Drainage Manual

A Water Resources Technical Publication

U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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A guide to integrating plant, soil, and water relationships for drainage of irrigated lands.

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PREFACE

It has been said of world irrigation, "It is a modern science—the science of survival." A prime ingredient of this science is the development and maintenance of a root zone having a balance of moisture, air, and salts favorable for plant growth. Drainage is one of the essential activities needed to provide such a balance.

Where man has practiced irrigation agriculture successfully, he has enlarged his territory, supported increasing populations, lived in better health, and made great strides culturally. Where drainage has been overlooked or neglected, man's development and his civilization have failed. Lack of adequate drainage has probably been the greatest single cause of failure on irrigation projects throughout the world. History has shown repeatedly that excess water and salt must be removed from soils for irrigation to be permanently successful. If irrigation is the science of survival of man, it can be added that drainage provides for the survival of irrigation. The fundamental measure of the importance of drainage is the benefit provided by irrigation itself.

Drainage of irrigated lands by the Bureau of Reclamation began shortly after passage of the Reclamation Act in 1902. However, not until the late 1940's and early 1950's did engineers in the Bureau of Reclamation begin pioneering efforts to develop the technology of drainage of irrigated lands into a modern engineering science.

This manual contains the engineering tools and concepts that have proven useful in planning, constructing, and maintaining drainage systems for successful long term irrigation projects. The manual is not a textbook. Mathematical and experimental development of the engineering tools has generally not been included. Indeed, not even all the innovative ways to use the tools are included. The manual provides drainage engineers a ready reference and guide for making accurate estimates of drainage requirements. Design and construction criteria, if followed with reason, will result in reliable drainage systems for irrigated areas.

All the methods and techniques covered in the manual have proven to be very satisfactory through observed field conditions on irrigated lands throughout the world. Some methods have a more elegant development and basis in science than others, but all have been designed to solve practical problems in the field.

The manual contains techniques developed over the last 25 years by personnel in the Bureau of Reclamation. Messrs. R. J. Winger, Jr., L. D. Dumm, J. N. Christopher, W. F. Ryan, and G. P. Brunskill have been primary contributors of the new concepts.

Mathematical and computer treatment for the concepts were chiefly rendered by R. E. Glover, W. T. Moody, and R. W. Ribbens; A. J. Cunningham, Jr., made significant contributions to the second edition revisions. E. J. Carlson and E. R. Zeigler provided valuable research.

Field evaluation and application has been the main responsibility of field offices and crews. Without their dedicated efforts, many of the concepts would have remained little more than theoretical guesswork. Our special thanks to those directing these evaluations: D. A. Barker, K. G. Bateman, M. D. J. Batista, W. C. Bell, Keith Campbell, C. L. Christensen, D. A. DeBruyn, R. J. Efferts, R. R. Frogge, J. E. Fuller, H. T. Hardman, P. J. Kennedy, W. A. Lidster, R. O. Lunde, C. R. Maki, A. E. Mathison, John Monteith III, P. M. Myers, G. E. Neff, H. R. Nelson, C. A. Neumann, N. E. Noyes, P. J. Pehrson, J. A. Pugsley, G. D. Sanders, J. M. Schaack, H. A. Schweers, W. O. Watson, R. H. Weimer, and John Williford.

The relationships of drainage to land classification and project economics were developed through the efforts of J. T. Maletic, W. B. Peters, Edmund Barbour, and their staffs. Major contributions to the overall presentations in the manual were made by C. R. Maierhofer, W. H. Yarger, R. J. Winger, Jr., J. N. Christopher, and R. D. Mohr.

We gratefully acknowledge contributions to the development of drainage concepts used in this manual made by personnel of the Soil Conservation Service, Agricultural Research Service, and the many colleges and universities. Occasional references to proprietary materials or products in this publication must not be construed in any way as an endorsement, as Reclamation cannot endorse proprietary products or processes of manufacturers or the services of commercial firms for advertising, publicity, sales, or other purposes.

For this Second Edition of the Drainage Manual, the metric unit system has been added to the U.S. customary unit system to comply with U.S. Government requirements and for the benefit of those who prefer working with the metric system. Personnel of the Drainage/Seepage Section, Ground Water Branch, Denver Office of the Bureau of Reclamation were responsible for making these additions throughout the manual as well as for checking and updating all chapters in the manual.

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Chapter I

INTRODUCTION

1-1. General.—A prