and combining Eqs. 19.14a and 19.17 for $T_R \leq 1.0/(1.0 - LR_t)$ gives:

$$D_g = \frac{100\, D_n}{\mathrm{EU}(1.0 - LR_t)} \tag{19.18b}$$

The *gross seasonal volume of irrigation water required*, V_s, can now be computed by:

$$V_s = \frac{D_g A}{K} \tag{19.19}$$

where V_s = gross seasonal volume of irrigation water required, ha-m (A-ft), A = area irrigated, ha (A), and K = conversion constant, which is 1000 for metric units (12 for English units).

PLANNING FACTORS

Sample Calculations to illustrate the use of Eqs. 19.9 through 19.19 developed in the sections on Net Water Requirements and Gross Irrigation Requirements are presented in Chapter 21. This has been done to save space and time, because the individual computations are simple and straightforward. However, before proceeding to these computations the important issues of emitter selection and related design criteria are covered in Chapter 20.

Design Data Form

The data that must be collected prior to beginning the design computations are summarized in the "Trickle Irrigation Design Data" form presented as Fig. 19.5. This form was developed as a guide to organize the gathering of necessary field and equipment data.

I. PROJECT				
II. WATER AND LAND				
(a) Field number				
(b) Field area – ha (A)	A			
(c) Effective rain – mm (in.)	R_n			
(d) Residual soil water – mm (in.)	M_s			
(e) Water supply – L/s (gpm)				
(f) Water storage – ha-m (A-ft)				
(g) Water quality – dS/m (mmhos/cm)	EC_w & SAR			
(h) Water quality classification				
III. SOIL AND CROP				
(a) Soil texture				
(b) Available water capacity – mm/m (in./ft)	W_a			
(c) Soil depth – m (ft)				
(d) Soil limitations				
(e) Management allowed deficiency – %	MAD			
(f) Crop				
(g) Plant spacing – m x m (ft x ft)	$S_p \times S_r$			
(h) Plant root depth – m (ft)	Z			
(i) Percent shaded area – %	P_d			
(j) Average peak ET – mm/day (in./day)	U_d			
(k) Seasonal water requirement – mm (in.)	U			
(l) Leaching requirement – ratio	LR_t			
IV. EMITTER				
(a) Type				
(b) Outlets per emitter				
(c) Pressure [head] – kPa [m] (psi [ft])	P [H]			
(d) Rated discharge @ H – L/hr (gph)	q			
(e) Discharge exponent	x			
(f) Coefficient of variability	v			
(g) Discharge coefficient	K_d			
(h) Connection loss equivalent – m (ft)	f_e			

FIG. 19.5. Trickle Irrigation Design Data Form.

Design Factors Form

Prior to designing the hydraulic network, a number of computations must be made. These include the emitter spacing, average emitter discharge, average

I. PROJECT				
II. TRIAL DESIGN				
(a) Emission point layout				
(b) Emitter spacing – m x m (ft x ft)	$S_e \times S_l$			
(c) Emission points per plant	N_p'			
(d) Percent wetted area – %	P_w			
(e) Maximum net depth – mm (in.)	d_x			
(f) Average peak transpiration – mm/day (in./day)	T_d			
(g) Maximum interval – days	f_x			
(h) Irrigation frequency – days *	f'			
(i) Net depth per irrigation – mm (in.)	d_n			
(j) Assumed uniformity – %	EU			
(k) Gross depth per irrigation – mm (in.)	d			
(l) Gross water per plant – L/day (gal/day)	G			
(m) Application time – hr	T_a			
III. FINAL DESIGN				
(a) Application time – hr	T_a			
(b) Irrigation interval – days *	f'			
(c) Gross depth per irrigation – mm (in.)	d			
(d) Average emitter discharge – L/hr (gph)	q_a			
(e) Average emitter head – m (ft)	H_a			
(f) Allowable head variation – m (ft)	ΔH_s			
(g) Emitter spacing – m x m (ft x ft)	$S_e \times S_l$			
(h) Percent wetted area – %	P_w			
(i) Number of stations	N_s			
(j) System capacity – L/s (gpm)	Q_s			
(k) Seasonal efficiency – %	E_s			
(l) Seasonal irrigation – ha-m (A-ft)	V_s			
(m) Seasonal operation – hr	O_t			
(n) Total dynamic head – m (ft)	TDH			
(o) Actual uniformity – %	EU			
(p) Net application rate – mm/hr (in./hr)	I_n			

* Use a 1-day interval during the design process, then adjust f' up to f_x by multiplying the design T_a by the final f'.

FIG. 19.6. Trickle Irrigation Design Factors Form.

emitter pressure head, allowable head variation, and the hours of operation per season. The steps for developing these factors are outlined in the "Trickle Irrigation Design Factors" form presented as Fig. 19.6. This data sheet is a useful guide and provides a convenient place to record results of various trial and final computations.

REFERENCES

Ayers, R. S., and D. W. Westcott. 1985. Water quality for agriculture. *Food and Agricultural Organization of the United Nations, Irrigation and Drainage Paper* 29, Rev. 1.

Karmeli, D., and J. Keller. 1975. *Trickle Irrigation Design*. Glendora, California: Rain Bird Sprinkler Manufacturing Corp.

Keller, J., and D. Karmeli. 1974. Trickle irrigation design parameters. *ASAE Transactions* 17(4):678-684.

Roth, R. L. 1974. Soil Moisture distribution and wetting pattern from a point source. Proceedings Second International Drip Irrigation Congress, Aug. 1974. San Diego, California, pp. 246-251.

Schwartzmass, M., and B. Zur. 1985. Emitter spacing and geometry of wetted soil volume. *Journal of Irrigation and Drainage Engineering ASCE* 112(3):242-253.

Sharples, R. A., D. E. Rolston, J. W. Biggar, and H. I. Nightingale. 1985. Evaporation and soil water balances of young trickle-irrigated almond trees. In *Drip/Trickle Irrigation in Action*, Proceedings of Third International Drip/Trickle Irrigation Congress, Nov. 1985, 18-21, Fresno, California, vol. II, pp. 792-797. St. Joseph, Michigan: American Society of Agricultural Engineers.

RECOMMENDED READING

Benami, A., and A. Ofen. 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).

Bowman, R. S., and F. S. Nakayama. 1986. Design principles: Salt distribution. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama, and D. A. Bucks, pp. 117-141. Amsterdam: Elsevier.

Hoffman, G. J. 1986. Management principles: Salinity. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama, and D. A. Bucks, pp. 345-362. Amsterdam: Elsevier.

Warrick, A. W. 1986. Design principles: Soil water distribution. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama, and D. A. Bucks, pp. 93-116. Amsterdam: Elsevier.

20

Emitter Selection and Design Criteria

Selecting an emitter requires a combination of objective and subjective judgments. Along with the related requirements for water treatment, selection of an appropriate emitter is the most nebulous aspect of the trickle irrigation design process. The selection process is not simply a matter of following a checklist of instructions. The consequences of one decision will alter the assumptions used in making other decisions.

Efficiency of the designed system depends largely on the emitter selection. Some characteristics of emitters that affect system efficiency are:

- Variations in rate of discharge due to manufacturing tolerances;
- Closeness of discharge-pressure relationship to design specifications;
- Emitter discharge exponent;
- Possible range of suitable operating pressures;
- Loss of pressure on lateral lines caused by the emitters' connections;
- Susceptibility to clogging, siltation, or accretion of chemical deposits; and
- Stability of the discharge-pressure relationship over a long time.

The choices of discharge, spacing, and the emitter itself are major items in system planning. They are dictated partly by physical data, and also by such factors as emitter placement, type of operation, lateral diameter, and user preference. Selection of emitters requires four steps. First evaluate and choose the general type of emitter that best fits the needs of the area to be wetted. Then, according to the system's required discharge, spacing, and other planning considerations, choose the specific emitter needed. Third, determine the required discharge, q_a, and pressure head, H_a, for the average emitter. Fourth, determine what variation in subunit pressure head, ΔH_s, is allowable and will give the desired uniformity of emission, EU.

EMITTER FLOW THEORY

Emitters dissipate the pressure in the pipe distribution network as the water flows from the lateral hoses into the atmosphere. The pressure is dissipated either by individual small-diameter orifices, a series of such orifices, vortex chambers, short tubes, long tubes, or tortuous flow paths. A general knowledge of the theory of emitter design as it applies to different methods of dissipating pressure is helpful in the selection process.

Long-Path Emitters

Most loss of head in a smooth long-path emitter (Fig. 20.1) occurs in the long-flow path where the flow is laminar. The length of path needed for a given loss of head at a known discharge for long-path emitters is computed by:

$$l_c = \frac{H\, g\, D^4 \pi}{K\, q\, \nu} \tag{20.1}$$

where

l_c = length of the flow path in the emitter, m (ft)
H = working pressure head of (or head loss through) the emitter, m (ft)
g = acceleration of gravity, 9.8 m/s^2 (32.2 ft/s^2)
D = cross-sectional diameter of the flow path, mm (in.)
K = conversion coefficient, 3.56×10^7 for metric units; (98.6 for English units)
q = emitter discharge, L/hr (gph)
ν = kinematic viscosity of the water, m^2/s (ft^2/s), which is 1.00×10^{-6} m^2/s at 20°C (1.075×10^{-5} ft^2/s at 68°F)

Equation 20.1 shows that H is directly proportional to l_c and is inversely proportional to the fourth power of D, or conversely, l_c depends on the fourth

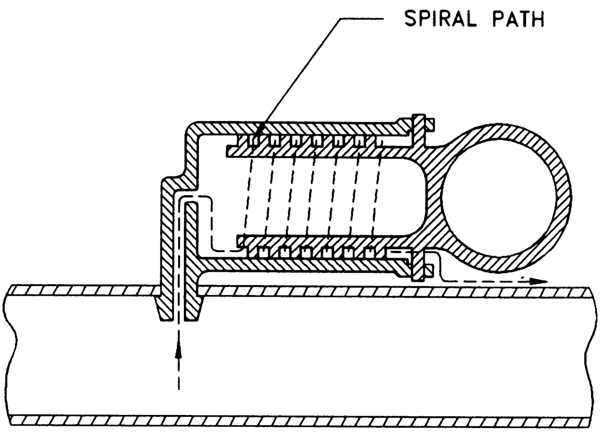


FIG. 20.1. Cross Section of a Long-spiral-path Emitter That Can Be Opened for Ease of Cleaning.

power of D. Hence, any change in the diameter, D, considerably influences the head loss, H, and the length of the flow path, l_c. Because ν decreases as temperature increases, l_c also needs to increase for higher temperatures or decrease for lower temperatures. Equation 20.1 also shows that the rate of discharge is directly proportional to H for laminar flow. Therefore, laminar flow emitters are quite sensitive to pressure differences within the system.

Equation 20.1 is based on the assumption that the cross section of flow is circular. For the same cross-sectional area and length of flow path, the discharge in an equilateral triangular cross section is greater than in a semicircular one. The discharge in a square flow cross section is greater than in other rectangular sections having the same cross-sectional area and length of path, but is less than for a circular cross section.

The spiral effects and other irregularities in long-tube emitters (see Fig. 20.1) create considerable turbulence; therefore, the characteristics of the emitter head loss deviate considerably from what Eq. 20.1 describes. If there are enough irregularities the emitter is classed as a tortuous-path emitter.

Tortuous- and Short-Path Emitters

Tortuous-path emitters have relatively long flow paths. Pressure head is lost by a combination of wall friction, sharp bends, contractions, and expansions. Some tortuous-path emitters look similar to ordinary long-path emitters. However, their flow channel is typically shorter, and the cross section is larger for the same head, H, and discharge, q. Because the flow regime is almost fully turbulent, the q varies more nearly as the $\sqrt{H}$ than directly with H and is nearly independent of ν.

Short-path emitters generally behave like orifice emitters because the entrance characteristics (losses) dominate the flow regime in the short-tube section. However, many short-path emitters are pressure-compensating.

Orifice Emitters

The class called *orifice emitters* includes many drip and spray emitters and also single-chamber line-source tubing. In a nozzle or orifice emitter, water flows through a small-diameter opening or series of openings where most of the pressure head is lost. The flow regime is fully turbulent, and the discharge of the emitter, q, is given as:

$$q = K\, a' K_q \sqrt{2gH} \tag{20.2}$$

where a' = orifice flow cross-sectional area, mm^2 (in.^{2}), K = conversion constant, 3.6 for metric units (187 for English units), and K_q = coefficient of discharge for an outlet that depends on the characteristics of the orifice or nozzle and ranges from 0.6 to 1.0. The cross-sectional area needed for the desired relationship between q and H depends on K_q. For a sharp orifice it is close to 0.6 and for a tapered nozzle it approaches 1.0.

Twin-Chamber Tubing

Most of the loss of pressure head in twin-chamber tubing (see Fig. 20.2) occurs in the inner orifice. The discharge from the inner orifice can be computed by modifying Eq. 20.2 to:

$$q = K\,a'K_q\,\sqrt{2g(H - H')} \tag{20.3a}$$

where H' = working pressure head in the secondary chamber, m (ft), and K = conversion constant, 3.6 for metric units (187 for English units). Normally the orifices in both the main chamber and secondary chamber have the same diameter, and there are from three to six secondary orifices for each main orifice. Therefore, H' can be computed by: $H' = H/(1 + n_o^2)$ and substituting in Eq. 20.3a gives:

$$q = K\,a'K_q\,\sqrt{2gH\,n_o^2/(1 + n_o^2)} \tag{20.3b}$$

where n_o = number of secondary orifices for each main orifice.

Vortex Emitters and Sprayers

The vortex emitter (or sprayer) has a flow path containing a round cell that causes circular flow. The circular motion is achieved by having the water enter tangentially to the outer wall. This produces a fast rotational motion, creating a vortex at the center of the cell. Consequently, both the resistance to the flow and the head loss in the vortex emitter are greater than for a simple orifice

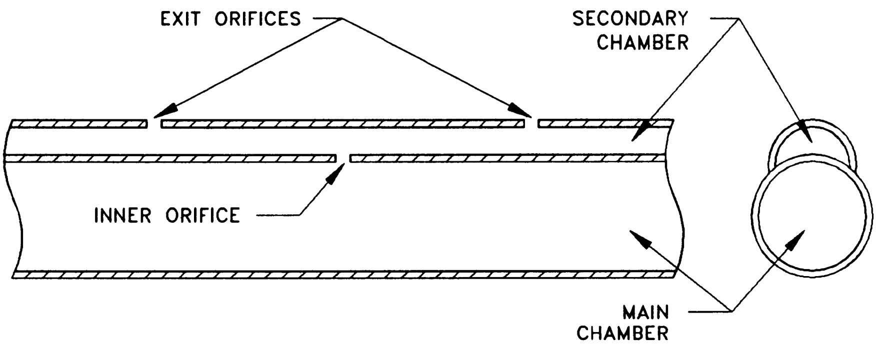


FIG. 20.2. Twin-chamber Trickle Irrigation Tubing.

having the same diameter. Vortex emitters can be constructed to give a q vs. H relationship of approximately:

$$q = K\, a' K_q \sqrt{2g}\, H^{0.4} \tag{20.4}$$

where K = conversion constant, 3.6 for metric units (187 for English units).

The discharge exponent of approximately $x = 0.4$ for vortex emitters is an advantage over the $x = 0.5$ for simple orifice emitters (see Eq. 20.2). Larger openings that are less susceptible to clogging may be used. Furthermore, variations in emitter operating pressures due to elevation differences and pipe friction cause smaller variations in the discharge from vortex emitters.

Compensating Emitters

Compensating emitters are constructed to yield a nearly constant discharge over a wide range of pressures. Long-path, short-path, and orifice-type (see Fig. 20.3) compensating emitters are available. The constant discharge (or compensating feature) is achieved by using a resilient material in the flow path. This is acted on by the line pressure so that the flow cross section decreases as the pressure increases. A peculiar problem of compensating emitters is that the resilient material may distort over time. This can gradually squeeze off the flow even though pressure remains constant.

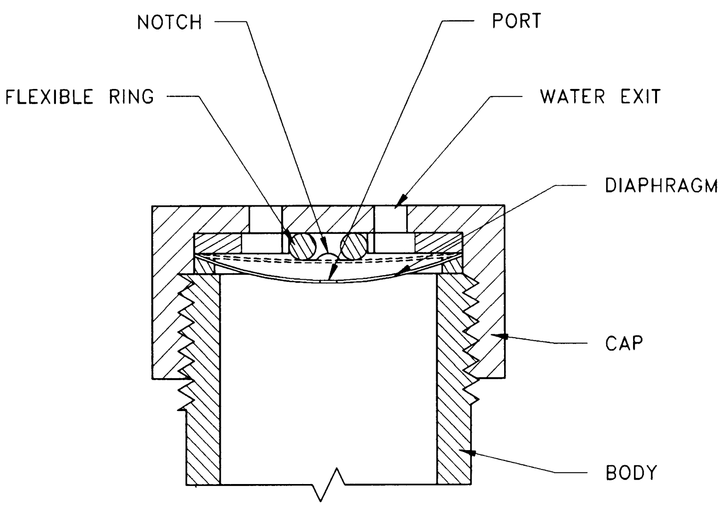


FIG. 20.3. Cross-section of a Flushing and Flow Compensating Emitter (Source: Karmeli and Keller, 1975 (Fig. 2.6)).

The general equation for orifice-type and short-tube compensating emitters is:

$$q = K\,a'K_q\,\sqrt{2g}\,H^x \tag{20.5}$$

where K = conversion constant, 3.6 for metric units (187 for English units), and x = an exponent that varies from 0.5 to 0.0, depending on the characteristics of the flow section and resilient material.

Flushing Emitters

The two types of self-flushing emitters are on-off flushing and continuous flushing. On-off-flushing emitters flush for only a few moments each time the system is started and again when it is shut off. They are typically of the compensating type (see Fig. 20.3).

Continuous-flushing emitters are constructed so they can eject relatively large particles during operation. They do this by using relatively large-diameter flexible orifices in series to dissipate pressure. As shown in Fig. 20.4, particles larger than the diameter of the orifices are ejected by a local increase of pressure as the particles reach each successive flexible orifice.

Because the orifices are flexible they expand under pressure, and the discharge equation can be approximated by:

$$q = K\,a'K_q\,\sqrt{2g}(H/n')^{0.7} \tag{20.6}$$

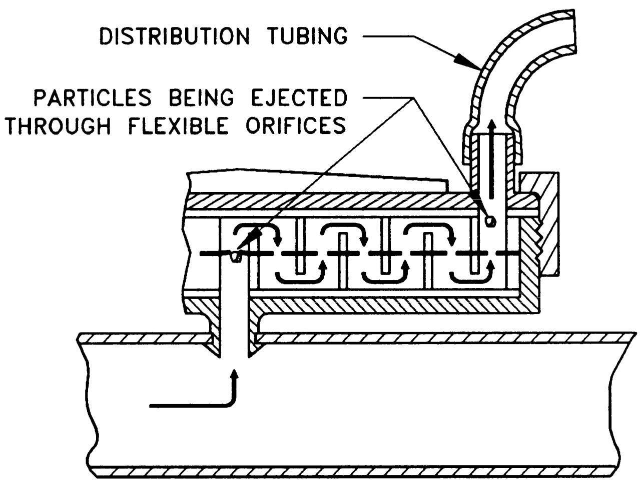


FIG. 20.4. Cross-section of Continuous Flushing Emitter.

and for an emitter that has a series of rigid orifices:

$$q = K\, a' K_q \sqrt{2gH/n'} \tag{20.7}$$

where K = conversion constant, 3.6 for metric units (187 for English units), and n' = number of orifices in series.

Comparing Eqs. 20.6 and 20.7 shows that continuous-flushing orifice emitters are more sensitive to pressure changes than the on-off type. Furthermore, because of the effect that temperature has on the orifice material, they are almost as sensitive to temperature as long-path emitters.

Calculation of Emitter Discharge Exponent

Over the desired range of discharge, the q vs. H relationship of most emitters can be characterized, as described earlier, by Eq. 17.1:

$$q = K_d H^x$$

where K_d = constant of proportionality (discharge coefficient) that characterizes each emitter, and x = emitter discharge exponent.

To determine K_d and x, the discharge from an emitter at two different operating pressures must be known. From q_1 at H_1 and q_2 at H_2, the exponent x may be determined analytically by:

$$x = \frac{\log (q_1/q_2)}{\log (H_1/H_2)} \tag{20.8}$$

A similar result can be determined graphically by measuring the slope of the line that connects the two H and q values plotted on log-log graph paper (Fig. 20.5). The value of x can be used in Eq. 17.1 to solve for K_d.

Sample Calculation 20.1. *Determine the discharge exponent and discharge coefficient from discharge-versus-pressure-head data for a vortex emitter, and find the pressure head required to produce any given discharge.*

GIVEN: Emitter discharges, q, at operating heads, H:
q_1 = 3.0 L/hr at H_1 = 5.0 m; and q_2 = 4.0 L/hr at H_2 = 10.0 m.

FIND: Discharge exponent, x;
Discharge coefficient, K_d; and
Head at which q = 5.0 L/hr.

CALCULATIONS: Either the numerical or graphical method can be used. *Numerical method* by Eq. 20.8:

$$x = \frac{\log(3.0/4.0)}{\log(5.0/10.0)} = 0.42$$

and by Eq. 17.1:

$$K_d = \frac{q}{H^x} = \frac{4}{(10.0)^{0.42}} = 1.52$$

Check:

$$q_1 = 1.52 \times (5.0)^{0.42} = 3.0 \text{ L/hr}$$

The head at which $q = 5.0$ L/hr is found by rearranging Eq. 17.1 to:

$$H_3 = \left(\frac{q_3}{K_d}\right)^{1/x} = \left(\frac{5}{1.52}\right)^{1/0.42} = 17.0 \text{ m}$$

Graphical method by Fig. 20.5:

$$x = 0.42; \text{ and } K_d \simeq 1.5$$

When $q_3 = 5.0$ L/hr, $H_3 = 17.0$ m.

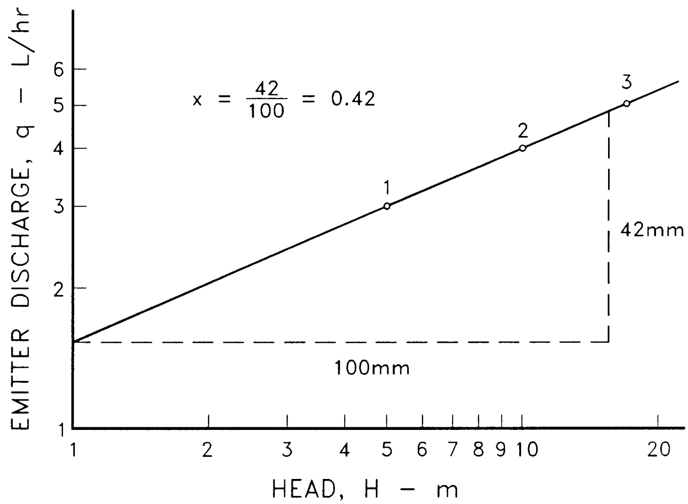


FIG. 20.5. Graphical Method for Determining the Exponent x for Sample Calculation 20.1.

CRITERIA FOR SELECTING EMITTERS

The quality and safety of trickle systems are affected directly by: the emitter design and quality; the percentage area wetted; allowable variations of pressure; adequacy of filtration; degree of automation; and reliability of the management, labor, power, and water supplies. Two very important items are the percentage area wetted and the reliability of the emitter against clogging and malfunctioning.

Initially, selection of an emitter depends on the soil to be wetted, plant requirements for water, emitter discharge, quality of the water, and the terrain of a particular location. The choice of a particular emitter should follow a detailed evaluation of the various features discussed below. Evaluation must include cost of the emitter and risks inherent in the system. Generally, the emitters that offer the more desirable features and pose the fewest system risks are more expensive. An emitter's performance characteristics will influence the estimated cost of the pipe network and filtration system. Therefore, the original choice may need to be reevaluated before an emitter for the system is finally selected.

A sound design objective for a trickle system is to provide a sufficient number of emission points to wet between one-third and one-half of the horizontal cross-section of area of the potential root system. There is some interaction between the rate of emitter discharge and area wetted at each emission point. However, the density of emission points required to obtain $P_w > 33\%$ can usually be based on an assumed discharge rate of 4 L/hr (1 gph) using the procedures described in Chapter 19.

The volume of water required for maximum growth increases as the plant grows. Economic advantages for perennial crops, in terms of lower initial installation costs and water savings, can be achieved by installing fewer emitters at first. The number of emitters can be increased later as required for each stage of growth. However, the initial pipe network must be designed to meet the final needs of the mature plants.

Although they are difficult to achieve, an ideal set of emitters should have the following attributes:

- Durability;
- Low cost;
- Reliable performance with a relatively low rate of discharge that is reasonably uniform among all emitters within the system despite: variances in tolerances inherent in manufacturing, expected differences in pressure head due to friction loss and elevation, and expected changes in temperature of the water; and
- Relatively large and/or self-flushing passageways to reduce or prevent clogging.

General Suitability of Emitters

Suitability refers to how well the emitter will fit into the particular design and match the spacing and water requirements of the crop. Emission devices are available that emit water at individual points or along the length of a line. The point-source devices come with one, two, or multiple outlets for the water. When more than one outlet is provided, distribution tubing is generally used to deliver the water from the emitter location to the desired discharge location as on Fig. 20.6A.

Single-outlet emitters can be used to irrigate small spots, or can be arranged around larger plants (see Fig. 17.5) to serve the same function as dual- or multiple-outlet emitters or sprayers (see Fig. 20.6B). Dual-outlet emitters are often used for vines; multiple-outlet emitters are used in orchards where larger trees may each require several emission points.

Emitters that have more than one outlet are more expensive than single-outlet emitters, but the higher cost is not in proportion to the number of outlets. For instance, a dual-outlet emitter is probably more expensive than a comparable single-outlet emitter, but less expensive than two single-outlet emitters. Thus emitters that have more outlets are generally less expensive per outlet.

For row crops, such as strawberries or vegetables, line-source tubing fits well with the cropping pattern, because it provides the desired linear wetted strip (see Fig. 20.7). Cost is especially important in row crop irrigation because serving the closely spaced plants requires a large amount of line-source tubing. Point-source emitters may also be appropriate for irrigating row crops. But emitters must be closely spaced along the laterals to generate the necessary strip pattern of wetting. Therefore, only the least expensive emitters are normally considered for irrigating row corps. However, improved performance characteristics and longer life could justify the use of the more expensive point-source emitters.

Besides fitting in with the intended cropping pattern, the system of emitters chosen must be able to deliver the required discharge at the desired pressure. There are many emission points within a field. Therefore, even a small difference between the actual and required emitter discharge rates can result in a significant difference in the required pump and pipe sizes.

Sensitivity to Clogging

To achieve the low rates of discharge required in trickle irrigation, the cross sections of the flow channels must be within the range of 0.25 and 2.5 mm (0.01 and 0.10 in.). Necessity for such small channels makes all emitters susceptible to clogging. Therefore, careful filtration of all irrigation water is required. The usual recommendation is to remove all particles larger than one-tenth the diameter of the emitter passageway, as discussed earlier. Even this is

A. Lateral with Multiple-outlet Drip Emitter

B. Spray Emitter on Underground Lateral

FIG. 20.6. Trickle Irrigation Systems with Individual Emitters Operating in Orchards.

FIG. 20.7. Trickle Systems with Line Source Tubing to Irrigate Row Crops Without Mulch (Top) and With Plastic Mulch (Bottom).

not sufficient for long-path emitters because of sedimentation along the passageway, which can cause slow clogging over a period. However, for some flushing-type emitters less filtration is required.

Sensitivity to clogging is a very important consideration when selecting an emitter. Two critical parameters related to clogging susceptibility are the size of the flow passage and the velocity of the water through the passage. The relation between the passage cross section and the passage's susceptibility to clogging is:

- *Very sensitive*—less than 0.7 mm (0.028 in.);
- *Sensitive*—0.7 to 1.5 mm (0.028 to 0.060 in.); and
- *Relatively insensitive*—larger than 1.5 mm (0.060 in.) or continuously flushing emitters (see Fig. 20.4).

The velocity of water through the passage is probably as important as the passage dimensions. Velocities ranging from 4 to 6 m/s (13 to 20 ft/s) have resulted in reduced clogging.

An emitter's discharge is usually rated at a reference temperature of 20°C (68°F) and pressure of 105 to 210 kPa (15 to 30 psi). This is equivalent to a range of pressure heads from approximately 10 to 20 m (33 to 66 ft). However, line-source tubing is usually rated at less than 105 kPa (15 psi). The flow cross section in an orifice emitter is very small, 0.2 to 0.6 mm (0.008 to 0.024 in.), for reference discharges of 2 to 10 L/hr (0.5 to 2.5 gph). Therefore, orifice emitters tend to clog easily. The flow cross section in a long-path emitter is considerably larger, 0.5 to 1.4 mm (0.02 to 0.055 in.) for discharges of 2 to 8 L/hr (0.5 to 2.0 gph). Their larger passageway reduces the likelihood of clogging. But when flow velocity is low, silt and mineral deposits along the flow path can cause slow clogging as mentioned earlier.

Clearly an easy way to ascertain an emitter's sensitivity to clogging is to consider the manufacturer's recommendations for filtration. The greater the sensitivity, the finer the recommended filtration should be. Of course user experience based on experience with various emitters in use locally is also a valuable gauge of filtration requirements.

To reduce danger of clogging, some emitters have been designed to include *capability for flushing*. These features range from those that automatically flush at start-up and shut down (see Fig. 20.3) to those that flush continually (see Fig. 20.4) as discussed earlier. To be effective, the short-flush type requires a minimum velocity and duration of flush. If the flushing control mechanism depends on gravity, it must be kept upright in the field. The continual-flushing emitters have a series of orifices in a resilient material designed to dissipate pressure. When clogging occurs, line pressure builds up behind the particle and forces the orifice to expand and let the particle pass through.

Recent experience with line-source tubing has shown that the tendency to clog can be reduced significantly by regular lateral flushing. Two types of lateral flushing are used. One has independent, automatically actuated spring-loaded valves. The other has small hydraulic valves actuated by an external control system. Even in situations with good-quality water, lateral flushing provides an added safety factor for continual operation of a system. Therefore, provisions should be made for flushing all emitter laterals, especially if nonflushing emitters are selected.

Coefficients of Variation

It is impossible to manufacture any two items exactly alike. The small differences between what appear to be identical emitters may cause significant variations in discharge. This is because the critical dimensions of the emitter flow passage are small and difficult to manufacture precisely. Very small variations in passage size, shape, and surface finish can result in large relative variations from the nominal emitter dimensions. Emitters that have an elastomer to provide pressure-compensating or flushing ability are inherently difficult to manufacture with uniform dimensions and characteristics. The amount of difference in discharge characteristics depends on the emitter's design, the materials used in its construction, and the precision with which it is manufactured.

The *coefficient of manufacturing variation for the emitter*, v, is used as a measure of the anticipated variations in discharge for a sample of new emitters. The value of v should be available from the manufacturer. However, it can be determined from the discharge data of a sample set of at least 50 emitters operated at a reference pressure head. It is calculated by:

$$v = \frac{\sqrt{(q_1^2 + q_2^2 \ldots + q_n^2 - nq_a^2)/(n-1)}}{q_a} \tag{20.9a}$$

which is:

$$v = \frac{\text{sd}}{q_a} \tag{20.9b}$$

where

v = coefficient of manufacturing variation for the set of emitters in which $q_1, q_2, \ldots, q_n$ are individual emitter discharge rates, L/hr (gph)
n = number of emitters in the sample
q_a = average emitter discharge rate for the sample, $(q_1 + q_2 \ldots + q_n)/n$, L/hr (gph)
sd = estimated standard deviation of the discharge rates of the population, L/hr (gph)

The v is a very useful parameter with rather consistent physical significance, because the discharge rates for emitters at a given pressure are essentially normally distributed. The physical significance of v is derived from the classic bell-shaped normal distribution curve in which:

- Essentially all of the observed discharge rates fall within $(1 \pm 3v)q_a$;
- Approximately 95% of the discharge rates fall within $(1 \pm 2v)q_a$;
- The average of the low one-fourth of the discharge rates is approximately equal to $(1 - 1.27v)q_a$; and
- Approximately 68% of the discharge rates fall within $(1 + v)q_a$.

Thus, for an emitter having $v = 0.06$, which is average, and $q_a = 4$ L/hr (1.0 gph), 95% of the discharges can be expected to fall within the range of 3.52 and 4.48 L/hr (0.88 and 1.12 gph), and the average discharge of the low one-quarter will be approximately 3.70 L/hr (0.92 gph).

As a general guide, manufacturing variability can be classified in accordance with Table 20.1, which comes from Soloman, 1979. A lower standard is used for line-source tubing, because it is difficult to keep the variation and price both low. Nevertheless, because line-source outlets are usually closely spaced, row crop production is relatively insensitive to moderate variations in the discharge between adjacent outlets.

The *system coefficient of manufacturing variation*, v_s, is a useful concept, because more than one emitter or emission point may be used per plant (Soloman, 1979). In such an installation, the variations in flow rate for all emitters around the plant generally compensate for one another. One emitter might have a high flow rate and another would probably have a lower flow rate. On the average, the variations in the total volume of water delivered to each plant is less than might be expected from considering v alone. The v_s may be characterized by:

$$v_s = \frac{v}{\sqrt{N'_p}} \tag{20.10}$$

Table 20.1. Classification of emitter coefficient of manufacturing variation, v[1]

Classification (quality)	Drip & spray emitters	Line-source tubing
Excellent	$v < 0.05$	$v < 0.1$
Average	$0.05 < v < 0.07$	$0.1 < v < 0.2$
Marginal	$0.07 < v < 0.11$	
Poor	$0.11 < v < 0.15$	$0.2 < v < 0.3$
Unacceptable	$0.15 < v$	$0.3 < v$

[1]This has basically been adapted in the Engineenng Practice Standard: ASAE EP405.1

where: v_s = system coefficient of manufacturing variation, and N'_p = number of emitters from which each plant receives water.

Line-source systems may have only one outlet per plant; however, because of the close spacing of outlets, (see Fig. 20.7), each plant may receive its water from two outlets in which case $N'_p = 2$. If multioutlet emitters with small-diameter distribution tubing are used (see Figs. 17.5E and 20.6A), the proper value of N'_p depends on the design of the individual emitter. If one common loss element serves several outlets, $N'_p = 1$. If there is a separate pressure loss passageway for each outlet, then there are really multiple emitters in a single housing, and N'_p is the number of outlets. It should be emphasized that *v is a property of the emitter alone, and v_s is a property of the trickle irrigation system as a whole.*

Sprayers (see Fig. 20.6 B) must apply a relatively uniform depth of application to the directly wetted soil surface. Some variation is acceptable, but extensive differences in water distribution within the main wetted area are not. Such variations can cause runoff, which would not result from a uniform application at the same average rate. Furthermore, the distribution of soil moisture is likely to be unacceptable when the depth of application varies by a factor of more than 2 : 1 between points 1 m (3 ft) or farther apart.

Discharge versus Pressure Relationships

The relation between pressure head and discharge is an important characteristic of emitters. Figure 17.1 shows this relationship for various types of emitters. The emitter discharge exponent, x, measures the flatness of the discharge-pressure curve. It clearly demonstrates the desirability of an emitter that has a discharge-pressure curve with a low x. Compensating emitters have the lowest x values. But they all have some physical part that responds to pressure, and their long-range performance is not always reliable. Furthermore, compensating emitters often have a high v, and their performance may be affected by temperature and material fatigue.

On *undulating terrain* the design of a high-uniformity system is constrained by the pressure sensitivity of the emitters. Compensating emitters or the use of pressure-regulated flow into short laterals provide immediate solutions. Another potential solution is to use various sizes of emitters to compensate for the variations in pressure caused by changes in elevation. However, using more than one size of emitter in the field is generally considered impractical.

The *lateral length*, even on smooth fields, must be kept reasonably short to avoid excessive differences in pressure. Factors that affect the maximum recommended length are the: discharge per unit length; desired emission uniformity; flow characteristics of the emitter selected; lateral layout pattern; terrain; and lateral pipe diameter. In most installations, field dimensions and cultural practices are deciding factors for determining the length.

For *laminar flow emitters*, the relation between the discharge and the operating pressure is nearly linear. Therefore, the variations in operating pressure head within the system should be held within about ± 5% pressure head variation around the desired average called for.

For *turbulent-flow emitters*, the change in discharge varies with the square root of the pressure head; i.e., $x = 0.5$. Consequently, the pressure must be increased four times to double the flow. Therefore, the pressure head in systems using turbulent flow emitters is often allowed to vary by about ± 10% of the desired average.

Flow-compensating emitters provide varying degrees of flow regulation with x values between 0.0 and 0.4. For complete flow regulation, $x = 0.0$. However, complete flow regulation where increases in pressure would not increase flow might be undesirable. This is because there would be no flexibility to adjust for underdesign or for discharge decreases due to slow clogging or emitter deterioration. When x is between 0.2 and 0.35, considerable regulation is achieved (i.e., a 50% head differential would cause only an 8 to 15% variation in discharge). However, there is still some flexibility for adjusting the discharge rate. Compensating emitters (and pressure regulators) are valuable chiefly for use on hilly sites where it is impractical to design for uniform pressures along the laterals and manifolds.

Temperature versus Discharge Relationships

An emitter may be sensitive to water temperature for any of three reasons (see Table 20.2). With most long-path emitters the discharge depends on the viscosity of the water, which changes with temperature. Most emitters are somewhat sensitive to water temperature because of dimensional changes in the flow passages. The discharge from emitters with parts made of resilient material (e.g., pressure-compensating emitters) may vary due to changes in material characteristics caused by temperature changes.

There is a difference between the air temperature and that of the water in the pipe, especially if the lateral pipe lies in the sun. As water moves through the system it changes temperature (usually warming) toward the ends of laterals. For laminar flow emitters the decrease in viscosity resulting from this warming may compensate for the usual decrease in pressure toward the ends of laterals. For a long-path emitter with $x = 0.8$, the increase in discharge due to decreased ν is roughly 1% for each 2°C (3.6°F) increase in water temperature. For a tortuous-path emitter with $x \approx 0.6$, the increase is roughly 1% for each 4°C (7.2°F).

Connection Losses

The three main types of lateral connections are in-line, on-line, and on-line riser, as shown in Fig. 17.4. On-line connections can be used with above-ground

Table 20.2. Characteristics of emission devices based on test data[1]

Emission Device[2]	x^3	v^4	TDR[5] 45°C	TDR[5] 65°C	MFPD[6] mm	Flushing ability
Orifice						
Vortex/Orifice	0.42	0.07	0.92	0.88	0.6	None
Multiple flexible	0.7	0.05	1.04	1.07	-	Continuous
orifices	0.7	0.07	1.04	1.07	-	Continuous
Ball & slotted	0.50	0.27	1.15	1.21	(0.3)	Automatic
seat	0.49	(0.25)	0.83	0.79	(0.3)	Automatic
Compensating ball	0.15	0.35	0.85	0.81	0.3	Automatic
& slotted seat	0.25	0.09	0.90	0.89	(0.3)	Automatic
Capped orifice	0.56	(0.05)	(1.03)	(1.05)	1.0	None
sprayers	0.53	(0.05)	(1.03)	(1.05)	1.5	None
Long-path						
Small tube	0.70	0.05	1.08	1.13	1.0	None
	0.80	0.05	1.16	1.22	1.0	None
Spiral path	0.75	0.06	1.19	1.18	0.8	Manual
	0.65	0.02	(1.10)	(1.15)	0.7	None
Compensating	0.40	0.05	1.19	1.33	(0.8)	None
	0.20	0.06	1.11	1.24	(0.8)	Automatic
Tortuous	0.50	(0.08)	1.40	1.70	0.8	None
	0.65	0.02	1.08	1.14	(1.0)	None
Short-path						
Groove & flap	0.33	0.02	1.00	1.00	0.3	Automatic
Slot & disc	0.11	0.10	1.06	1.08	0.3	Automatic
Line-source						
Porous pipe	1.0	0.40	2.70	3.80	-	None
Twin-chamber	0.61	0.17	(1.05)	(1.10)	(0.4)	None
	0.47	(0.10)	(1.04)	(1.08)	(0.4)	None

[1]Test data based on a standard operating temperature of 20°C (68°F). Numbers in parentheses are estimates. Data are adapted from Solomon (1977).
[2]Double entries indicate different devices of the same general type.
[3]x is the emitter discharge exponent (Eq. 17.1).
[4]v is the coefficient of manufacturing variation of the emitter (Eq. 20.9).
[5]Temperature-discharge ratio, TDR, is the ratio of the emitter discharge at a temperature warmer than 20°C (68°F)
[6]Minimum flow path dimension—not meaningful with continuous flushing.

or shallow-buried laterals. On-line risers are used with underground laterals. However, they are cost-effective only where they provide agronomic advantages and the emitter spacing is wide.

Stress cracking may occur where the emitter barbs stretch the lateral wall and cause premature aging. This results in leakage, and in extreme cases the emitter connections may rupture. To prevent this potential hazard, on-line emitters should be carefully connected to the lateral. Holes for the barbs should have smooth edges and be properly sized and punched out. In-line emitters should be provided with compression barbs or compression ring fittings that fit over the outside of the lateral.

The *emitter connection friction loss as an equivalent length of lateral*, f_e, is a useful value for estimating the friction loss of laterals. The f_e depends on the size and type of barb and inside diameter of the lateral. Figure 20.8 gives estimated f_e values for in-line emitters and three sizes of on-line emitter barbs for laterals with different inside diameters (Watters and Keller, 1978).

Performance of Emitters

Test data for a number of emitters are presented in Table 20.2 (Solomon, 1977). All tests were made with clean water at a standard temperature of 20°C (68°F) using new emitters obtained from retail outlets. Data listed in parentheses are not based on actual measurements, but are estimates of probable values.

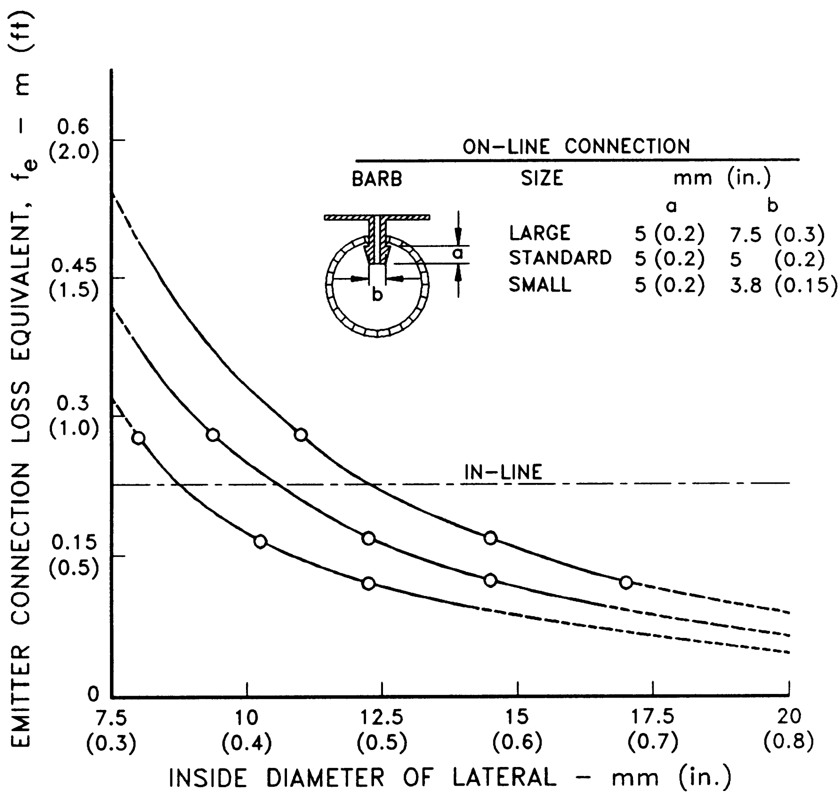


FIG. 20.8. Emitter Connection Loss, f_e, Values for Different Barb Sizes and Lateral Diameters.

Generalizing from these data should be done with caution. Emitters of the same design may have quite different performance characteristics. Their performance depends on the materials used in their construction and the care and precision with which they were manufactured. Nevertheless, the data in Table 20.2 provide a useful guide for the probable characteristics and important features of the various types of emitters.

Selecting for Discharge

Average emitter discharge versus pressure relationships and recommended operating pressure ranges should be available from the equipment suppliers. Usually emitters are specified in terms of their rated average discharge at some standard pressure head along with their discharge exponents. The number of different emitter sizes in terms of rated discharge is limited. The most common size of point-source emitters is 4 L/s (1 gph), and a few manufacturers also produce 2, 6, or 8 L/s (0.5, 1.5 or 2.0 gph) emitters. Therefore, to obtain the needed design flexibility in terms of discharge per unit area of field depending solely on the availability of different-sized emitters is not sufficient. The emitter spacing and operating time must also be adjusted.

EMITTER DISCHARGE AND HEAD REQUIREMENTS

After selecting a trial emitter, let q_a be the rated emitter discharge, then determine the duration of application for the periods of peak use by:

$$T_a = \frac{G}{N_p q_a} \tag{20.11}$$

where

T_a = irrigation application time required during the peak use period, hr/day
G = gross volume of water required per plant (or unit length of row) per day during the peak-use period, L/day (gal/day)
N_p = is the number of emitters per plant
q_a = average emitter discharge, L/hr (gph)

Average Emitter Discharge, q_a

The *maximum number of hours of operation per day* should not exceed 90% of the available time (i.e., 21.6 hr/day). This is necessary to allow some margin of safety for system failure or other unexpected downtime. However, systems should be operated as nearly continuously as is practical—at least 12 hr/day—to keep investment costs low. Furthermore, systems can be subdivided only into whole numbers (1, 2, 3, . . . , N_s) of operating stations, N_s (see Fig. 17.2). Thus the gross daily water required per plant, G, must be provided in no more than $21.6/N_s$ hr or less. With these concepts in mind, determine the number of stations needed. Then select a reasonable value for T_a so $12 < T_a < 21.6$ hr/day and rearrange Eq. 20.11 to compute a new q_a.

If the preliminary value of T_a computed by Eq. 20.11 is greater than 21.6 hr/day (even for a single station system), the emitter discharge would need to be increased above the rated discharge. When the amount of increase exceeds the recommended operating range or requires too much pressure, either larger emitters or more emitters per plant are required. Decision strategies for other preliminary T_a values are:

- If $T_a \approx 21.6$ hr/day, use a one-station system, $N = 1$, select $T_a \leq 21.6$ hr/day, and adjust q_a accordingly;
- If $T_a \approx 10.8$ hr/day use $N = 2$, select $T_a \leq 10.8$ hr/day, and adjust q_a accordingly; and
- If $12 < T_a < 18$ hr/day, it may be desirable to use different emitters or a different number of emitters per plant to operate closer to 90% of the time, providing this will reduce investment costs.

Average Emitter Pressure Head, H_a

The *average emitter pressure head*, H_a, that will give the desired q_a can be determined from the basic emitter discharge specifications. When the published data for the emitter are given as a series of pressure heads versus discharges, x must first be determined by Eq. 20.8. Next determine H_a directly by rearranging Eq. 17.1 to obtain:

$$H_a = H\left(\frac{q_a}{q}\right)^{1/x} \tag{20.12a}$$

or first determine K_d and then H_a by rearranging Eq. 17.1 as needed:

$$H_a = \left(\frac{q_a}{K_d}\right)^{1/x} \tag{20.12b}$$

DESIGN EMISSION UNIFORMITY, EU

It is necessary to know the efficiency of the irrigation system, so the relation between gross irrigation amounts and net additions to the root zone can be established. Emission uniformity is important, because it is one of the two components of irrigation efficiency; the other is various losses that occur during operation of the system. Emission uniformity can be calculated from field test data by using Eq. 17.2.

In the design phase, it is not possible to measure the rates of emission of the intended system. The variation to be expected in emission rates must be estimated by some analytical procedure. Unfortunately, it is not practical to consider all the influencing factors, such as full or partial clogging, changes in water temperature and aging of emitters in a formula for emission uniformity. It is not possible to look at a design and compute or even satisfactorily estimate the unpredictable variations in emission rates these factors may cause. However, the other items can be known. The manufacturer should provide information about the relation of pressure to rate of emission and also about manufacturing variability for the emitter. Topographic data from the intended site and a hydraulic analysis of the proposed pipe network can give the needed information about expected variations in pressure.

The basic concept and formulas used for emission uniformity were initially published by Keller and Karmeli (1974). The basis of their formulas is the ratio of the lowest emission rate to the average emission rate. This process treats emission rates below the average as being more important than those above. It treats the lowest emission rates as being more important than those merely somewhat below the average. This seems reasonable for evaluation, because trickle irrigation applies reduced amounts of water to only a portion of the plant's root zone. For trickle irrigation in particular it is more important to worry about underwatering than overwatering.

To estimate the emission uniformity for a proposed design, the following formula was developed (Karmeli and Keller, 1975):

$$EU = 100\left(1.0 - 1.27\frac{v}{\sqrt{N'_p}}\right)\frac{q_n}{q_a} \tag{20.13a}$$

or

$$EU = 100\,(1.0 - 1.27\,v_s)\frac{q_n}{q_a} \tag{20.13b}$$

where

EU = design emission uniformity, %
v = emitter coefficient of manufacturing variation from the manufacturer or by Eq. 20.9
v_s = system coefficient of manufacturing variation by Eq. 20.10
N'_p = minimum number of emitters from which each plant receives water
q_n = minimum emission rate computed from the minimum pressure in the

subunit or system, based on the nominal flow rate versus pressure curve, L/hr (gph)

q_a = average or design emission rate, L/hr (gph)

The ratio q_n/q_a expresses the relationship between the minimum and average emission rates resulting from pressure variations within the subunit or system. The factor $(1.0 - 1.27\ v_s)$ adjusts for the additional nonuniformity caused by anticipated manufacturing variations between individual emitters as discussed earlier.

ALLOWABLE HEAD VARIATION, ΔH_s

Figure 20.9 shows a schematic of the distribution of pressure head in a simple subunit. Figure 20.10 shows an example of the combined effect of pressure head and manufacturing variations on individual emitter discharges (Karmeli and Keller, 1975). The particular example depicted is for a subunit on a level field with constant-diameter manifolds and laterals in which ΔH_s = 3.0 m (10 ft) when H_a = 12.2 m (40 ft). This gives a subunit head-loss ratio of 0.25. The emitter characteristics are: q_a = 3.60 L/hr (0.95 gph) at H_a = 12.2 m (40 ft); x = 0.72; and v = 0.033.

In Fig. 20.10 the region of emitter discharges is bounded on the sides by the minimum and maximum pressures in the subunit. The bottom and top of the

$$\Delta H_s = (H_m - H_n)$$

$$(\Delta H_m)_a = (\Delta H_s - \Delta H_l)$$

$$\Delta H_l = (H_l - H'_n)$$

$$H_n \approx (H_m - \Delta H_m - \Delta H_l)$$

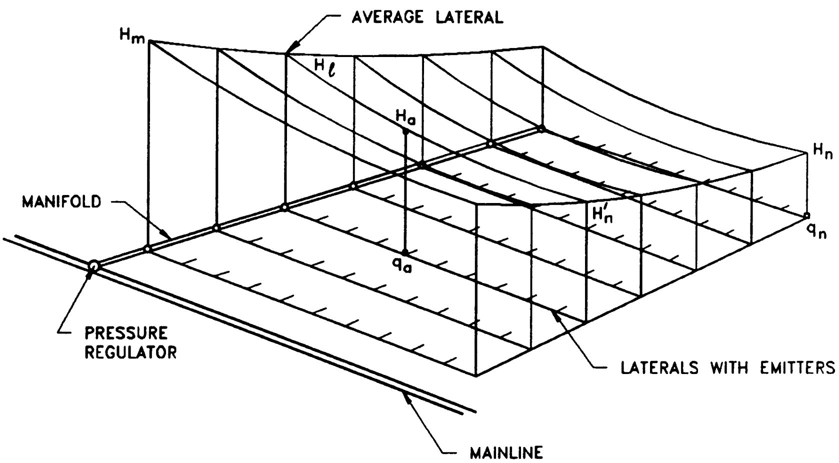


FIG. 20.9. Distribution of Pressure Head in a Subunit.

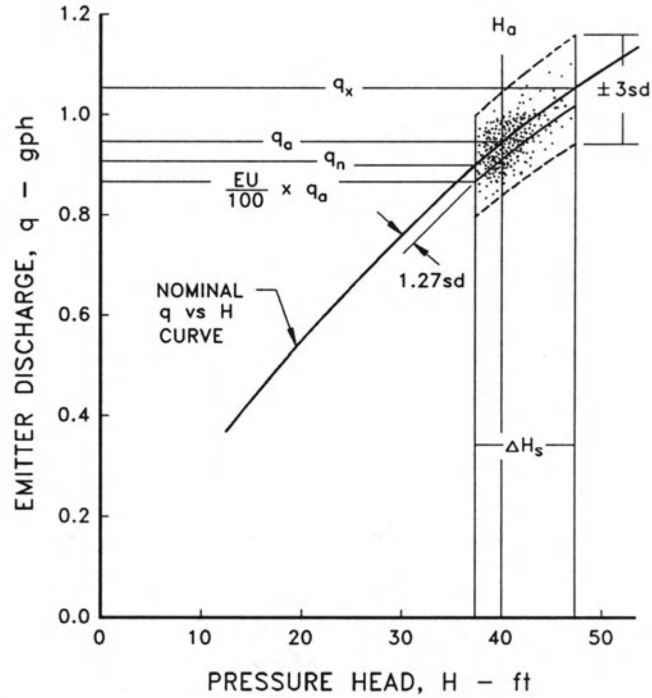


FIG. 20.10. Combined Effect of Pressure Head and Manufacturing Variations on the Discharges from Individual Emitters (Source: Karmeli and Keller, 1975 (Fig. 4.1)).

region are bounded by the minimum and maximum discharge expected from a test sample of emitters at each possible operating pressure. The change in operating pressure head ΔH_s in the subunit on a level field is caused by the friction loss. The average pressure head, H_a, which gives the average emitter discharge, q_a, is not midway between the extremes of pressure. This is because loss of pressure is greatest in the first part of constant-diameter manifolds and laterals where the flow rate is highest (see Fig. 20.9).

Sample Calculation 20.2. *Determine emission characteristics and EU in a subunit.*

GIVEN: The emitter characteristics depicted in Fig. 20.10, where,

$$q_a = 0.95 \text{ gph at } H_a = 40.0 \text{ ft}$$
$$\Delta H_s = 10 \text{ ft and } H_n = 37.5 \text{ ft, therefore, } H_x = 47.5 \text{ ft}$$
$$x = 0.72, \text{ and } v = 0.033$$

FIND: The minimum and maximum nominal emitter discharges, q_n and q_x, the emission uniformity, EU, of the subunit for $N'_p = 1$, and the net design q.

CALCULATIONS: Using Eq. 17.1:

$$K_d = \frac{0.95}{(40)^{0.72}} = 0.067$$

$$q_n = 0.067\,(37.5)^{0.72} = 0.91 \text{ gph}$$

$$q_x = 0.067(47.5)^{0.72} = 1.08 \text{ gph}$$

From Eq. 20.13a:

$$\text{EU} = 100\left(1.0 - 1.27\,\frac{0.033}{\sqrt{1}}\right)\frac{0.91}{0.95} = 92\%$$

Therefore, the net design q is:

$$q = q_a\,\text{EU}/100 = 0.95 \times 0.92 = 0.87 \text{ gph}$$

The application uniformity within a subunit is equivalent to the EU, because all the emitters are operated for the same application time, T_a. Selecting the ideal design EU requires economic tradeoffs between factors. These are: costs of systems having various EU values; water and water-related costs; sensitivity of crop yield and quality to nonuniform irrigation; and market values of the crop. A complete economic analysis involving these factors is required to determine the optimal EU in any specific situation. Unfortunately, there is seldom sufficient data for such an economic analysis. Therefore, for design purposes standard recommended EU values are generally used in conjunction with Eq. 20.13. The EU values recommended by the American Society of Agricultural Engineers in ASAE Standard EP 405.1 for different site conditions are presented in Table 20.3.

The minimum emitter discharge that will satisfy the desired EU value can be determined by solving Eq. 20.13 for q_n, i.e., using the q_a determined from Eq. 20.11 and the v_s for the selected emitter and layout.

The pressure head, H_n, which gives q_n for the selected emitter, can be determined from Eq. 17.1. From H_a and H_n the allowable variation in subunit pressure head, ΔH_s, can be computed for design purposes by:

$$\Delta H_s = 2.5(H_a - H_n) \tag{20.14}$$

where

ΔH_s = allowable variation in subunit pressure head that will give an EU reasonably close to the desired design value, m (ft)

Table 20.3 Recommended ranges of design emission uniformities, EU[1]

Emitter type	Emitters per plant	Topography	EU range, %
Point-source	≥ 3	Uniform[2]	90 to 95
Point-source	<3	Uniform	85 to 90
Point-source	≥ 3	Undulant[3]	85 to 90
Point-source	<3	Undulant	80 to 90
Spray	All	Uniform	90 to 95
Spray	All	Undulant	85 to 90
Line-source	All	Uniform	80 to 90
Line-source	All	Undulant	70 to 85

[1]Adapted in part from: American Society of Agricultural Engineers, 1988, Design and installation of microirrigation systems, ASAE Engineering Practice Standard: ASAE EP405.1.
[2]Uniform with slopes $\leq 2\%$.
[3]Undulant or steep with slopes $> 2\%$.

H_a = pressure head that will give the q_a required to satisfy Eq. 20.11, m (ft)
H_n = pressure head that will give the q_n required to satisfy Eq. 20.13 with the design EU, m (ft)

To satisfy the design EU, the pressure head must be held between H_n and $(H_n + \Delta H_s)$. If the calculated ΔH_s is too small to allow for both pipe friction and elevation differences, adjustments are necessary. The options are: to select another emitter that has a lower v, x, or both; use more emitters per plant to increase N_p; use a different emitter or rearrange the system to get a higher H_a; or relax the design EU requirement.

SYSTEM DISCHARGE

It is necessary to determine the system capacity and operating time per season to design a pumping plant and pipeline network that are economical and efficient.

Total System Capacity, Q_s

The system capacity is simply the maximum number of emitters operating at any given time multiplied by q_a. For balanced systems the capacity with any emitter layout can be computed by:

$$Q_s = K \frac{A}{N_s} \frac{N_p q_a}{S_p S_r} \tag{20.15a}$$

For uniformly spaced laterals that supply uniformly spaced emitters:

$$Q_s = K \frac{A}{N_s} \frac{q_a}{S_e S_l} \tag{20.15b}$$

where

Q_s = total system capacity, L/s (gpm)
K = conversion constant, 2.778 for metric units (726 for English units)
A = field area, ha (A)
N_s = number of operating stations

For line-source tubing where the discharge per standard length of tubing is given (rather than q_a per outlet), replace q_a/S_e in Eq. 20.15b with Q_a per m (ft).

Operating Time per Season, O_t

The pump operating time per season, O_t, can now be estimated from the volume of irrigation water required per season, V_s, determined by Eq. 19.19 and the system capacity, Q_s:

$$O_t = K \frac{V_s}{Q_s} \tag{20.16a}$$

or approximated by:

$$O_t \approx 1.1\ T_a \frac{D_n}{T_d} \tag{20.16b}$$

where O_t = average pump operating time per season, hr, and K = conversion constant, 2778 for metric units (5430 for English units).

Operational Considerations

Some systems require extra capacity because of anticipated slow changes in q_a with time. Decreases in q_a can result from such things as slow clogging due to sedimentation in long-path emitters or compression of resilient parts in compensating emitters. Increases in q_a can result from mechanical fatigue of the flexible orifices in continuously and periodic-flushing emitters or increases in minor leakage due to fatigue in emitters and tubing.

Both decreases and increases in q_a necessitate periodic cleaning or replacement of emitters. To compensate for a decrease in discharge rate, the system must be operated at either a higher pressure or for a longer time during each irrigation application. To prevent the need for frequent cleaning or replacement of emitters, where decreasing discharge rates are a potential problem, the system should be designed with 10 to 20% extra capacity. (By following the recommended design procedure based on a maximum of 21.6 hr/day operation during the peak use period, 10% extra capacity is already available.) A possible alternative is to provide sufficient reserve operating pressure so the pressure can be increased as required to hold q_a constant until the emitter discharge characteristics have degenerated by 10 to 20%.

Providing extra system capacity necessitates increasing the pump and pipe sizes, whereas providing reserve operating pressure requires only a slightly larger pump. Consequently, the added initial cost of providing reserve pressure is less than the cost of providing extra capacity. Nonetheless, systems that have extra capacity do have greater ability to make up for unavoidable interruptions before the emitter discharge has decreased. Furthermore, they can also handle situations when emitter discharge or minor leakage increases q_a and their energy requirements are lower.

NET APPLICATION RATE, I_n

The net application rate, I_n, is the application rate to the plants that receive water at the lowest application rate. The I_n is important for irrigation scheduling, because it is needed to calculate the number of hours the system must operate to apply a specific minimum depth or volume of water.

The I_n is a function of the minimum expected emitter discharge rate, q_n, and thus cannot be computed until the hydraulic network has been designed. The q_n is a function of the minimum expected pressure head, H_n, in the system, and it can be computed by rearranging Eq. 17.1 to give:

$$q_n = q_a \left(\frac{H_n}{H_a}\right)^x \tag{20.17}$$

The H_n in subunits or systems where the friction head loss is greater than the head gain due to elevation drop (which is the case for most systems) can be computed by:

$$H_n = (H_m - \Delta H_m - \Delta H_l) \tag{20.18}$$

where: H_m = manifold inlet pressure head, m (ft), ΔH_m = difference in pressure head along the manifold, m (ft), and ΔH_l = difference in pressure head along the laterals, m (ft).

Steep, downhill manifolds and laterals where the friction loss is less than the pressure gain due to elevation will have lower inlet pressures than intermediate pressures. In such cases, H_n must be determined by either inspection of the graphical solutions or mathematical analysis presented in Chapters 22 and 23.

With an estimate of q_n, the final design EU for the system can be computed by Eq. 20.13. Then the net application rate for the system can be computed by:

$$I_n = K \frac{\mathrm{EU}}{100} \frac{N_p q_a}{S_p S_r} \tag{20.19}$$

where I_n = net application rate, mm/hr (in./hr), and K = conversion constant, 1.0 for metric units (1.604 for English units)

The Trickle Irrigation Design Factors Form, Fig. 19.6, provides a convenient place to record the results of the computations outlined in this chapter. Three major Sample Calculations are presented in the following chapter. Individual Sample Calculations are presented for drip, spray, and line-source trickle systems. They were developed to demonstrate how to apply the information and equations presented up to this point.

REFERENCES

Karmeli, D., and J. Keller. 1975. *Trickle Irrigation Design*. Glendora, California: Rain Bird Sprinkler Manufacturing Corp.

Keller, J., and D. Karmeli, 1974. Trickle irrigation design parameters. *ASAE Transactions* 17(4):678–684.

Solomon, K. 1977. Performance characteristics of different emitter types. Proceedings 7th International Agricultural Plastics Congress, April 1977. San Diego, California, pp. 97–107.

Solomon, K. 1979. Manufacturing variation of trickle emitters. *ASAE Transactions* 22(5):1034–1038, 1043.

Watters, G. Z., and J. Keller. 1978. Trickle irrigation tubing hydraulics. *ASCE Technical Paper* No. 78-2015.

RECOMMENDED READING

James, L. G. 1988. *Principles of Farm Irrigation System Design*. New York: Wiley.

Van Bernuth, R. D., and K. Solomon. 1986. Design principles: Emitter construction. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama and D. A. Bucks, pp. 27–52. Amsterdam: Elsevier.

21

Trickle System Design Strategy

Several important design criteria affect trickle irrigation system efficiency. The most important of these are:

- Efficiency of filtration;
- Permissible variations of pressure head;
- Base operating pressure to be used;
- Degree of control of flow or pressure;
- Relation between discharge and pressure at the pump or hydrant supplying the system;
- Allowance for temperature correction for long-path emitters;
- Chemical treatment to dissolve or prevent mineral deposits:
- Use of secondary safety screening;
- Incorporation of flow monitoring; and
- Allowance for reserve system capacity or pressure to compensate for reduced flow due to clogging.

Strategies for use in designing trickle systems to meet the criteria presented in Chapters 19 and 20 follow. The strategies are best presented by examples and depend somewhat on the type of trickle system being considered. Therefore, three extensive Sample Calculations are presented to provide examples for drip, spray, and line-source trickle systems. Because Sample Calculation 21.1 for drip systems is presented first, it contains the most detail. In Sample Calculations 21.2 and 21.3, only the additional new concepts specific to line-source and spray systems are elaborated.

In addition to illustrating the general process for designing a drip irrigation system, the following procedures are emphasized in Sample Calculation 21.1:

1. Selecting the emitter (or emission point) spacing, the duration of application, the number of stations, and the average emitter discharge and operating pressure head (S_e, S_l, T_a, N_s, q_a, and H_a, respectively);
2. Determining the allowable variation in pressure head that will produce the desired uniformity of emission (ΔH_s).

Once the basic system design has been decided upon, such details as pipe sizing and pump selection must be worked out. Procedures for tackling these problems are covered in Chapter 8 on pipeline hydraulics, Chapters 22 through 24 on trickle irrigation pipeline networks, and Chapter 12 on pump selection. In Chapters 22, 23, and 24 the following are emphasized: for Sample Calculation 21.1:

3. Positioning of manifolds and designing laterals for level and sloping rows;
4. Designing the manifold and selecting economical pipe sizes for both manifolds and main lines; and
5. Computing system capacity and total dynamic operating head requirements.

Sample Calculation 21.1. *Determining the drip irrigation system design factors for an orchard.*

GIVEN: A typical almond (deciduous) orchard in the Central Valley of California. The data that must be collected prior to beginning the design computations are summarized in the Trickle Irrigation Design Data form, see Fig. 21.1, column headed *DRIP*. The orchard layout is shown in Fig. 21.2.

FIND: The design factors for a drip irrigation system and fill in the column headed *DRIP* in Fig. 21.3 through Part III-m.

CALCULATIONS: The general sequence of steps for developing the factors is outlined in Fig. 21.3. the form for the Trickle Irrigation Design Factors. This form provides a useful guide and a convenient place to record results of the various trial and final computations.

Emitter Spacing, S_e.—Field observations of emitter wetting patterns were made at several sites with trickle irrigation systems in the same area. The wetted diameter, w, produced by emitters discharging 1.0 gph was between 8 and 9 ft. To have a continuous wetted strip, the spacing between emitters in the row should not exceed 80% of the wetted diameter. Therefore, for the 24-ft tree spacing, a uniform emitter spacing of $S_e = 6.0$ ft was selected. (Table 19.1 could have been used for predicting the area that would be wetted by an emitter; however, field test data or observations at existing systems are preferable.)

Percent Area Wetted, P_w.—Try a single line of emitters with an emitter spacing of $S_e = 6.0$ ft and lateral spacing the same as the row spacing, $S_l = S_r = 24$ ft. Then the number of emitters per tree for the tree spacing of $S_p = 24$ ft in the row would be:

$$N_p = \frac{S_p}{S_e} = \frac{24}{6} = 4$$

		DRIP	LINE-SOR	SPRAY
I. PROJECT				
II. WATER AND LAND				
(a) Field number	(Fig.)	21.2	21.4	21.5
(b) Field area – ha (A)	A	115.7	4.7	32.2
(c) Effective rain – mm (in.)	R_n	3.7	3.5	39.0
(d) Residual soil water – mm (in.)	M_s	0	2.0	1.7
(e) Water supply – L/s (gpm)		800	200+	pit
(f) Water storage – ha-m (A-ft)		–	–	–
(g) Water quality – dS/m (mmhos/cm)	EC_w & ~~SAR~~	1.4	1.0	0.3
(h) Water quality classification		Good	Good	Excellent
III. SOIL AND CROP				
(a) Soil texture		Silt loam	Clay loam	Fine sand
(b) Available water capacity – mm/m (in./ft)	W_a	1.8	2.1	0.7
(c) Soil depth – m (ft)		10	6+	10.0
(d) Soil limitations		none	none	none
(e) Management allowed deficiency – %	MAD	30	30	30
(f) Crop		Almonds	Tomatoes	Citrus
(g) Plant spacing – m x m (ft x ft)	$S_p \times S_r$	24 x 24	3 x 5	15 x 25
(h) Plant root depth – m (ft)	Z	6	2.5	6.0
(i) Percent shaded area – %	P_d	66	50	75
(j) Average peak ET – mm/day (in./day)	U_d	0.28	0.28	0.22
(k) Seasonal water requirement – mm (in.)	U	36.7	25.0	48.0
(l) Leaching requirement – ratio	LR_t	0.10	0.04	0.02
IV. EMITTER				
(a) Type		Vortex	Mono-wall tubing	280° spray
(b) Outlets per emitter		1	1	1
(c) Pressure [head] – kPa [m] (psi [ft])	P [~~H~~]	15.0	4.0	25.0
(d) Rated discharge @ H – L/hr (gph)	q	1 0	0 39	11 3
(e) Discharge exponent	x	0 42	0 48	0 556
(f) Coefficient of variability	v	0.07	0.12	0.042
(g) Discharge coefficient	K_d	0.32	0.20	1 89
(h) Connection loss equivalent – m (ft)	f_e	0.4	N/A	0.4

FIG. 21.1. Trickle Irrigation Design Data in English Units.

Assuming an average wetted width of $w = 8.5$ ft, by Eq. 19.3 the percentage area wetted would be:

$$P_w = \frac{N_p S_e w}{S_p S_r} 100 = \frac{4 \times 6.0 \times 8.5}{24 \times 24} = 35\%$$

This meets the design criteria of having $P_w > 33\%$.

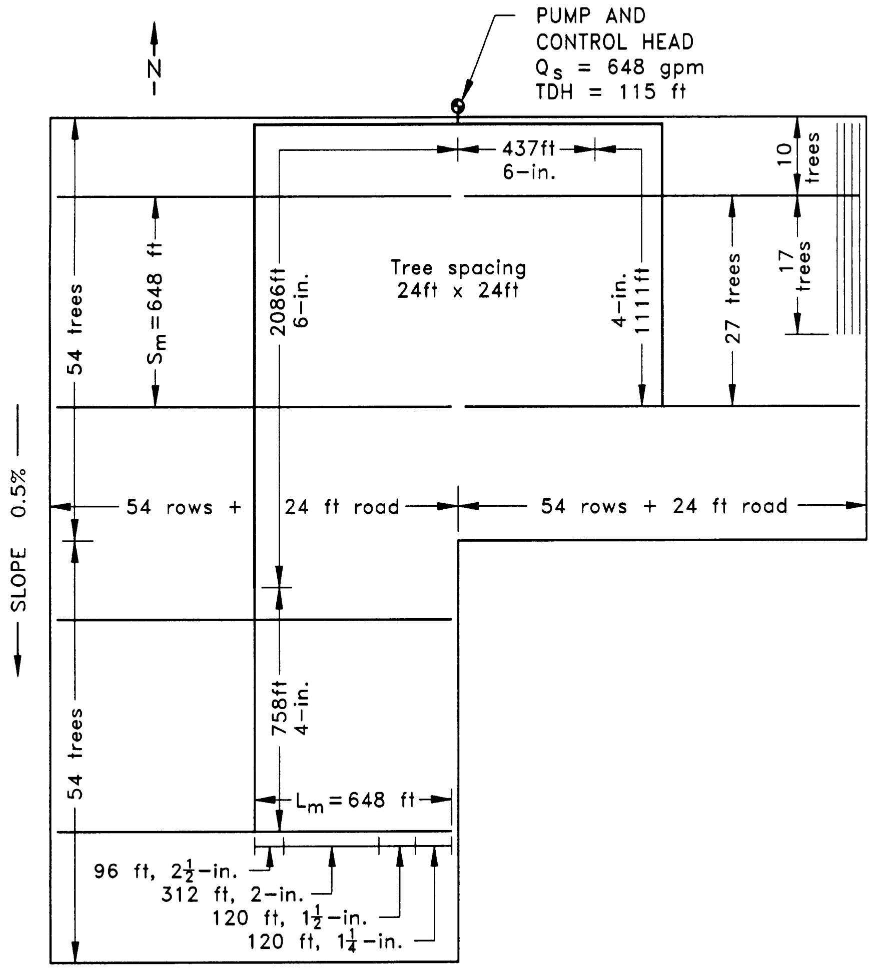


FIG. 21.2. Orchard Layout with Sample Design for a Drip Irrigation System (Lateral Lines are 0.58-Inch PE, Manifolds are SDR 26 PVC, and Main Lines are SDR 41 PVC).

Maximum Net Depth, d_x.—Using $P_w = 35\%$ and the values from Fig. 21.1 of MAD = 30%, $W_a = 1.8$ in./ft, and $Z = 6$ ft in Eq. 19.12 gives:

$$d_x = \frac{\text{MAD}}{100}\frac{P_w}{100}W_a Z = \frac{30}{100} \times \frac{35}{100} \times 1.8 \times 6 = 1.15 \text{ in.}$$

Average Peak Transpiration Rate, T_d.—From Fig. 21.1 the average peak consumptive use rate of almonds computed by conventional means is $U_d = 0.28$ in./day. From field observation the percent area shaded is $P_d = 66\%$. Thus, the average peak transpiration rate under drip irrigation would be:

$$T_d = U_d\left[0.1(P_d)^{0.5}\right] \qquad (19.9)$$

$$= 0.28\left[0.1(66)^{0.5}\right] = 0.23 \text{ in./day}$$

I. PROJECT		DRIP	LINE-SOR	SPRAY
II. TRIAL DESIGN				
(a) Emission point layout		St. Line	Line	St. Line
(b) Emitter spacing – m x m (ft x ft)	$S_e \times S_l$	6 x 24	1 5 x 5	15 x 25
(c) Emission points per plant	N_p	4	2	1
(d) Percent wetted area – %	P_w	35	100	46
(e) Maximum net depth – mm (in.)	d_x	1.15	1.6	0.58
(f) Ave. peak transpiration – mm/day (in./day)	T_d	0.23	0.20	0.19
(g) Maximum interval – days	f_x	5	8	3
(h) Irrigation frequency – days *	f'	1	1	1
(i) Net depth per irrigation – mm (in.)	d_n	0.23	0.20	0.19
(j) Assumed uniformity – %	EU	90	80	90
(k) Gross depth per irrigation – mm (in.)	d	0.26	0 25	0 22
(l) Gross water per plant – L/day (gal/day)	G	93 3	2.34	51 4
(m) Application time – hr	T_a	23 3	3.00	4 55
III. FINAL DESIGN				
(a) Application time – hr	T_a	21.0	3 00	4 50
(b) Irrigation interval – days *	f'	1	1	1
(c) Gross depth per irrigation – mm (in.)	d	0.26	0.25	0.22
(d) Average emitter discharge – L/hr (gph)	q_a	1.11	0.39	11 44
(e) Average emitter head – m (ft)	H_a	44.5	9.2	58 5
(f) Allowable head variation – m (ft)	ΔH_s	16 0	5.8	13 2
(g) Emitter spacing – m x m (ft x ft)	$S_e \times S_l$	6 x 24	1 5 x 5	15 x 25
(h) Percent wetted area – %	P_w	35	100	46
(i) Number of stations	N_s	1	1	4
(j) System capacity – L/hr (gpm)	Q_s	648	177	178
(k) Seasonal efficiency – %	E_s	90	80	77
(l) Seasonal irrigation – ha-m (A-ft)	V	319	7.0	22 5
(m) Seasonal operation – hr	O_t	2680	215	686
(n) Total dynamic head – m (ft)	TDH	115	82	140
(o) Actual uniformity – %	EU	91 5	81	90
(p) Net application rate – mm/hr (in./hr)	I_n	0.0113	0.068	0 044

* Use a 1-day interval during the design process, then adjust f' up to f_x by multiplying the design T_a by the final f'.

FIG. 21.3. Trickle Irrigation Design Factors in English Units.

Maximum Irrigation Interval, f_x.—Using the values computed above in Eq. 19.13b gives:

$$f_x = \frac{d_x}{T_d} = \frac{1.15}{0.23} = 5 \text{ days}$$

Irrigation Interval, f'.—For design purposes it is usually most convenient to assume $f' = 1$ day because management can easily adjust the interval up to $f' = f_x$ by multiplying the design application time, T_a, by the selected f'.

Net Depth per Irrigation, d^n.—For daily irrigation, $f' = 1$ day, $d_n = T_d = 0.23$ in. (see Eq. 19.13a).

Emission Uniformity, EU.—From Table 20.3, EU = 90% is a reasonable design target value for the specific site conditions and emitter selected. Selecting the EU at the lower end of the range gives a larger allowable pressure variation in the hydraulic design. This in turn should give a lower cost pipe network than one with a smaller pressure variation.

Leaching Requirement, LR_t.—Obtain the min $EC_e = 1.5$ mmhos/cm and max $EC_e = 7$ mmhos/cm for almonds from Table 19.2. With proper leaching, yields will not be reduced, since $EC_W <$ min EC_e, i.e., $1.4 < 1.5$ mmhos/cm. By Eq. 19.8 the leaching requirement is:

$$LR_t = \frac{EC_W}{2(\max EC_e)} = \frac{1.4}{2(7)} = 0.10$$

Gross Depth per Irrigation, d.—From the peak period transpiration ratio values presented in Table 19.3, $T_r = 1.00$ for the deep-rooted almond trees on the medium-textured (silt loam) soil. This implies that there should be no unavoidable losses to satisfy the leaching requirement. However, since $LR_t \leq 0.10$, use Eq. 19.15a to find:

$$d = \frac{d_n T_r}{\mathrm{EU}/100} = \frac{0.23}{90/100} = 0.26 \text{ in.}$$

Gross Water Required per Plant per Day, G.—Using Eq. 19.16a obtain:

$$G = K \frac{d}{f} S_p S_r \tag{19.16a}$$

$$= 0.623 \times \frac{0.26}{1} \times 24 \times 24$$

$$= 9.3. \text{ gal/day}$$

Application Time, T_a.—Using the rated emitter discharge of $q = 1.0$ gph in Eq. 20.11:

$$T_a = \frac{G}{N_p q_a} = \frac{93.3}{4 \times 1.0} = 23.3 \text{ hr/day}$$

To allow for a reasonable degree of safety, T_a should be reduced to no more than 90% of 24 hr = 21.6 hr/day. Since T_a is nearly 24 hr, only one station, $N_s = 1$, will be used for the system.

Average Emitter Discharge, q_a.—Letting $T_a = 21.0$ hr and rearranging Eq. 20.11 gives:

$$q_a = \frac{G}{N_p T_a} = \frac{93.3}{4 \times 21.0} = 1.11 \text{ gph}$$

Average Emitter Pressure Head, H_a.—A vortex emitter with a rated $q = 1.0$ gph at 15 psi $= 34.7$ ft and $x = 0.42$ is specified in Fig. 21.1. The pressure head that will give $q_a = 1.11$ gph can be computed by Eq. 20.12a as:

$$H_a = H\left[\frac{q_a}{q}\right]^{1/x} = 34.7\left[\frac{1.11}{1.00}\right]^{1/0.42} = 44.5 \text{ ft}$$

Allowable Subunit Head Variation, ΔH_s.—A subunit is that part of the system beyond the last pressure-regulation point. For example, if a valve is used to adjust the inlet pressure to a manifold that has no other pressure regulator, the area served by the manifold is a subunit. The object is to limit the pressure variation within each subunit, so the actual emission uniformity will equal or exceed the target design value of EU $= 90\%$.

Rearranging Eq. 20.13a, to determine the minimum permissible discharge q_n, for the vortex emitters with $v = 0.07$ and $n_p = N'_p = 4$ gives:

$$q_n = \frac{q_a \text{ EU}/100}{1.0 - 1.27\, v/\sqrt{N'_p}} \qquad (20.13a')$$

$$= \frac{1.11 \times 90/100}{(1.0 - 1.27 \times 0.07/\sqrt{4})} = 1.04 \text{ gph}$$

The minimum permissible pressure head, H_n, that will give q_n can be found by replacing q and H in Eq. 20.12a with q_n and H_n and rearranging to obtain:

$$H_n = H_a\left[\frac{q_n}{q_a}\right]^{1/x} = 44.5\left[\frac{1.04}{1.11}\right]^{1/0.42} = 38.1 \text{ ft}$$

And by substituting into Eq. 20.14:

$$\Delta H_s = 2.5(H_a - H_n) = 2.5(44.5 - 38.1) = 16.0 \text{ ft}$$

Total System Capacity, Q_s.—The design layout has uniformly spaced laterals that supply uniformly spaced emitters. Therefore, with $A = 115.7$ A, by Eq. 20.15b:

$$Q_s = K\frac{A}{N_s}\frac{q_a}{S_e S_l} = 726\,\frac{115.7}{1}\,\frac{1.11}{6 \times 24} = 647 \text{ gpm}$$

Seasonal Irrigation Efficiency, E_s.—From Table 19.4 the potential seasonal transpiration ratio $T_R = 1.00$. This assumes careful irrigation scheduling and no runoff or pipeline leakage. Since $T_R \le 1.0/(1.0 - LR_t)$, i.e., $1.0 <$

1.0/(1.0 − 0.10), by Eq. 19.14a:

$$E_s = \text{EU} = 90\%$$

Gross Seasonal Volume of Irrigation Water Required, V_s.—To compute V_s, first the net and gross seasonal irrigation depths, D_n and D_g, must be computed. Substituting design data values from Fig. 21.1 into Eq. 19.11 gives:

$$D_n = (U - R_n - M_s)[0.1(P_d)^{0.5}] \tag{19.11}$$

$$= (36.7 - 3.7)[0.1(66)^{0.5}] = 26.8 \text{ in.}$$

Because $T_R \leq 1.0/(1.0 - LR_t)$, by Eq. 19.18b:

$$D_g = \frac{D_n}{(1.0 - LR_t)\ \text{EU}/100} = \frac{26.8}{(1.0 - 0.1)90/100} = 33.1 \text{ in.}$$

And by Eq. 19.19:

$$V_s = \frac{D_g A}{K} = \frac{33.1 \times 115.7}{12} = 319 \text{ A-ft}$$

Operation Time per Season, O_t.—The number of hours the pumping plant must be operated per year can now be computed by Eq. 20.16a:

$$O_t = K\frac{V_s}{Q_s} = 5430\,\frac{319}{647} = 2680 \text{ hr}$$

Sample Calculation 21.2. *Determining line-source system design factors for a tomato crop.*

GIVEN: A typical, staked tomato field in Texas. The data that should be collected prior to beginning the design are summarized in the Trickle Irrigation Design Data Form, Fig. 21.1, under the column headed *LINE-SOR*. The field layout is shown in Fig. 21.4 and the mono-wall line-source tubing has an 18-in. outlet spacing.

FIND: The line-source trickle irrigation system design factors, and fill the column headed *LINE-SOR* in Fig. 21.3 through Part III-m. In addition to illustrating the general line-source irrigation design process, the following procedures will be emphasized in Chapters 22 and 23 using this example layout:

1. Calculating EU for line-source tubing; and
2. Graphical design of downhill manifold so that friction slope closely follows ground slope.

CALCULATIONS: The design computations that follow are as brief as possible, except for concepts that were not included in Sample Calculation 21.1 covering drip systems.

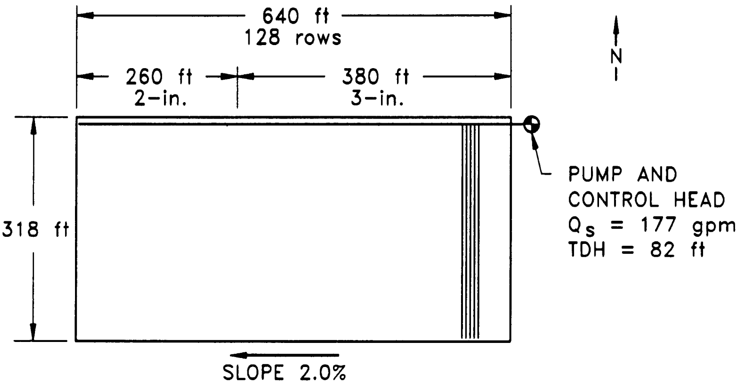


FIG. 21.4. Tomato Field with Line-source Drip Irrigation System (Lateral Lines are Single Chamber 0.625-Inch ID PE Tubing Which Discharges 26 gph/100 ft; The Manifold is Buried PVC Pipe).

For a small field with a large water supply, it is not necessary to compute all the design factor elements included in Fig. 21.3. This is because the entire system can be operated simultaneously, and the irrigation takes only about 3 hr per day. Thus full irrigation could be achieved with a water delivery rate one-sixth as large or six times as much land could be irrigated with the same water supply. If either were so, all the design factor elements would be needed. Therefore, the column headed *LINE-SOR* in Fig. 21.3 has been filled out through Part III-m, and a brief summary of the computations is included.

Percentage Wetted, P_w.—From Table 19.1, for a fine-textured stratified shallow soil, $w = 5.0$ ft and by Eq. 19.3 with $S_e = 1.5$ ft:

$$P_w = \frac{2 \times 1.5 \times 5}{3 \times 5} \times 100 = 100\%$$

Maximum Net Depth of Irrigation, d_x.—By Eq. 19.12:

$$d_x = \frac{30}{100} \times \frac{100}{100} \times 2.1 \times 2.5 = 1.6 \text{ in.}$$

Average Peak Daily Rate of Transpiration, T_d.—By Eq. 19.9, assuming $P_d = 50\%$:

$$T_d = 0.28\left[0.1(50)^{0.5}\right] = 0.20 \text{ in./day}$$

Maximum Irrigation Interval, f_x.—By Eq. 19.13b:

$$f_x = \frac{1.6}{0.20} = 8 \text{ days}$$

Leaching Requirement, LR_t.—From Table 19.2, for tomatoes, max EC_e = 12.5 mmhos/cm and by Eq. 19.8 with EC_w = 1.0 mmhos/cm:

$$LR_t = \frac{1.0}{2(12.5)} = 0.04$$

Gross Daily Irrigation Depth, d'.—For $T_r = 1.00$ from Table 19.3 and a target EU = 80% from Table 20.3 use Eq. 19.15b (since $LR_t < 0.10$) to find:

$$d' = \frac{0.20 \times 1.00}{80/100} = 0.25 \text{ in.}$$

Gross Water Required per Plant per Day, G.—By Eq. 19.16b:

$$G = 0.623 \times 0.25 \times 3.0 \times 5.0 = 2.34 \text{ gal/day}$$

Application Time, T_a.—Using the rated outlet discharge of $q = 0.39$ gph in Eq. 20.11:

$$T_a = \frac{2.34}{2 \times 0.39} = 3.0 \text{ hr/day}$$

Final Design Part III of Fig. 21.3.—Under the column headed *LINE-SOR*, lines a), b), c), d), e), g), and h), in the Final Design, Part III of Fig. 21.3 are repeats of the data already computed. This is because no adjustment in the application time was called for.

Allowable Subunit Head Variation, ΔH_s.—There are only two outlets per plant. However, the water spread is over 4 ft, so each tomato plant will have access to water from at least three outlets. Thus, $N_p' = 3$, and by rearranging Eq. 20.13a find:

$$q_n = \frac{0.39 \times 80/100}{(1.0 - 1.27 \times 0.12/\sqrt{3})} = 0.34 \text{ gph}$$

The pressure head H_n that gives $q_n = 0.34$ gph can now be determined from Eq. 20.12a using the rated $q = 0.39$ gph at 4 psi = 9.2 ft:

$$H_n = 9.2\left[\frac{0.34}{0.39}\right]^{1/0.48} = 6.9 \text{ ft}$$

And by Eq. 20.14:

$$\Delta H_s = 2.5(9.2 - 6.9) = 5.8 \text{ ft}$$

Total System Capacity, Q_s.—For a single-station design with $N_s = 1$, the system capacity by Eq. 20.15b is:

$$Q_s = 726 \times \frac{4.70}{1} \times \frac{0.39}{1.5 \times 5.0} = 177 \text{ gpm}$$

Seasonal Irrigation Efficiency, E_s.—From Table 19.3 with excellent scheduling and operation, $T_R = 1.00$ for the fine-textured 2.5-ft root zone. Since $T_R < 1/(1.0 - LR_t)$, use Eq. 19.14a to obtain:

$$E_s = \text{EU} = 80\%$$

Gross Seasonal Volume of Irrigation, V_s.—First by Eq. 19.11 find the net seasonal irrigation depth:

$$D_n = (25.0 - 3.5 - 2.0)\left[0.1(50)^{0.5}\right] = 13.8 \text{ in.}$$

Then by combining Eqs. 19.18b and 19.19 find:

$$V_s = \frac{D_n A}{K(1.0 - LR_t)\ \mathrm{EU}/100}$$

$$= \frac{13.8 \times 4.70}{12(1.0 - 0.04)80/100} = 7.0 \text{ A-ft}$$

Operating Time per Season, O_t.—By Eq. 20.16a:

$$O_t = \frac{5430 \times 7.0}{177} = 215 \text{ hr}$$

Sample Calculation 21.3. *Determining spray system design factors for a citrus grove in a humid area.*

GIVEN: A typical citrus grove on a homogeneous, fine sandy soil in Florida. The data that must be collected prior to beginning the design computations are summarized in the Trickle Irrigation Design Data Form, Fig. 21.1, under the column headed *SPRAY*. The field layout is shown in Fig. 21.5.

FIND: The spray irrigation system design factors, and fill the column headed *SPRAY* in Fig. 21.3 through Part III-m. In addition to illustrating the general spray irrigation design process, the following procedures will be emphasized in Chapters 22 and 23 using this example layout:

1. Manifold spacing for multistation systems;
2. Economic pipe sizing for tapered manifolds (both graphical and adjusted economic chart method solutions) on rectangular fields; and
3. Pipe sizing for tapered manifolds on nonrectangular fields.

CALCULATIONS: Example design computations that were elaborated in Sample Calculation 21.1 for drip system design are presented in a briefer form in the sections that follow. The values obtained for the spray irrigation design factors are presented in Fig. 21.3 under the column headed *SPRAY*.

The particular spray emitter selected wets a butterfly-shaped pattern (see Fig. 20.6B) that can be approximated by a circle with two 40° pie-shaped wedges removed. (The wedges are opposite each other and result from water being deflected by supports that hold a miniature deflector cap above a small vertical nozzle.) The diameter of the wetted circle and the nozzles' discharge are both

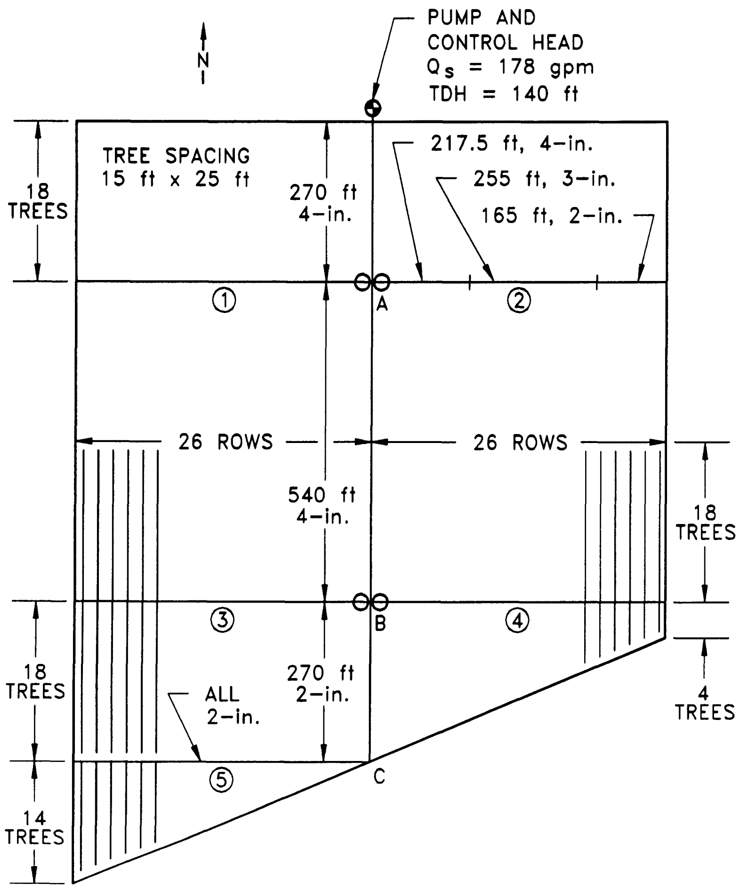


FIG. 21.5. Citrus Grove with Spray Irrigation System (Lateral Lines Are 0.70-Inch PE and Manifolds and Main Lines are PVC Pipe).

functions of the operating pressure. From information provided by the manufacturer, the emitter exponent and coefficient of discharge are $x = 0.556$ and $K_d = 1.89$, respectively. The relationship between pressure and the wetted diameter is as shown in Fig. 21.6.

Percent Area Wetted, P_w.—The surface area, A_s, directly wetted by the spray when operating at the rated pressure of 25 psi has a diameter of 14.5 ft. With the two 40° pie-shaped wedges removed this gives:

$$A_s = \frac{(14.5)^2 \pi}{4} \times \frac{280}{360} = 128 \text{ ft}^2$$

The wetted soil area is larger than the surface area wetted because there is some outward soil water movement, as shown in Fig. 19.4. To estimate the total wetted soil area add a strip $S_e'/2$ wide around the perimeter of the wetted sur-

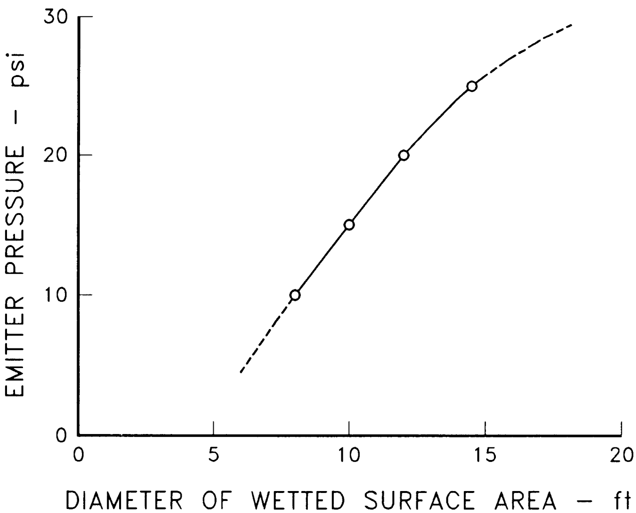


FIG. 21.6. A Plot of Spray Diameter Versus Emitter Pressure. Developed from Manufacturer's Data for a 0.04-Inch Diameter Orifice.

face soil, *PS*. For butterfly-type wetting patterns, *PS* can be assumed equal to the circumference of the full circle thus:

$$PS = 14.5\pi = 46 \text{ ft}$$

From Table 19.1 $S_e' = 2.0$ ft for a homogeneous coarse-textured soil, and the percentage of soil area wetted would be:

$$P_w = \frac{N_p A_s + (S_e' \times PS)/2}{S_p S_r} \times 100 \tag{19.5}$$

$$= \frac{1 \times 128 + (2.0 \times 46)/2}{15 \times 25} \times 100 = 46\%$$

Maximum Net Depth of Irrigation, d_x.—By Eq. 19.12:

$$d_x = \frac{30}{100} \times \frac{46}{100} \times 0.7 \times 6.0 = 0.58 \text{ in.}$$

Average Peak Daily Rate of Transpiration, T_d.—By Eq. 19.9:

$$T_d = 0.22[0.1(75)^{0.5}] = 0.19 \text{ in./day}$$

Maximum Irrigation Interval, f_x.—By Eq. 19.13b:

$$f_x = 0.58/0.19 = 3 \text{ days}$$

Leaching Requirement, LR_t.—From Table 19.2 for citrus, the max $EC_e = 8$ mmhos/cm, and by Eq. 19.8 with $EC_w = 0.3$ mmhos/cm:

$$LR_t = \frac{0.3}{2(8)} = 0.02$$

Gross Daily Irrigation Depth, d'.—Obtain $T_r = 1.05$ from Table 19.3 and a design target EU = 90% from Table 20.3; substituting in Eq. 19.15b (since $LR_t < 0.10$:) gives:

$$d' = \frac{0.19 \times 1.05}{90/100} = 0.22 \text{ in.}$$

Gross Water Required per Plant per Day, G.—By Eq. 19.16b:

$$G = 0.623 \times 0.22 \times 15 \times 25 = 51.4 \text{ gal/day}$$

Application Time, T_a.—Using the rated spray emitter discharge of $q = 11.3$ gph in Eq. 20.11:

$$T_a = \frac{51.4}{1.0 \times 11.3} = 4.55 \text{ hr/day}$$

Round off to 4.5 hr/day, and use $N_s = 4$ to give 18 hr/day operation.

Average Emitter Discharge, q_a.—Letting $T_a = 4.5$ hr and rearranging Eq. 20.11 gives:

$$q_a = \frac{51.40}{1.0 \times 4.5} = 11.4 \text{ gph}$$

Average Emitter Pressure, P_a.—By substituting P_a for H_a in Eq. 20.12b:

$$P_a = \left[\frac{q_a}{K_d}\right]^{1/x} = \left[\frac{11.4}{1.89}\right]^{1/0.556} = 25.3 \text{ psi; } H_a = 58.5 \text{ ft}$$

Allowable Subunit Head Variation, ΔH_s.—By Eq. 20.13a:

$$q_n = \frac{11.4 \times 90/100}{(1.0 - 1.27 \times 0.042/\sqrt{1.0})} = 10.8 \text{ gph}$$

The P_n that gives q_n can be found by using Eq. 20.12b as above:

$$P_n = \left[\frac{10.8}{1.89}\right]^{1/0.556} = 23.0 \text{ psi}$$

And by Eq. 20.14 using pressures P_a and P_n in place of H_a and H_n gives:

$$\Delta H_s = 2.31\,[2.5(25.3 - 23.0)] = 2.31\,(5.7 \text{ psi}) = 13.2 \text{ ft}$$

Total System Capacity, Q_s.—For $N_s = 4$ and $A = 32.2$ A the system capacity by Eq. 20.15b is:

$$Q_s = 726 \times \frac{32.2}{4} \times \frac{11.4}{15 \times 25} = 178 \text{ gpm}$$

Seasonal Irrigation Efficiency, E_s.—Entering Table 19.4 midway between the coarse and very coarse soil texture columns for humid zones and for a root depth over 5 ft, find $T_R = 1.15$. Adding 0.05 for spray emitters in humid areas gives $T_R = 1.20$. Since the unadjusted $T_R > 1.0/(1.0 - LR_t)$, i.e., $1.15 > 1.0/(1.0 - 0.02) = 1.02$, use Eq. 19.14b to compute E_s as:

$$E_s = \frac{\text{EU}}{T_R(1.0 - LR_t)} = \frac{90}{1.20(1.0 - 0.02)} = 77\%$$

Gross Seasonal Volume of Irrigation Water Required, V_s.—The net annual irrigation depth by Eq. 19.11 is:

$$D_n = (48.0 - 39.0 - 1.7)\left[0.1 + (75)^{0.5}\right] = 6.3 \text{ in.}$$

Since the unadjusted $T_R > 1.0/(1.0 - LR_t)$, combine Eqs. 19.18a and 19.19 to obtain:

$$V_s = \frac{D_n T_R A}{K\,\text{EU}/100} = \frac{6.3 \times 1.20 \times 32.2}{12 \times 90/100} = 22.5 \text{ A-ft}$$

Operating Time per Season, O^t.—By Eq. 20.16a:

$$O_t = \frac{5430 \times 22.5}{178} = 686 \text{ hr}$$

22

Trickle Lateral Design

In trickle irrigation systems the lateral lines are the pipes on which the emitters are installed. They receive water from the manifolds and are usually made of plastic tubing ranging in diameter from 12 mm ($\frac{1}{2}$ in.) to 25 mm (1 in.). Laterals with only one diameter tubing (nontapered) are normally recommended to simplify installation and maintenance and provide better flushing characteristics.

Many systems have pairs of laterals extending in opposite directions from the manifold. In this chapter, simple graphical and numerical solutions are developed for both single and pairs of nontapered laterals on uniform slopes.[1] The development includes sections on the hydraulics of small-diameter plastic tubing.[2]

The procedures include determining such lateral characteristics as: flow rate and inlet pressure; locating and spacing the manifolds, which in effect sets the lateral lengths; and estimating the differences in pressure within laterals.

On fields where the average slope in the direction of the laterals is less than 3%, it is usually most economical to supply laterals to both sides of each manifold. The manifold should be positioned so that, starting from a common manifold connection, the minimum pressures along the pair of laterals (one to either side of the manifold) are equal. Thus, on level ground the laterals to either side of the manifold should have equal lengths.

Where the ground slopes in the direction of the laterals (rows), the manifold should be shifted uphill as in Fig. 17.3. The effect is to shorten the upslope laterals and lengthen the downslope laterals so the combination of pipe-friction losses and elevation differences is in balance. This shifting can be determined either graphically or numerically.

Spacing of manifolds is a compromise between field geometry and lateral hydraulics. To set practical limits for preliminary design purposes, it is useful to limit the lateral pressure head difference, ΔH_l, to 0.5 ΔH_s, where the manifold plus attached laterals make up a subunit. The ΔH_l for a given manifold

[1]The graphical procedures used herein were first presented by Keller (1980). The numerical design concepts were first presented by Keller and Rodrigo (1979).

[2]The concepts on small-pipe hydraulics and emitter-connection losses were originally presented by Watters and Keller (1978).

spacing and set of lateral specifications is about the same for level fields as for laterals that have slopes of as much as 2.5%. Utilizing this fact helps in computing the manifold spacing, S_m, and in designing the layout of the pipeline network.

GENERAL LATERAL CHARACTERISTICS

The following general characteristics of laterals are necessary or useful for the design process.

Hydraulics

For small-diameter, smooth pipes the Darcy-Weisbach and Blasius equations can be combined to give accurate predictions of friction-head loss (see Chapter 8). A simple equation that approximates the loss through smooth plastic pipes and hoses less than 125 mm (5 in.) in diameter is:

$$J = \frac{100 h_f}{L} = K \frac{Q^{1.75}}{D^{4.75}} \tag{8.7a}$$

where:

J = head loss gradient, m/100 m (ft/100 ft)
h_f = pipe friction head loss, m (ft)
K = constant, 7.89×10^7 for metric units (0.133 for English units) for water at 20°C (68°F)
Q = flow rate, L/s (gpm)
L = pipe length, m (ft)
D = inside diameter of pipe, mm (in.)

Each emitter connection causes some additional head loss. It is convenient to take the emitter-connection-friction loss into account as an equivalent length of lateral, f_e, using values from Fig. 20.8. To obtain an equivalent head-loss gradient, J can be adjusted to:

$$J' = J \frac{S_e + f_e}{S_e} \tag{22.1}$$

where:

J' = equivalent head loss gradient of the lateral with emitters, m/100 m (ft/100 ft)

S_e = spacing between emitter connections along the lateral, m (ft)
f_e = emitter connection loss as an equivalent length of lateral, m (ft)

The head loss for laterals that have evenly spaced outlets with uniform discharge from each outlet can be estimated by modifying Eq. 8.8 to:

$$h_f = J'FL/100 \tag{22.2}$$

where F = reduction coefficient to compensate for the discharge along the pipe.

Values of F for laterals with a flow-rate exponent of $b = 1.75$ and different numbers of outlets are given in Table 8.7. Usually it can be assumed that $F = 0.36$, because trickle laterals almost always have more than 15 outlets.

Dimensionless Friction Relationship

The general friction-loss characteristics for multioutlet pipelines are given in Eqs. 8.10a and 8.10b for any flow exponent b. The form of these equations presented below is developed specifically for small-diameter plastic hoses and pipes with barbed emitters. They assume $b = 1.75$ and account for f_e and are extensively used in the following trickle lateral design procedures.

The flow rate at any point along a lateral is $(x/L)Q$. Replacing Q in Eq. 8.7a with $(x/L)Q$ and combining with Eq. 22.1 gives the equivalent head-loss gradient at x equal to $J'(x/L)^{1.75}$. Substituting into Eq. 22.2 and replacing L with $(x/L)L$ gives:

$$h_{fx} = J'F\frac{L}{100}\left[\frac{x}{L}\right]^{2.75} \tag{22.3a}$$

which can be reduced (by combining with Eq. 22.2) to:

$$h_{fx} = h_f\left[\frac{x}{L}\right]^{2.75} \tag{22.3b}$$

where:

h_{fx} = friction-head loss from x to the closed end, m (ft)
x = distance from the closed end of a lateral, m (ft)
L = length of the lateral, m (ft)
h_f = friction-head loss in lateral with length L, m (ft)

Equation 22.3 defines the pipe friction curve for small-diameter plastic pipe. A dimensionless pipe-friction curve can be made by plotting the dimensionless ratios of x/L and dividing the corresponding h_{fx} by the total friction loss, h_f, as shown in Fig. 8.2. Table 8.8 lists the data points for this curve.

Tapered Laterals

Usually constant-diameter laterals are used because they are convenient to install and to maintain, but tapered laterals may be less expensive. Tapered laterals are sometimes needed on steep slopes. This is because the increase in pressure due to the slope would result in too much pressure at the end of a constant-diameter lateral. However, it is impractical to use tapered laterals with more that two pipe diameters for trickle systems.

For tapered laterals, h_f can be computed in a three-step process:

Step 1. Compute h_f by Eq. 22.2 for the full length of the lateral using J' for the larger diameter pipe.

Step 2. Compute the difference in h_f values between using J' values for the small- and large-diameter pipes and L and F for the length of small-diameter pipe in Eq. 22.2.

Step 3. The h_f for the tapered lateral will equal the h_f found in Step 1 plus the difference in the two h_f values found in Step 2.

The important thing to note in computing h_f for tapered laterals is that all computations involving Eq. 22.2 must include the closed end of the lateral. This is because the equation is based on the assumptions that: the discharges from all outlets are equal; and no water flows beyond the last outlet of the pipe section being considered. For further details on design of multioutlet pipeline, refer to Chapter 9 on Set Sprinkler Lateral Design.

Length and Flow Rate

When two laterals extend in opposite directions from a common manifold outlet, they are referred to as a *pair* of laterals. For example, the laterals in Figs. 17.2 and 17.3 are paired. The length of a pair of laterals, L_p, is equal to the manifold spacing, S_m. When considering the length of a single lateral that extends in only one direction from a manifold the length is designed as L.

The hydraulic design is based on the lateral or pair of laterals with the average flow rate in each subunit. The subunit is made up of a manifold and the laterals it supplies, as shown in Figs. 17.3 and 20.9. The flow rate into the average lateral is equal to the number of emitters along it times the average emitter discharge (based on all the emitters) within the subunit; thus:

$$Q_l = \frac{N q_a}{60} = \frac{L}{S_e} \frac{q_a}{60} \tag{22.4}$$

where:

Q_l = average lateral flow rate, L/min (gpm)
N = number of emitters along the lateral
q_a = average emitter discharge, L/hr (gph)
S_e = spacing of emitters along the lateral, m (ft)

It is important to note that *the following lateral design strategy is based on the lateral or pair of laterals having the average lateral discharge, Q_l, and corresponding inlet pressure head, H_l.* Therefore, q_a and H_a are the average emitter discharge and its related pressure head for the lateral as well as for the subunit or system (see Fig. 20.9). However, the minimum pressure head for the average lateral, H'_n, is greater than the minimum pressure head for the subunit or system, H_n, unless each lateral forms a subunit with its inlet pressure or flow regulated.

MANIFOLD SPACING

The spacing between manifolds, S_m, in orchards should be such that adjacent manifolds are a whole number of tree spacings, S_p, apart. Furthermore, it is most convenient to have the same manifold spacing throughout the field. The procedure for selecting the manifold spacing is:

Step 1. Inspect the field layout and select a reasonable manifold spacing, S_m, in accordance with the criteria listed above.

Step 2. Determine the lateral pipe-friction loss with laterals half as long as S_m by Eq. 22.2.

Step 3. Assume the lateral friction loss, $h_f = \Delta H_l$, i.e., the field is level, and compare with ΔH_s from Eq. 20.14. If h_f is much larger than 0.5 ΔH_s, the manifold spacing should be decreased. If it is much smaller, the manifold spacing may be increased. Once the friction-head loss for a given length of lateral has been computed, the head loss for any other length of lateral can be determined by rearranging Eq. 8.10 to obtain:

$$(h_f)_b \simeq (h_f)_a \left(\frac{(L)_b}{(L)_a}\right)^{2.75} \tag{22.5a}$$

Or conversely, the length of lateral that will give any desired head loss can be determined by:

$$(L)_b \simeq (L)_a \left(\frac{(h_f)_b}{(h_f)_a}\right)^{1/2.75} \tag{22.5b}$$

where $(L)_a$ and $(L)_b$ = original and new lateral pipe length, m (ft), and $(h_f)_a$ and $(h_f)_b$ = original and new lateral pipe friction losses, m (ft).

After selecting the manifold spacing the next step is to determine the optimum system layout and lateral inlet locations. Then determine the required inlet pressure head, H_l, and pressure head difference along the average lateral, ΔH_l (see Fig. 20.9). This can be done for constant-diameter laterals on uniform slopes using either the graphical or numerical methods that follow. (The graphical and numerical methods for tapered manifolds presented in Chapter 23 can also be applied to tapered laterals.) The graphical solutions are useful for visualizing the friction and elevation head relationships and can be adapted for non-uniform slopes; however, the numerical solutions are much quicker and simpler.

GRAPHICAL HYDRAULIC SOLUTIONS

There are two types of laterals, each of which requires a different graphical solution. They are: single laterals that extend in only one direction from the manifold (see Fig. 21.4); and pairs of laterals that extend in opposite directions (uphill and downhill or one to either side along the contour) from the manifold (see Figs. 21.2 and 21.5). The graphical solutions are presented for both these cases to demonstrate the conceptual development of the simplified numerical solutions for constant-diameter laterals that follow.

Single Lateral

When the manifold feeds single laterals, the graphical solution is quite simple. The hydraulic characteristics of the laterals can be determined by using the friction curve presented in Fig. 8.2 and drawing the groundline. In Fig. 8.2, the solid curved line represents the friction curves for lateral flow from right to left, and the dashed line represents the last half of the curve for flow from left to right.

This general friction curve can be adapted to a specific problem by setting the intercept of the friction curve (at $x/L = 1.0$) equal to h_f for a lateral with a specific diameter, flow rate, number of outlets, and length. The groundline for a uniform sloping field can be represented by a straight-sloping line on the graph, with the vertical distance (between $x/L = 0.0$ and $x/L = 1.0$) equal to $\Delta El/h_f$, where ΔEl is the difference in elevation.

The general hydraulic characteristics of uphill ($s > 0$), horizontal ($s = 0$), and two downhill ($s < 0$) laterals are presented in Fig. 22.1. Each lateral has the same average emitter pressure head, H_a. In all cases, the average head loss for the friction curve and the average elevation are the same.

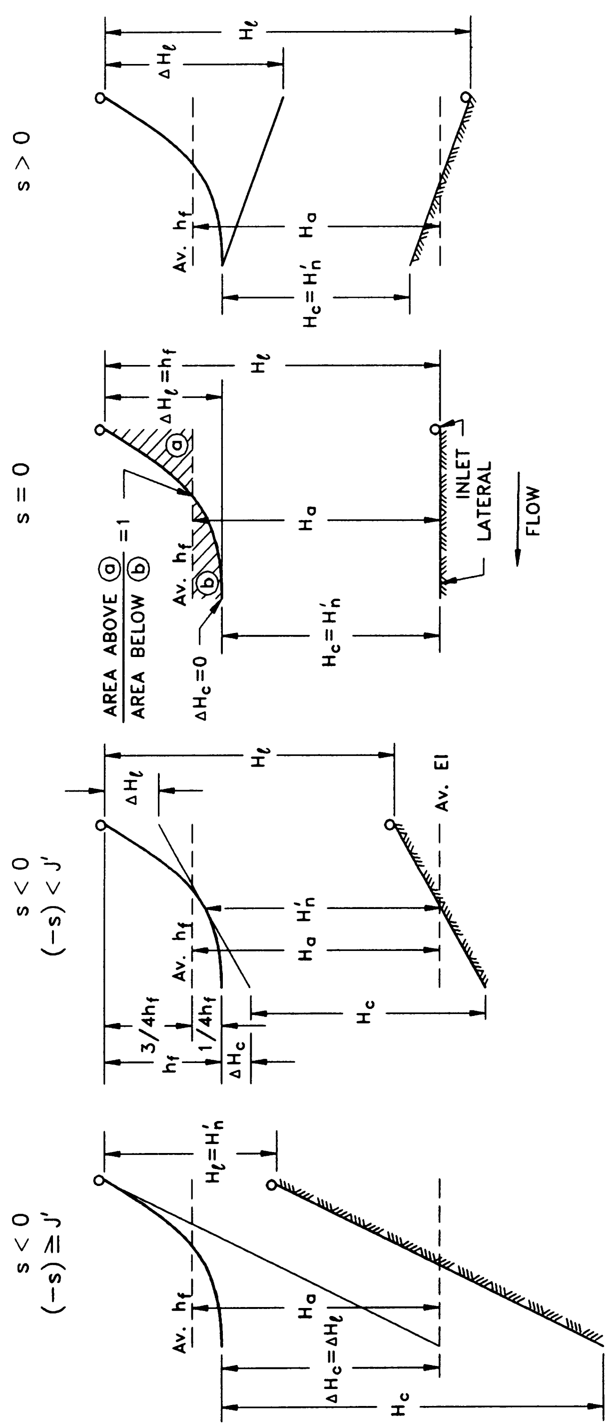


FIG. 22.1. Hydraulic Characteristics of Downhill ($s < 0$), Horizontal ($s = 0$), Uphill ($s > 0$) Laterals with the Same Average Emitter Pressure Head in which H_a = Average, H_n = Minimum, H_c = Closed End and H_1 = Inlet Pressure Head.

The average ordinate of the friction curves, which is equivalent to the average friction-head loss, h_{fa}, can be computed from Eq. 22.3b by integrating from x/L equals 0 to 1:

$$h_{fa} = \int_0^1 h_f \left[\frac{x}{L}\right]^{2.75} \mathrm{d}\frac{x}{L} = \frac{1}{3.75} h_f = 0.267 h_f$$

It can also be obtained graphically from Fig. 8.2, in which the area bounded by the solid curve and the horizontal axis between the values of x/L from 0 to 1 is 0.267. Therefore, the average ordinate (the area divided by the length ratio $x/L = 1$) is also equal to 0.267. Furthermore, from Eq. 22.3b, the location where the average ordinate intersects the friction curve is at:

$$x = (0.267)^{1/2.75} = 0.62L$$

Thus, at a distance of 0.38 L from the inlet of the pipe, $100(1.0 - 0.267) = 73\%$ of the head loss will have occurred. For a lateral on level ground the average pressure point will be at this distance from the inlet. (Typically 0.40 L and 75% are used for convenience of calculation and practical purposes.)

The average pressure head for the emitter along a lateral, H_a, is equal to the difference between the average ordinate of the friction curve and the average elevation. This is shown in Fig. 22.1 for laterals on various slopes. (Note that the average ordinates of the friction curves are independent of the slope and the average elevation is at the midpoint of the uniform slopes.) In mathematical form the various pressure heads depicted in Fig. 22.1 can be represented by a modified version of Eq. 9.2b, which is:

$$H_l = H_a + kh_f + 0.5\Delta El \tag{22.6}$$

where:

H_l = average lateral inlet pressure head, m (ft)
H_a = average emitter pressure head, m (ft)
k = 0.75 for constant-diameter laterals and 0.63 for tapered laterals with two pipe sizes
h_f = head loss due to pipe friction in the lateral, m (ft)
ΔEl = difference in elevation between the closed and inlet ends, which is (+) for laterals running uphill from the inlet and (−) for downhill laterals, m (ft)

To compute the variation in pressure head along the lateral, first determine ΔH_c (as in Step 6 for the graphical solution for pairs of laterals, which follows), and then find the minimum pressure head along the average lateral by:

$$H_n' = H_l - (h_f + \Delta El) - \Delta H_c \tag{22.7}$$

where H_n' = minimum pressure head along the average lateral, m (ft), and ΔH_c = difference between the downhill (closed) end and minimum pressure heads, m (ft). Then determine the difference between maximum and minimum pressure heads along the average lateral by (see Fig. 22.1):

$$\Delta H_l = H_l - H_n'; \qquad \text{if } (H_l - H_n') \geq \Delta H_c \tag{22.8a}$$

$$\Delta H_l = \Delta H_c; \qquad \text{if } (H_l - H_n') < \Delta H_c \tag{22.8b}$$

where ΔH_l = difference or variation in pressure heads along the average lateral, m (ft).

Pairs of Laterals

When the manifold feeds pairs of laterals, the best manifold position is the location that will result in the same minimum pressure in the uphill and downhill laterals. This position will depend on the ground slope and the friction losses in the laterals. The length of both laterals should be the same on level fields where $s = 0$. In all other cases the manifold should be shifted uphill, in an attempt always to find a position that balances the differences in elevations and pressure losses in the uphill and downhill laterals.

Figure 8.2 can be used as a basis for a graphical solution to obtain the best manifold position. To do this the following steps should be taken:

Step 1. Determine the h_{fp} for a single lateral equal in length to the manifold spacing, S_m, which is the total length of each pair of laterals, L_p.

Step 2. Place an overlay on Fig. 8.2, and make a tracing of the horizontal boundaries, as shown in Fig. 22.2. Then draw a line on the overlay representing the ground surface for the pair of laterals, such that the absolute difference in elevation, ΔE_p is properly scaled. To do this on the overlay $\Delta E_p'$ is equal to $\Delta E_p / h_{fp}$ and the groundline slopes downward to the left. The prime (′) denotes a scalar value that must be multiplied by h_{fp} to obtain the actual value.

Step 3. Slide the overlay down so the solid line friction curve on Fig. 8.2 is tangent to the groundline, and then trace the downhill lateral friction curve on the overlay, as shown in Fig. 22.2.

Step 4. Locate the "best" manifold positions by moving the overlay down farther until the dashed friction curve (right-hand curve on Fig. 8.2) coincides with the ground line at $x/L_p = 1.0$. The dashed curve in Fig. 22.2 represents the uphill lateral, and the intersection between the two curves is the "exact" manifold location, x/L_p, that will give the same minimum uphill and downhill pressures.

Step 5. Adjust the manifold location uphill by as much as three-fourths of a tree spacing, S_p, or downhill by as much as one-fourth S_p to fall

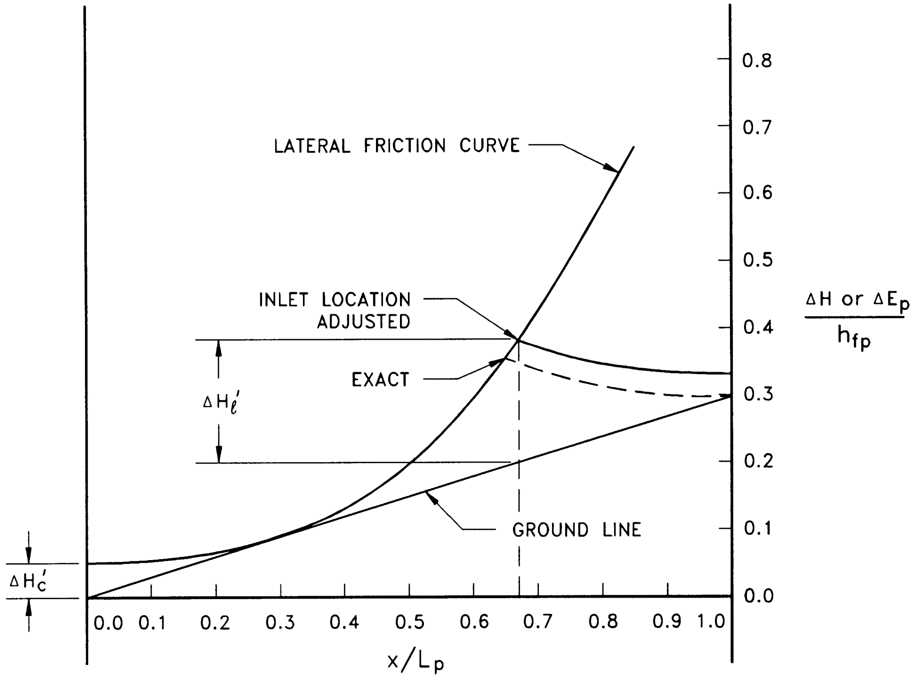


FIG. 22.2. Graphical Solution (Overlay) for Locating the Best Manifold Position for Feeding a Pair of Laterals Laid on a Slope. Vertical Scaler Values Denoted by a Prime (′) Must Be Multiplied By h_{fp} To Obtain Actual Values.

midway between two tree locations. The effect of this adjustment is shown by the solid uphill friction curve in Fig. 22.2.

Step 6. Determine the difference between the inlet and minimum pressure heads, ΔH_l. This can be done by multiplying the vertical scalar distance, $\Delta H_l'$, from the intersection of the uphill and downhill friction curves to the ground surface by h_{fp}. Determine the difference between the downhill end and minimum pressure heads, ΔH_c, in a similar manner.

Step 7. Determine the inlet pressure, H_l, that will give the desired average weighted pressure head, H_a, for the pair of laterals as follows:

a. Estimate the location of the average ordinate of the friction-head loss curve for both the uphill and downhill laterals, as described earlier in connection with Fig. 22.1 for a single lateral. The upper two dashed horizontal lines in Fig. 22.3 represent the average ordinates for the uphill and downhill portions of the laterals. The weighted average ordinate of the friction curves, which is the average head loss of the pair of laterals represented by the upper solid horizontal line in Fig. 22.3 can

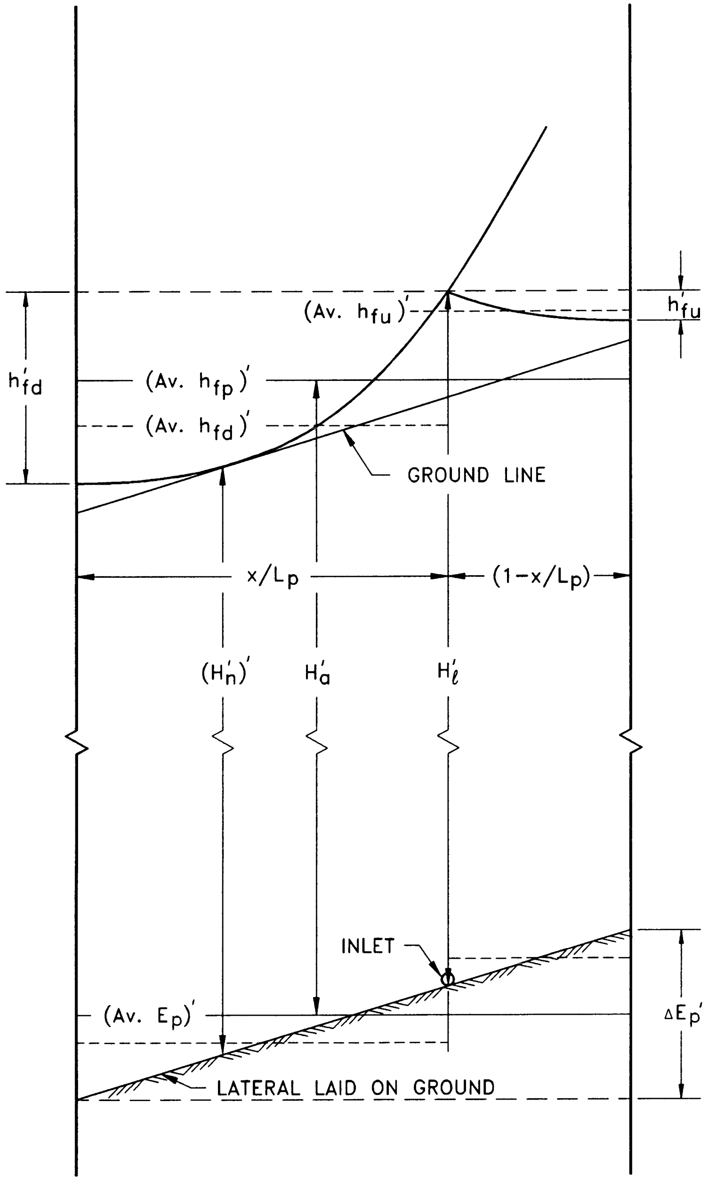


FIG. 22.3. Graphical Overlay Showing the Average Elevations and Friction Head Losses for the Uphill and Downhill Laterals (Dashed Horizontal Lines) and for the Pair of Laterals (Solid Horizontal Lines). Vertical Scaler Values Denoted by a Prime (′) Must Be Multiplied By h_{fp} To Obtain Actual Values.

be computed by:

$$(\text{Av. } h_{fp})' = (\text{Av. } h_{fd})' \left(\frac{x}{L_p}\right) + (\text{Av. } h_{fu})' \left(1 - \frac{x}{L_p}\right)$$

where:

$(\text{Av. } h_{fp})'$ = scalar weighted average head loss for the average pair of laterals

$(\text{Av. } h_{fd})'$ = scalar average head loss for the average downhill lateral

$(\text{Av. } h_{fu})'$ = scalar average head loss for the average uphill lateral

$\dfrac{x}{L_p}$ = adjusted best manifold position ratio from graphical solution

Then, multiply the scalar $(\text{Av. } h_{fp})'$ by h_{fp} to obtain the actual Av. h_{fp}.

b. Find the weighted average elevation. This is the midpoint elevation along the pair of laterals (see Fig. 22.3).

c. The H_l that will give the desired H_a can now be determined graphically from the vertical scalar distances H_l' and H_a' in Fig. 22.3 by:

$$H_l = (H_l' - H_a')h_{fp} + H_a$$

where H_l' = vertical scalar distance from the intersection of the friction curves to the ground surface line in Fig. 22.3, and H_a' = vertical scalar distance between the $(\text{Av. } h_{fp})'$ and $(\text{Av. } El_p)'$ lines in Fig. 22.3.

NUMERICAL SOLUTIONS

The numerical solutions derived below give: simplified dimensionless procedures for determining the best x/L_p for pairs of average laterals; and the required H_l to obtain a desired H_a and the resulting H_n' for both single and pairs of average laterals. The ground slope must be fairly uniform, so it can be represented by a straight line, for the numerical procedures to apply.

Single Laterals

For single laterals compute J' by Eqs. 8.7a and 22.1, and obtain F from Table 8.7. The lateral head loss, h_f, can be calculated by substituting these values into Eq. 22.2. The lateral inlet pressure head, H_l, that gives the desired H_a can be computed by Eq. 22.6.

Minimum Emitter Pressure Head. To calculate H_n' by Eq. 22.7, ΔH_c must be determined. Figure 22.1 shows that H_n' is at the closed end and $\Delta H_c = 0$ when the lateral is level or uphill, i.e., $s \geq 0$. It also shows that for steep downhill laterals with $(-s) \geq J'$, H_n' is at the inlet end; thus, $\Delta H_c = (-\Delta El) - h_f$.

For downhill laterals with $(-s) \leq J'$, the location of H_n' must be determined to compute ΔH_c; see Figs. 22.1 and 22.4. The location of H_n' is where the friction curve or head-loss gradient between two consecutive outlets, j', is equal to the ground slope, i.e., $j' = (-s)$. The location of this point, y, can be obtained from Eq. 22.3a by noting that $h_{fx} \simeq j'Fx/100$, solving for j', and replacing j' with $(-s)$. (This is based on the assumption that the F factor for the portion of lateral between the closed end and y is approximately equal to the F for the whole lateral.) Therefore:

$$(-s) = \frac{-\Delta El}{L/100} = j' = J'(y)^{1.75}; \qquad \text{for } s < 0 \text{ and } (-s) \leq J'$$

which can be rearranged and combined with Eq. 22.2 to obtain:

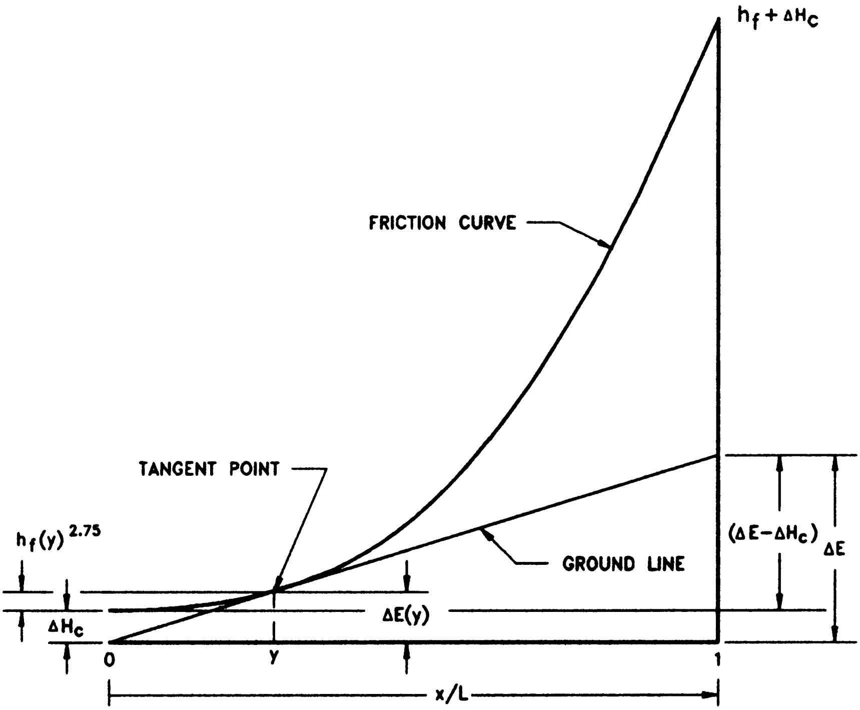


FIG. 22.4. Sketch Showing Relationships Used in Numerical Solution for Single Laterals When $s < 0$.

$$y = \left(\frac{-s}{J'}\right)^{1/1.75} = \left(\frac{-s}{J'}\right)^{0.57} = \left(F\frac{\Delta E}{h_f}\right)^{0.57} \qquad (22.9)$$

where:

s = ground slope in the direction of flow, which is (−) for downhill and (+) for uphill, percentage
J' = equivalent head-loss gradient of the lateral with emitter-connection losses, m/100 m (ft/100 ft)
j' = equivalent head-loss gradient between two consecutive outlets along the lateral, m/100 m (ft/100 ft)
y = value of x/L where the friction curve and groundline have the same slope
ΔE = absolute difference in elevation between the two ends of the lateral, m (ft)

Note that ΔEl is positive (+) for uphill and negative (−) for downhill, and ΔE is always positive because it denotes the absolute difference in elevation.

Referring to Fig. 22.4, equations for ΔH_c can now be developed using Eq. 22.3b for the friction-loss component as follows:

$$\Delta H_c = \Delta E(y) - h_f(y)^{2.75} \qquad (22.10)$$

Replacing y with its equivalent value from Eq. 22.9 gives:

$$\Delta H_c = \Delta E\left(F\frac{\Delta E}{h_f}\right)^{0.57} - h_f\left(F\frac{\Delta E}{h_f}\right)^{1.57} \qquad (22.11)$$

The minimum lateral pressure head in the average lateral, H_n', can be determined for single laterals on the different slope relationships depicted in Fig. 22.1 as follows:

(i) Where $s \geq 0$, $\Delta H_c = 0$ and from Eq. 22.7:

$$H_n' = H_l - (h_f + \Delta El) = H_l - h_f - \Delta E; \qquad \text{for } s \geq 0 \qquad (22.12)$$

(ii) Where $s < 0$ and $(-s) \leq J'$, combining Eqs. 22.7 and 22.11 gives:

$$H_n' = H_l - h_f + \Delta E - \Delta E\left(F\frac{\Delta E}{h_f}\right)^{0.57} + h_f\left(F\frac{\Delta E}{h_f}\right)^{1.57}$$

which can be reduced to:

$$H_n' = H_l - \beta'(h_f); \qquad \text{for } s < 0 \text{ and } (-s) \leq J' \qquad (22.13)$$

by letting:

$$\beta' = 1.0 - \frac{\Delta E}{h_f} + \frac{\Delta E}{h_f}\left[F\frac{\Delta E}{h_f}\right]^{0.57} - \left[F\frac{\Delta E}{h_f}\right]^{1.57}$$

Noting that $(F^{0.57} - F^{1.57}) \simeq 0.36$ for five or more outlets:

$$\beta' = 1.0 - \frac{\Delta E}{h_f} + 0.36\left[\frac{\Delta E}{h_f}\right]^{1.57} \qquad (22.14)$$

where β' = inlet to minimum pressure head adjustment factor for single laterals.

Values of β' computed by Eq. 22.14 for different $\Delta E/h_f$ values are given in Table 22.1.

(iii) When $s < 0$ and $(-s) \geq J'$, the minimum pressure head will be located at the lateral inlet, $\Delta H_c = (-\Delta El) - h_f$, and Eq. 22.7 reduces to:

$$H'_n = H_l; \qquad \text{for } s < 0 \text{ and } (-s) \geq J' \qquad (22.15)$$

In all three of the above cases of single laterals, the pressure head at the closed end (see Fig. 22.1) is:

$$H_c = H_l - h_f - \Delta El \qquad (22.16)$$

where H_c = pressure head at the closed end of the lateral, m (ft).

Table 22.1. Values of β' for use in Eq. 22.13 when computing H'_n for the average single lateral, where $s < 0$ and $(-s) \leq J'$

$\frac{\Delta E}{h_f}$	β'	$\frac{\Delta E}{h_f}$	β'	$\frac{\Delta E}{h_f}$	β'
0.00	1.00	0.80	0.45	1.60	0.15
0.05	0.95	0.85	0.43	1.65	0.14
0.10	0.91	0.90	0.41	1.70	0.13
0.15	0.87	0.95	0.38	1.75	0.12
0.20	0.83	1.00	0.36	1.80	0.11
0.25	0.79	1.05	0.34	1.85	0.10
0.30	0.75	1.10	0.32	1.90	0.09
0.35	0.72	1.15	0.30	1.95	0.08
0.40	0.69	1.20	0.28	2.00	0.07
0.45	0.65	1.25	0.26	2.05	0.06
0.50	0.62	1.30	0.24	2.10	0.06
0.55	0.59	1.35	0.23	2.15	0.05
0.60	0.56	1.40	0.21	2.20	0.04
0.65	0.53	1.45	0.20	2.30	0.03
0.70	0.51	1.50	0.18	2.40	0.02
0.75	0.48	1.55	0.17	2.75	0.01

Pairs of Laterals

The following relationships, which are depicted in Fig. 22.5, will be used in developing the numerical solution for the average pair of laterals:

$$H_l = H_a + \alpha h_{fp} - (Y - 0.5)\Delta E_p \tag{22.17}$$

and

$$H_n' = H_l - \beta h_{fp} \tag{22.18}$$

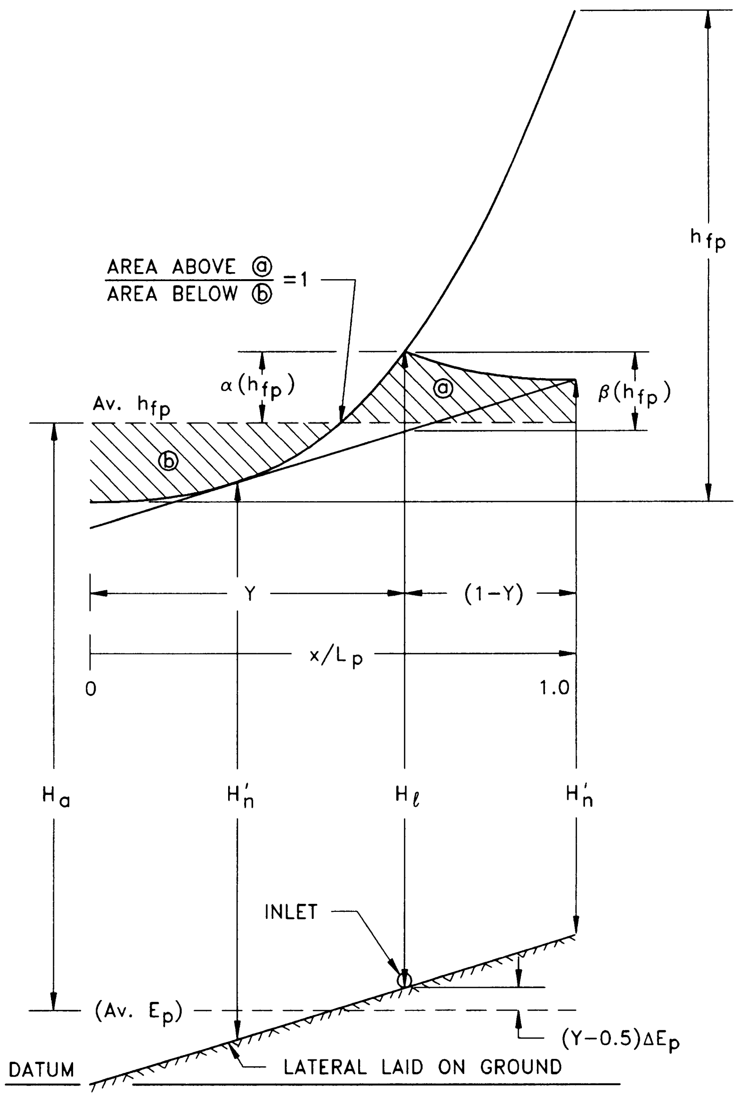


FIG. 22.5. Sketch Showing Relationships Between H_a, H_n', H_l and h_{fp} For a Pair of Laterals.

where

H_l = inlet pressure head for the pair of laterals, m (ft)
H_a = average emitter pressure head, m (ft)
α = average to inlet pressure-head-adjustment factor for pairs of laterals
h_{fp} = head loss due to friction for a single lateral with a total length and flow equal to that of the pair of laterals, m (ft)
Y = manifold position ratio, x/L_p, which gives the same minimum uphill and downhill pressure head along a pair of laterals
ΔE_p = absolute difference in elevation between the outer ends of the pair of laterals, m (ft)
H_n' = minimum pressure head along the average pair of laterals, m (ft)
β = inlet to minimum pressure-head-adjustment factor for pairs of laterals

Best Manifold Position. The value of x/L_p at the "best" manifold position as shown in Fig. 22.6 is developed from Eqs. 22.3b, 22.9, and 22.10. The pressure head along the pipe friction curve that gives H_n' for the downhill lateral using the datum as in Fig. 22.5 is:

$$(H)_d = h_{fp}\left(\frac{x}{L_p}\right)^{2.75} + \Delta H_c + H_n' \tag{22.19}$$

and for the same value of H_n' at the end of the uphill lateral it is:

$$(H)_u = h_{fp}\left(1 - \frac{x}{L_p}\right)^{2.75} + \Delta E_p + H_n' \tag{22.20}$$

where x = distance from the downhill end of a pair of laterals, m (ft), and L_p = total length of a pair of laterals that extend in opposite directions from a common manifold outlet, m (ft).

The "best" manifold position, Y, is located where the uphill and downhill friction curves intersect (see Figs. 22.5 and 22.6). Its location can be determined by equating Eqs. 22.19 and 22.20:

$$h_{fp}\left[\frac{x}{L_p}\right]^{2.75} + \Delta H_c = h_{fp}\left[1 - \frac{x}{L_p}\right]^{2.75} + \Delta E_p$$

Letting $Y = x/L_p$ and rearranging gives:

$$\frac{\Delta E_p - \Delta H_c}{h_{fp}} = Y^{2.75} - (1 - Y)^{2.75} \tag{22.21}$$

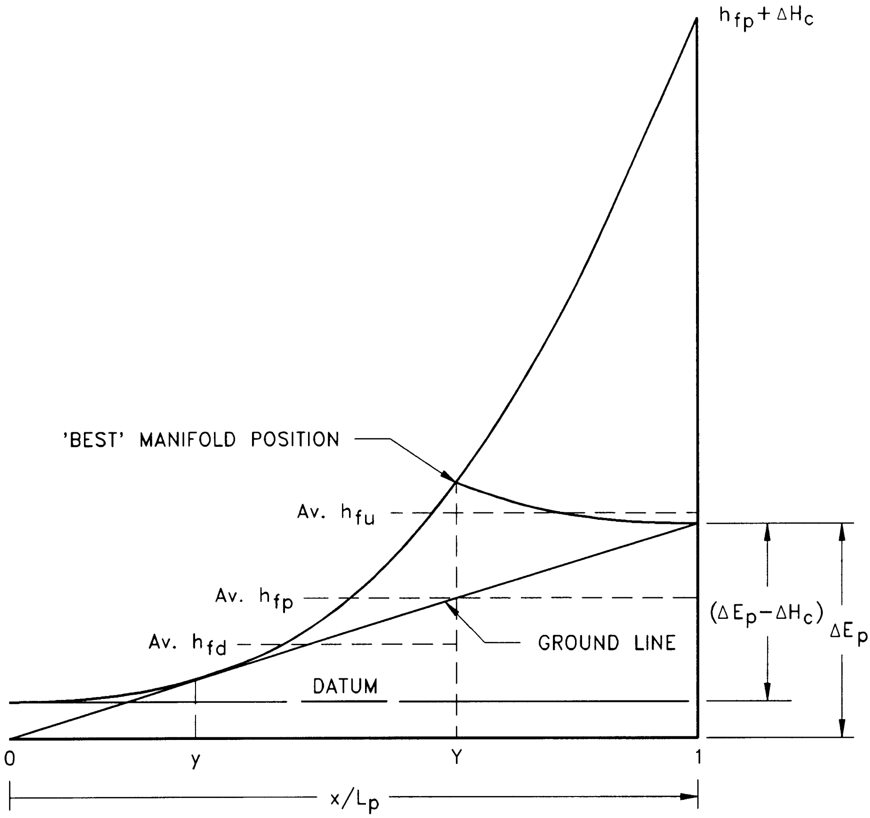


FIG. 22.6. Relationships Used in Developing Numerical Solutions for a Pair of Laterals in Which L_p Represents the Total Length of the Pair of Laterals.

where Y = manifold position ratio, x/L_p, which gives the same minimum uphill and downhill-pressure head along a pair of average laterals.

To solve for Y as a function of $\Delta E_p/h_{fp}$, the left-hand terms of Eq. 22.21 can be expanded to give:

$$\frac{\Delta E_p - \Delta H_c}{h_{fp}} = \frac{\Delta E_p}{h_{fp}} - \frac{\Delta E_p}{h_{fp}} \cdot \frac{\Delta H_c}{\Delta E_p}$$

Replacing ΔH_c with Eq. 22.10 in which h_f is now h_{fp} and rearranging gives:

$$\frac{\Delta E_p - \Delta H_c}{h_{fp}} = \frac{\Delta E_p}{h_{fp}} - \frac{\Delta E_p}{h_{fp}}\left[y - \frac{h_{fp}}{\Delta E_p}(y)^{2.75}\right]$$

$$= \frac{\Delta E_p}{h_{fp}}(1 - y) + y^{2.75}$$

where y = value of x/L_p where the pipe friction loss gradient and ground slope are the same along the downhill lateral.

Replacing y (see Fig. 22.6) with its equivalent value from Eq. 22.9 for a pair of laterals with the inlet at the uphill end gives:

$$\frac{\Delta E_p - \Delta H_c}{h_{fp}} = \frac{\Delta E_p}{h_{fp}}\left[1 - \left[F\frac{\Delta E_p}{h_{fp}}\right]^{0.57}\right] + \left[F\frac{\Delta E_p}{h_{fp}}\right]^{1.57}$$

And replacing the first term in Eq. 22.21 with this gives:

$$\frac{\Delta E_p}{h_{fp}}\left[1 - \left[F\frac{\Delta E_p}{h_{fp}}\right]^{0.57}\right] + \left[F\frac{\Delta E_p}{h_{fp}}\right]^{1.57} = Y^{2.75} - (1 - Y)^{2.75} \tag{22.22a}$$

A computer program was used to solve Eq. 22.22a for Y, by trial and error, for values of $\Delta E_p/h_{fp}$ from 0 to 2.75 and values of F representing from 9 to an infinite number of outlets. From Table 22.2 it is apparent that Y is practically independent of F. Therefore, the corresponding values for $F = 0.36$ can be used universally with little error. By letting $F = 0.36$, Eq. 22.22a can be reduced to:

$$\frac{\Delta E_p}{h_{fp}} - 0.36\left[\frac{\Delta E_p}{h_{fp}}\right]^{1.57} = Y^{2.75} - (1 - Y)^{2.75} \tag{22.22b}$$

Table 22.2. Values of "best" manifold position, Y, for pairs of average laterals for various F values and elevation difference versus friction-head-loss relationships, $\Delta E_p/h_{fp}$

$\Delta E_p/h_{fp}$	Best manifold position-Y $F = 0.36$	0.39	0.41	$\Delta E_p/h_{fp}$	Y $F = 0.36$	0.39	0.41
0.0	0.50	0.50	0.50	1.0	0.85	0.85	0.85
0.1	0.56	0.56	0.56	1.2	0.89	0.89	0.89
0.2	0.60	0.60	0.60	1.4	0.92	0.92	0.92
0.3	0.65	0.65	0.65	1.6	0.94	0.94	0.95
0.4	0.69	0.69	0.69	1.8	0.96	0.96	0.97
0.5	0.72	0.72	0.72	2.0	0.98	0.98	0.98
0.6	0.75	0.75	0.75	2.2	0.99	0.99	0.99
0.7	0.78	0.78	0.78	2.4	1.00	1.00	1.00
0.8	0.81	0.81	0.81	2.6	1.00	1.00	1.00
0.9	0.83	0.83	0.83	2.75	1.00	1.00	1.00

Inlet Pressure Head. The value of H_l for a pair of laterals can be determined by Eq. 22.17 (see Fig. 22.5). The following procedure is used to compute values for α as a function of $\Delta E_p / h_{fp}$ for use in Eq. 22.17. First determine the numerically weighted average pressure head along the pipe friction loss curves for a pair of laterals as depicted in Fig. 22.5. In this case the datum is assumed to be the pressure head at the lower end of the pair of laterals as depicted in Fig. 22.6. This is also zero on the ordinate of the friction curves and the average h_{fp}, (Av. h_{fp}), can be approximated by:

$$(\text{Av. } h_{fp}) = \tfrac{1}{4} h_{fp} Y^{2.75}(Y) + \left[(\Delta E_p - \Delta H_c) + \tfrac{1}{4} h_{fp}(1 - Y)^{2.75}\right](1 - Y)$$

Replacing $(\Delta E_p - \Delta H_c)$ by Eq. 22.21 and rearranging gives:

$$(\text{Av. } h_{fp}) = \tfrac{1}{4} h_{fp}\left[Y^{3.75} + (1 - Y)^{3.75}\right] + h_{fp}\left[Y^{2.57} - (1 - Y)^{2.75}\right](1 - Y) \qquad (22.23)$$

The average difference in elevation along the pair of laterals is:

$$(\text{Av. } \Delta E_p) = 0.5 \Delta E_p$$

Furthermore, it can be shown from Fig. 22.5 that:

$$H_l = H_a - (\text{Av. } h_{fp}) + h_{fp} Y^{2.75} + 0.5 \Delta E_p - E_p Y \qquad (22.24)$$

Substituting the (Av. h_{fp}) value obtained from Eq. 22.23 in Eq. 22.24 gives:

$$H_l = H_a + h_{fp}\left\{ Y^{2.75} - \tfrac{1}{4}\left[Y^{3.75} + (1 - Y)^{3.75}\right] - \left[Y^{2.75} - (1 - Y)^{2.75}\right](1 - Y)\right\} - (Y - 0.5)\Delta E$$

And comparing this equation with Eq. 22.17:

$$\alpha = Y^{2.75} - \tfrac{1}{4}\left[Y^{3.75} + (1 - Y)^{3\ 75}\right] - \left[Y^{2\ 75} - (1 - Y)^{2.75}\right](1 - Y)$$

which reduces to:

$$\alpha = \tfrac{3}{4}\left[Y^{3.75} + (1 - Y)^{3.75}\right] \qquad (22.25)$$

Equation 22.25 was solved for different values of Y to obtain the α values in Table 22.3. The data compiled in Table 22.3 also show the corresponding $\Delta E_p/h_{fp}$ values (see Table 22.2) for $F = 0.36$.

Minimum Pressure Head. The value of H_n' along the average pair of laterals can be determined by Eq. 22.18. The following procedure was used to compute the values for β presented in Table 22.3 as a function of $\Delta E_p/h_{fp}$.

The minimum pressure head is the same in the uphill and downhill lateral (see Fig. 22.5). In the uphill lateral $\Delta H_c = 0$, and by Eqs. 22.3b and 22.7 H_n' is:

$$H_n' = H_l - (1 - Y)\Delta E_p - (1 - Y)^{2.75} h_{fp}$$

Comparing this equation with Eq. 22.18:

$$\beta = (1 - Y)\frac{\Delta E_p}{h_{fp}} + (1 - Y)^{2.75} \tag{22.26}$$

Table 22.3. Values of "best" fit manifold position[1] and α and β for use in Eqs. 22.17 and 22.18 to determine the inlet and minimum pressure heads for average pairs of laterals, H_l and H_n', for various elevation difference versus friction loss relationships, $\Delta E_p/h_{fp}$

$\Delta E_p/h_{fp}$	Y	α	β	$\Delta E_p/h_{fp}$	Y	α	β
0.00	0.50	0.11	0.15	1.00	0.85	0.42	0.15
0.05	0.53	0.11	0.15	1.05	0.86	0.43	0.15
0.10	0.56	0.12	0.15	1.10	0.87	0.45	0.14
0.15	0.58	0.13	0.15	1.15	0.88	0.47	0.14
0.20	0.60	0.14	0.16	1.20	0.89	0.49	0.13
0.25	0.63	0.15	0.16	1.25	0.90	0.50	0.13
0.30	0.65	0.16	0.16	1.30	0.91	0.52	0.12
0.35	0.67	0.18	0.16	1.35	0.91	0.53	0.12
0.40	0.69	0.19	0.17	1.40	0.92	0.55	0.11
0.45	0.71	0.21	0.17	1.50	0.93	0.58	0.10
0.50	0.72	0.23	0.17	1.60	0.94	0.60	0.09
0.55	0.74	0.25	0.17	1.70	0.95	0.63	0.08
0.60	0.75	0.26	0.17	1.80	0.96	0.65	0.07
0.65	0.77	0.28	0.17	1.90	0.97	0.67	0.06
0.70	0.78	0.30	0.17	2.00	0.98	0.69	0.05
0.75	0.80	0.32	0.16	2.10	0.98	0.70	0.04
0.80	0.81	0.34	0.16	2.20	0.99	0.72	0.03
0.85	0.82	0.36	0.16	2.30	0.99	0.73	0.02
0.90	0.83	0.38	0.16	2.40	1.00	0.74	0.01
0.95	0.84	0.40	0.15	2.75	1.00	0.75	0.00

[1]For $F = 0.36$.

Equation 22.26 was solved for different $\Delta E_p / h_{fp}$ and corresponding Y values to obtain the β values presented in Table 22.3.

Simple Numerical Solution of Pairs of Laterals. The Y, α, and β values in Table 22.3 can be used along with Eqs. 22.17 and 22.18 to simplify the design process. First calculate $\Delta E_p / h_{fp}$, in which h_{fp} is the friction loss for a single lateral with the same length as the pair of laterals, and ΔE_p is the difference in elevation between its ends. Then enter Table 22.3 with $\Delta E_p / h_{fp}$ and obtain the corresponding values for Y, α, and β.

The best manifold position will be at a distance $x = Y L_p$ from the downstream end of the pair of laterals (see Fig. 22.5). The inlet pressure head to the pair of laterals, H_l, can be computed by Eq. 22.17 using the desired emitter operating pressure head, H_a, and value for α obtained from Table 22.3. Then the minimum pressure along the average pair of laterals, H_n', can be computed by Eq. 22.18 using the values for H_l obtained above and β from Table 22.3.

Sample Calculation 22.1. *Designing the single laterals for the line-source system presented in Sample Calculation 21.2.*

GIVEN: The data in the column headed "*LINE-SOR*" of the Trickle Irrigation Design Data and Factors Forms (see Figs. 21.1 and 21.3) for the tomato system layout shown in Fig. 21.4. Two sizes of tubing are available; their inside diameters are 0.625 and 0.824 in.

Single-chamber tubing is recommended for this design, because it can be flushed. Some clogging problems are anticipated because the irrigation water contains 3 ppm of iron, even though chlorination will be used.

As there is a large water supply, it was decided to simplify operation and maintenance by having only one operating station. Furthermore, the farmer wanted the tomato rows to run east-west and to have the manifold buried along the west side of the field. This established the system layout as shown in Fig. 21.4.

FIND: The recommended tubing size and the required average lateral inlet pressure head.

CALCULATIONS: The lateral line design procedures for all trickle irrigation systems are essentially the same. The procedure includes determining the manifold spacing, manifold layout, lateral size (or sizes in the case of tapered laterals), and maximum variation of pressure head along the laterals. The lateral flow rate is:

$$Q_l = \frac{L}{S_e} \frac{q_a}{60} \tag{22.4}$$

$$= \frac{318}{1.5} \times \frac{0.39}{60} = 1.38 \text{ gpm}$$

The friction-head-loss gradient for 1.38 gpm through the smaller diameter tubing by Eq. 8.7a is:

$$J = 0.133 \frac{(1.38)^{1.75}}{(0.625)^{4.75}} = 2.18 \text{ ft}/100 \text{ ft}$$

Because the orifices in the tubing wall are the emitters, $f_e = 0$. Therefore, $J = J'$, and the lateral head loss due to pipe friction is:

$$h_f = J'FL/100 \tag{22.2}$$

$$= 2.18 \times 0.36 \times 318/100 = 2.50 \text{ ft}$$

As a general design guideline, the allowable subunit head variations ΔH_s can be allocated equally between the lateral and manifold head variations; ΔH_l and ΔH_m. The laterals are on the contour, so $s = 0$ and the maximum head difference along the laterals $\Delta H_l = h_f = 2.5$ ft. This is less than half of $\Delta H_s = 5.8$ ft (from Fig. 21.3). Therefore, the 0.625-in. tubing should be satisfactory.

To provide the desired average emitter pressure head $H_a = 9.2$ ft, the required average lateral's inlet pressure head should be approximately:

$$H_l = H_a + 0.75h_f + \Delta El \tag{22.6}$$

$$= 9.2 + 0.75 \times 2.5 + 0 = 11.1 \text{ ft}$$

Sample Calculation 22.2. *Design the pairs of laterals for the spray system presented in Sample Calculation 21.3.*

GIVEN: The data in the columns headed "*SPRAY*" in Figs. 21.1 and 21.3 for the citrus grove with the tree rows running north-south, as shown in Fig. 21.5. The inside diameters of the lateral tubing available are 0.58 and 0.70 in., and the sprayers are connected to the lateral with large barbs.

FIND: A satisfactory manifold spacing and layout and design of the lateral lines for it.

CALCULATIONS: A manifold spacing must be selected to establish the lateral length. The factors considered when selecting the manifold spacing are: h_f, which should be $\approx 0.5\Delta H_s$; field dimensions and slopes in the direction of the rows; number of operating stations; and available lateral pipe sizes and costs.

There must be at least four manifolds to serve the four stations, $N_s = 4$, determined in the design factor computations. The tree rows run north and south, and there is no dominant slope. Therefore, the manifolds should run east and west. No adjustments in manifold position or main line position are necessary to compensate for slope effects.

There are 52 rows of trees with an average of 72 trees per row spaced at S_p

= 15 ft. A main line can be placed running north-south and located midway between the east and west boundaries of the grove. The two pairs of manifolds plus a fifth manifold for the small triangular section in the southwest corner can be served from it, as shown in Fig. 21.5.

The grove can then be irrigated in four equal-sized blocks by operating manifolds 1, 2, and 3 independently and 4 and 5 simultaneously. With this layout the spacing between the pairs of manifolds, which is also twice the average length of the laterals in the rectangular subsections, is the average number of trees along the rows between the manifolds times S_p:

$$S_m = (72 \times 15)/2 = 540 \text{ ft}$$

The h_f for the level laterals with 0.58-in. hose and serving 18 trees to either side of each manifold can now be determined as follows. By Eq. 22.4 for q_a = 11.4 gph and $N = 18$:

$$Q_l = 18 \times 11.4/60 = 3.42 \text{ gpm}$$

Using Eq. 8.7a, the head-loss gradient in the 0.58-in. tubing would be:

$$J = 0.133 \frac{(3.42)^{1.75}}{(0.58)^{4.75}} = 15.2$$

From Fig. 20.8 the connection loss, $f_e = 0.5$ ft, for the large barbs in 0.58-in. tubing. With one sprayer per tree $S_e = S_p = 15$ ft, and the equivalent head-loss gradient for flow through the tubing with barbed inserts is:

$$J' = J \frac{S_e + f_e}{S_e} \tag{22.1}$$

$$= 15.2(15 + 0.5)/15 = 15.7 \text{ ft}/100 \text{ ft}$$

Using Eq. 22.2 to find the head loss due to pipe friction in the lateral lines with $L = 270$ ft and $F = 0.37$ (from Eq. 8.9c or $F = 0.36$ from Table 8.7) gives:

$$h_f = 15.7 \times 0.37 \times 270/100 = 15.7 \text{ ft}$$

This exceeds the allowable pressure-head variation in the subunits, which is $\Delta H_s = 13.2$ ft.

The h_f should be limited to about half of ΔH_s or 6.6 ft. Either the laterals need to be shortened or larger tubing should be used. The length of lateral that would give $h_f = 6.6$ ft can be computed by:

$$(L)_b = (L)_a \left(\frac{(h_f)_b}{(h_f)_a} \right)^{1/2.57} \tag{22.5b}$$

$$= 270 \left(\frac{6.6}{15.7} \right)^{1/2.75} = 197 \text{ ft}$$

This would require dividing the field to operate with either three or six stations. Neither is satisfactory because three stations would require only 13.5 hr of operation per day and six stations would require 27 hr per day.

Repeating the earlier computations with 0.70-in. tubing gives:

$$J' = 6.23\left(\frac{15 + 0.4}{15}\right) = 6.40$$

$$h_f = 6.40 \times 0.37 \times 270/100 = 6.4 \text{ ft}$$

This is about half of ΔH_s and therefore acceptable for the four-station layout shown in Fig. 22.5.

Because the citrus grove is nearly level, manifolds 1, 2, and 3 should be laid out to serve pairs of laterals of equal length as shown in Fig. 22.5. Furthermore, the maximum pressure head variations along the level laterals, $\Delta H_l = h_f = 6.4$ ft.

The h_f of one lateral of the pair being known, the common lateral inlet pressure head for the pairs of single-diameter laterals on the level field can be determined directly by Eq. 22.6. Therefore, to obtain the desired $H_a = 58.5$ ft with $h_f = 6.4$ ft, the average lateral's inlet pressure head should be:

$$H_l = 58.5 + 0.75 \times 6.4 = 63.3 \text{ ft}$$

Sample Calculation 22.3. *Positioning the manifolds and designing pairs of laterals for the drip system presented in Sample Calculation 21.1.*

GIVEN: The data in the column headed "*DRIP*" of the Trickle Irrigation Design Data and Design Factors Forms (see Figs. 21.1 and 21.3) for the almond orchard layout shown in Fig. 21.2. The inside diameter of the selected lateral tubing is 0.58 in., and the emitters have standard-sized barbs.

FIND: A recommended manifold layout and design for the average lateral pairs.

CALCULATIONS: The procedure for selecting the manifold spacing on sloping fields is essentially the same as for level fields. To balance the ΔH_s equally between ΔH_l and ΔH_m let $h_f \approx 0.5\ \Delta H_s$. With only one operating station, balancing the system flow rate is not a constraint.

Inspection of the orchard layout in Fig. 21.2 shows that three pairs of manifolds, each serving rows of 54 trees, would meet the above criteria with the fewest number of manifolds. This requires two pairs of manifolds for the west 80-A portion and one for the east 40-A portion. The h_f for laterals serving 27 trees on either side of each manifold can be calculated using Eq. 22.4 as follows:

$$L = 27\ S_p = 27 \times 24 = 648\ \text{ft}$$

$$Q_l = \frac{648}{6} \times \frac{1.11}{60} = 2.00\ \text{gpm}$$

The equivalent friction-loss gradient for the 0.58-in. tubing as computed by using Eqs. 8.7a and 22.1 is:

$$J' = 5.95(6.0 + 0.4)/6.0 = 6.35\ \text{ft}/100\ \text{ft}$$

and by Eq. 22.2:

$$h_f = 6.35 \times 0.36 \times 648/100 = 14.8\ \text{ft}$$

This is almost equal to the ΔH_s of 16 ft and would leave too little margin for differences in pressure head in the manifolds. The lateral length that would produce $h_f = 0.5\Delta H_s = 8.0$ ft can be found directly by using Eq. 22.5b to obtain:

$$L = 648 \times \left(\frac{8.0}{14.8}\right)^{1/2.75} = 518\ \text{ft}$$

This would give a manifold spacing of 1036 ft for pairs of laterals. With S_p = 24 ft, the length of the west side of the field is $108 \times 24 = 2592$ ft. Thus, three pairs of manifolds would be sufficient for it, but the east side would require two pairs of manifolds.

To simplify construction and improve water distribution, six equally spaced pairs of manifolds were selected so:

$$S_m = 27\ \text{trees} \times 24 = 648\ \text{ft}$$

Thus, the average lateral length, L, will be 324 ft. The head difference along each pair of laterals can be estimated by using Eq. 22.5a to obtain:

$$h_f = 14.8\left[\frac{324}{648}\right]^{2.75} = 2.2\ \text{ft}$$

The next design step is to determine the "best" manifold positions so the up- and downhill minimum lateral pressure heads, h_n, will be equal. The graphical method is useful to demonstrate the process, but the numerical procedure is much quicker.

The first step of the *graphical method* is to compute the friction head loss h_{fp} = 14.8 ft for a single lateral with a length L_p = 648 ft equal to the length of the pair of laterals. (This happened to be calculated in the process of determining S_m.) Then determine the absolute difference in elevation along the pair of laterals as:

$$\Delta E_p = |sL_p/100| = 0.5 \times 648/100 = 3.24\ \text{ft}$$

and the ratio:

$$\Delta E_p/h_{fp} = 3.24/14.8 = 0.22$$

Design Steps 1 through 5 were followed in constructing Fig. 22.7. The exact best manifold location is at $x/L_p = 0.61$. This falls at $0.61 \times 27 = 16.5$ tree spacings from the closed end, which places it under the 17th tree, because the trees are in the middle of the spacings. Therefore, it should be moved to $x/L_p = 17/27 = 0.63$, so it falls midway between the 17th and 18th trees from the lower side of each subunit, as shown in Fig. 21.2.

In accordance with Step 6 (see Fig. 22.7):

$$\Delta H_l = 0.18 h_{fp} = 0.18 \times 14.8 = 2.6 \text{ ft}$$

By following Step 7 and completing this graphical solution (see Fig. 22.3), the average pair of laterals' inlet pressure head that gives H_a is:

$$H_l = (H_l' - H_a')h_{fp} + H_a$$

$$= (3.15 - 3.00)14.8 + 44.5 = 46.7 \text{ ft}$$

The sloping line above the friction curves and parallel to the groundline in Fig. 22.7 is the proposed upper limit of pressure head variation, which is $0.5\Delta H_s$. The scalar distance for the head variation limit line is:

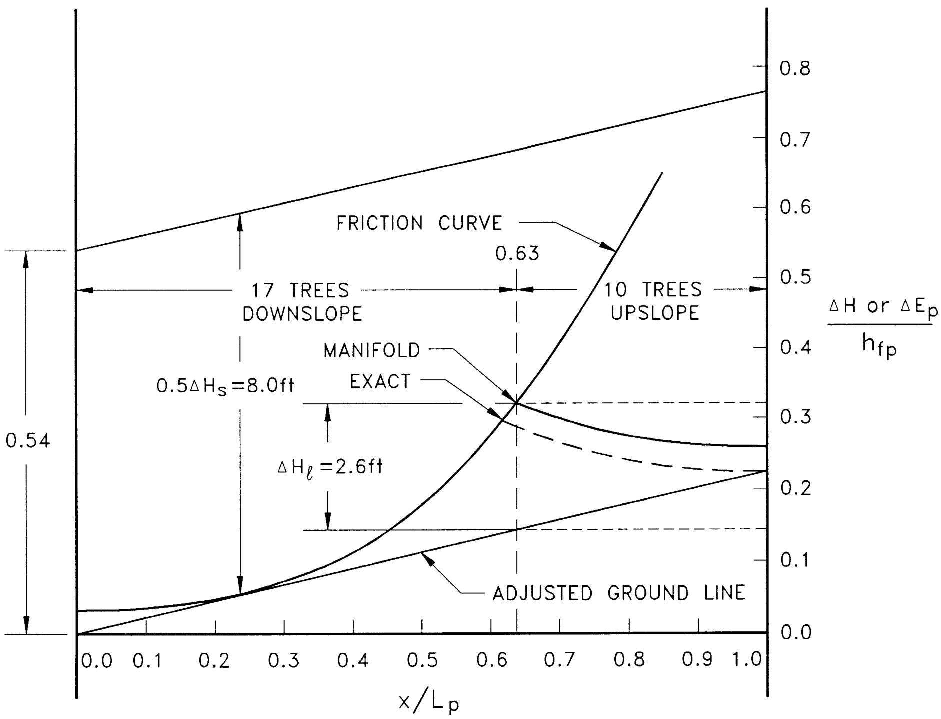


FIG. 22.7. Friction Curve Overlay to Demonstrate Graphical Solution for Manifold Positioning and ΔH_l.

$$0.5\Delta H_s/h_{fp} = 0.5 \times 16/14.8 = 0.54$$

As long as the pipe friction curves fall below this line, $\Delta H_l < 0.5\Delta H_s$.

The *numerical method* begins the same as the graphical method. However, after determining $\Delta E_p/h_{fp} = 0.22$, Table 22.3 can be used to obtain the best manifold position, $Y = 0.61$ (or it can be determined by trial and error using Eq. 22.22b). This is the same value that was obtained for the graphical solution. According to the same logic as before, the manifold position should be located between the 17th and 18th trees from the lower side of each subunit, as shown in Fig. 21.2.

With $\alpha = 0.15$, obtained from Table 22.3, the inlet pressure head required for the average pair of laterals can be computed by:

$$H_l = H_a + \alpha h_{fp} - (y - 0.5)\Delta E_p \tag{22.17}$$

$$= 44.5 + 0.15 \times 14.8 - (0.61 - 0.5)3.2$$

$$= 46.4 \text{ ft}$$

The minimum pressure head along the lateral can now be computed using $\beta = 0.16$ from Table 22.3 and H_l from above to obtain:

$$H_n' = H_l - \beta h_{fp} \tag{22.18}$$

$$= 46.4 - 0.16(14.8) = 44.0 \text{ ft}$$

and the pressure head variation along the laterals is:

$$\Delta H_l = H_l - H_n' = 46.4 - 44.0 = 2.4 \text{ ft}$$

Because equal manifold spacings will be used and the field has a uniform slope, the lateral design criteria will be the same for all the subunits in Fig. 21.2.

The location of the minimum pressure head, H_n' along the average pair of laterals is at $x/L_p = y$. It can be determined by using Eq. 22.9 with h_{fp} in place of h_f to obtain:

$$y = \left(\frac{-s}{J'}\right)^{0.57} = \left(\frac{0.5}{6.35}\right)^{0.57} = 0.24$$

or:

$$y = \left(F\frac{\Delta E}{h_{fp}}\right)^{0.57} = (0.36 \times 0.22)^{0.57} = 0.24$$

REFERENCES

Keller, J. 1980. Trickle irrigation, Chapter 7 of Section 15. In *Irrigation*. Washington, D.C. USDA-SCS Nat. Eng. Handbook.

Keller J., and J. Rodrigo. 1979. Trickle irrigation lateral design. *ASAE Technical Paper* No. 79-2570.

Watters, G. Z., and J. Keller. 1978. Trickle irrigation tubing hydraulics. *ASAE Technical Paper* No. 78-2015.

RECOMMENDED READING

Benami, A., and A. Ofen. 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).

Wu, I. P., H. M. Gitlin, K. H. Solomon, and C. A. Saruwatari. 1986. Design principles: System design. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama and D. A. Bucks, pp. 53–92. Amsterdam: Elsevier.

23

Trickle Manifold Design

Trickle irrigation manifolds are multioutlet pipelines like laterals. They differ from laterals in that flow rates are much higher, and usually they are tapered, having up to four different pipe sizes. This is done to economize on pipe costs and to keep the pressure head variations within the desired limits.

Manifold design procedures covered in this chapter include determining the following manifold characteristics: flow rate, inlet location, pipe sizes to keep within the desired pressure head differential, and inlet pressure needed to give the desired average emitter discharge. (Similar procedures are also useful for designing fixed sprinkler laterals for buried plastic systems.)

Where the average slope along the manifolds is $<3\%$, it is usually most economical to have manifolds extending in both directions from the main line. The main line should be positioned so the minimum pressure along each leg of a pair of manifolds extending from a common outlet is about equal. On level ground the length of the legs should be equal. On sloping fields the uphill legs should be shorter than the downhill legs so the friction losses plus elevation differences are in balance.

CHARACTERISTICS OF MANIFOLDS

Manifolds are usually tapered and designed to use two, three, or four different pipe sizes. To assure adequate flushing, the smallest pipe should be no less than one-half the diameter of the largest. The velocity should be limited to about 2 m/s (7 ft/s) in manifolds. This is higher than the 1.5 m/s (5 ft/s) often recommended for main lines, because the outlets along the manifold are always open, and thus water hammer shock is dampened.

Allowable Pressure Head Loss

The main line and manifold layout is a compromise between field geometry and manifold hydraulics. The allowable manifold pressure head variation (see Fig. 20.9) is:

$$(\Delta H_m)_a = \Delta H_s - \Delta H_l \tag{23.1}$$

where

$(\Delta H_m)_a$ = allowable manifold pressure head variation that will satisfy the desired emission uniformity, m (ft)
ΔH_s = allowable subunit pressure head variation that will give the desired design uniformity, m (ft)
ΔH_l = pressure head variation along the average lateral, m (ft)

For simplification the hydraulic design procedure is based on assuming an average emitter discharge, q_a, throughout the subunit served by each manifold. Thus for manifolds serving rectangular subunits, the lateral flow rate, Q_l, is assumed to be constant.

Length

Where two manifolds extend in opposite directions from a common main line outlet, they will be referred to as a *pair* of manifolds. Where only one manifold is connected to an outlet, it will be referred to as a *single-manifold configuration*. The length of a single manifold is usually equal to:

$$L = (N_r - 0.5)S_r \tag{23.2a}$$

The length of a pair of manifolds is usually equal to:

$$L_p = (N_r - 1)S_r \tag{23.2b}$$

where:

L = length of a single manifold, m (ft)
L_p = length of a pair of manifolds, m (ft)
N_r = number of rows (or laterals) served from a common main line outlet (or by a manifold)
S_r = row (or lateral) spacing, m (ft)

Main Line Position

To optimize hydraulic design, the inlet to pairs of manifolds should be located so the minimum pressure along the uphill manifold equals that of the downhill manifold. However, field boundaries, roadways, structures, existing facilities, or such topographic features as drains must also be considered. Furthermore, sometimes manifolds making up pairs are operated individually, and to balance system flow rates the main line must be positioned accordingly.

The inlet location, $Y = x/L_p$, that will balance the minimum uphill and downhill pressures is precise for manifolds (or laterals) having a single pipe size, as demonstrated in Chapter 22. However, for tapered manifolds the location depends on the selection of pipe sizes and lengths. Figure 23.1 can be used as a guide for selecting the inlet location of both single-pipe-size and tapered manifolds. In Fig. 23.1 the upper curve for use with single-pipe-size manifolds is a plot of the best fit lateral inlet, Y, values from Table 22.3. The curve for tapered manifolds is based on assuming equal average friction-loss slopes for both the uphill and downhill manifold sections. This is based on the logic of using similar pipe diameters for similar flow rates. Thus, for pairs of tapered manifolds it was assumed that:

$$\frac{(\Delta H_m)_a + Y\Delta E}{Y} = \frac{(\Delta H_m)_a - (1 - Y)\Delta E}{(1 - Y)}$$

And rearranging gives:

$$\frac{\Delta E}{(\Delta H_m)_a} = \frac{2Y - 1}{2Y(1 - Y)} \tag{23.3}$$

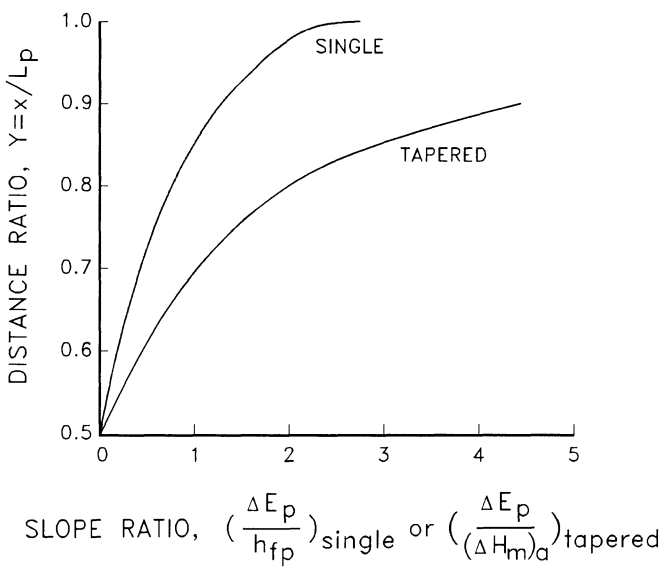


FIG. 23.1. Graph for Selecting the Inlet Location for a Pair of Manifolds with Single or Tapered Pipes.

where:

Y = best inlet position, x/L_p, for a pair of tapered manifolds
ΔE = absolute difference in elevation between the opposite ends of a pair of manifolds, m (ft)
$(\Delta H_m)_a$ = allowable manifolds pressure head variation, m (ft)

Optimizing the location of main lines serving pairs of sloping manifolds can result in both better uniformity and considerable pipe cost savings. The cost savings result from splitting the flow. Thus the maximum flow rate in the manifold sections is reduced so more smaller diameter pipe can be used.

Inlet Pressure

As a rule the main pressure-control (adjustment) points for a trickle irrigation system are at the manifold inlets that form the system subunits. Therefore, the manifold inlet pressures must be known for proper system management and for determining the total dynamic head required for the system. The manifold inlet pressure head for rectangular subunits can be computed by a modified version of Eq. 22.6:

$$H_m = H_l + k\,h_f + 0.5\Delta El \tag{23.4a}$$

$$= H_l + \Delta H_{m-l} \tag{23.4b}$$

where:

H_m = manifold inlet pressure head, m (ft)
H_l = average lateral inlet pressure head computed by the methods presented in Chapter 22 to give H_a, m (ft)
ΔH_{m-l} = amount the manifold inlet pressure head differs from the average lateral inlet pressure head, H_l, m (ft)
k = 0.75 for manifolds with one pipe size, 0.63 for two pipe sizes, and 0.5 for three or more pipe sizes
h_f = manifold pipe friction head loss, m (ft)
ΔEl = elevation difference between the closed and inlet ends of a manifold, which is positive (+) for uphill and negative (−) for downhill manifolds, m (ft)

FRICTION LOSS BY GRAPHICAL METHOD

The pressure head loss, h_f, due to pipe friction in tapered manifolds can be determined graphically using standard unit friction-loss curves.

Standard Friction Curves

Figure 23.2 was developed to provide the basis for a graphical solution that simplifies h_f calculations for tapered manifolds (Keller, 1980). It is a plot of head-loss curves for manifolds made up of different nominal pipe diameters and was inspired by the works of Herbert (1971), Jobling (1976), and Wu and Gitlan (1977). The curves are for standard PVC thermoplastic IPS pipe with the minimum recommended SDR ratings presented in Chapter 8. In common terms, the 1- through 2 1/2-in. curves are for class 160 pipe, the 3-in. curve is for class 125 pipe, and the 4-in. curve is for class 100 pipe.

The standard manifold unit friction-loss curves in Fig. 23.2 are presented in English units. They represent the friction loss along manifolds with outlets discharging 2 gpm (0.13 L/s) and spaced 2 ft (0.6 m) apart to serve either single or pairs of laterals. Data for the curves were computed in a step-by-step fashion using Eq. 8.7a to calculate the friction head loss between each outlet.

Figure 23.2 is satisfactory for obtaining sufficiently accurate estimates of h_f for manifold outlet discharges up to 4 gpm (0.25 L/s). This covers most trickle

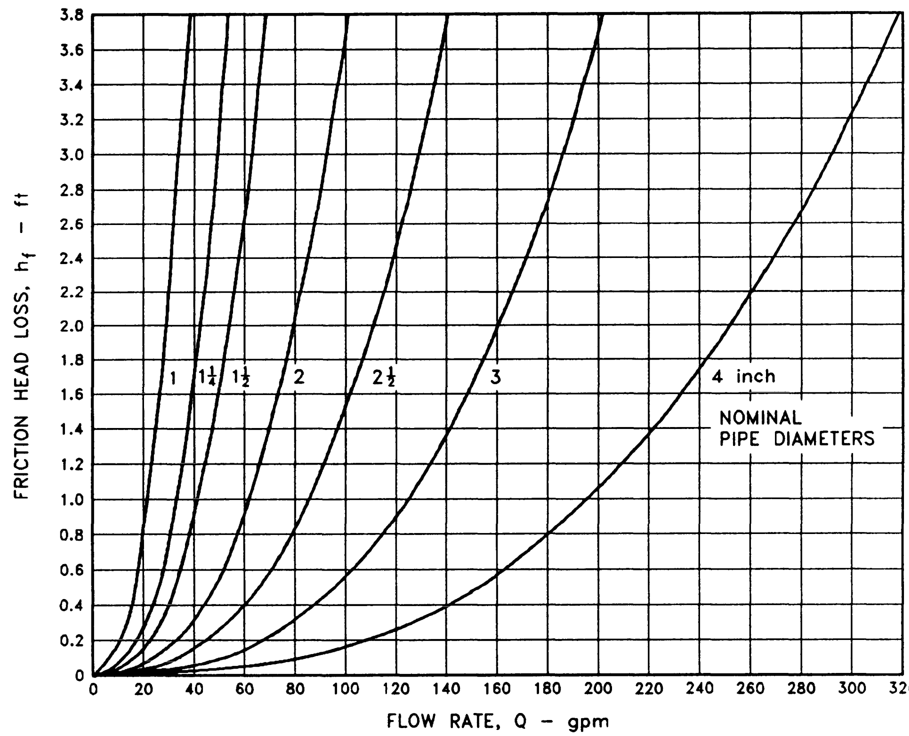


FIG. 23.2. Standard Manifold Unit Friction Loss Curves for PVC Thermoplastic IPS (Minimum Recommended SDR) Pipe with Outlets Discharging 2.0 gpm and Spaced 2 ft Apart.

systems. Different sets of curves should be generated for higher outlet discharges or for pipe with different internal diameters, such as standard metric sizes.

The curves in Fig. 23.2 are referred to as *unit* friction-loss curves, because the spacing between outlets is such that the discharge per outlet divided by the spacing equals 2 gpm/2 ft = 1.0 gpm/ft. The reason a different set of curves should be used for outlets discharging > 4.0 gpm is because the discharge per outlet affects the proportional shape of the curves. A set of curves based on outlets 6 ft apart and discharging 6 gpm would be satisfactory for outlet discharges between 4 and 8 gpm.

An entirely different set of unit friction-loss curves is needed for metric units and standard metric-sized pipe. Because 2.0 gpm ≈ 8 L/min or roughly 0.1 L/s, a similar set of standard unit friction-loss curves for metric units should be based on outlets discharging 8.0 L/m (or 0.10 L/s) and spaced 8.0 m (or 0.10 m) apart.

Because the h_f values in Fig. 23.2 are based on 1.0 gpm/ft, they must be adjusted by a scale factor to reflect the actual manifold discharge per unit length. Thus, to determine the actual head loss represented by the graphical values:

$$h_f = (S_l/Q_l)h'_f = k_m h'_f \tag{23.5a}$$

or

$$h_f = (L/Q_m)h'_f = k_m h'_f \tag{23.5b}$$

where:

h_f = pipe friction head loss, m (ft)
k_m = scale factor for adjusting manifold pressure head values taken from a standard unit friction curve
h'_f = scalar value of friction loss from standard unit friction loss curve, m (ft)
S_l = spacing between lateral outlets along the manifold, m (ft)
Q_l = lateral discharge or flow rate, L/s (gpm)
L = length of manifold, m (ft)
Q_m = manifold discharge or flow rate, L/s (gpm)

The graphical method for determining h_f in manifolds with any combination of the pipe sizes presented in Fig. 23.2 is as follows:

- First lay a piece of tracing paper on the figure and draw lines along the abscissa and ordinant;

- Draw vertical lines at the flow rates representing the desired divisions between successive pipe sizes;
- Trace the curve for the smallest desired pipe between the origin and the flow rate at which the next larger pipe should begin;
- Then slide the overlay down, so the end of this curve for the smaller pipe coincides with the curve for the next larger pipe and trace its curve to the next pipe size change point; and
- Repeat this process until the set of curve segments reaches the manifold flow rate, Q_m.

This series of head-loss segments represents the head loss along the tapered manifold. To convert the graphical h_f' to the actual h_f values multiply them by the scale factor, k_m, from Eq. 23.5 so:

$$h_f = k_m(h_f' \text{ from graph overlay})$$

HEAD VARIATIONS

To estimate the manifold pressure head variation, ΔH_m, for level or uphill manifolds:

$$\Delta H_m = h_f + s(L/100); \qquad \text{for } s \geq 0 \tag{23.6a}$$

For downhill manifolds ΔH_m can be determined graphically, as described later, or when $\Delta E < h_f$, it can be approximated by:

$$\Delta H_m \approx h_f + \left[s\left(1.0 - \frac{0.36}{n}\right)\frac{L}{100}\right]; \qquad \text{for } s < 0 \tag{23.6b}$$

where:

ΔH_m = pressure head variation along the manifold, m (ft)
h_f = pressured head loss in the manifold due to pipe friction, m (ft)
s = slope of the manifold, which is positive (+) for uphill and negative (−) for downhill, percentage
L = length of the manifold, m (ft)
n = number of pipe sizes used in the manifold

When using Eq. 23.6a the h_f and L are the friction head loss and length of the uphill portion of manifold respectively; and when using Eq. 23.6b they are the h_f and L values for the downhill portion.

MANIFOLD DESIGN

The design strategy for pairs of manifolds is to first select the manifold inlet location. Then treat the two manifold sections separately and adjust their common inlet pressure head to give the desired H_l and H_a.

The hydraulic design procedure can be carried out using graphical or numeric methods. There are two numerical design methods. One is based on the universal economic pipe-size selection chart presented in Chapter 8 and the other uses a hydraulic grade line, HGL, fitting procedure.

Pipe-Sizing Criteria

Selecting pipe sizes for tapered manifolds involves three criteria:

1. Economics of the pipe's initial cost balanced against pumping cost over the expected life of the pipe.
2. A balance between friction loss, change in elevation, and allowable variation in pressure.
3. Maximum permissible velocity.

Pipe sizes selected on the basis of economics are considered acceptable if variations in pressure do not exceed allowable limits. If the limits of pressure variation are exceeded, the pipe sizes can be changed to keep within the allowable limits by balancing pipe friction with changes in elevation. However, the maximum permissible velocity controls minimum pipe sizes regardless of other criteria.

There are two approaches for selecting the diameters and lengths of pipe to be used for a trickle irrigation manifold. One is to analyze the head-loss distribution through a series of pipe diameters and design the manifold so the pressure head remains within certain limits, as is done with the graphical and HGL-fitting procedures. The second option is to employ the economic analysis concepts introduced in Chapter 8 to select pipe diameters. The best approach depends primarily on the effect manifold design will have on system uniformity and on the expected cost of the pipe and energy required to operate the system.

Graphical Design Procedure

The graphical design procedure can be used for single-pipe-size or tapered manifolds on uniform or irregular topography, but they must serve laterals with uniform spacing and discharge. It provides a quick means to carefully design manifolds for trickle irrigation systems and simplifies the investigation of alternative designs. Thus the most promising individual designs for each specific manifold in the system can be selected.

The graphical design procedure uses standard manifold curves like Fig. 23.2 as follows:

Step 1. The h_f values are plotted for a 1 gpm/ft average manifold discharge. Therefore, the $(\Delta H_m)_a$ and ΔE values must be adjusted to compensate for the difference between the standard curves and the manifold under study. To do this divide them by k_m, the scale factor computed using Eq. 23.5. In the text that follows, these scalar values will be designated with a prime (′) as $(\Delta H_m)'_a$ and $\Delta E'$.

Step 2. Place a transparent overlay on the standard manifold curves, then trace the horizontal and vertical axes, and draw a vertical line at Q_m (see Fig. 23.3).

Step 3. Draw a line representing the ground slope on the overlay (see Fig. 23.3). The groundline will be between (0, 0) and $\Delta E'$ at Q_m for downhill manifolds on uniform slopes; the horizontal axis for level manifolds; and between $\Delta E'$ at $Q = 0$ and 0 at $Q = Q_m$ for uphill manifolds (where $\Delta E'$ represents the scalar value of the absolute difference in elevation between points).

Step 4. Place a point on the overlay $Q = Q_m$ line at $[(\Delta H_m)'_a + \Delta E']$ for downhill manifolds and at $(\Delta H_m)'_a$ for level and uphill manifolds (see Fig. 23.3). This represents the *maximum hydraulic grade point* along the manifold and it is always at the inlet end. Also draw a line through this point and parallel to the groundline for downhill manifolds (see Fig. 23.3 A).

Step 5. Slide the overlay down to the first friction curve that does not dip below the groundline when it passes through the maximum hydraulic grade point on the Q_m line. Then sketch this pipe friction curve on the overlay and note the diameter of the pipe (see Fig. 23.3). This will be the inlet and largest diameter of pipe in the manifold.

Step 6. For level and downhill manifolds on uniform slopes draw a straight line on the overlay from the origin and tangent to the friction curve drawn in Step 5. For irregular downslopes, draw the line so it is tangent to both the friction curve and the groundline. For uphill manifolds draw the line from $Q = 0$ at $\Delta E'$ and tangent to the friction curve from Step 5. This will be the lower limit hydraulic grade line for all the friction-loss-curve segments making up a tapered manifold (see Fig. 23.3).

Step 7. Tapered manifolds may have up to four segments with different pipe diameters; the smallest being no less than half the inlet diameter found in Step 5. The appropriate length of each of the pipe sections can be determined as follows:

   i. Sketch sections of the friction curves for the different pipe diameters so they are tangent to the hydraulic grade line drawn in

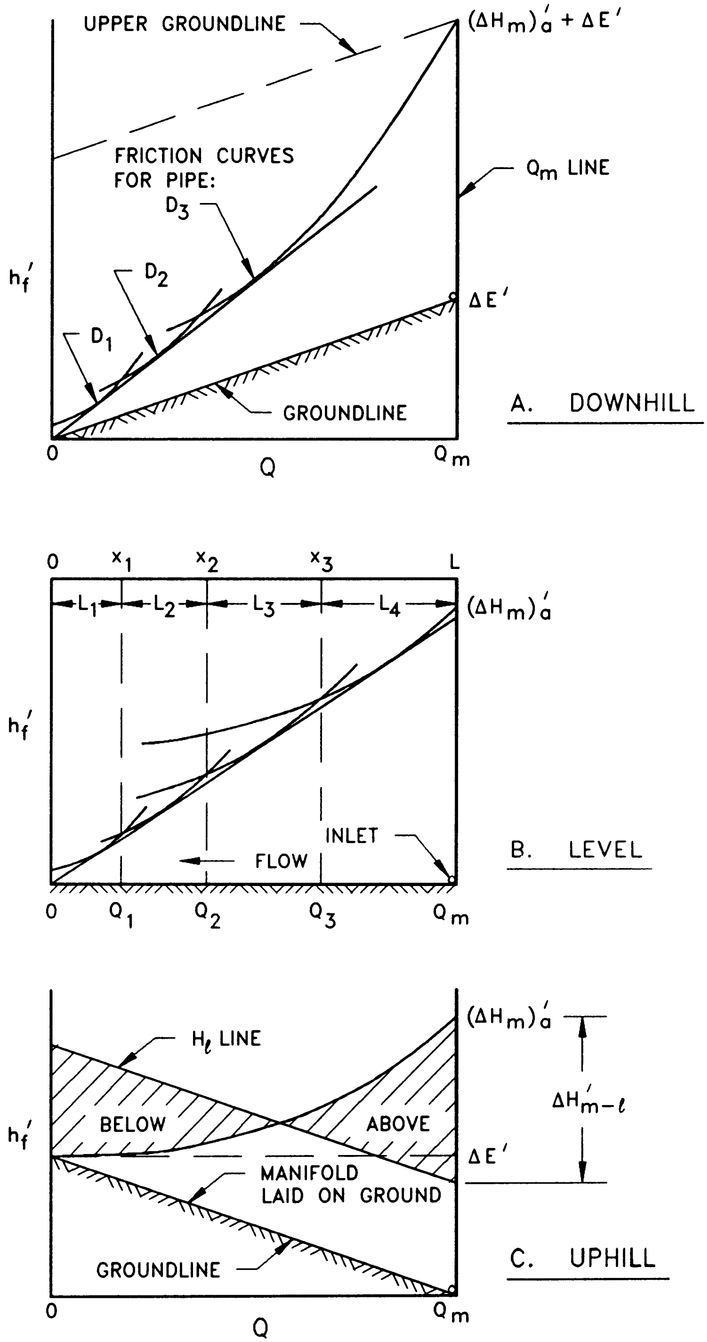


FIG. 23.3. Sketches to Demonstrate Graphical Manifold Design Process and Terminology.

Step 6 (see Figs. 23.3 A and B). (For the uphill manifold shown in Fig. 23.3 C, the friction loss curve for the pipe diameters selected in Step 5 required all the available head loss, so only one pipe diameter is used.)

ii. For downhill manifolds select the pipe sizes so the intersections between the friction curves for adjacent pipe diameters fall below the "upper groundline" drawn in Step 4 (see Fig. 23.3 A).

iii. The flow rates where pipe size changes should occur are at the intersection between the friction curves for adjacent pipe diameters. The length of each size of pipe, beginning with the smallest diameter pipe, can be determined by:

$$L_D = \frac{Q_i - Q_o}{Q_m} L \tag{23.7}$$

where:

L_D = length of pipe with diameter D, m (ft)
Q_i = flow rate into pipe with diameter D, L/s (gpm)
Q_o = flow rate out of pipe with diameter D, L/s (gpm)
Q_m = manifold flow rate, L/s (gpm)
L = length of manifold used in computing Q_m, m (ft)

Step 8. Estimate ΔH_{m-l} (the amount the manifold inlet pressure head, H_m, differs from the average lateral line inlet pressure head, H_l) for use in Eq. 23.4b. The ΔH_{m-l} is represented by the distance along the Q_m line between H_m and a line parallel to the ground slope representing the manifold's average outlet pressure head when it is equal to the H_l that gives the design q_a.The average lateral inlet pressure head or H_l line is positioned so the areas between it and the friction curve are the same above and below. To aid in locating the H_l line, place the transparent overlay on a piece of grid paper. Adjust the overlay (counting squares) until the above conditions are satisfied (see Fig. 23.3 C).

Step 9. Compute the manifold inlet pressure head, H_m, using Eq. 23.4b with ΔH_{m-l} from Step 8. For pairs of manifolds that operate simultaneously from the same regulating value, use the sum of the weighted (by length) uphill and downhill H_m values to compute the system H_m, i.e.:

$$\text{System } H_m = Y(H_m)_d + (1 - Y)(H_m)_u$$

Economic Chart Design Procedure

An economic pipe-size-selection chart, such as Fig. 8.7, can be used to select pipe sizes and lengths for manifolds serving rectangular subunits. The chart being used should be constructed for the desired pipe materials and wall thickness (or pressure ratings). (Figure 8.7 is designed for PVC thermoplastic IPS pipe with the same internal diameters used in constructing Fig. 23.2.) The general procedure for using the economic chart is presented in Chapter 8.

The procedure for the economic chart method for designing tapered manifolds is as follows:

Step 1. Compute the equivalent annual cost of escalating energy per water horsepower, E', by Eq. 8.19.

Step 2. Determine the system flow rate adjustment factor, A_f, by Eq. 8.20.

Step 3. Calculate the adjusted system flow for entering the chart, Q_s' by:

$$Q_s' = A_f Q_m \tag{8.21}$$

For a pair of manifolds use the flow rate in the downhill or longest manifold.

Step 4. Enter the vertical axis of the economic chart (see Fig. 8.7) with Q_s', and draw a horizontal line across the graph. The flow rates at the diameter change points can be read directly along the bottom axis. Where the Q_s' line intersects the upper limit of each pipe-size region is where the next larger pipe diameter should begin. Select no more than four different pipe sizes, so the smallest pipe is no less than half the diameter of the largest pipe.

Step 5. Use Eq. 23.7 to determine the length of each pipe segment beginning with the smallest diameter pipe.

Step 6. Determine the head loss, h_f, in the tapered manifold either by the *graphical method* using the unit friction-loss curves (see Fig. 23.2) or numerically. The *numerical method* presented for tapered sprinkler laterals in Chapter 9 can also be used for trickle irrigation system manifolds. However, a more convenient way to compute h_f for manifolds with more than two segments with different diameters of pipe can be derived using the same basic concepts. The resulting general equation for use with the flow rates at the diameter change points is:

$$h_f = \frac{FLK}{100\,Q_m}\left(\frac{(Q_1)^a}{(D_1)^c} + \frac{(Q_2)^a - (Q_1)^a}{(D_2)^c} \cdots + \frac{(Q_n)^a - (Q_{n-1})^a}{(D_n)^c}\right) \tag{23.8a}$$

And when the lengths of each pipe size are known:

$$h_f = \frac{FK}{100}\left(\frac{Q_m}{L}\right)^b \cdot \left(\frac{(x_1)^a}{(D_1)^c} + \frac{(x_2)^a - (x_1)^a}{(D_2)^c} \cdots + \frac{(x_n)^a - (x_{n-1})^a}{(D_n)^c}\right) \tag{23.8b}$$

where:

F = multiple outlet pipe friction reduction coefficient, which equals $1/a$
L = length of manifold, m (ft)
K = metric or English conversion constant from Eq. 8.7a or 8.7b
$a = 1.0 + b$
b = flow rate exponent from Eq. 8.7a or 8.7b
c = pipe diameter exponent from Eq. 8.7a or 8.7b
Q_1 = input flow rate to smallest pipe, L/s (gpm)
D_1 = diameter of smallest pipe, mm (in.)
Q_2 = input flow rate to next larger pipe, L/s (gpm)
D_2 = diameter of next larger pipe, mm (in.)
$Q_n = Q_m$ = maximum flow rate in largest pipe, L/s (gpm)
D_n = diameter of largest pipe, mm (in.)
$x_1 = L_1$ = length of smallest diameter pipe
$x_2 = L_1 + L_2$ = length of smallest plus length of next larger diameter pipe, m (ft)
$x_n = L$ = length of manifold, m (ft)

Figure 23.3 B shows the relationships between terms used for the lengths and flows through the different segments of pipe in a tapered manifold.

Step 7. Estimate the manifold pressure head variation, ΔH_m. If the *numerical method* was used to determine h_f, this can be done using Eq. 23.6a for uphill and level manifolds or Eq. 23.6b for downhill manifolds. If the *graphical method* was used to determine h_f, then ΔH_m can be determined by Eq. 23.6a for level and uphill laterals. However, for downhill manifolds use the following graphical procedure to determine it:

i. Draw a ground slope line on the overlay from the origin at (0, 0) to $\Delta E/k_m = \Delta E'$ at $Q = Q_m$. This represents the ground slope drawn to the same scale as the pipe-friction-curve segments.
ii. Draw another line parallel to the ground slope line and tangent to the lowest pressure head point along the sequential set of pipe-friction-curve segments on the overlay.
iii. Measure the maximum scalar distance between this "tangent" line and the (segmented) manifold friction curve and multiply it by k_m to obtain ΔH_m.

Step 8. If $\Delta H_m \leq 1.1\ (\Delta H_m)_a$, then the design is considered to be satisfactory. If $\Delta H_m > 1.1\ (\Delta H_m)_a$, then the manifold pipe sizes should be adjusted to reduce h_f. Small adjustments can usually be made by inspection. For large adjustments calculate a modified system flow rate Q_s'' for reentering the economic pipe-size selection chart. For level or uphill manifolds using the h_f computed in Step 6, let:

$$Q_s'' = \frac{h_f}{(\Delta H_m)_a - \Delta E} Q_s' \tag{23.9a}$$

and for downhill manifolds let:

$$Q_s'' = \frac{h_f}{(\Delta H_m)_a + (1.0 - 0.36/n)\Delta E} Q_s' \tag{23.9b}$$

Step 9. Repeat Steps 4 through 8 beginning with the modified system flow rate, Q_s'', until $(\Delta H_m)_a$ has been satisfied as specified in Step 8.

Step 10. Compute the manifold inlet pressure head, H_m, using Eq. 23.4a with the h_f from Step 6. For pairs of manifolds that operate simultaneously from the same regulating value, use the sum of the weighted (by length) uphill and downhill H_m, values to compute the system H_m as in *Step 9 of the graphical design procedure*.

Nonrectangular Subunits

The design procedure for tapered manifolds is similar for both rectangular and nonrectangular subunits. However, the reduction coefficient, F, used to compute friction loss in multiple-outlet pipelines must be adjusted for the subunit shape. The shape factor for the manifold is defined as:

$$S_f = \frac{(Q_l)_c}{(Q_l)_a} \tag{23.10}$$

where:

S_f = subunit shape factor

$(Q_l)_c$ = flow rate into the lateral (pair) at the closed end of the manifold, L/s (gpm)

$(Q_l)_a$ = average lateral (pair) flow rate along the manifold, L/s (gpm)

To compute the pressure head loss due to pipe friction in a nonrectangular manifold, modify Eq. 8.8a to:

$$h_f = J\, F_s F\, L/100 \tag{23.11}$$

where F_s = shape-adjustment factor for pipe-friction loss in manifolds serving nonrectangular subunits from Fig. 23.4 or Eq. 23.12.

Figure 23.4 is a plot of manifold pipe-friction-adjustment factors, F_s, for various configurations of trapezoidal subunits (Keller, 1980). It was generated by comparing multiple-outlet, pipe-friction-reduction coefficients for nonrectangular subunits with coefficients (F values) for rectangular subunits. Computations were made using a stepwise procedure. Manifolds with different numbers of outlets were analyzed, and the F_s values were found to be almost constant when there were more than 10 outlets. A regression analysis of the data (Lopez, 1985) gives:

$$F_s = 0.38 S_f^{1.25} + 0.62 \tag{23.12}$$

To use graphical methods to compute the friction head loss, h_f, for nonrectangular manifolds would require making special plots of the friction curves for each shape factor. Since nonrectangular subunits are uncommon, it is preferable to compute h_f by the numerical method presented in Chapter 9 for tapered sprinkle laterals. To do this Eq. 23.11 should be used in place of Eq. 8.8a to compute the h_f for each segment of pipe. The S_f and consequently F_s will be different for

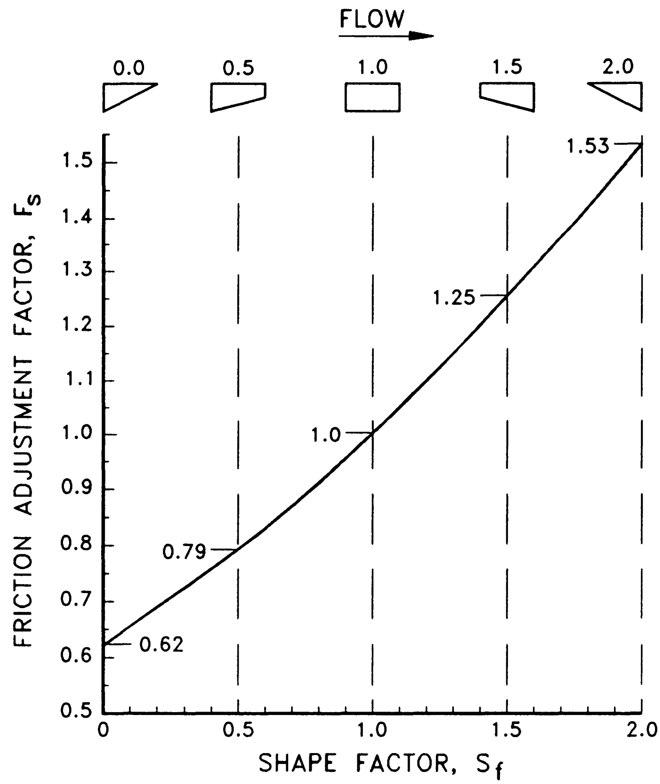


FIG. 23.4. Manifold Pipe Friction Adjustment Factors for Trapezoidal Shaped Subunits.

each segment of pipe. This is because the $(Q_l)_a$ for each segment of pipe (which must always include the closed end of the manifold) will be different. The pipe-friction loss in the tapered manifold can then be computed by a procedure similar to that given in conjunction with Eq. 9.4a.

Estimating h_f

The pressure head loss due to pipe friction, h_f'', can be estimated from the h_f of a similar manifold (or lateral) by:

$$h_f'' = h_f \frac{L''}{L} \frac{F_s''}{F_s} \left(\frac{Q_m''}{Q_m}\right)^{1.8} \tag{23.13}$$

in which the terms with double primes (h_f'', L'', F_s'', and Q_m'') designate the manifold for which the estimate is being made.

The estimated h_f'' will be quite accurate as long as the proportional lengths of the different sizes of pipe in the two tapered manifolds is constant, and the difference between the friction adjustment factors, F_s, is less than 0.25. If the lengths and subunit shapes are the same, the discharges can vary over a wide range without reducing the accuracy of the h_f'' estimate.

HGL Design Method

The friction curves for the set of pipe segments represented in parts A and B of Fig. 23.3 are tangent to a uniform hydraulic grade line, HGL. The HGL is specific for the manifold conditions and the design procedure. Instead of the graphical procedure described above, the numerical method described below (Boswell, 1985) can be used to obtain similar results.

Like the graphical procedure, the numerical method can be used only for manifolds serving equally spaced laterals having equal flow rates. Also it can be used only to find sets of friction-loss curves that are tangent to uniform hydraulic grade lines, HGLs. (Although not discussed above, the graphical procedure can be used to follow nonuniform HGLs.)

Basic Equations. The elevation of the hydraulic grade at any point x along a pipe friction curve that is tangent to an HGL with slope s can be computed directly. This can be done by adding h_{fx} to ΔH_c (see Fig. 22.4) to obtain:

$$H_x = h_{fx} + \Delta H_c \tag{23.14a}$$

where:

H_x = hydraulic grade at point x along a pipe-friction curve that is tangent to the HGL, m (ft)

h_{fx} = friction head loss from point x on a multiple outlet pipeline to the closed end, m (ft)
ΔH_c = difference between the closed end and minimum pressure head along a multiple outlet pipeline, m (ft)

Replacing h_{fx} with Eq. 22.3 and ΔH_c with Eq. 22.11 and rearranging gives:

$$H_x = \frac{JFL}{100}\left(\frac{x}{L}\right)^{1/a} + \frac{S^{a/b}}{J^{1/b}} \cdot \frac{L(1 - F)}{100} \tag{23.14b}$$

in which:

$$J = K\frac{(Q_m)^b}{D^c}; \qquad F = 1/a, \text{ and } a = (1 + b)$$

where:

F = multiple-outlet, friction-reduction coefficient used in Eq. 8.8 or 22.2 for a large number of outlets
L = manifold length, m (ft)
x = distance from the closed end of the manifold, m (ft)
S = absolute slope of the HGL to which the pipe friction curve is tangent, %
J = friction-head-loss gradient, m/100 m (ft/100 ft)
K = metric or English conversion constant from Eq. 8.7a or 8.7b
Q_m = manifold flow rate, L/s (gpm)
D = inside diameter of the pipe, mm (in.)
b and c = flow rate and pipe diameter exponents from Eq. 8.7a or 8.7b

There will be a series of intersections between pipe-friction-loss curves for adjacent segments with different pipe sizes (or diameters), as shown in Fig. 23.3. The locations of these intersections must be determined to select the necessary length of each size of pipe and find the pressure head variation along the manifold.

When the friction curves for different sizes of pipe are tangent to the HGL defined by S, their intersections are fixed. The intersection between the friction curves for a pipe with diameter D_1 and the next larger pipe with diameter D_2 will be at:

$$x_1 = \phi\left(\frac{b(D_2)^{c/b} - b(D_1)^{c/b}}{(D_1)^{-c} - (D_2)^{-c}}\right)^{1/a} \tag{23.15}$$

in which:

$$\phi = (L/Q_m)(S/K)^{1/b}$$

Design Steps. For the HGL method, the following steps should be used to design tapered manifolds serving rectangular subunits.

Step 1. Set H_x at the inlet end of the manifold at $(\Delta H_m)_a + \Delta E$ for downhill, at $(\Delta H_m)_a$ for level, and $(\Delta H_m)_a - \Delta E$ for uphill manifolds.

Step 2. For downhill manifolds find the smallest diameter pipe that gives $S \geq 100\ \Delta E/L$ using Eq. 23.14b with $x = L$ and H_x from Step 1. For level or uphill manifolds find the smallest diameter pipe that gives $h_f \leq H_x$ from Step 1. Then, determine the absolute percentage slope of the HGL to which the pipe friction curve is tangent, $S \geq 0$, using this diameter in Eq. 23.14b with $x = L$ and H_x from Step 1. With $x = L$ and H_x set, Eq. 23.14b can be rearranged as follows to simplify solving for S:

$$S = \left(\frac{(H_x - JFL/100)J^{1/b}}{(L/100)(1 - F)} \right)^{b/a} \tag{23.16}$$

Step 3. Starting with the D found in Step 2 as the inlet end pipe diameter, select up to three smaller pipe diameters, with the smallest no less than one-half the inlet pipe diameter.

Step 4. Using the S value from Step 2 in determining ϕ for use in Eq. 23.15, find the intersections between the friction-loss curves for the pipe diameters selected in Step 3.

Step 5. Determine the length of each size of pipe as follows, beginning with the smallest diameter pipe, L_1:

$$L_1 = (x_1 - 0)$$

$$L_2 = (x_2 - x_1)$$

$$L_n = (L - x_{n-1})$$

Step 6. Determine the head loss due to pipe friction, h_f, for the manifold by Eq. 23.8b.

Step 7. Determine the manifold inlet pressure required, H_m, by Eq. 23.4a.

Nonrectangular manifolds can be handled using this HGL design method by incorporating F_s into Eqs. 23.14a and the numerical method for computing h_f presented in Chapter 9. However, this is a tedious process, because F_s will be different for each pipe size segment. The shape factor, S_f, and consequently F_s, is dependent not only on the shape of the subunit, but also on the length and position of each pipe size segment. Therefore, it would be easier to compute h_f

step-by-step using Eq. 8.7a to calculate the friction head loss between each outlet.

Sample Calculation 23.1. *Designing the manifold for the line-source system presented in Sample Calculations 21.2 and 22.1 using the graphical method.*

GIVEN: From Sample Calculation 21.2, $\Delta H_s = 5.8$ ft (see Fig. 21.3); and from Sample Calculation 22.1 $\Delta H_l = 2.5$ ft. The manifold serves $N = 128$ laterals spaced at $S_l = S_r = 5$ ft and discharging $Q_l = 1.38$ gpm.

FIND: A suitable manifold design for the line-source system layout shown in Fig. 21.4; the manifold inlet pressure head H_m; and the best manifold inlet position, assuming the inlet location is not fixed (as in Fig. 21.4).

CALUCLATIONS: The allowable manifold pressure head variation is:

$$(\Delta H_m)_a = \Delta H_s - \Delta H_l$$

$$= 5.8 - 2.5 = 3.3 \text{ ft} \qquad (22.1)$$

Three possible manifold configurations that will stay within the small allowable $(\Delta H_m)_a = 3.3$ ft on the relatively steep 2% slope are:

i. A carefully tapered manifold for which the friction slope closely follows the ground slope.
ii A manifold with headers and pressure (or flow) regulators, as shown in Fig. 17.3.
iii. A manifold with flow regulators or jumper tubes of various lengths for each lateral to compensate for pressure variations along it.

It was decided that a carefully tapered manifold would be ideal for meeting the farm's long-term requirements providing the desired design precision could be achieved, i.e., EU of at least 80%. A tapered-manifold system should be cheaper, simpler, and more durable than a system requiring flow or pressure regulators. The graphical or HGL method is better than the economic chart method when designing tapered downhill manifolds with a small $(\Delta H_m)_a$, because the ΔH_m can be more accurately controlled. For this example the *graphical method* will be used. The manifold discharge is:

$$Q_m = N_l Q_l = 128 \times 1.38 = 177 \text{ gpm}$$

and its length is:

$$L = N_r S_r = 128 \times 5 = 640 \text{ ft}$$

because the length to the first outlet is a full (rather than a half) row spacing.

The difference in elevation along the downhill manifold is:

$$\Delta E = Ls = 640 \times 2/100 = 12.8 \text{ ft}$$

The next step is to convert $(\Delta H_m)_a$ and ΔE to scalar values for use with design overlays on Fig. 23.2. First determine k_m by Eq. 23.5b:

$$k_m = L/Q_m = 640/17 = 3.6$$

Then in accordance with Step 1 of the graphical procedure:

$$(\Delta H_m)'_a = 3.3/3.6 = 0.92 \text{ ft}$$

$$\Delta E' = 12.8/3.6 = 3.56 \text{ ft}$$

$$[(\Delta H_m)'_a + \Delta E'] = 0.92 + 3.56 = 4.48 \text{ ft}$$

Following *Steps 2 through 7* of the *graphical method*, construct the standard unit friction-curve overlay shown in Fig. 23.5. The sloping dashed line beginning at $Q = 0$ and $h'_f = (\Delta H_m)'_a = 0.92$ ft and the groundline define the design limits of pressure head variations. Any combination of different diameters of pipe that produce a set of friction curves falling between these parallel lines will satisfy the specified design conditions. However, the procedure outlined in *Steps 2 through 7* used in developing Fig. 23.5 provided a convenient way to obtain a satisfactory economic manifold design.

The length of the sections of pipe between the intersections of the pipe-friction curves in Fig. 23.5 can be computed by Eq. 23.7. For example, for the tapered manifold with four different diameters, the length of 2-in. pipe is:

$$L_D = \frac{56 - 32}{177} 640 = 87 \text{ ft}$$

A summary of the lengths of the four different pipe diameters is:

Pipe size (in.)	Inlet flow rate (gpm)		Length (ft)	Weight (lb)
$1\frac{1}{2}$	32		116	32
2	56		87	36
$2\frac{1}{2}$	95		141	87
3	177		296	219
		Totals	640	374

A simpler manifold configuration is a combination of 2- and 3-in. pipe as indicated by the curve extensions that cross at $Q = 71$ gpm in Fig. 23.5. A summary of the design with two pipe sizes is:

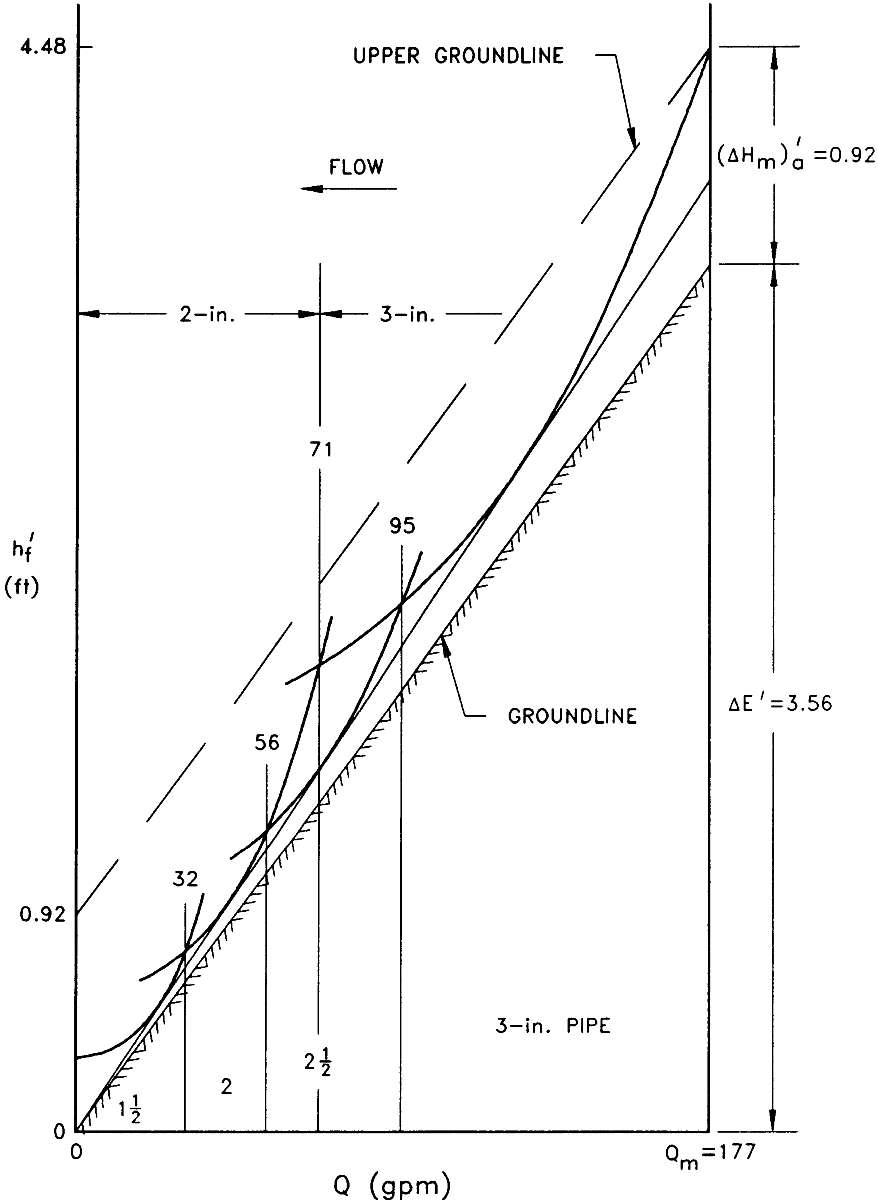


FIG. 23.5. Friction Curve Overlay to Demonstrate Graphical Method for Designing a Tapered Manifold Using Standard Unit Friction Loss Curves.

Pipe size (in.)	Inlet flow rate (gpm)		Length (ft)	Weight (lb)
2	71		257	107
3	177		383	284
		Totals	640	391

The weight of the pipe required for the design with four pipe sizes is 17 lb less; thus, the pipe itself would cost less. However, the extra complications associated with the installation of four rather than two different sizes of pipe would more than offset this cost differential, so the design with two pipe sizes was selected. The lengths should be adjusted to the nearest lateral spacing; thus there should be 260 ft of 2-in. and 380 ft of 3 in. pipe (see Fig. 21.4.)

In accordance with *Step 8* the amount the *manifold inlet pressure* differs from the average lateral inlet pressure head can be estimated graphically as demonstrated on Fig. 23.6 for the 2- and 3- in. pipe size design. The line parallel to and above the ground slope line is the average lateral inlet pressure head line (or H_l line). It is positioned so the cross hatched areas (defined by it and the 2- and 3-in. pipe friction curves) above and below it are approximately equal. The manifold inlet pressure is 0.54 graph units above it. Therefore, $\Delta H'_{m-l} = 0.54$, so $\Delta H_{m-l} = 3.6 \times 0.54 = 1.9$; and in accordance with *Step 9*:

$$H_m = H_l + \Delta H_{m-l}$$

$$= 11.1 + 1.9 = 13.0 \text{ ft} \qquad (23.4b)$$

The *manifold inlet is fixed at the uphill end* because of the water supply location. However, if the manifold inlet could be at any point along its length the *best inlet position* could be estimated using Eq. 23.3 or Fig. 23.1. To do this first compute:

$$\frac{\Delta E}{(\Delta H_m)_a} = \frac{12.8}{3.3} = 3.9$$

Then from Fig. 23.1:

$$Y = x/Lp \approx 0.9$$

Sample Calculation 23.2. *Manifold design for the drip system presented in Sample Calculations 21.1 and 22.3 using the economic chart design method.*

GIVEN: From Sample Calculation 21.1, $\Delta H_s = 16.0$ ft and $O_t = 2680$ hr/season (see Fig. 21.3), and from Sample Calculation 22.3 $H_l = 46.4$ ft and $\Delta H_l = 2.6$ ft. Each manifold serves $N = 54$ pairs of laterals spaced at $S_l = S_r$

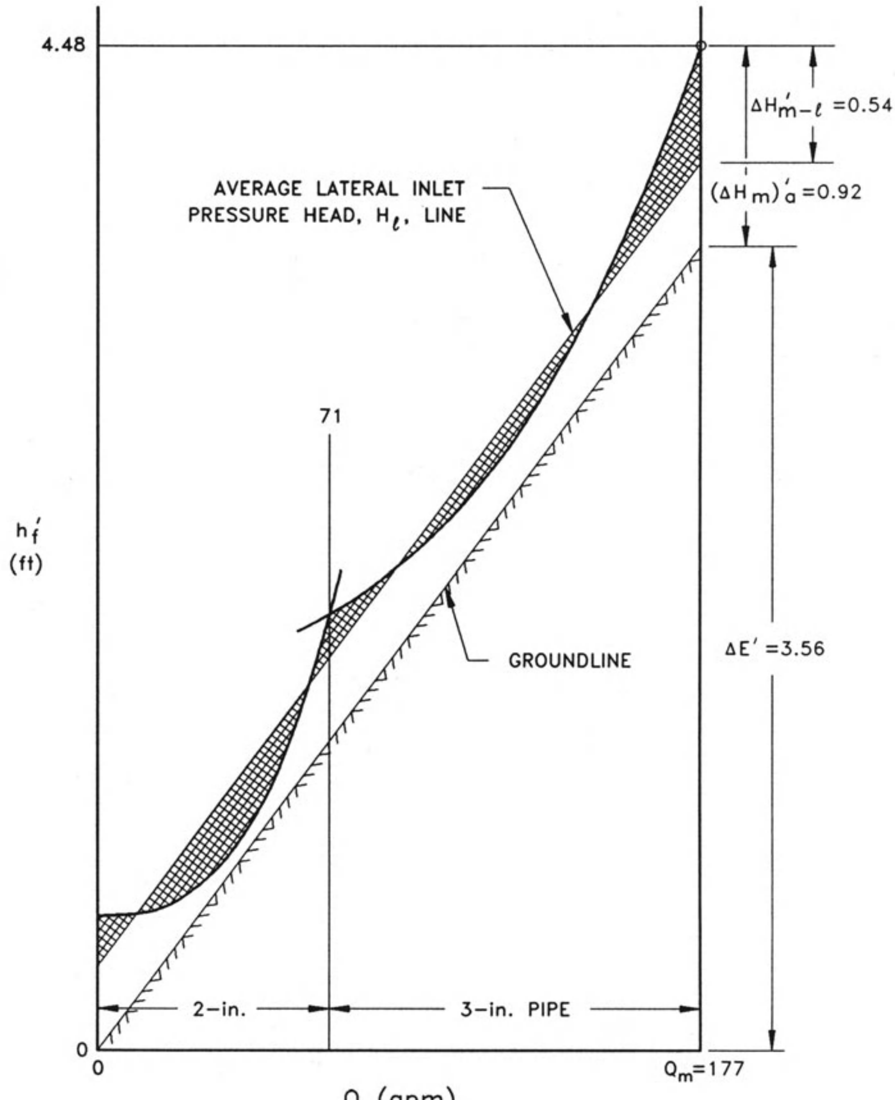


FIG. 23.6. Friction Curve Overlay to Demonstrate Graphical Method for Determining the Difference Between H_m and H_1, ΔH_{m-l}.

= 24 ft and discharging $(Q_l)_p = 2.0$ gpm per pair of laterals. The economic factors include:

Pump efficiency is $E_p = 75\%$;

Fuel cost per unit of brake power output is $C_f = \$0.036/\text{hp-hr}$;

Escalating energy cost factor is $EAE(9) = 1.594$;
Capital recovery factor for $n = 20$ years and $i = 20\%$ is $CRF = 0.205$; and
Plastic pipe cost per unit weight is $C_p = \$0.99/\text{lb}$.

FIND: A suitable manifold design for the system layout shown in Fig. 21.2 and the required manifold inlet pressure heads.

CALCULATIONS: For economic reasons and acceptable ΔH_s values, pairs of manifolds extending in opposite directions from a common main line connection should not exceed a total length of 450 m (1500 ft). Therefore, parallel main lines are needed. Main lines should be positioned so, starting from a common main line connection, along each pair of manifolds the minimum pressures are equal. Since the ground is level in the direction of the manifolds, they should also be of equal length (see Fig. 21.2).

Access roads replace the center row of trees in the west 80 A and in the east 40 A. Therefore, the length of each manifold is:

$$L = N_l S_l = 27 \times 24 = 648 \text{ ft}$$

And their flow rate is:

$$Q_m = N_l (Q_l)_p = 27 \times 2.0 = 54 \text{ gpm}$$

All the manifolds serve subunits with similar configurations, and extra pressure head can be used to reduce the pipe sizes in them all. Therefore, the standard manifold flow rate, $Q_m = 54$ gpm, should be used in place of Q_s for determining the initial adjusted system flow rate, Q_s'. The Q_s' is needed for entering the universal economic pipe-size-selection chart, Fig. 8.7, for which $K_{uc} = 0.001$.

To determine Q_s', first compute the annual cost of the escalating energy per water horsepower in accordance with *Step 1* of the *economic design method*, by:

$$E' = \frac{O_t \, C_f \, EAE(e)}{E_p/100} \tag{8.19}$$

$$= \frac{2680 \times 0.036 \times 1.594}{75/100} = \$205/\text{hp-year}$$

Then determine the flow-rate-adjustment factor (*Step 2*) by:

$$A_f = \frac{K_{uc} \, E'}{CRF \, C_p} \tag{8.20}$$

$$= \frac{0.001 \times 205}{0.205 \times 0.99} = 1.01$$

And (*Step 3*) by Eq. 8.21:

$$Q'_s = A_f Q_m = 1.01 \times 54 = 55 \text{ gpm}$$

Enter the vertical axis of Fig 8.7 with $Q'_s = 55$ gpm and draw a horizontal line (*Step 4*). Then note the flow rate along the horizontal axis where the 55-gpm line intersects the upper limit of each pipe size region. The flow-rate values should be adjusted to the nearest whole number of lateral connections. Then the length of each section of pipe can be computed using Eq. 23.7 (*Step 5*). A summary of the diameter and length of the pipe sections making up the manifold is:

Pipe diameter		Flow rate		Section
Nominal (in.)	D (in.)	Chart (gpm)	Adjusted (gpm)	length (ft)
$1\frac{1}{4}$	1.532	10.5	10	120
$1\frac{1}{2}$	1.754	20.2	20	120
2	2.193	45.0	46	312
$2\frac{1}{2}$	2.655	54.0	54	96

The head loss due to pipe friction can be computed using Eq. 23.8a with: $F = 0.36$; $L = 648$ ft; $K = 0.133$; $Q_m = 54$; $a = 1 + 1.75 = 2.75$; $c = 4.75$; and the above adjusted flow rates and respective inside pipe diameters, D (*Step 6*). Thus:

$$h_f = \frac{0.36 \times 648 \times 0.133}{100 \times 54}\left(\frac{10^{2.75}}{(1.532)^{4.75}} + \frac{20^{2.75} - 10^{2.75}}{(1.754)^{4.75}} + \frac{46^{2.75} - 20^{2.75}}{(2.193)^{4.75}} + \frac{54^{2.75} - 46^{2.75}}{(2.655)^{4.75}}\right) = 7.5 \text{ ft}$$

The head variation along the manifold can now be computed by Eq. 23.6a (*Step 7*). Since there is no slope along the manifolds:

$$\Delta H_m = h_f = 7.5 \text{ ft}$$

This is less than the allowable difference in manifold pressure head, which, using Eq. 23.1 is:

$$(\Delta H_m)_a = \Delta H_s - \Delta H_l = 16.0 - 2.6 = 13.4 \text{ ft}$$

Therefore, no further pipe-size adjustments are necessary for the critical manifolds. (To increase the inlet pressure head to the critical manifolds would necessitate increasing the total dynamic head of the system.) Adjustments could be made using the procedure outlined in *Step 8* to take advantage of any extra head available for all noncritical manifolds after completing the main line design.

The *manifold inlet pressure head* that will give an average lateral inlet pressure head of $H_l = 46.4$ ft can now be computed (*Step 9*) by:

$$H_m = H_l + k\,h_f + 0.5\,\Delta El \tag{23.4}$$

$$= 46.4 + 0.5 \times 7.5 + 0 = 50.2 \text{ ft}$$

since $k \approx 0.5$ for tapered manifolds with three or more pipe diameters.

Because the 12 manifolds are similar (see Fig. 21.1), the same pipe sizes and inlet pressures can be used for them all to standardize the layout (see Fig. 21.1.)

Sample Calculation 23.3. *Manifold design for the spray system presented in Sample Calculations 21.3 and 22.2, using the HGL design method.*

GIVEN: From Sample Calculation 21.3, $\Delta H_s = 13.2$ ft, and the number of operating stations, $N_s = 4$ (see Fig. 21.3); and from Sample Calculation 22.2, $H_l = 63.3$ ft, and $\Delta H_l = 6.4$ ft for the laterals serving rectangular subunits. In the rectangular subunits each manifold serves pairs of laterals spaced at $S_l = S_r = 25$ ft and discharging $2\,Q_l = 2 \times 3.42 = 6.84$ gpm per pair of laterals. Some of the subunits are not rectangular and will have shorter pairs of laterals with lower discharges.

FIND: A suitable design for the manifolds in the system layout shown in Fig. 21.5 and the required manifold inlet pressure heads using the *HGL design method.* (Assume 2.5 in. diameter pipe is not available.)

CALCULATIONS: As the field is nearly level, the main line should be placed down the center. Thus it will supply equal-length manifolds to the east and west and the number of laterals served by each manifold is $N_l = 26$, as shown in Fig. 21.5. The manifolds should also be placed along the center of each rectangular subunit.

The length of each manifold is:

$$L = (N_l - 0.5)S_l \tag{23.2a}$$

$$= (26 - 0.5)25 = 637.5 \text{ ft}$$

The flow rate into each of the manifolds serving the rectangular subunits is:

$$Q_m = N_l(Q_l)_p = 26 \times 6.84 = 178 \text{ gpm}$$

Manifold 4 (see Fig. 21.5) has a flow rate of:

$$(Q_m)_4 = 26 \times 6.84 \left(\frac{36 + 22}{2 \times 36}\right) = 143 \text{ gpm}$$

And manifold 5, serving the small triangular area, requires:

$$(Q_m)_5 = 26 \times 6.84 \left(\frac{14 + 0}{72}\right) = 35 \text{ gpm}$$

The design calls for four operating stations. The operating sequence that will give an equal flow rate for each station is:

Station	Manifold	Q_s (gpm)
I	(1)	178
II	(2)	178
III	(3)	178
V	(4 & 5)	143 + 35 = 178

The same manifold configurations can be used for all three rectangular subunits. However, manifolds 4 and 5 will require different configurations.

The steps for the HGL design procedure are essentially the same as for the graphical procedure. By Eq. 23.1:

$$(\Delta H_m)_a = \Delta H_s - \Delta H_l = 13.2 - 6.4 = 6.8 \text{ ft}$$

Since the manifolds are level, by *Step 1* of the *HGL design method*, H_x = 6.8 ft. In accordance with *Step 2*, the smallest diameter pipe that will give $h_f \leq H_x$ can be found by combining Eqs. 8.7a and 8.8a to obtain:

$$H_x = h_f = K \frac{Q_m^{1.75}}{D^{4.75}} FL/100$$

Then solving for D to obtain:

$$6.8 = 0.133 \frac{(178)^{1.75}}{D^{4.75}} \times 0.36 \times 637.5/100$$

$$D = 3.5 \text{ in.}$$

The inside diameter of 3-in. pipe is 3.284 in. and of 4-in. is 4.280 in. Therefore, the largest pipe in the tapered manifold must be 4-in.

Then, in accordance with *Step 2*, the slope, S, of the HGL that is tangent to the D = 4-in. pipe friction curve can be computed by Eq. 23.16. To do this first determine J by Eq. 8.7a:

$$J = \frac{K(Q_m)^b}{D^c} = \frac{0.133 \times (178)^{1.75}}{(4.280)^{4.75}} = 1.155$$

Then letting $x = L = 637.5$ ft; $H_x = 6.8$ ft; $a = 2.75$; and $F = 1/2.75$ in Eq. 23.16 gives:

$$S = \left(\frac{(6.8 - 6.375 \times 1.155/2.75)(1.155)^{1/1.75}}{6.375(1 - 1/2.75)} \right)^{1.75/2\ 75}$$

$$= 1.065\%$$

In accordance with *Step 3*, the tapered manifold may have 2-, 3-, and 4-in. pipe. The locations of the intersections between the friction curves for the different sizes of pipe should be determined next. This can be done using Eq. 23.15 (*Step 4*). For example, the intersection between the friction curves for 2-in. ($D_2 = 2.193$ in.) and 3-in. ($D_3 = 3.284$ in.) can be found as follows:

$$\phi = (L/Q_m)(S/K)^{1/b}$$

$$= (637.5/178) \times (1.065/0.133)^{1/1.75}$$

$$= 11.756$$

and

$$c/b = 4.75/1.75 = 2.714$$

$$a = 1 + b = 2.75$$

Then using Eq. 23.15 to obtain:

$$x_2 = 11.756 \left(\frac{1.75(D_3)^{2.71} - 1.75(D_2)^{2.71}}{(D_2)^{-4.75} - (D_3)^{-4.75}} \right)^{1/2\ 75}$$

$$= 165 \text{ ft}$$

Repeating the above computations for 3- and 4-in. ($D_4 = 4.280$ in.) pipe gives:

$$x_3 = 420 \text{ ft}$$

In accordance with *Step 5*, the required lengths of pipe are:

$$L_2 = 165 \text{ ft}$$

$$L_3 = 420 - 165 = 255 \text{ ft}$$

$$L_4 = 637.5 - 420 = 217.5 \text{ ft}$$

At this point the head loss due to pipe friction can be determined (*Step 6*) using Eq. 23.8b with $F = 1/a$ to obtain:

$$h_f = \frac{1 \times 0.133}{2.75 \times 100} \times \left(\frac{178}{637.5}\right)^{1.75}$$

$$\times \left(\frac{(165)^{2.75}}{(2.193)^{4\ 75}} + \frac{(420)^{2\ 75} - (165)^{2.75}}{(3.284)^{4.75}} + \frac{(637.5)^{2\ 75} - (420)^{2.75}}{(4.280)^{4.75}} \right) = 6.15 \text{ ft}$$

The inlet pressure head for manifolds (1), (2), and (3) serving the rectangular subunits can now be determined (*Step 7*) by:

$$H_m = H_l + k\,h_f + 0.5\,\Delta El \tag{23.4a}$$

$$= 63.3 + 0.5 \times 6.15 + 0 = 66.4 \text{ ft}$$

For comparison, the graphical design procedure was employed to produce the overlay presented in Fig. 23.7. The scale factor $k_m = 3.65$ and $H'_x = 1.86$. The intersection between the 2- and 3-in. pipe friction curves occurs at $Q_2 = 46$ gpm which gives:

$$x_2 = (46/178)637.5 = 164 \text{ ft}$$

The intersection between the 3- and 4-in. curves is at $Q_3 = 115$ gpm, which gives $x_3 = 412$ ft. Furthermore, the pipe friction loss is:

$$h_f = (1.86 - 0.19) \times 3.65 = 6.10 \text{ ft}$$

These are practically the same as the values obtained by the HGL method.

Manifolds (4) and (5) serve *nonrectangular subunits* (see Fig. 21.5). The shape factor for manifold (4) with $(Q_m)_4 = 143$ gpm is:

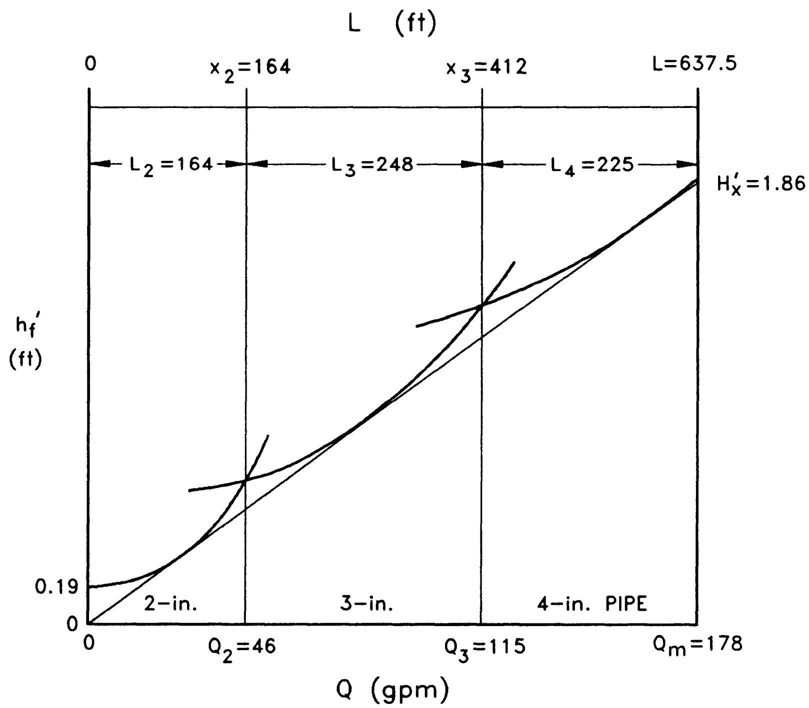


FIG. 23.7. Friction Curve Overlay for Graphical Design of a Manifold Serving a Rectangular Subunit for the Spray System.

$$(S_f)_4 = \frac{(Q_l)_c}{(Q_l)_a} = \frac{(22/36)6.84}{143/26} = 0.76 \qquad (23.10)$$

And for manifold (5) with $(Q_m)_5 = 35$ gpm it is:

$$(S_f)_5 = \frac{(14/36)6.84}{35/26} = 1.98$$

The friction-loss adjustment factor for the two manifolds can now be determined by:

$$(F_s)_4 = 0.38\ S_f^{1.25} + 0.62 \qquad (23.12)$$

$$= 0.38(0.76)^{1.25} + 0.62 = 0.89$$

And:

$$(F_s)_5 = 0.38(1.98)^{1\ 25} + 0.62 = 1.51$$

Using the same configuration for manifold (4) as for the rectangular subunits would make installation more convenient. Equation 23.13 can be used to estimate the h_f, for the difference between the friction-adjustment factors is only $1.0 - 0.88 = 0.12$. Thus:

$$(H_f)_4 = \frac{L_4}{L}\frac{(F_s)_4}{F_s}\left[\frac{(Q_m)_4}{Q_m}\right]^{1\ 8} h_f \qquad (23.13)$$

$$= \frac{637.5}{637.5} \times \frac{0.88}{1.0} \times \left[\frac{143}{178}\right]^{1.8} \times 6.15 = 3.6 \text{ ft}$$

For simplicity and better flushing, manifold (5) could be constructed of all 2-in.-diameter pipe. This would give a head loss of:

$$(h_f)_5 = F_s F(L/100)J \qquad (23.11)$$

$$= \frac{1.51}{2.75} \times \frac{637.5}{100} \times 0.133\ \frac{(35)^{1.75}}{(2.193)^{4.75}} = 5.6 \text{ ft}$$

This is acceptable because $(\Delta H_m)_a = 6.8$ ft. The required inlet pressure heads for manifolds (4) and (5) by Eq. 23.4a would be:

$$(H_m)_4 \approx 63.3 + 0.5 \times 3.6 = 65.1 \text{ ft}$$

and

$$(H_m)_5 \approx 63.3 + 0.75 \times 5.6 = 67.5 \text{ ft}$$

REFERENCES

Boswell, M. J. 1985. *Micro-Irrigation Design Manual*, pp. (6)27–30. El Cajon, California: James Hardie Irrigation.

Herbert, E. 1971. *Hydraulic Design—The Use of Polyplot*. Melbourne, Australia: Imperial Chemical Hydraulics of Australia and New Zealand Ltd.

Jobling, G. A. 1976. *Trickle Irrigation Design Manual, Part 2*. New Zealand Agricultural Engineering Institute, Miscellaneous Publication No. 7.

Keller, Jack. 1980. Trickle irrigation manifold design. *ASAE Technical Paper* No. 80-2058.

Lopez, Rodriqo J. 1985. Design of manifolds for non-rectangular drip irrigation manifolds. In *Drip/Trickle Irrigation in Action*, Proceedings of Third International Drip/Trickle Irrigation Congress, Nov. 18–21 1985. Fresno, California, Volume II, pp. 878–886. St. Joseph, Michigan: American Society of Agricultural Engineers.

Wu, I. P., and H. Gitlin. 1977. Design of drip irrigation lines with varying pipe sizes. *Journal of the Irrigation and Drainage Division*, ASCE 103(IR4): 499–503.

RECOMMENDED READING

Benami, A., and A. Ofen. 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).

Boswell, M. J. 1985. *Micro-Irrigation Design Manual*. El Cajon, California: James Hardie Irrigation.

Wu, I. P., H. M. Gitlin, K. H. Solomon, and C. A. Saruwatari. 1986. Design principles: System design. In *Trickle Irritation for Corp Production*, eds. F. S. Nakayama and D. A. Bucks, pp. 53–92. Amsterdam: Elsevier.

24

Trickle System Design Synthesis

The prescribed strategy for designing trickle irrigation systems focuses on providing an optimum supply of water to each plant. The focal point is the spacing, amount, and uniformity of the discharge from the emitters after the decision to proceed has been made. The design process involves three phases.

The *first phase of the design process* is presented in Chapters 19, 20, and 21. Chapter 19 covers the specific planning factors related to trickle systems. Chapter 20 covers emitter selection and setting up the design criteria and parameters for the hydraulic network. Chapter 21 presents a synthesis of this first phase of the design strategy. This is done in the form of sample calculations for a drip, a line-source, and a spray-type trickle irrigation system.

The *second phase of the design process* involved dividing the system into subunits and designing the hydraulic networks for each of them. Each subunit is made up of a manifold that serves a set of laterals supplying emitters. The hydraulic design of the laterals is covered in Chapter 22. The design of the manifolds that serve them is covered in Chapter 23. The three sample calculations are continued through the subunit design phase in Chapters 22 and 23.

The *third and final phase of the process* is to design the main line network that supplies the subunits, the control head, and the pressurized water supply. This remains to be done, but before proceeding it would be useful to reflect on the steps leading up to this point.

The objective of the first two design phases is to have very uniform emitter discharges throughout the system with a minimum of pressure- or flow-regulation points. In addition the spacing between emission points and the discharge should be sufficient to meet optimum plant water requirements. A brief summary of these first two phases is to be followed by a presentation of the design procedures for the third phase. Sample calculations to complete the drip, line-source, and spray system designs are also included to demonstrate the final design procedures.

DESIGN PARAMETERS

Trickle irrigation systems are usually designed and managed to deliver frequent, light applications of water and to wet only a portion of the soil surface. Therefore, the procedures used for other methods must be adjusted to compute water- and salinity-control requirements, irrigation depth, and frequency.

Soil Wetting

Trickle irrigation systems normally wet only a portion of the horizontal, cross-sectional area of soil, as depicted in Fig. 2.7. The percentage wetted area, P_w, compared to the entire cropped area, depends on the volume and rate of discharge at each emission point, spacing of emission points, and type of soil being irrigated.

No single "right" or proper minimum value for P_w has been established. Nevertheless, systems having high P_w values provide more stored water (a valuable protection in the event of system failure). Consequently, they should be easier to schedule and bring more of the soil system into action for storage and supply of nutrients. A reasonable design objective of such widely spaced crops as vines, bushes, and trees is to wet at least one-third and as much as two-thirds of the potential horizontal cross-sectional areas of the root systems, i.e., $33\% < P_w < 67\%$. However, in closely spaced crops with rows and emitter laterals spaced less than 1.8 m (6.0 foot) apart, P_w often approaches 100%.

Procedures for estimating P_w are presented in Chapter 19. These procedures cover the effects of different emitter layout and discharge configurations on various types of soil.

Salinity Control

Crop yields should equal or slightly exceed those produced under other methods of irrigation with good-quality irrigation water. However, when water is of poor quality, yields under trickle irrigation are usually considerably higher than under other methods, but not as high as from good-quality water. This yield advantage is because the salts remain diluted by the continuous high soil moisture resulting from frequent replenishment of the water lost by evapotranspiration. Frequent sprinkle irrigation applications might give similar results, but leaf burn would be a problem for many crops if the water were saline.

A procedure for allowing for leaching requirements in determining system capacities and management criteria is presented in Chapter 19. It also includes a means for estimating the reduction in yield for various crops resulting from water of various qualities.

Water Requirements

The plant canopies of young and wide-spaced crops shade only a portion of the soil surface area and intercept a portion of the incoming radiation. Conventional estimates of water requirements of young crops assume that part of the applied water will be lost to nonbeneficial, consumptive use. This loss is through evaporation from the wetted soil surface or through transpiration from undesirable vegetation.

Trickle irrigation reduces evaporation losses to a minimum, so transpiration by the crop accounts for practically all of the water consumed. Therefore, estimates of consumptive use that assume the entire field surface will be wetted should be modified for trickle irrigation.

The transpiration rate under trickle irrigation is a function of the conventionally computed consumptive use rate and the extent of the plant canopy. Furthermore, since only part of the soil is wetted under many trickle systems, the conventionally computed soil moisture storage capacity must be adjusted accordingly.

Chapter 19 covers the methods and design criteria for dealing with the above effects on system design capacities and irrigation scheduling. It provides a means for designing the net system water requirements, determining the system efficiency, and developing the gross water requirements.

Emitter Selection

Emitter selection requires a combination of objective and subjective judgments. Along with the related requirements for water treatment, selection of an appropriate emitter is the most nebulous aspect of the trickle irrigation design process. The selection process is not simply a matter of following a checklist of instructions because the consequences of one decision will alter the assumptions used in making other decisions.

The quality and safety of trickle systems are affected directly by: the emitter design and quality; the percentage area wetted, allowable variation of pressure; adequacy of filtration; degree of automation; and reliability of the management, labor, power, and water supplies. The two most important of these items are the percentage area wetted and the reliability of the emitter against clogging and malfunctioning.

Initially, selection of an emitter depends on the soil to be wetted, the plant requirement for water, the emitter discharge, the quality of the water, and the terrain of a particular location. The choice of a particular emitter should follow a detailed evaluation of the various features discussed in Chapter 20. Evaluation must include cost of the emitter and risks inherent in the system. Generally, the emitters that offer the more desirable features and pose the fewest system risks cost more per unit. Whatever emitter is considered initially will influence the estimated cost of the pipe network and filtration system; the original choice may need to be reevaluated before an emitter for the system is finally selected.

Although they are difficult to attain, an ideal emitter should have the following attributes:

- Durability;
- Low cost;

- Reliable performance with a relatively low rate of discharge that is reasonably uniform among all emitters within the system despite the variances in tolerance inherent in manufacturing and the expected differences in pressure head due to friction loss and elevation; and
- Relatively large and/or self-flushing passageways to reduce or prevent clogging.

Emission Uniformity

It is necessary to know the efficiency of the irrigation system so the relationship between gross irrigation amounts and net additions to the root zone can be established. Emission uniformity, EU, is important because it is one of the two components of irrigation efficiency; the other is various losses that occur during operation of the system.

In the design phase, it is not possible to measure the emission rates throughout the intended system. The variation to be expected in emission rates must be estimated by some analytical procedure. Unfortunately, it is not practical to consider all the influencing factors in a design formula for emission uniformity.

It is not possible to look at a design and compute or even satisfactorily estimate the unpredictable variations in emission rates that may be caused by such factors as full or partial clogging, changes in water temperature, and aging of emitters. However, the other items in emission uniformity can be known. The manufacturer should provide information about the relation of pressure to rate of emission and also about manufacturing variability for the emitter. Topographic data from the intended site and a hydraulic analysis of the proposed pipe network can give the needed information about what variations in pressure to expect.

A means for estimating the design EU and guidelines for establishing reasonable EU design criteria are presented in Chapter 20. Furthermore, a procedure for determining the allowable pressure head variation that will result in the desired EU is presented.

Design Criteria

The choices of discharge, spacing, and the emitter itself are major items in system planning. They are dictated partly by physical data, and also by such factors as emitter placement, type of operation, lateral diameter, and user preference.

Setting up the design criteria and parameters for the hydraulic network requires four steps. First is evaluation and choice of the general type of emitter that best fits the needs of the area to be wetted. Then, according to the system's required discharge, spacing, and other planning considerations, choose the specific emitter needed. Third, determine the discharge, q_a, and pressure head, h_a of the average emitter. Fourth, determine what variation in subunit pressure head, ΔH_s, is allowable and will give the desired EU.

The sample calculations presented in Chapter 21 demonstrate the process for doing this. The data that must be collected prior to beginning the design computations are summarized in the "Trickle Irrigation Design Data" form presented as Fig. 19.5 or 21.1. This form was developed as a guide to organize the gathering of necessary field and equipment data.

The computations include the emitter spacing, average emitter discharge, average emitter pressure head, allowable head variation, and the hours of operation per season. The steps for developing these factors are outlined in the "Trickle Irrigation Design Factors" form presented as Fig. 19.6 or 21.3. This data sheet is a useful guide and provides a convenient place to record results of various trial and final computations.

Some systems require extra capacity because of anticipated slow changes in the average emitter discharge, q_a, with time. Decreases in q_a can result from such things as slow clogging due to sedimentation in long-path emitters or compression of resilient parts in compensating emitters. Increases in q_a can result from mechanical fatigue of the flexible orifices in continuously and periodic-flushing emitters or increases in minor leakage due to fatigue in emitters and tubing.

Both decreases and increases in q_a necessitate periodic cleaning or replacement of emitters. To compensate for a decrease in discharge rate, the system must either be operated at a higher pressure or for a longer time during each irrigation application. To prevent the need for frequent cleaning or replacement of emitters, where decreasing discharge rates are a potential problem, the system should be designed with 10 to 20% extra capacity. A possible alternative is to provide sufficient reserve operating pressure so the pressure can be increased as required to hold q_a constant until the emitter discharge characteristics have degenerated by 10 to 20%.

SUBUNIT DESIGN

First the subunits must be configured and then the hydraulic networks designed for each of them. The procedures for doing this are straightforward, but sometimes one or two trials are necessary before a suitable subunit configuration and hydraulic solution is found.

Subunit Layout

The layout of the subunits depends on the following:

- Plant and emitter spacing;
- Average emitter flow rate and allowable pressure head variations;
- Desired number of operating stations;
- Overall length of plant rows in the field or subsets of it;
- Number of plant rows in the field or subsets of it; and
- Field topography and boundaries.

The final layout is usually a compromise in an effort to satisfy a number of design objectives that are not always compatible. Ideally the final subunit layout should lead to:

- A minimum number of subunits and pressure- or flow-control points;
- A convenient and economical main line layout to serve them;
- Having the same total system flow rate for each operating station;
- Subunit configurations that are uniform in size and shape;
- Single-pipe-size (not tapered) laterals made up of 12-mm ($\frac{1}{2}$-in.) hose or at most 20-mm ($\frac{3}{4}$-in.) hose;
- Tapered manifolds with 100-mm (4-in.) and smaller pipe; and
- Pressure head variations that do not exceed the allowable variation established for meeting the desired emission uniformity.

Some strategies for achieving these multiple objectives are given in conjunction with the sample calculations presented in Chapters 22 and 23. Rectangular subunits are the norm and are the easiest to design. The subunit (in the lateral direction) is governed by: the emitter spacing and discharge; the economics of using small-diameter lateral hose; the allowable pressure head variations; the length of the rows and number of operating stations; and the topography.

The subunit width (in the manifold direction) is governed by similar criteria. It is governed by the lateral spacing and discharge; the economics of using small-diameter pipe; the allowable pressure head variations; the number of laterals and number of operating stations; and the topography of the field.

Lateral Design

The laterals must be designed before the manifolds can be designed. Normally they should have only one size of pipe. On fields where the average slope in the direction of the laterals is less than 3%, it is usually most economical to supply laterals to both sides of each manifold. The manifold should be positioned so that, starting from a common manifold connection, the minimum pressures along the pair of laterals (one to either side of the manifold) is equal. Thus, where the ground slopes in the direction of the laterals (rows) the manifold should be shifted uphill. The effect is to shorten the upslope laterals and lengthen the downslope laterals so the combination of pipe-friction losses and elevation differences are in balance.

Selecting the length of the subunits and consequently the length of the laterals is a compromise between field geometry and lateral hydraulics. To set practical limits for preliminary design purposes it is useful to limit the lateral pressure head difference, ΔH_l, to 0.5 ΔH_s, where the manifold plus attached laterals make up a subunit. The ΔH_l for a given manifold spacing and set of lateral specifications is about the same for level fields as for laterals that have slopes of as much as 2.5%. Using this fact helps in developing the subunit layout.

Both graphical and numerical solutions for designing laterals are developed and presented in Chapter 22. The procedures are based on an analysis of an average lateral having the desired average emitter discharge, q_a and include determining such lateral characteristics as: flow rate, Q_l, and inlet pressure; H_l; locating and spacing the manifolds, which in effect sets the lateral lengths; and estimating the differences in pressure within laterals, ΔH_l. Examples of the application of these procedures for various site conditions are given in the progressive sample calculations for the drip, line-source, and spray systems. In some instances the first trial solution is not workable and must be revised.

Manifold Design

Trickle irrigation manifolds differ from laterals in that flow rates are much higher and they are usually tapered, with up to four different pipe sizes. This is done to economize on pipe costs and to keep the pressure head variations within the desired limits. However, to assure adequate flushing, the smallest pipe should be no less than half the diameter of the largest.

As with laterals, where the average slope along the manifolds is less than 3%, it is usually most economical to have manifolds extending in both directions from the main line. The main line should be positioned so the minimum pressure along each length of a pair of manifolds extending from a common outlet is about equal. Therefore, on sloping fields the uphill manifolds should be shorter than the downhill manifolds, so the combination of friction losses and elevation differences is in balance.

The main line and manifold layout is a compromise between field geometry and manifold hydraulics. The allowable manifold pressure head variation is ΔH_s minus the pressure head difference ΔH_l that must be allowed for the laterals.

For simplification the hydraulic design procedure is based on assuming an average emitter discharge, q_a, throughout the subunit served by each manifold. Thus for manifolds serving rectangular subunits the lateral flow rate is assumed to be constant.

Manifold design procedures are covered in Chapter 23. They include determining the following manifold characteristics: flow rate, Q_m; inlet location; pipe sizes to keep within the desired pressure head differential; and inlet pressure, H_m, needed to give the desired average emitter discharge, q_a. Both graphical and numerical procedures are presented for handling the pipeline hydraulics. The numerical procedures lead to direct (rather than iterative) solutions for manifolds on uniform slopes.

The three progressive sample calculations are continued for the manifold design to demonstrate the application of the various design procedures. This completes the design of the drip, line-source, and spray systems through the layout of the subunits and the hydraulic networks within them. The fourth and final phase of the process is to design the system to supply filtered water to the subunits.

MAIN LINE DESIGN AND TDH

The control head components and main pipe network must be specified to complete the trickle system design (see Figs. 2.6 and 17.2). To design the pumping plant the total dynamic head, TDH, that will be required at the system inlet must be estimated. Therefore, the anticipated head loss through the control head and main pipe network must be determined.

Control Head

The control head must be located upstream from all manifolds. It is the assemblage of special equipment needed to protect the system against clogging, provide fertigation, and monitor system performance. The most important items at the control head are the filtration system, injection equipment, and flow meter.

Clogging of emitters is the most difficult problem encountered in the operation of trickle irrigation systems. Emitters can be clogged by particles or by precipitates or bacterial slimes resulting from dissolved calcium or other salts in the water supply. Filtering and keeping contaminants out of the system are the main defense against clogging caused by mineral and organic particles. Periodic chemical injections are often necessary to dissolve mineral precipitates and prevent the growth of algae and slimes.

In arid areas fertigation is necessary to supply sufficient fertility, especially nitrogen, for fields irrigated with drip or line-source trickle systems. This is because dry fertilizer broadcast over the soil surface will not be moved into the plant root zone by the irrigation water. The same type of equipment can be used to inject either fertilizer solutions or chemicals that help prevent emitters from clogging.

Details about chemigation in general and fertigation in particular are presented in Chapter 16. Clogging and filtration are covered in Chapter 18. Usually about 70 kPa (10 psi) must be provided to compensate for the pressure loss across the filter system just prior to flushing.

Where chemical injection pumps are used, there is no pressure loss associated with the process. However, differential pressure injectors require approximately 14 kPa (2 psi) of pressure difference. This will be lost unless a venture pipe section is used. The loss caused by an appropriately sized flow meter will also be about 14 kPa (2 psi).

Main Line Design

The main lines must supply filtered and treated water to each subunit. It is usually best to lay out the main lines so: the flow is split or divided as close to the supply end as possible; their direction is parallel or at right angles to the rows; they are not looped; and their length is as short as practical.

Similar procedures can be used for designing main lines for trickle systems as those presented in Chapters 8 and 10 for sprinkle systems. A means for adjusting the inlet pressure head, H_m, to each subunit should be provided, so the pressure head can vary along the main line without adversely affecting emission uniformity.

The basic requirements for a good main line design are to follow life-cycle, economic, pipe-size-selection procedures and keep flow velocities below 1.5 m/s (5 ft/s). Either a Universal Economic Pipe-Selection Chart like Fig. 8.7, or the completely numerical method presented in Chapter 8 can be used in the design process.

The economic design procedure using Fig. 8.7 involves the following steps:

Step 1. Select the main line layout and compute the total system capacity, Q_s. This is equal to the sum of the subunit flow rates divided by the number of operating stations.

Step 2. Determine the adjusted system flow rate, Q_s', for entering Fig. 8.7. This is done by Eq. 8.21 using Eqs. 8.19 and 8.20 to compute A_f.

Step 3. Enter the vertical axis of Fig. 8.7 with Q_s' and select an economic pipe size for the flow, Q, in each section of main line pipe. (To hold velocities below 1.5 m/s (5 ft/s), stay within the solid vertical boundary lines.)

Step 4. Compute the head loss due to pipe friction, h_f, in each section of main line for each operating station by Eqs. 8.7 and 8.8.

Step 5. Determine the pressure head difference, H_{fe}, due to pipe friction, h_f, and elevations, ΔH_e, between the control head and each manifold inlet:

$$H_{fe} = h_f + \Delta H_e$$

Step 6. Compute $(H_m + H_{fe})$ for each manifold. The manifold with the largest value establishes the required discharge pressure at the control head. This will be referred to as the *critical manifold inlet*, and the sections of main line leading to it as the *critical main line section*.

Step 7. The critical section of main line cannot be changed without increasing the required inlet pressure. However, the pipe sizes in other parts of the main line system can be reduced (trimmed). The objective of trimming is to save on capital costs without increasing inlet pressure requirements or reducing any manifold inlet pressure heads below their respective design H_m.

The trimming procedure presented in Sample Calculation 10.3 and Eq. 10.5 can be employed for trickle systems. The amount the pipe-friction loss can be increased, Δh_f, for use in Eq. 10.5 can be determined for noncritical main line sections by:

$$\Delta h_f = (H_m + H_{fe})_c - (H_m + H_{fe}) \tag{24.1}$$

where:

Δh_f = the amount the pipe friction loss can be increased, m (ft)
H_m = manifold inlet pressure head, m (ft)
H_{fe} = the pressure head required to overcome pipe friction and elevation differences between the control head and a manifold, m (ft)

Use the $(H_m + H_{fe})$ values computed in Step 6, with the subscript c designating the critical manifold. (The parentheses without a subscript is the manifold supplied by the section of main line to be trimmed.)

Total Dynamic Head

The total dynamic head, TDH, or system inlet head, can now be computed. The procedure for computing the TDH is essentially the same for trickle as for sprinkle systems (see Chapter 11). Figure 11.3 shows a schematic representation of the various elements that make up the TDH.

Determining the TDH for trickle systems is straightforward. It is the sum of the following:

- Dynamic lift;
- Supply system losses;
- Control head losses;
- $(H_m + H_{fe})_c$;
- Miscellaneous losses in subunits;
- Friction loss safety factor, which is 10% of the sum of friction head losses; and
- Pressure head allowance for emitter deterioration.

The best way to present the process for designing the main line and determining the TDH is through examples. Thus, the sample calculations for the three trickle systems will be completed in the order of main line design complexity.

Sample Calculation 24.1. *Determining the TDH for the line-source system presented in the progressive set of sample calculations 21.2, 22.1, and 23.1.*

GIVEN: From the sample calculations: Q_s = 177 gpm; H_a = 9.2 ft; and H_m = 13.0 ft. The system will have a simple screen filter with a maximum expected pressure loss of 5 psi and a differential pressure fertilizer injector across a flow

Table 24.1. TDH for the line-source irrigation system

Item		Head, ft
Dynamic lift from well		35.0
Control Head losses		
Filter (5 psi × 2.31 ft/psi)	11.6	
Flow meter (2 psi × 2.31)	4.6	
Plumbing system	3.0	
		19.2
$(H_m + H_{fe})_c$		13.0[1]
Miscellaneous losses in subunit		
Pressure control value	4.6	
Safety screens at lateral inlets	2.3	
		6.9
Friction-loss safety factor: $0.1(19.2 + 13.0 + 6.9 - H_a - \Delta H_e)$		3.0[2]
Allowance for emitter deterioration: $0.5H_a = 0.5 \times 9.2$		4.6
	TDH	82 ft

[1] $H_{fe} = 0.0$.
[2] $H_a = 9.2$ ft; and $\Delta H_e = 0.0$.

meter that has a 2-psi loss. Water will be supplied from a shallow well with a dynamic lift of 35 ft when discharging 177 gpm.

FIND: The TDH required for the system.

CALCULATIONS: For this particular system there is only a short section of main line (see Fig. 21.3). This should be 3-in. pipe. Furthermore, a pressure- or flow-regulating valve should be installed at the pump discharge for safety purposes. This is necessary because the line-source tubing could be easily ruptured by high-pressure surges during start-up. The TDH required for the installation can now be computed. It is the sum of the various pressure head, miscellaneous friction loss, and safety factor values listed in Table 24.1.

Sample Calculation 24.2. *Main line design and determining the TDH for the spray system presented in the progressive set of sample calculations 21.3, 22.2, and 23.3.*

GIVEN: From the sample calculations: $Q_s = 178$ gpm; $H_a = 58.5$ ft; $(H_m)_5 = 67.5$ ft; and $O_t = 686$ hr. The economic factors for the PVC main line pipe selection are the same as those given for the drip systems in sample calculation 23.2. Water will be supplied from a canal, and the dynamic lift is 9.0 ft. The control head will have a media filter with a maximum expected pressure loss of 10 psi, a chemical injection pump, and a flow meter that causes a 2-psi pressure loss.

FIND: The optimum economic pipe sizes for the main line and the TDH for the system.

CALCULATIONS: The highest main line friction loss will occur for station IV when manifolds (4) and (5) are in operation. This is because all stations have the same flow rate and the field is nearly level. Figure 24.1 shows the main line layout and the flow rates in it when station IV is operating. The flow rate from the pump to manifold (4), P to B, is 178 gpm; and from manifold (4) to manifold (5), B to C, it is 35 gpm.

To determine Q_s' for entering the economic pipe-selection chart, Fig. 8.7, A_f must first by computed. All the economic pipe-selection factors are the same as for the drip system. Therefore, A_f can be computed by multiplying the A_f =

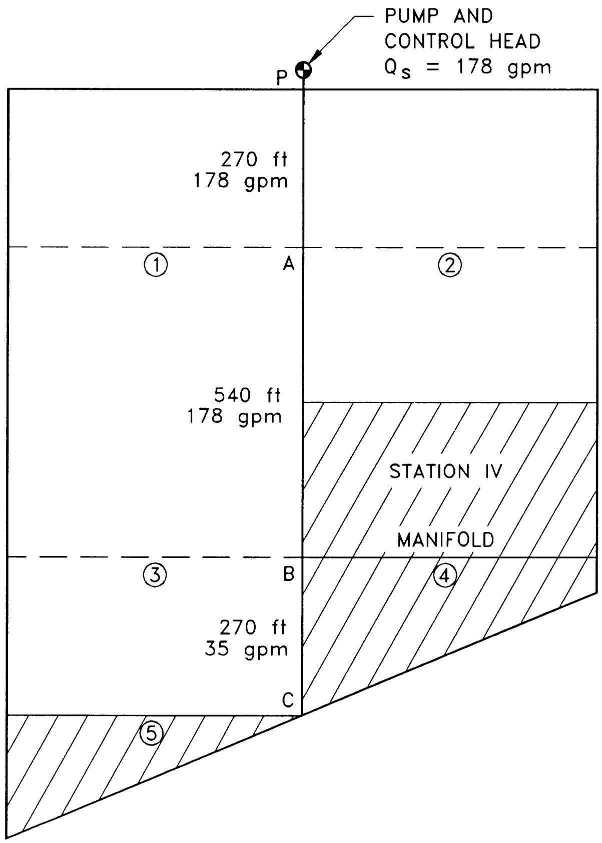


FIG. 24.1. Main Line Layout for Spray Irrigation System.

Table 24.2. Main line friction losses for the spray irrigation system

Section	Flow (gpm)	Pipe (in.)	J	$\frac{L}{100}$	h_f (ft)
P-A	178	4[1]	1.17	2.70	3.2[2]
A-B	178	4[1]	1.17	5.40	6.3
B-C	34	2	1.49	2.70	4.0
				Total	13.5

[1]Pipe selection controlled by 5-ft/s velocity restriction (solid vertical lines in Fig. 8.7).
[2]$h_f = J(L/100)$

1.01 for the drip system (see sample calculation 23.2) by the ratio of the respective operating hours per year for the spray and drip systems (686/2680) to obtain:

$$A_f = 1.01 \times 686/2680 = 0.26$$

Then by Eq. 8.21:

$$Q_s' = A_f Q_s = 0.26 \times 178 = 46 \text{ gpm}$$

Enter Fig. 8.7 with $Q_s' = 46$ gpm to select the most economical pipe size for each main line section. Then compute the h_f for each section of pipe using J values from Table 8.3. Table 24.2 gives a summary of this process.

Because the difference in elevation between P and C is 0, $H_{fe} = h_f = 13.5$ ft. Because $(H_m)_5 = 67.5$ ft is the highest manifold inlet pressure head required, it is the critical manifold and:

$$(H_m + H_{fe})_c = 67.5 + 13.5 = 81.0 \text{ ft}$$

The TDH for the system can now be computed as shown in Table 24.3.

Sample Calculation 24.3. *Designing the main line, determining the TDH, actual uniformity and net application rate for the drip system.*

GIVEN: The following data from the progressive set of drip system sample calculations 21.1, 22.3, and 23.2: $Q_s = 648$ gpm; $H_a = 44.5$ ft; $H_m = 50.2$ ft; $A_f = 1.01$; $\Delta H_l = 2.6$ ft; $\Delta H_m = 7.5$ ft; and the information in Figs. 21.1 and 21.3.

The pumping lift is 8.0 ft, and the suction assembly losses will be 2.0 ft. An automatic back-flushing filter set to flush when the pressure differential reaches 10 psi (23.1 ft) will be used. A fertilizer-injection pump will be used, and the various control head and miscellaneous subunit losses (presented later in Table 24.6) were taken from manufacturers' or standard charts.

Table 24.3. TDH for the spray irrigation system

Item		Head, ft
Dynamic lift of pump discharge		9.0
Supply system head requirements		—
Control head losses:		
Filter (10 psi × 2.31)	23.1	
Flow meter (2 psi × 2.31)	4.6	
Plumbing system	4.6	
		32.3
$(H_m + H_{fe})_c$		81.0[1]
Miscellaneous losses in subunits:		
Manifold value and regulator	6.9	
Lateral risers and hose bibs	2.3	
Lateral inlet safety screen	2.3	
		11.5
Friction-loss safety factor		
$0.1(32.3 + 81.0 + 11.5 - H_a - \Delta H_e)$		6.6[2]
Allowance for emitter deterioration		0.0
	TDH	140 ft

[1] $H_{fe} = 0.0$.
[2] $H_a = 58.5$ ft, and $\Delta H_e = 0.0$.

FIND: The optimum economic pipe sizes for the main line, the TDH, the actual EU, and the net application rate for the system.

CALCULATIONS: The following procedures use the seven steps presented in the text for using the economic pipe-selection chart. The system is designed to have all the emitters operating simultaneously for it is a single-station system. Figure 24.2 shows the main line layout, section lengths, and flow rates in each of them.

The first trial set of main line pipe sizes should be selected from the economic pipe-size-selection chart, Fig. 8.7. The flow is divided immediately after it leaves the control head. Therefore, the Q_s' for entering the chart should be based on the larger of the two branch flow rates. Thus, in accordance with *Steps 1 and 2*, by Eq. 8.21:

$$Q_s' = 1.01 \times 432 = 436 \text{ gpm}$$

Enter the vertical axis of Fig. 8.7 with 436 gpm, and select the economical size of PVC pipe for each main line section (*Step 3*). Then use Eqs. 8.7a and 8.7b to determine the h_f for each section of pipe in accordance with *Step 4* of the mainline design procedure as outlined in Table 24.4.

Then compute H_{fe} between the control head and each manifold inlet in ac-

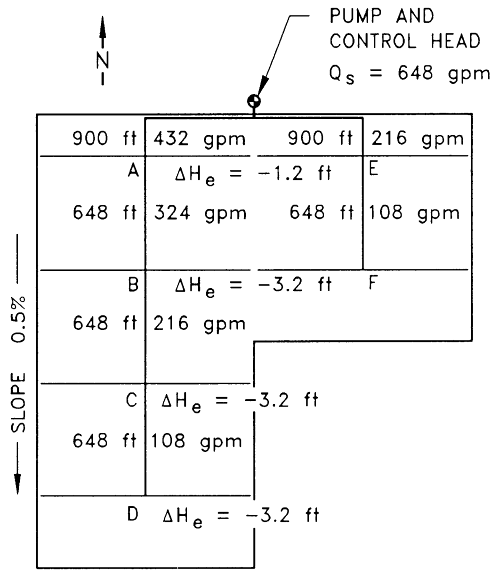


FIG. 24.2. Main Line Layout for Drip Irrigation System.

cordance with *Step 5*, as outlined in Table 24.5. The h_f values for each pipe section are taken from Table 25.4 and the ΔH_e values are shown in Fig. 24.2.

As all the manifolds required the same inlet pressure head there is no need to compute $(H_m + H_{fe})$ for each of them. The $H_{fe} = 7.4$ ft for the pipe section P-B is the highest. Thus, in accordance with *Step 6*, the critical manifold inlet is at B. The pump must supply 7.4 ft of pressure head to overcome pipe friction and elevation along the main lines. Because all the manifolds require the same inlet pressure head, if the required $H_m = 50.2$ ft is supplied at point B, all other manifold inlet pressure head requirements will be more than satisfied.

Following the procedure outlines in *Step 7*, the pipe sections downstream

Table 24.4. Head losses due to pipe friction in main line sections for the drip irrigation system

Section	Flow (gpm)	Pipe (in.)[1]	J	$\frac{L}{100}$	h_f (ft)
P-A	432	6	0.92	9.00	8.3
A-B	324	6	0.54	6.48	3.5
B-C	216	6	0.26	6.48	1.7
C-D	108	4	0.48	6.48	3.1
P-E	216	6	0.26	9.00	2.3
E-F	108	4	0.48	6.48	3.1

[1]The inside diameters of the 4- and 6-in. pipes are 4.280 in. and 6.301 in. (see Table 8.5).

Table 24.5. Friction loss from P to each manifold inlet, H_{fe}, for the drip irrigation system

Section	h_f (ft)	ΔH_e (ft)	$(h_f + \Delta H_e)$ (ft)	Section P-to	H_{fe} (ft)
P-A	8.3	−1.2	7.1	A	7.1
A-B	3.5	−3.2	0.3	B	7.4[1]
B-C	1.7	−3.2	−1.5	C	5.9
C-D	3.1	−3.2	−0.1	D	5.8
P-E	2.3	−1.2	1.1	E	1.1
E-F	3.1	−3.2	−0.1	F	1.0

[1]Critical H_{fe}.

from B and from P to F can be trimmed so $H_{fe} = 7.4$ ft for the manifolds at C, D, E, and F. The allowable increase in friction head in section B-C can be determined by Eq. 24.1 as:

$$(\Delta h_f)_{\text{B-C}} = (50.2 + 7.4)_{\text{B}} - (50.2 + 5.9)_{\text{C}} = 1.5 \text{ ft}$$

This unnecessary gain in pressure head can be eliminated to reduce pipe costs by replacing some of the 6-in. pipe with 4-in. pipe in section B-C. The exact length of the smaller pipe, L_s, that will increase the head loss by 1.5 ft can be determined by:

$$L_s = \frac{100\,\Delta h_f}{J_s - J_b} \tag{10.5}$$

$$(L_4)_{\text{B-C}} = \frac{100 \times 1.5}{1.62 - 0.26} = 110 \text{ ft}$$

With 538 ft of 6-in. and 110 ft of 4-in. pipe in section B-C, $(H_{fe})_{\text{C}} = 7.4$ ft. This will also increase H_{fe} to the manifold inlets at D by 1.5 ft to give $(H_{fe})_{\text{D}} = 7.3$ ft. This is so close to the critical $H_{fe} = 7.4$ ft that tapering section C-D is not warranted. Therefore, branch P-D should have 2086 ft of 6-in. pipe and 758 ft of 4-in. pipe, as shown in Fig. 21.2.

In a similar manner, the east branch of the system can be tapered as follows:

$$(\Delta h_f)_{\text{P-E}} = (7.4)_{\text{B}} - (1.1)_{\text{E}} = 6.3 \text{ ft}$$

And:

$$(L_4)_{\text{P-E}} = \frac{100 \times 6.3}{1.62 - 0.26} = 463 \text{ ft}$$

Therefore, branch P-F should have 437 ft of 6-in. pipe and 1111 ft of 4-in. pipe, as shown in Fig. 21.2.

Table 24.6. TDH for the drip irrigation system

Item	Head, ft	
Dynamic lift to pump discharge:		
Lift	8.0	
Suction friction loss	2.0	
Valve and screen losses	—	
		10.0
Supply system head requirements:		
Pipe friction	—	
Elevation	—	
Miscellaneous	—	
		—
Control head losses:		
Filter and screens	23.1	
Flow meter	3.0	
Main control valves	2.3	
Chemical/fertilizer injector	—	
Plumbing system	4.6	
		33.0
$(H_m + H_{fe})_c$ (50.2 + 7.4) =		57.6
Miscellaneous subunit losses:		
Manifold		
Riser and control valve	3.0	
Pressure/flow regulator	—	
Safety screen	—	
Lateral or header		
Risers and hose bibs	2.3	
Pressure/flow regulator	—	
Safety screen	2.3	
		7.6
Friction loss safety factor at 10%		6.7
Allowance for emitter deterioration		0.0
	TDH	115 ft

The TDH for the system can now be computed as outlined in Table 24.6. The table contains a comprehensive list of miscellaneous head-loss and safety-factor items that should be considered when computing the TDH. However, a number of the items listed in Table 24.6 are not pertinent to this design and have been left blank.

The friction loss safety factor is 10% of the sum of all friction losses, which is computed as follows:

Suction friction loss	2.0 ft
Control head losses	33.0
Miscellaneous subunit losses	7.6
$(H_m + H_{fe})$	57.6
$-(\Delta H_e)$	+10.8
$-H_a$	$\underline{-44.5}$

10% of Total = 0.1 (66.5) = 6.7 ft

The flow characteristics of the vortex emitters used for this design are not expected to change with time. Therefore, no additional or extra pressure head will be allowed for emitter deterioration.

The final system design layout is shown in Fig. 21.2. The design data are presented in Figs. 21.1 and 21.3. These three figures, along with a brief writeup of the system specifications and a bill of materials, form the complete design package.

For scheduling irrigation, the net system application rate should be computed. To do this the final system EU must be determined as follows. First compute the approximate H_n by:

$$H_n \approx (H_m - \Delta H_m - \Delta H_l) \tag{20.18}$$

$$= (50.2 - 7.5 - 2.6) = 40.1$$

Then determine q_n/q_a by:

$$\frac{q_n}{q_a} = \left(\frac{H_n}{H_a}\right)^x \tag{20.17}$$

$$= (40.1/44.5)^{0.42} = 0.957$$

And compute the actual design EU by Eq. 20.13a to obtain:

$$\text{EU} = 100(1.0 - 1.27 \times 0.07/\sqrt{4}) \times 0.957$$

$$= 91.5\%$$

The net design application rate can now be computed by:

$$I_n = K \frac{\text{EU}}{100} \frac{N_p q_a}{S_p S_r} \tag{20.19}$$

$$= 1.604 \frac{91.5}{100} \times \frac{4 \times 1.11}{24 \times 24} = 0.0113 \text{ in./hr}$$

EVALUATING FIELD PERFORMANCE

Once a trickle system has been put in operation, its field test emission uniformity, EU′, can be determined using Eq. 17.2. Merriam and Keller (1978) presented a systematic strategy for estimating a system or subunit's average emitter discharge, q_a, and the average discharge of the lowest one-fourth of the emitter discharges, q_n', for use in Eq. 17.2. The procedure for using their field-evaluation technique requires a rather large amount of field data to obtain accurate results; thus it is quite time-consuming.

Bralts and Edwards (1986) and Bralts (1986) presented a field-evaluation strategy that reduces the amount of data necessary. It allows for partitioning the emitter discharge variations into those caused by pressure variations in the hydraulic network and those caused by emitter performance components. Their procedure gives a statistical uniformity based on the coefficient of variation of emitter flows.

It can be modified to give a field emission uniformity similar to that given by using Eq. 17.2. Where the variation in emitter discharges is uniformly distributed throughout the field, the resulting statistical field test emission uniformity is:

$$\mathrm{EU}'' = 100\left\{1 - 1.27\left[(v_e)^2/N_p' + x^2(v_p)^2\right]^{0.5}\right\} \tag{24.2a}$$

And where it is due to a mixture of partial and full plugging and the variation is apt to be concentrated at specific locations, such as near the distal ends of manifolds and laterals:

$$\mathrm{EU}'' = 100\left\{1 - 1.27\left[(v_e')^2 + x^2(v_p)^2\right]^{0.5}\right\} \tag{24.2b}$$

where:

- EU″ = statistical field test emission uniformity, percentage
- v_e = field coefficient of emitter discharge variation for individual emitters operating at the same actual or simulated pressure head
- N_p' = number of emitters each plant receives water from
- x = emitter discharge exponent
- v_p = coefficient of variation of emitter operating pressure heads through the field or subunit under study
- v_e' = field coefficient of discharge variation between the composite discharges from sets of emitters serving individual plants when operating at the same actual or simulated pressure head

Clemmens (1987) developed a statistical analysis to compare various trickle uniformity coefficients. He established the independence of the variations due to emitter performance and those due to hydraulic effects. The results of his analysis demonstrate that for emitters with the same coefficient of variation the design emission uniformity, EU, computed by Eq. 20.13 is always a few percentage points less than EU′ computed by Eq. 17.2. Thus, the use of EU for design purposes should give conservative results, except where a moderate to high level of emitter plugging is anticipated.

Emitter Plugging

Solomon (1985) developed a simulation model to study the effect of the various equipment, system, and other factors known to influence emission uniformity. He found that their order of importance, beginning with the most important, was: plugging; number of emitters per plant; emitter coefficient of variation; emitter exponent; emitter flow response to water temperature; subunit pressure differences; pressure regulator coefficient of variation; rates of manifold to lateral friction loss; and degree of manifold taper. The ranking is not absolute, for it depends on the range of values associated with each parameter. However, the parameter values analyzed were chosen to represent the range of values usually encountered in practice, so the ranking as presented should be correct for most circumstances.

Since plugging affected the emission uniformity the most, he analyzed a wide range of different emitter-plugging conditions representative of field conditions. His analysis indicated that relatively uniformly distributed partial plugging should not appreciably affect EU′. However, where some emitters are fully plugged or there is a mixture of full and partial plugging, the EU′ would be lowered significantly. For example, for his medium levels of plugging, EU′ decreased by roughly 6% and for his high levels of plugging, by roughly 12%.

In view of the above, when problems with plugging are anticipated, the initial system EU, Q_s, and TDH should be adjusted accordingly. For uniform partial plugging this can be done by designing the system to provide the capacity to increase the average hours of operation per day or the TDH as q_a decreases. Compensating for some fully plugged emitters is more difficult because it reduces overall emission uniformity and system performance in unpredictable ways. This must be corrected by cleaning or replacing the affected emitters.

REFERENCES

Bralts, V. F. 1986. Operational principles: Field performance and evaluation. In *Trickle Irrigation for Crop Production*, eds., F. S. Nakayama and D. A. Bucks, 216–240. Amsterdam: Elsevier.

Bralts, V. F., and D. M. Edwards. 1986. Field evaluation of drip irrigation submain units. *ASAE Transactions* 29(6):1659–1664.

Clemmens, A. J. 1987. A statistical analysis of trickle irrigation uniformity. *ASAE Transactions* 30(1):169–175.

Merriam, J. L., and J. Keller. 1978. *Farm Irrigation System Evaluation*, 3rd ed., Logan, Utah: Agricultural and Irrigation Engineering Department, Utah State University.
Solomon, K. H. 1985. Global uniformity of trickle irrigation systems. *ASAE Transactions* 28(4):1151–1158.

RECOMMENDED READING

Benami, A., and A. Ofen. 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).
Howell, T. A., D. A. Bucks, D. A. Goldhamer, and J. M. Lima. 1986. Management principles: Irrigation scheduling. In *Trickle Irrigation for Crop Production*, eds., F. S. Nakayama and D. A. Bucks, pp. 241–279. Amsterdam: Elsevier.
James, L. G., and W. M. Shannon. 1986. Management principles: Flow management. In *Trickle Irrigation for Crop Production*, eds. F. S. Nakayama and D. A. Bucks, pp. 280–316. Amsterdam: Elsevier.
Nir, D. 1982. Drip irrigation. In *CRC Handbook of Irrigation Technology*, ed. H. J. Finkel, Volume I, pp. 247–298. Boca Raton, Florida: CRC Press.

IV

SYSTEM SELECTION

25

Pressurized Irrigation System Selection

Insightful selection of the system type and layout that best meet the goals and objectives of an irrigated agricultural development is of primary importance in the overall design process. A system may be well designed but inappropriate if another one would be better suited for the development.

Knowledge of the characteristics of the various system types and layouts and the capability to produce optimal designs are essential parts of the system selection process, but they are not sufficient. In addition it is necessary to select from the many candidates the "best type and configuration" of irrigation system for each specific site situation.

To do this effectively requires a systematic procedure (see Figs. 1.2 and 1.3.) that has five essential steps:

Step 1. Identification of the development goals and impacts;
Step 2. Definition of the site conditions (physical and institutional);
Step 3. Prescreening to select the set of most promising, adaptable system types and configurations;
Step 4. Detailed design and economic analysis of the preselected types; and
Step 5. Comparison of the results of Step 4 for each system type against the goals defined in Step 1 for making the final selection.

Though the selection procedure presented herein is for pressurized systems, it includes all potential irrigation methods, at least through the prescreening phase. Too often a valid, and possibly the best, choice is overlooked due to the designer's predisposition to certain irrigation methods.

IDENTIFICATION OF DEVELOPMENT GOALS

It may seem that the obvious goal of all irrigation development is economic efficiency or maximum economic return. It is true that most development projects seek economic efficiency, but other economic goals may be important. Furthermore, both beneficial and adverse social and environmental impacts may be associated with the development and should not be ignored.

Economic Goals

In terms of economic efficiency, a project may focus on either maximum return on investment (maximum benefit/cost, B/C, ratio) or maximum net benefits (B-C) from the development. The difference between these goals is illustrated in Fig. 25.1 (Keller, et al., 1988). The range of project development options may be described as a continuum as depicted by the curved line in Fig. 25.1. (This is an oversimplification, for the options would typically define an envelope rather than a line, but it is useful for illustration purposes.)

Referring to Fig. 25.1, the maximum return on investment is the point on the curve where the B/C ratio is greatest. This is defined by the point where a straight line drawn through the origin is tangent to the B/C ratio (or development) curve.

The maximum net benefits occur at the point where a line parallel to the B/C = 1 line is tangent to the B/C continuum curve. The curve usually crosses the B/C = 1 line twice. The space between these two lines is the economic development space, as indicated by the shaded area in Fig. 25.1. Any development option falling within this space will produce economic benefits that exceed costs.

When comparing systems, both the beneficial and adverse social and environmental impacts must be identified and properly accounted for in the analysis. The economic efficiency may be constrained by social or environmental impacts that outweigh the production economic goals. This may require selecting a system with less than maximum economic efficiency.

Economic constraints may have an impact on economic efficiency and play an important role in selecting the system type and configuration. For example, if capital is limited, minimizing the initial cost is necessary. Consequently, even though it may appear more efficient to use life-cycle costing and increase the initial cost to reduce operating costs, this may not be acceptable.

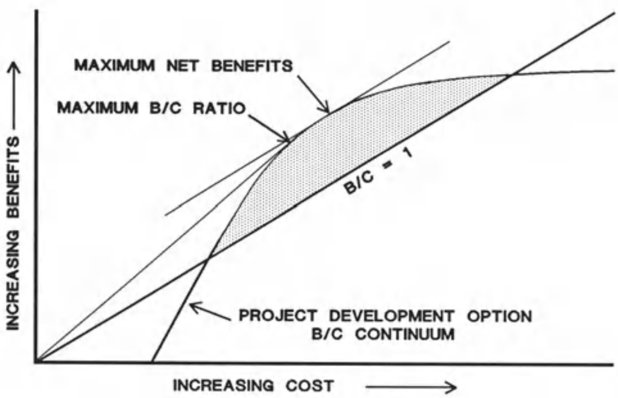


FIG. 25.1. Benefit/Cost Envelope for Economic Analysis of Development Projects.

Social and Environmental Issues

Social goals are those that would impact some segment of society in ways related to the project development. Some of the more obvious beneficial social impacts or goals include providing jobs for unemployed people, providing locally produced food to limit imports, and using locally produced equipment.

Environmental goals relate to the beneficial impacts of irrigation on the environment, and environmental constraints relate to the adverse impacts. Water-quality issues may require limitation of runoff or deep percolation from the project site. There may be a desire to provide wildlife habitat in conjunction with irrigated agriculture. Any potential environmental relationship could impact the design of the system, if a particular environmental goal or constraint is set.

These social and environmental goals and impacts may be defined economically by assigning a value or cost to them. For example, they may be expressed in terms of government subsidies or monetary penalties. In such cases, the social and environmental impacts can be incorporated directly into the economic analysis. However, if these impacts cannot be defined in economic terms, the allowable levels of impact, either positive or negative, must be established and incorporated in the overall analysis.

SITE CONDITIONS

Proper irrigation system selection requires a general understanding of the local site. Knowledge of many physical site conditions is also necessary for the design process. For each particular system type and configuration, both the design considerations for the site conditions and the overall impact on them are important in the selection process.

For system selection, data on institutional, economic, and physical site conditions are important. Inadequate understanding or lack of consideration for institutional and economic conditions could lead to the failure of a system that may physically appear to be well designed.

Becoming familiar with a particular area requires collecting detailed information on the local conditions, but this may need to be done only once. However, when working in a new area, consideration should again be given to collecting the needed information in each information category as a beginning step in the system selection and design process.

Institutional Considerations

The institutional considerations important to irrigation system selection are those that relate to people-system interactions. These considerations are often poorly addressed in the selection process and are the most difficult to quantify. However, they may override economic or even physical considerations and should be addressed early in the selection process.

Political, legal, and regulatory issues are of primary importance. Included are such issues as land reform, water rights, containment of runoff and drainage waters, taxation, financial incentives from governments, and zoning or construction permit requirements. These issues should be fully understood at the beginning of the selection process.

Because many pressurized systems involve moderate to high levels of both hardware and software sophistication, availability of parts and technical support for maintenance is important. The capability of the farmer to repair the equipment, as well as the availability of repair and maintenance services, should be understood and considered.

Although labor is often considered an economic issue, it is also an institutional issue, in terms of the availability and reliability of both laborers and managers. If adequate labor is not available or dependable, then irrigation systems that minimize labor requirements would be attractive, even if an economics analysis would suggest otherwise. Understanding the characteristics of the labor pool is important. This applies not only to availability and reliability, but also to the level of training and skill required to operate the different types of systems.

The institutional considerations for system selection in *developing countries* is of paramount importance, especially for the modern, more complicated systems. Concerns of special interest in developing nations are: divisibility (capability of irrigating small tracts); maintenance costs and parts requirements; risks of complete and lengthy breakdowns; the skill level required for operation and maintenance; and the overall ruggedness or durability of the irrigation equipment.

Table 25.1 developed by Keller (1990) characterizes the various types of systems in terms of these factors for developing countries. The table is also useful in the more developed countries when types of systems new to a particular region are being considered. Amplifications for the terms used to categorize the factors affecting system selection in Table 25.1 are:

> *Divisibility* (or suitability/adaptability for small land holdings) of each application technology is an important consideration. Some are inherently suitable for use only, or at best most, on relatively large fields, compared with the typical farm/plot size of 0.2 to 5 ha in most developing countries. Furthermore, even for those technologies that can easily be subdivided to fit any size of field, it may be difficult or very expensive to use them on very small irregularly shaped holdings. The three categories of divisibility used in Table 25.1 are: *total*, for techniques that can be economically fitted to any size of plot; *partial*, for techniques that can be divided with difficulty or at high expense; and *no*, used for techniques that realistically are suitable only for

Table 25.1. Factors affecting the selection of different types of modern irrigation systems for use in developing countries

Method and type	Factors affecting system selection: Divisibility	Maintain by	Risk	Mgt and O&M: Skill	Mgt and O&M: Effort	Ruggedness
Surface						
Canal-feed						
Basin	Total[1]	Grower	Low	Master	5	Lasting
Border	Total[1]	Farmer	Low	Master	6	Lasting
Furrow	Total[1]	Farmer	Low	Medium	10	Lasting
Pump/pipe-feed						
Basin (level)	Partial[1]	Shop	Med	Master	3	Robust
Border	Partial	Shop	Med	Master	3	Robust
Furrow	Partial[1]	Shop	Med	Master	6	Robust
Sprinkle						
Lateral						
Hand-move	Total	Shop	Med	Simple	9	Durable
End-tow	Partial	Shop	Med	Medium	5	Durable
Side-roll	Partial	Shop	High	Medium	6	Durable
Side-move	No	Agency	High	Master	5	Fragile
Hose-fed/pull	Total[1]	Farmer	Med	Simple	10/7	Durable
Traveling-gun	Partial	Agency	High	Master	4	Sturdy
Center-pivot	No	Agency	High	Complex	1	Sturdy
Linear-moving	No	Agency	High	Complex	2	Sturdy
Solid-set						
Portable	Total[1]	Shop	Med	Medium	5	Durable
Permanent	Total[1]	Farmer	Med	Medium	1	Durable
Trickle						
Point-source						
Drip	Total[1]	Grower	High	Complex	2	Fragile
Spray	Total[1]	Grower	Med	Complex	2	Durable
Bubbler	Total[1]	Grower	Low	Complex	4	Robust
Hose-basin	Total[1]	Farmer	Low	Simple	10	Robust
Line-source						
Reusable	Total[1]	Grower	High	Complex	5	Fragile
Disposable	Total[1]	Grower	High	Complex	3	Fragile

[1] Well-adapted for irregularly shaped fields.

large fields. In addition to divisibility, superscript numerals show those systems that can be easily designed for irregularly shaped fields.

Organizational requirements are a function of the divisibility of the field application system and whether the pressurized water-delivery system could best be operated by a group or agency. Systems with *total* divisibility can be operated independently by each *private* farmer. Totally divisible application systems supplied by a shared pump and/or distribution network should be considered as only partially divisible. With *partial* divisibility *group* (or cooperative) effort is usually required. The direct operational assistance of an *agency*, such as a large cooperative or irrigation department, is usually needed to manage and operate an application system with *no* divisibility. The same holds true where a very large pumping plant and pressurized distribution are used.

Maintain by is a category that gives some feeling for the complexity of the technologies in terms of overall physical sustainability. It does this by indicating who has the capacity to maintain, in a practical sense, the operability of the irrigation application equipment so it will function adequately over its full economic life. *Farmer* is used for equipment that can be rather easily maintained and fixed by the ordinary farmers who raise traditional crops. *Grower* is used for equipment that can be maintained at the farm level, but requires advanced skills normally associated with producers of high-value crops. *Shop* is used to indicate the need for typical local merchants having some special but limited facilities and technical capacity to repair the equipment. *Agency* is used to indicate that very specialized equipment, facilities, and skills are needed to keep the equipment in operation. Irrespective of whom maintains the application equipment, shops or agencies are normally required to maintain engine-driven pumps.

Risk is a category that addresses the issue of potential crop loss due to equipment "breakdown." The *low* risk category is used for surface systems that are not vulnerable to breakdown at the field level. *Medium* is used for all pressurized systems where parts of the water-application equipment can break or malfunction without jeopardizing the rest of the system. *High* is used for both trickle irrigation systems, which require microfiltration, and irrigation machines. Malfunctions of the filter can be disastrous for trickle systems and thus for the crops. Machinery malfunctions can cause the whole application system to shut down. If this occurs at a critical growth stage and is not immediately repaired, the entire crop may be lost.

Management, operation, and maintenance are closely linked for the various irrigation techniques. They should be considered together in terms of: management difficulty; what must be available and known; the nature of the skills needed for operation and maintenance; levels of support required for service and spare parts; and what other agricultural technologies will be needed to make irrigation cost-effective and sustainable.

In Table 25.1 management complexity is categorized according to the *skill* level necessary to realize reasonable application efficiencies. It also relates to the skill and support services necessary to maintain and service the equipment to keep it operable for its anticipated economic life. *Simple* is used to indicate only elementary skills are necessary. *Medium* indicates that considerable skill is needed to manage and operate the equipment properly. *Master* indicates that much hands-on field experience is needed to manage the flows, spread the water, and do the irrigating (or move and position the equipment). *Complex* indicates that sophisticated technical skills and reading ability are necessary to operate and service the irrigation equipment effectively.

The *Effort* levels of 1 through 10 indicate the relative management time and labor required to manage, operate, and maintain the various types of irrigation application systems. The numbers give a rough quantitative indication of the average number of days/hectare (half-days/acre) per month of operation required.

Ruggedness is included in Table 25.1 to give a sense of the durability of the water conveyance and application equipment needed for each irrigation technique. Thus, it is also an indication of the likelihood of breakdowns and level of spare parts and services needed. *Lasting* is reserved for the gravity surface systems, because they never actually break down to the point of being completely inoperable. *Robust* systems depend on low-pressure pipe distribution systems, but have few mechanical or intricate parts (other than a pump if gravity pressure is not sufficient) and practically never break down. *Durable* systems seldom have breakdowns and do not require very careful handling, but some spare parts and service facilities are necessary. *Sturdy* systems, however, are irrigation machines that require careful handling and maintenance and considerable spare parts and service backup to continue functioning. *Fragile* systems have delicate components that malfunction when improperly handled or maintained and require a considerable inventory of spare parts, but only limited service facilities.

Physical Conditions

The most obvious site parameters, and usually the easiest to quantify, are the physical conditions. Physical site information is needed for both selection and design. An understanding of how the systems or configurations under consideration relate differentially to the physical conditions, along with identifying their impact on system design and performance, is of basic importance in the selection process. A checklist of physical conditions to be considered is presented in Table 25.2. Detailed discussions of many of them are presented elsewhere in this text. For discussions of the others see especially Jensen (1980) or Bliesner and Merriam (1988).

Economic Considerations

As discussed earlier, economic efficiency is central to the irrigation system selection process. Although other considerations are important, most, if not all, have economic implications. Regardless of the importance of the noneconomic goals and impacts of an irrigation development, some economic analysis will be required.

Table 25.2. Physical site conditions to consider in irrigation system selection

Crops	*Land*
Crops grown	Field shape
Crop rotation	Obstructions
Crop height	Topography
Cultural practices	Soil—texture
Disease potential	—uniformity
Pests	—depth
Water requirements	—intake rate
Climate modification	—water capacity
	—erodibility
Water supply	—salinity
Source	—drainability
Quantity	—bearing strength
Quantity—salinity	Flood hazard
—sediments	Water table
—organics	
Reliability	*Climate*
Delivery—schedule	Precipitation
—frequency	Temperature
—rate	Frost conditions
—duration	Humidity
	Wind

Before making the economic analysis the data required to define the conditions under which the economic analyses will be made must be assembled. The economic data required fall into two general categories, site-dependent and system-dependent.

Site-Dependent. Site-dependent economic parameters are those that will not be influenced by the system, but are necessary to determine the relative economics of the development. They include: interest rate (real and nominal); labor costs; energy costs; the energy inflation factor; general inflation factor; property taxes (on irrigation equipment); water costs; the land value; and the return to irrigation for each crop.

Interest rates are often categorized as *real* or *nominal. Nominal rates* are the current rates of interest charged by the lending institution that will provide credit. The rate includes an inflationary component and a risk, management, and profit component. The *real rate* is inflation-free; therefore, it is less than the nominal rate by the long-term inflation rate. The real rate is usually in the range from 5 to 7%.

The real rate of interest is used to determine the annualized cost of capital expenditures that tend to appreciate, such as land values and permanent improvements to the land, like land-leveling. The nominal rate is used to determine the annualized cost of capital expenditures that depreciate or reach technical obsolescence with little or no salvage value.

Including the land value is necessary when comparing systems that do not irrigate all the land to systems that do, such as a center-pivot compared with hand-move sprinkle systems. Crop returns to irrigation are the net returns after all production and land costs have been deducted. The net returns are needed to compare the relative economics of systems with sufficiently different water-application uniformities or characteristics to impact yield.

The energy inflation factor is simply the expected inflation rate for energy over the economic life of the system. It is used to compute the equivalent annual energy cost factor, *EAE*, discussed in Chapter 8 and is important for balancing capital and operating costs, i.e., life-cycle costing.

Inflation factors should also be included for other irrigation input costs, such as for labor and water. Such inflation factors are used in the same way as the energy inflation factor. The long-term general inflation rate is usually adequate for this purpose.

System-Dependent. The system-dependent parameters are those that relate directly to the system and are identified individually for each system. They include: system component costs; system component lives; and labor, energy and maintenance costs. The physical life of some components may be longer than the expected technical life due to obsolescence of the irrigation technology. In such cases, it is practical to use an expected economic life equal to the technical life rather than the full physical life. For example, PVC pipe has a very long physical life (50 to 100 years), but the system may become obsolete in a much shorter time.

Labor and energy requirements are not economic parameters directly. However, they are necessary for determining the system operation economics and are included here for that reason.

SYSTEM PRESCREENING

After the goals and potential impacts have been defined and the site data collected, all the various types of systems can be screened for suitability (see Fig. 1.2). The detailed selection process is too time-consuming to be practical for this step. Furthermore, it is not necessary to do for some types of systems, for they will not fit the site conditions or satisfy the goals.

As experience is gained for a particular location, it becomes easier to eliminate unsuitable systems during prescreening. This is possible as more information is gained about the likelihood of a particular system type and configuration fitting local site conditions. However, care should be taken to not overreact and eliminate systems that may be likely candidates, but were eliminated in a previous analysis under different conditions.

The prescreening process is one of matching the capabilities of the potential irrigation systems to the physical site conditions and the goals and impacts of the development. It is best demonstrated by example.

Sample Calculation 25.1 *Prescreening of irrigation systems for a given set of development goals and site conditions in the United States.*

GIVEN:

Development Goals: Economic—Maximum net benefits
Social—None
Environmental—No runoff allowed

Site Conditions:
Institutional: No crop subsidies
Maintenance support available
Labor is available and reliable

Crops:	Spring wheat	Potatoes	Dry beans
Field sizes	64 ha	64 ha	64 ha
Water Requirements			
Net irrigation	355 mm	535 mm	355 mm
Peak demand	7.0 mm/day	8.0 mm/day	7.5 mm/day
Peak month	June	July	July
Root zone depth	1.2 m	1.0 m	1.0 m
MAD	50%	40%	50%

Land:
Field shape, obstructions, and topography: see Fig. 25.2

Soils:

Texture	Sandy loam
Uniformity	Uniform with a few small areas of loam
Intake rate	15–20 mm/hr
Water holding cap.	125 mm/m
Depth	>1.5 m
Salinity	No restriction
Drainage	No restriction
Erodibility	Moderate (wind)
Bearing strength	Good

Water supply:

Source	Irrigation district canal (see Fig. 25.2)
Quantity	1.8×10^6 m^3
Rate	190 Lps
Frequency	Continuous
Salinity	500 dS/cm, no specific ion concerns
Sediment	Low to moderate turbidity
Organics	High, algae
Reliability	No shortages expected

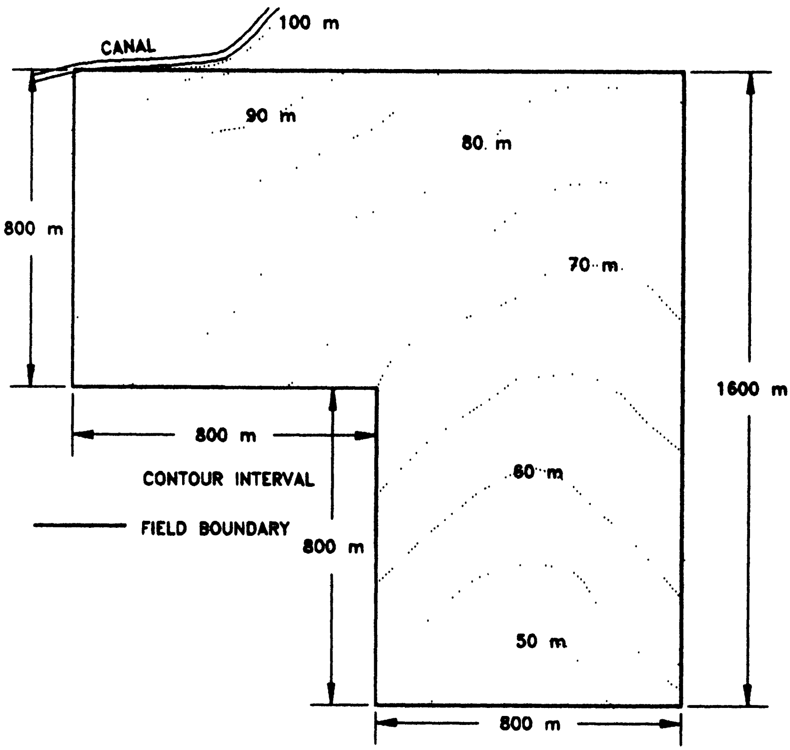


FIG. 25.2. Dimensions and Topography of Field to Be Irrigated.

Climate:

Precipitation	50 mm per irrigation season
Temperature	High desert climate
Humidity	Low
Wind	5–10 km/hr

Economic Factors:

Interest rate	10% nominal, 6% real	
Fuel inflation rate	7%	
Labor inflation rate	4%	
Water inflation rate	4%	
Labor cost	\$5.00 per hr	
Energy source	Electric, \$0.07/kWh	
Water cost	\$16.00/1000 m^3	
Raw land cost	\$700/ha	
Property tax	None on irrigation equipment	
Crop returns:	Wheat	\$125/ha
	Potatoes	\$1230/ha
	Beans	\$500/ha

FIND: Preselected irrigation systems that have potential for meeting the development goals stated for the given site conditions.

ANALYSIS: The preselection process is one of elimination. Localized irrigation methods can be eliminated. They are not adaptable for such field crops as wheat and are generally not economical for lower value row crops.

Because surface runoff from the property will not be allowed, any surface irrigation method will require a runoff-recovery system. Also, for surface methods the relatively steep slopes and irregular topography will necessitate significant leveling. The high-intake-rate soils and high level of erodibility are also constraints for surface irrigation methods. Given these limitations, surface methods are not considered adaptable.

Because this is an arid climate and relatively high-value, water-sensitive crops will be grown, traveling-gun sprinklers are not recommended. This is because of their high energy requirements and the low water-application uniformity that would result from the relatively high winds at the site.

Set systems would be adaptable to the site. Hand-move (see Fig. 2.2) laterals would be a likely choice for further analysis. A combination of hand-move and solid-set (see Fig. 4.6), using solid-set on the potatoes, would be another option. Side-roll laterals (see Fig. 4.2) would also be a likely consideration. However, end-tow laterals (see Fig. 4.1) are not well adapted to these crops.

Center-pivot systems (see Fig. 2.1) would be another likely choice, given the high-intake-rate soils and relatively steep, irregular topography. The field shapes are also suitable for pivots. Both standard center-pivots and ones with corner arms could be considered.

Linear-moving systems are not readily adaptable because of the field shape and steep topography. It would be possible to use one linear and one pivot. Due to the topography the linear would need to be hose-fed and may have some operating problems. At least part of the linear would have to be in wheat to provide sufficient capacity for the potatoes and beans. This would complicate cultural practices, for the crops would need to be split under the pivot to maintain the specified crop rotation.

In summary the five choices that warrant further consideration are: hand-move laterals; hand-move laterals and some solid-set; side-roll laterals; and center-pivot lateral with and without corner arms.

DETAILED DESIGN AND ECONOMIC ANALYSIS

After the potentially suitable systems have been selected, the next step is to complete and compare detailed analyses for each of them. Where there are several potentially suitable systems, this should be done as a two-phase process with a less detailed analysis in the first phase.

After completing the layout and design, a full economic analysis for each system, within the constraints and goals selected, is required to make comparisons and the final selection. This economic analysis is most easily completed on an annualized cost basis. The costs and returns resulting from each system can then be compared (see Fig. 25.1).

After the designs are completed and the total annualized costs for each system are established, the economic returns expected from each of them must then be determined to compute their B/C ratio.

Annualized Costs

The annualized *capital costs* are computed using the capital recovery factor, *CRF*, (Eq. 8.13), for the life of the various system components and the nominal interest rate. Each component will have a capital recovery factor associated with its life, as indicated in Table 25.3. For capital items such as land cost that do not depreciate, use the inflation free or ''real'' interest rate to account for the fact that they will retain their value.

The expected average *annual maintenance costs* over the life of the components should be estimated and included in the analysis. Table 25.3 lists some typical maintenance costs expressed as a percentage of the original capital costs for major system components. Use these values if local experience is not available.

Energy costs should be computed from the annual energy requirements of the system and the local energy cost for the source of power used. When the energy costs over the life of the project are annualized, they should be adjusted to account for inflation. To do this use the equivalent annual energy cost factor, $EAE(e)$, (Eq. 8.13) as indicated in Chapter 8 (see Eq. 8.12) when comparing the relative economics of two or more systems.

When estimating the economic benefits expected from a given development, the EAE factor should be used only if inflation is included when estimating the other annual costs and crop returns. Inflation being difficult to forecast with any accuracy, the preferred method for estimating the expected net benefits from the system option selected is to leave inflation out of all costs.

The *cost of labor* to operate each system type must also be included. Typical labor requirements for the various system types are given in Table 25.4. These values are expressed in man-hours per irrigation per hectare (1 A = 0.4 ha) for in-season costs.

Because costs are associated with starting the systems at the beginning of each season and storing them at the end of the season, pre- and postseason costs are also shown. The operating cost of some systems is not easily computed on a per-irrigation basis. The footnotes to the table explain the exceptions. The labor time requirements shown are for operation only and do not include maintenance. Maintenance labor is covered in the maintenance cost percentages given in Table 25.3.

Table 25.3. Typical economic lives and maintenance costs for irrigation system components

System type and component	Economic life, yr[1,2]	Annual maintenance, % of cost[2]
Sprinkle		
Lateral		
Hand-move	15	2
End-tow	10	3
Side-roll	15	2
Side-move	15	4
Hose-fed/pull	5/20	3
Traveling-gun	10	6
Center-pivot		
Standard	15	5
with/corner	15	6
Linear-moving	15	6
Solid-set		
Portable	15	2
Permanent	20	1
Trickle		
Point-source		
Drip	10/20	3
Spray	10/20	3
Bubbler	15	2
Hose-basin	7/20	2
Line-source		
Reusable	10/20	3
Disposable	1/20	3
Other components		
Buried PVC main line	20–40	1
Steel main line	10–20	1
Aluminum main line	10–20	2
Electric pumps	15	3
Diesel/gas pumps	10	6
Wells	25	1

[1]Where two lives are shown with a slash, the first number is for above-ground components and the second for below-ground components.
[2]These values are approximate and were taken in part from Keller (1990) and Bliesner and Merriam (1988). Local experience and local operating conditions should be considered when available.

Labor costs should be computed using the local cost for labor and the man-hours per hectare shown. When systems with large differences in labor requirement are compared, the expected inflation in labor cost should be used. The equivalent annual labor cost over the life of the project may be computed by

Table 25.4. Average operating labor requirements for sprinkle and trickle irrigation systems

System type	Pre- & postseason time required,[1] man-hr/ha	Time required per irrigation, man-hr/ha
Sprinkle		
Center-pivot	0.12	0.05[2]
Linear-moving		
Ditch-fed	0.12	0.10
Hose-fed	0.15	0.15
Pipe-fed	0.12	0.07
Side-move	0.49	0.62
Side-roll	0.25	0.86
Traveling-gun	0.25	0.62
Hand-move	0.25	1.73
Hose-fed/pull	0.25	2.00/1.25
Solid-set		
Portable	2.47[3]	0.15
Permanent	0.25	0.15
Trickle		
Point-source		
Drip	0.25	0.05[4]
Spray	0.25	0.05[4]
Line-source		
Reusable	3.40	0.05[4]
Disposable	2.00[5]	0.05[4]

[1] The amount shown for each pre- or postseason operation.
[2] Assumes 1 in. net application or greater.
[3] Add 2.47 hr for each midseason move.
[4] Computed using 1 hr/day for each 60 ha and 2-day irrigation interval.
[5] Assumes tubing is laid during planting by machine.

using Eq. 8.12 with the expected labor inflation rate, rather than the energy inflation rate. This factor could be termed the *equivalent annual labor cost factor*, $EAL(e)$. Care should be taken to use expected long-term inflation rates to avoid bias in the analysis. When estimating the expected net returns from a given option, the discussion under energy costs applies.

Other annual irrigation costs include taxes on irrigation equipment, if any, and water costs. If these costs are inflating, they should be adjusted for inflation for comparison purposes, as discussed above.

Returns

The returns to the project should be computed for the crops to be grown. If the systems being compared have markedly different water-application uniformities, then the yield impact of these uniformity differences should be estimated by the methods presented by Hill and Keller (1980) and in Chapter 6. For such cases, the estimated gross returns and production costs for each crop will be required. If yield expectations are not markedly different, as in the example below, then only the anticipated net return from each crop is required.

The net returns are computed by subtracting the estimated average annual costs from the average annual benefits. If the economic goal is to maximize net return, then the system with the largest net return (B − C) best meets the goal.

The benefit/cost, B/C, ratio is computed by dividing the annual benefits by the annual costs. If the goal is to maximize the return on investment, then the system that yields the highest B/C ratio best meets the goal. It is possible, even common, to have one system yield the largest net return and another have the highest B/C ratio.

The process for selecting systems based on an economic analysis is best illustrated with an example. For demonstration purposes, two of the preselected systems from Sample Calculation 25.1 will be analyzed.

Sample Calculation 25.2 *Compare the annualized system costs and returns for two potentially suitable systems from the preselection analysis.*

GIVEN:

- Site conditions from Sample Calculation 25.1.
- The most promising types of systems would be either side-roll or center-pivot laterals. The respective layouts for each type of system are shown in Figs. 25.3 and 25.4.
- The component costs, economic lives, other economic factors, and efficiencies are as follows:

Side-roll system:

Laterals	\$160,000 (15 yr)
Pumps	\$ 27,000 (15 yr)
Main line	\$124,800 (30 yr)
Energy use	137,000 kWh/yr
Coefficient of uniformity	85%
Peak efficiency	75%
Seasonal efficiency	70%
Irrigations per season	9
Irrigated area	194 ha

Center-pivot system:

Laterals	\$132,000 (15 yr)
Pumps	\$ 23,000 (15 yr)
Main line	\$ 93,800 (30 yr)
Energy use	65,500 kWh/yr
Coefficient of uniformity	90%

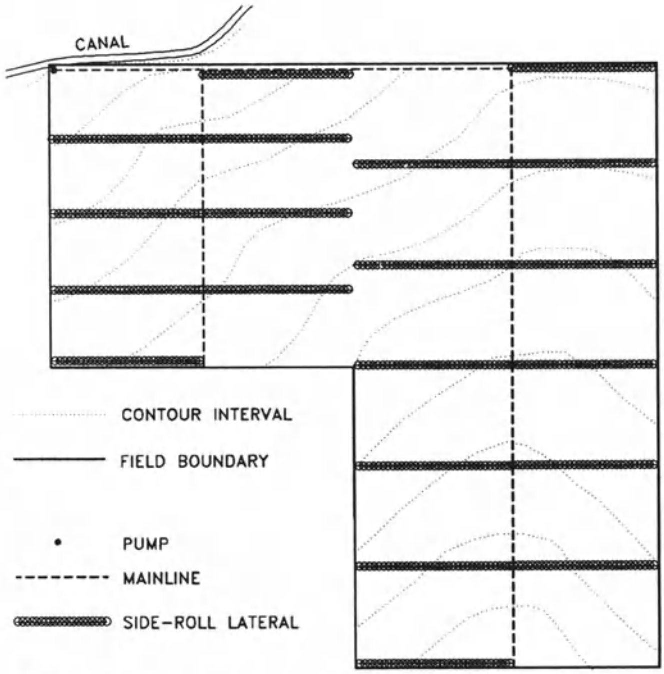


FIG. 25.3. Layout for Sample Side-roll Sprinkle Irrigation System.

Peak efficiency	80%
Seasonal efficiency	75%
Irrigations per season	16
Irrigated area	153 ha

FIND: The expected total annual cost of each option, and compare their net returns and B/C ratios.

ANALYSIS: Capital recovery factors for different economic lives with 10% nominal and 6% real interest are:

$$\text{CRF}(15\text{ yr}, 10\%) = \frac{(1 + 0.10)^{15}}{(1 + 0.10)^{15} - 1} \times 0.10 = 0.131$$

$$\text{CRF}(30\text{ yr}, 10\%) = 0.106$$

$$\text{CRF}(30\text{ yr}, 6\%) = 0.073$$

Side-Roll

Annual capital costs are (see Table 25.3 for economic life):

Laterals & pumps = 0.131 × $187,000	= $ 24,497	
Main line = 0.106 × $124,800	= 13,229	
Land (30 yr, 6%) = 0.073 × $700 × 194	= 9,913	
Total annual capital cost		= $ 47,639

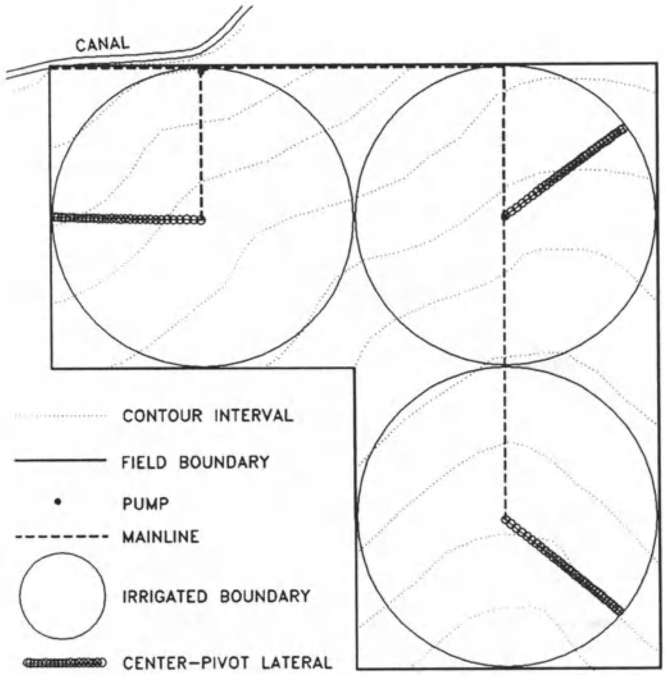


FIG. 25.4. Layout for Sample Center-pivot Irrigation System.

Annual maintenance costs are (see Table 25.3 for cost factors):

Pump maint. = 0.03 × 27,000	= \$ 810	
Lateral maint. = 0.02 × 160,000	= \$ 3,200	
Main line maint. = 0.01 × 124,800	= \$ 1,248	
Total annual maintenance cost		= \$ 5,258

Annual labor cost is (see Table 25.4):

Pre & postseason = 0.25 × 2 × 194	= 97 man-hr	
In-season = 0.86 × 9 × 194	= 1502 man-hr	
Total time	= 1599 man-hr	

$EAL(30 \text{ yr}, i = 10\%, e = 4\%) = 1.44$ (from Eq. 8.12)

Annual labor cost = 1.44 × \$5.00 × 1599 = \$ 11,513

Annual energy cost is:

$EAE(30 \text{ yr}, i = 10\%, e = 7\%) = 1.99$ (from Eq. 8.12)

Annual energy cost = 1.99 × \$0.07 × 137,000 = \$ 19,084

Annual water cost is:

$$\text{Volume} = \frac{(355 + 535 + 355)}{(3 \times 0.70)} \times 194 \times 10 = 1.15 \times 10^6 \text{ m}^3$$

$EAW = EAL = 1.44$

Annual water cost = 1.44 × \$16.00 × 1150 = \$ 26,496

Total annual cost of side-roll layout = \$109,990

$$\text{Gross return} = \frac{(125 + 1{,}230 + 500)}{3} \times 194 \quad = \$119{,}957$$

Net (B − C) return = \$119,957 − \$109,990 = \$ 9967

Benefit/cost(B/C) ratio =

\$119,957/\$109,990 = 1.09

Center Pivot

Annual capital costs are:

Laterals & pumps = 0.131 × \$155,000 = \$ 20,305

Main line (30 yr) = 0.106 × \$93,800 = 9,943

Land (30 yr, 6%) = 0.073 × \$700 × 194 = 9,913

Total annual capital cost = \$ 40,161

Annual maintenance costs are:

Pump maint. = 0.03 × 23,000 = \$ 690

Lateral maint. = 0.05 × 132,000 = \$ 6,600

Main line maint. = 0.01 × 93,800 = \$ 938

Total maintenance cost = \$ 8,228

Annual labor cost is:

Pre & postseason = 0.12 × 2 × 153 = 37 man-hr

In-season = 0.05 × 16 × 153 = 122 man-hr

Total time = 159 man-hr

EAL(30 yr, i = 10%, e = 4%) = 1.44 (from Eq. 8.12)

Annual labor cost = 1.44 × \$5.00 × 159 = \$ 1,145

Annual energy cost is:

EAE(30 yr, i = 10%, e = 7%) = 1.99 (from Eq. 8.12)

Annual energy cost = 1.99 × \$0.07 × 65,500 = \$ 9,124

Annual water cost is:

$$\text{Volume} = \frac{(355 + 535 + 355)}{(3 \times 0.75)} \times 153 \times 10 = 8.47 \times 10^5 \text{ m}^3$$

$EAW = EAL = 1.44$

Annual water cost = 1.44 × \$16 × 847 = \$ 19,515

Total annual cost of center-pivot layout = \$ 78,173

$$\text{Gross return} = \frac{(125 + 1{,}230 + 500)}{3} \times 153 \quad = \$ 94{,}605$$

Net (B − C) return = \$94,605 − \$78,173 = \$ 16,432

Benefit/cost (B/C) ratio = \$94,605/\$78,173 = 1.21

Final Selection

Since the project with center-pivots has the highest anticipated net return (\$16,432 compared with \$9,967) and meets the other development goals, it is a better choice than a project with side-roll laterals. It also has the highest B/C ratio (1.21 compared with 1.09) and lowest initial equipment cost (\$248,800 compared to \$311,800).

FINAL SELECTION

Final selection requires identification of the system that best meets the development goals. Since prescreening eliminates the systems that do not meet the noneconomic goals, the final selection usually reduces to the system that either returns the greatest net benefits or provides the best return on investment, depending upon the goal selected.

In some cases, there may be a clear choice. However, in many cases, given the uncertainties in the analysis, the goals may be met equally well by more than one system. In such cases, evaluation of the components of the design is necessary. In the example just completed, if the net returns had been close, the characteristics of the two systems could have been examined in detail to provide a basis for selecting one of them.

The center-pivot system requires less energy and much less labor. However, the maintenance requirements are more sophisticated and land is left unirrigated. An assessment of the risks associated with these items in terms of future operation would add information that would be relevant to the final selection. In all cases, the final selection should not be made by the designer alone. The comparative data for the better choices should be presented to the owner and/or operator and the decision made jointly.

In making the final selection, variations of the systems that appear to be the best may also be useful. For the example systems, splitting the pumping stations and pipelines between the two submains would have yielded lower energy costs, but would have increased capital costs. Adding corner-systems to the center-pivot laterals or examining an alternative means of irrigating the corners would be other logical options to explore.

By using the process outlined, the more promising alternatives can be explored and compared. By using the five-step procedure outlined above, the process of design and selection can be unified. The best system for meeting the desired development goals can be selected while also limiting adverse social and environmental impacts.

REFERENCES

Hill, R. W., and J. Keller. 1980. Irrigation system selection for maximum crop profit. *ASAE Transactions* 23(SW-2):366–372.

Keller, Jack. 1990. Modern irrigation in developing countries. Proceedings of the Fourteenth International Congress on Irrigation and Drainage, April 29–May 4, 1990, Rio de Janeiro, Brazil: In Press.

Keller, J., A. A. Keller, and R. D. Bliesner. 1988. Irrigation project simulation model for planning and design. Proceedings Sixth International Water Resources Association Congress on Water Resources, May 29–June 5, Ottawa, Canada, Volume III, pp. 127–136.

Bliesner, R., and J. Merriam (eds.) 1988. Selection of irrigation methods for agriculture. Report submitted by the On-Farm Irrigation Committee of ASCE, December 22.

RECOMMENDED READING

James, L. G. 1988. *Principles of Farm Irrigation System Design*. New York: Wiley.

Jensen, M. E. (Editor). 1980. *Design and Operation of Farm Irrigation Systems*, ASAE Monograph 3. St. Joseph, Michigan: American Society of Agricultural Engineers.

Steele, H. A., and G. A. Pavelis. 1967. Economics of irrigation policy and planning. In *Irrigation of Agricultural Lands*, eds. R. M. Hagan, H. R. Haise, and T. W. Edminister, pp. 174–189. Madison, Wisconsin: American Society of Agronomy.

Thompson, G. T., L. S. Spiess, and J. N. Krider. 1980. Farm resources and system selection. In *Design and Operation of Farm Irrigation Systems*, ASAE Monograph 3, ed. M. E. Jensen, pp. 45-73. St. Joseph, Michigan: American Society of Agricultural Engineers.

APPENDIX A

Suggested Further Reading

The focus of this book is on the synthesis aspects of irrigation system design, and it is unique in that respect. Coverage of basic background and analytical materials is more or less limited to what is needed to design sprinkle and trickle irrigation systems. Therefore, professionals and students may wish to have ready access to some of the more comprehensive reference texts on agroirrigation. Such texts contain additional details on the basic theories and analytical procedures used in synthesizing designs for irrigation systems. They also provide some additional insights to the design concepts and more extensive bibliographies covering the basic elements covered in the individual chapters of this text. Following is a listing of the more comprehensive of these reference and textbooks:

Benami, A., and A. Ofen, 1983. *Irrigation Engineering*. Haifa, Israel: Irrigation Engineering Scientific Publications (IESP).

Christiansen, J. E. 1942. *Irrigation by Sprinkling*. California Agric. Exp. Stn. Bull. 670. University of California, Berkeley.

Cuenca, R. H. 1989. *Irrigation System Design*. Englewood Cliffs, New Jersey: Prentice Hall.

Finkel, H. J. (Editor). 1982. *CRC Handbook of Irrigation Technology*, Volumes I and II. Boca Raton, Florida: CRC Press.

Hagan, R. M., H. R. Haise, and T. W. Edminister (Editors). 1967. *Irrigation of Agricultural Lands, Agronomy No. 11*. Madison, Wisconsin: American Society of Agronomy.

Hansen, V. E., O. A. Israelson, and G. E. Stringham, 1979. *Irrigation Principles and Practices*. New York: Wiley.

Hillel, D. (Editor). 1982. *Advances in Irrigation*. New York: Academic Press.

Hoffman, G. J., T. A. Howell, and K. H. Solomon (Editors). 1990 *Management of Irrigation Systems*. St. Joseph, Michigan: American Society of Agricultural Engineers. Forthcoming.

James, L. G. 1988. *Principles of Farm Irrigation System Design*. New York: Wiley.

Jensen, M. E. (Editor). 1980. *Design and Operation of Farm Irrigation Sys-*

tems, ASAE Monograph 3. St. Joseph, Michigan: American Society of Agricultural Engineers.

Merriam, J. L., and J. Keller. 1978. *Farm Irrigation System Evaluation*. 3rd ed., Logan, Utah: Agricultural and Irrigation Engineering Department, Utah State University.

Michael, A. M. 1978. *Irrigation Theory and Practices*. New Delhi: Vikas Publishing House.

Nakayama, F. S., and D. A. Bucks (Editors). 1986. *Trickle Irrigation for Crop Production*. Amsterdam: Elsevier.

Pair, C. H., W. H. Hing, K. R. Frost, R. E. Sneed, and T. J. Schilty (Editors). 1983. *Irrigation*, 5th edition. Arlington, Virginia: The Irrigation Association.

APPENDIX B

Glossary

A	design area, hectares (acres), ha (A)
A_f	system flow-rate adjustment factor for the Universal Economic Pipe-Selection Chart
A_s	area directly wetted by sprayer, m^2 (ft^2)
A_w	area wetted by a trickle irrigation emitter along a horizontal plane 15 to 30 cm (6 to 12 in.) below the soil surface, m^2 (ft^2)
a'	cross-sectional flow area, mm^2 ($in.^2$)
a	1.0 + flow rate exponent of pipe-friction equation
B	sprinkle nozzle diameter, mm (in.)
BP	brake power, kW (hp)
b	velocity or flow rate exponent of pipe-friction equation
C	Hazen-Williams pipe friction coefficient
c	pipe diameter exponent of pipe-friction equation
C_f	fuel cost per unit of brake-power output (unit cost of fuel)/(output per unit of fuel), \$/kW-hr (\$/hp-hr)
C_p	pipe cost per unit weight, \$/kg (\$/lb)
CE'	equivalent annual cost of escalating energy for overcoming head loss, \$
CI	spray coarseness index, %
CRF	capital recovery factor
CT	time allowed each cycle of linear-moving lateral for field drying and contingencies, min
CU	coefficient of uniformity, %
CU_a	coefficient of uniformity for alternate sets, %
D	inside diameter of pipe or outlet, mm (in.)
D_g	gross seasonal depth of irrigation water, mm (in.)
D_i	pump impeller diameter, mm (in.)
D_n	net seasonal irrigation depth required to meet consumptive use requirements, mm (in.)

d	gross depth of water application per irrigation, mm (in.)
d'	daily gross depth of water application required during peak-water-use period, mm (in.)
d_i	depth of water infiltrated for average sprinkle application rate, I, at time of ponding, T_p, mm (in.)
d_i'	average peak gross depth of infiltration required per day, mm (in.)
d_n	net depth of water application per irrigation to meet consumptive use requirements, mm (in.)
d_x	maximum net depth of water application per irrigation, mm (in.)
DE_{pa}	sprinkler distribution efficiency for percentage area, *pa*, adequately irrigated, %
DU	distribution uniformity, %
DU_a	distribution uniformity for alternate sets, %
E	present annual cost of energy per unit of water-power output, \$/kW-year (\$/hp-year)
E'	equivalent annual cost of escalating energy per unit of water-power output, \$/kW-year (\$/hp-year)
E_a	irrigation application efficiency, %
E_{pa}	application efficiency based on adequately irrigating a given percentage, *pa*, of the field area, %
E_h	irrigation application efficiency of lower half, %
E_p	pump efficiency, %
E_q	irrigation application efficiency of the low quarter, %
E_s	seasonal irrigation efficiency, %
$EAE(e)$	equivalent annualized cost of escalating energy factor
EC_e	electrical conductivity of saturated soil extract, dS/m (mmhos/cm)
EC_w	electrical conductivity of irrigation water, dS/m (mmhos/cm)
EC_{dw}	electrical conductivity of the drainage (deep percolation) water, dS/m (mmhos/cm)
ET	evapotranspiration or crop water-consumption-use rate, mm/day (in./day)
EU	design emission uniformity, %
EU′	field test emission uniformity, %
EU_p	design emission uniformity along a center-pivot lateral, %
e	expected annual rate of energy cost escalation, decimal
F	multiple-outlet pipe-friction reduction coefficient for laterals and manifolds
FC	field capacity, mm/m (in./ft)

F_f	Darcy-Weisbach pipe-friction factor
F_p	multiple-outlet pipe-friction reduction coefficient for center-pivot laterals to compensate for discharge along the pipe of length L_h
F_s	shape-adjustment factor for pipe-friction loss in manifolds serving nonrectangular trickle irrigation system subunits
f	operating time allowed for completion of one irrigation cycle, days or hr
f'	irrigation interval or frequency, days, hr, or min
f_e	emitter connection loss equivalent length, m (ft)
f_x	maximum irrigation interval, days or hr
G	gross volume of water required per plant during the peak use period, L/day (gal/day)
g	acceleration due to gravity, m/s^2 (ft/s^2)
H	operating pressure head, m (ft)
H_a	average sprinkler or emitter pressure head, m (ft)
H_c	pressure head at the closed end of a lateral (or manifold), m (ft)
H_e	static head due to elevation, m (ft)
H_{fe}	pressure head difference due to pipe friction and elevation between any two points in a system, m (ft)
H_j	pressure head at any radial (or linear) distance, r_j, (or l_j) from the inlet of a center-pivot (or linear-moving) lateral, m (ft)
H_l	average lateral inlet pressure head, m (ft)
H_m	manifold inlet pressure head, m (ft)
H_n	minimum pressure head, m (ft)
H'_n	minimum pressure head along the average single or pair of laterals in a subunit, m (ft)
H_r	height of riser, m (ft)
H_S	system inlet pressure head, m (ft)
H_s	system or subunit inlet pressure head, m (ft)
H_T	total pressure head required at a given point in a system, m (ft)
H_x	hydraulic grade at point x along a pipe-friction curve that is tangent to the HGL, m (ft)
HGL	hydraulic grade line
h_{cv}	head loss due to pressure- or flow-control device, m (ft)
h_f	head loss due to pipe or fitting friction, m (ft)
h_{fj}	pipe friction head loss between the inlet and radial distance r_j (or linear distance l_j) along a center-pivot (or linear-moving) lateral, m (ft)

$(h_f)_a$	allowable head loss due to pipe friction, m (ft)
h_{fp}	pipe friction head loss for a single lateral equal in length and flow to a pair of laterals, m (ft)
$(h_f)_s$	head loss due to pipe and fitting friction losses that are dependent on flow rate between the water source and pivoting elbow, m (ft)
h_{fx}	pipe friction head loss from a point x on a multiple-outlet line to the closed end, m (ft)
I	average application rate, mm/hr (in./hr)
I'	preliminary application rate or required average center-pivot application rate at radial distance $r_j = L$, mm/h (in./hr)
I_j	center-pivot average application rate at radial distance r_j, mm/hr (in./hr)
I_n	net application rate, mm/hr (in./hr)
I_t	approximate average application rate from an independent traveling or stationary gun sprinkler, mm/hr (in./hr)
I_x	maximum application rate, mm/hr (in./hr)
i	annual interest rate, decimal
J	pipe friction head-loss gradient, m/100 m (ft/100 ft)
J'	equivalent pipe friction head-loss gradient for laterals with emitter connection losses, m/100 m (ft/100 ft)
J_a	allowable pipe friction head loss gradient, m/100 m (ft/100 ft)
J_b or J_2	friction head-loss gradient in bigger diameter pipe, m/100 m (ft/100 ft)
J_s or J_1	friction head-loss gradient in smaller diameter pipe, m/100 m (ft/100 ft)
j'	equivalent pipe friction head-loss gradient along the lateral between emitters, m/100 m (ft/100 ft)
K	conversion constant that is equation-specific
K_{cp}	discharge coefficient of a center-pivot
K_d	outlet or sprinkler discharge coefficient
K_{ds}	lateral discharge-spacing coefficient
K_q	coefficient of discharge for an outlet, which ranges from 0.6 to 1.0, depending upon shape
K_r	resistance coefficient of fitting or valve
K_s	system discharge coefficient
k	special constant that is equation-specific
k_c	crop coefficient to relate the evapotranspiration of a specific crop to that of the reference crop ET

k_f	frequency factor to adjust standard crop-water-use values for high-frequency irrigation
k_m	scale factor for unit friction-loss curves for manifolds
k_p	time to ponding coefficient
k_y	specific yield response factor for each crop
L	length of pipe or radius irrigated in basic circular (center-pivot) field, m (ft)
L'	actual length of center-pivot lateral pipe, m (ft)
L_D	length of pipe with diameter D, m (ft)
L_f	length of field irrigated by a linear-moving lateral, m, (ft)
L_h	equivalent hydraulic length of a center-pivot lateral, m (ft)
L_N	net annual leaching requirement, mm (in.)
L_n	net leaching requirement for each irrigation, mm (in.)
L_p	total length of a pair of laterals (or manifolds) that extend in opposite directions from a common manifold (or main line) outlet, m (ft)
LR	leaching requirement ratio for sprinkle irrigation
LR_t	leaching requirement ratio for trickle irrigation
L_s or L_1	length of smaller diameter pipe, m (ft)
L_b or L_2	length of bigger diameter pipe, m (ft)
l_j	linear distance from lateral inlet to outlet under study, m (ft)
M	irrigation system cost, \$
MAD	management-allowed deficit, % or decimal
MAD_a	average management-allowed deficit for managing center-pivot irrigation, % or decimal
M_s	residual stored soil moisture from off-season precipitation, mm (in.)
m	mean depth of catch observations from uniformity test, mm (in.)
N	number of outlets on pipeline (may be laterals off main line or manifold, or sprinklers or emitters operating off laterals)
N_n	minimum average number of sprinklers operating
N_p	number of emitters per plant
N'_p	minimum number of emitters from which each plant receives water
N_r	number of rows (or laterals) served from a common main line outlet (or by a manifold)
N_s	number of operating stations
N_x	maximum usual number of sprinklers operating
n	number of observations in uniformity equations
n	number of years in life cycle for economic analysis

O_e	ratio of water effectively discharged through sprinkler orifices or nozzles to total system discharge
O_t	pump operating time per season or year, hr/year
P	outlet or sprinkler operating pressure, kPa (psi)
P_a	average sprinkle (or emitter) pressure, kPa (psi)
P_{cv}	pressure loss at the control valve, kPa (psi)
P_d	portion of soil surface shaded by plant canopies at midday, %
P_e	static pressure due to elevation, kPa, (psi)
P_f	pressure loss due to pipe friction, kPa, (psi)
$(P_f)_a$	allowable pressure loss due to pipe friction, m (ft)
P_h	pressure loss due to hoseline and/or hydrant friction, kPa (psi)
P_{fx}	pressure loss due to pipe friction between any point x on a multiple-outlet line and the closed end, kPa (psi)
P_j	pressure head at any radial (or linear) distance, r_j (or l_j), from the inlet of a center-pivot (or linear-moving) lateral, kPa (psi)
P_l	pressure required at average lateral inlet, kPa (psi)
P_m	pressure required at manifold inlet, kPa (psi)
P_n	minimum outlet pressure, kPa (psi)
P_r	pressure required to lift water up sprinkler riser, kPa (psi)
P_s	pressure required at submain, kPa (psi)
P_t	pressure loss through traveling sprinkler machine, kPa (psi)
P_x	maximum outlet pressure, kPa (psi)
P_w	percentage of soil area wetted by trickle irrigation, %
PET	an index dependent on the center-pivot lateral length, nozzling configuration, daily operating time, and application uniformity, mm/hr (in./hr)
PS	perimeter of area wetted by sprayer, m (ft)
$PW(e)$	present worth factor of escalating energy costs
p	time to ponding exponent
Q	flow rate in any pipe section of a system, L/s (gpm)
Q_b	discharge to basic circular (center-pivot) field, L/s (gpm)
Q_g	discharge from center-pivot end-gun or corner system, L/s (gpm)
Q_l	average lateral flow rate or discharge, L/s or L/min (gpm)
Q_m	manifold flow rate or discharge, L/s (gpm)
Q_j	center-pivot lateral flow rate at radial distance, r_j, L/s (gpm)
Q_s	total system discharge or capacity, L/s (gpm)
Q_s'	adjusted system flow rate for entering the Universal Economic Pipe-Selection Chart, L/s (gpm)

Q_w	well discharge, L/s (gpm)
q	outlet (sprinkler or emitter) discharge, L/s, L/min, or L/hr (gpm or gph)
q_a	average outlet (sprinkler or emitter) discharge, L/s, L/min, or L/hr (gpm or gph)
q_c	rate of injection of fertilizer or chemical solution into system, L/hr (gph)
q_g	discharge of part-circle sprinkler used to square off the radial application profile for the basic circular center-pivot field, L/s (gpm)
q_j	the required discharge at radius r_j, on a center-pivot lateral, L/s (gpm)
q_n	minimum nominal emitter discharge computed using the minimum design pressure, L/hr (gpm)
q_n'	average rate of discharge of the lowest one-fourth of the emitter discharge reading from field data, L/hr (gph)
q_x	maximum nominal emitter discharge, L/hr (gph)
R	maximum radius irrigated by a center-pivot when a corner system or end sprinkler is in operation, m (ft)
RPM	speed of rotation in revolutions per minute, rpm
RV	value of pipe size reduction, \$/m (\$/ft)
R_e	effective portion of applied water or natural precipitation, decimal
R_j	wetted radius of sprinkler or jet throw, m (ft)
R_n	effective rain during growing season, mm (in.)
R_w	radius from the pivot point to the location of the discharge weighted average elevation, m (ft)
R_y	Reynolds number
RT	total rest time per irrigation cycle for reversing machinery, fueling, and moving hoses as needed, min
r_e	radial distance from pivot to end drive unit, m (ft)
r_j	radius from pivot point to point under study, m (ft)
S	absolute slope of HGL, %
SG	specific gravity
SMD	soil moisture deficit, mm (in.)
S_e	outlet (sprinkler or emitter) spacings, m (ft)
S_e'	optimal emitter spacing, m (ft)
S_f	trickle irrigation subunit shape factor
S_l	spacing between laterals along main line or manifold, m (ft)
S_m	spacing of manifolds along main lines, m (ft)

S_r	row spacing, m (ft)
S_p	plant spacing in row, m (ft)
S_j	sprinkler spacing at radius r_j on a center-pivot lateral (which is equal to the average distance to the adjacent up-and downstream sprinklers), m (ft)
sd	standard deviation
SS	water-storage capacity at soil surface, mm (in.)
s	general slope of field which is negative (−) for downhill and positive (+) for uphill, % or decimal
T	average actual daily operating or watering time, hr/day or hr
TDH	total dynamic head, m (ft)
T_a	application time or duration of application during peak use period, hr
T_c	center-pivot cycle time or linear-moving lateral travel time along length of field while irrigating, hr
T_{cx}	maximum center-pivot lateral cycle time that does not cause runoff, hr
$(T_c)_n$	minimum time required for a linear-moving lateral to travel the length of the field, hr
T_d	average daily transpiration during peak-use period, mm (in.)
T_p	ponding time from beginning of sprinkling, min
T_R	seasonal transpiration ratio
T_r	peak-use period transpiration ratio
T_s	seasonal transpiration, mm (in.)
T_w	total watering (or irrigating) time per lateral cycle forth and back, min
U	conventionally estimated seasonal consumptive use for the mature crop with full canopy, mm (in.)
U_d	conventionally estimated average daily-consumptive-use rate during the peak use month, mm/day (in./day) or mm (in.)
u	desired dosage of chemical in irrigation water, ppm
V	velocity of flow, m/s (ft/s)
V_e	travel speed of end drive unit on center-pivot, m/min (ft/min)
V_i	linear-moving lateral travel speed when applying main irrigation application, m/min (ft/min)
V_j	travel speed at any radial distance r_j from the pivot point, m/min (ft/min)
V_r	linear-moving lateral travel speed when returning empty or applying light irrigation, m/min (ft/min)

V_s	gross seasonal volume of irrigation water, ha-mm (A-in.) or ha-m (A-ft) or m^3 (ft^3)
V_t	travel speed of traveling gun sprinkler, m/min (ft/min)
v	manufacturer's coefficient of variation of emitters
v'	coefficient of variation between lateral flow rates along a manifold with regulating valves at each lateral inlet
v_{dorc}	nominal variation of DU or CU, $\pm$%
v_s	system coefficient of manufacturing variation of emitters
W	towpath spacing, m (ft)
W_a	available water-holding capacity of the soil, mm/m (in./ft)
WP	water power, kW (hp)
WP	wilting point, mm/m (in. ft)
w	wetted width of soil water movement or water application pattern, m (ft)
w_j	wetted width of center-pivot water pattern at radial distance, r_j, m (ft)
$(w)_n$	minimum pattern width necessary to avoid runoff from a center-pivot or linear-moving lateral, m (ft)
X	absolute deviations of observations from the mean
x	outlet (sprinkler or emitter) discharge exponent
x	distance from closed end of pipe, m (ft)
Y	manifold (or main line) position ratio, x/L_p, which gives the same minimum uphill and downhill pressure head along a pair of laterals or manifolds
Y_a	estimated average crop yield, units/ha (units/A)
Y_p	expected or potential crop yield with no water deficit units/ha (units/A)
Y_r	relative yield, which is the ratio of the estimated yield with saline water to full potential yield under trickle irrigation
y	value of x/L where the pipe friction-loss gradient along a lateral and ground slope are the same
Z	plant root depth, mm (in.)
z	individual depth of catch observation from uniformity test, mm (in.)
α	average to inlet pressure head adjustment factor for pairs of laterals
β	inlet to minimum pressure head adjustment factor for pairs of laterals
β'	inlet to minimum pressure head adjustment factor for single laterals
ν	kinematic viscosity of water, m^2/s (ft^2/s)
ω	portion of circle receiving water, degrees

ΔE	absolute difference in elevation between the two ends of a lateral (or manifold), m (ft)
ΔEl	elevation difference which is positive (+) for uphill and negative (−) for downhill, m (ft)
ΔEl_p	maximum positive difference in ground elevation between the pivot and moving end of a center-pivot lateral, m (ft)
ΔE_p	absolute difference in elevation between the outer ends of a pair of laterals, m (ft)
ΔH	actual or allowable variation in pressure head, m (ft)
ΔH_c	difference between the downhill (closed) end and minimum pressure head along a lateral, m (ft)
ΔH_e	static pressure head difference between the inlet and downstream ends of a pipe section due to elevation difference, ΔEl, which is positive (+) for uphill and negative (−) for downhill, m (ft)
ΔH_l	difference in pressure head along an average lateral, m (ft)
ΔH_m	difference in pressure head along a manifold, m (ft)
ΔH_{m-l}	difference between H_m and H_l, m (ft)
$(\Delta H_m)_a$	allowable manifold pressure head variation that will satisfy the desired design emission uniformity, m (ft)
ΔH_s	allowable variation in pressure head for a subunit, m (ft)
Δh_f	actual or allowable pipe friction-head loss increase, m (ft)
$\Delta J_{(s-b)}$	difference in head loss gradient between adjacent (*s*maller and *b*igger) pipe diameters, m/100 m (ft/100 ft)
ΔP_e	static pressure difference between the inlet and downstream ends of a pipe section due to elevation difference, ΔEl, which is positive (+) for uphill and negative (−) for downhill, kPa (psi)

APPENDIX C

ASAE Standards for Pressurized Irrigation Systems

Following is an annotated listing of the American Society of Agricultural Engineers (ASAE) standards and engineering practices related to pressurized irrigation systems. These standards, whether industry, national, or international, provide guidance to consultants and irrigators regarding equipment, systems, and practices. This list was taken from:

Hahn, R. H., and E. E. Rosentreter. 1989. *ASAE Standards 1989*. St. Joseph, Michigan: American Society of Agricultural Engineers, 659 pages.

Individual ASAE standards, or the above book containing all of them, can be ordered by number from:

American Society of Agricultural Engineers
2950 Niles Road
St. Joseph, Michigan 49085-9695 USA
Phone (616) 429-0300   FAX (616) 429-3852

Individual standards cost $10.00 each ($5.00 to ASAE members), and the book of standards costs $89.00 list ($29.00 to ASAE members).

S261.7—Design and Installation of Nonreinforced Concrete Irrigation Pipe Systems, 5 pages. This standard deals with low- or intermediate-pressure concrete irrigation pipelines with vents or stands open to the atmosphere. The standard discusses load and pressure limits and suitable soil conditions. Requirements for stands, vents, anchors, and thrust blocks are given. Installation guidelines for placement, joints and contractions, curing and backfilling, and testing are given.

S263.3—Minimum Standards for Aluminum Sprinkler Irrigation Tubing, 2 pages. This standard prescribes minimum requirements for design, manufacture, and test for "Class 150" irrigation tubing to be used in systems where the operating pressure will not exceed 1000 kPa (145 psi).

EP285.7—Use of SI (Metric) Units, 9 pages. This engineering practice is a guide to the use of SI (metric) units, symbols, usage, and conversion factors for physical quantities.

S330.1—Procedure for Sprinkler Distribution Testing for Research Purposes, 3 pages. This standard has the following two purposes: (1) to provide a basis for the accumulation of data on the distribution characteristics of sprinklers; and (2) to provide a uniform method for the presentation of the data described above.

S376.1—Design, Installation and Performance of Underground, Thermoplastic Irrigation Pipelines, 11 pages. This standard pertains to thermoplastic pipe used underground for irrigation and is intended to: (1) provide minimum guidelines for engineers and others in planning, designing, and specifying thermoplastic pipe commonly used for irrigation. It is not intended as a complete specification nor a replacement for the judgment of personnel familiar with site conditions or other controlling factors; (2) consolidate applicable reference information and technical data in readily available form; (3) establish uniform standards for materials used in the manufacture of thermoplastic irrigation pipe and to promote uniformity in classifying, pressure-rating, testing, and marking the pipe; and (4) establish minimum requirements for the design, installation, and testing of pipelines that are necessary for the satisfactory performance and safe operating of the irrigation system and to prevent damage to the system.

S394—Specifications for Irrigation Hose and Couplings Used with Self-Propelled, Hose-Drag Agricultural Irrigation Systems, 2 pages. This standard establishes minimum performance levels, functional properties, and physical characteristics for new irrigation hose and couplings used with self-propelled, hose-drag (cable-drawn) agricultural irrigation systems (travelers).

S395—Safety for Self-Propelled, Hose-Drag Agricultural Irrigation Systems, 1 page. This standard is intended to improve the degree of personal safety for operators and others during the normal application, operation, and service of self-propelled, hose-drag (cable-drawn) agricultural irrigation systems (travelers).

S398.1—Procedure for Sprinkler Testing and Performance Reporting, 2 pages. This standard has the following three purposes: (1) to define a common test procedure for the collection of sprinkler test data, such as pressure, flow rate, and radius of throw, which may be used for the purpose of publishing performance specifications for sprinklers whose areas of coverage have uniform radii; (2) to provide methods for the interpretation of test data for sprinkler performance specifications, as derived from (1) above; and (3) to provide a method to readily distinguish which performance specifications have been developed using this procedure.

EP400.1—Designing and Constructing Irrigation Wells, 5 pages. This engineering practice is intended as a guide for preparing specifications for irrigation well construction, the objective being to obtain economical wells of high efficiency that are relatively sand-free and have a long projected life. In addition to this engineering practice, well design and construction should conform to all applicable health, safety, and other governmental regulations.

EP405.1—Design and Installation of Microirrigation Systems, 4 pages. The purpose of this engineering practice is to establish minimum recommendations for the design, installation, and performance of microirrigation (trickle irrigation) systems, including drip, subsurface, bubbler, and spray irrigation systems. This engineering practice should encourage sound system design and operation and enhance communication among involved personnel.

EP409.1—Safety Devices for Chemigation, 3 pages. This engineering practice specifies necessary safety devices to be used when injecting liquid chemicals into irrigation systems. Many irrigators apply fertilizers, herbicides, insecticides, fungicides, nematicides, and other chemicals through irrigation systems. This process is known as chemigation. Properly formulated chemicals can be uniformly and safely applied by injecting them into water flowing through a properly engineered irrigation and injection system.

S435—Drip/Trickle Polyethylene Pipe Used for Irrigation Laterals, 3 pages. This standard covers requirements and methods for testing of polyethylene (PE) pipe made in standard dimensions for drip irrigation. Included are criteria for classifying PE plastic materials and PE pipe, a system of nomenclature for PE plastic pipe, and requirements and methods of test for materials, workmanship, dimensions, sustained pressure, burst pressure, and environmental stress crack resistance. Methods of marking are also given.

S436—Test Procedure for Determining the Uniformity of Water Distribution of Center-Pivot, Corner-Pivot, and Moving Lateral Irrigation Machines Equipped with Spray or Sprinkler Nozzles, 2 pages. The purpose of this standard is to establish a uniform method of collecting water-distribution data in the field and calculating the coefficient of uniformity from the data.

S447—Procedure for Testing and Reporting Pressure Losses in Irrigation Valves, 2 pages. This standard has the following purposes: (1) to define a common procedure for testing and reporting the performance of all types of irrigation valves; and (2) to specify methods for the interpretation of valve-performance data.

EP458—Field Evaluation of Microirrigation Systems, 6 pages. The purpose of this engineering practice is to define practical engineering procedures for the field evaluation of existing microirrigation (trickle) systems. Practical field-evaluation procedures will encourage practicing engineers and irrigators to evaluate the adequacy of existing microirrigation systems based upon common procedures. This engineering practice is aimed at a base level evaluation; a more comprehensive evaluation may be appropriate under some circumstances.

Additional Standards under Development

Following is a list of standards and practices now being developed that may be available in the near future. The prefix X denotes that they are under development. It will be replaced by an S, if it is an engineering standard, or an EP if it is a practice.

X439—Procedure for Testing and Reporting Low-Pressure Spray Distribution Device Performance as Used on Mechanical-Move Irrigation Equipment.

X494—Traveler Irrigation Machines—Distribution Uniformity Test Method

X512—Standard for Inline Fixed-Pressure Regulators

X437—PVC Above-ground Irrigation Pipe

X438—Performance Standards for Irrigation Pumping Plants

X467—Irrigation Equipment Emitting-Pipe Systems

X468—Irrigation Equipment Emitters Specifications and Test Methods

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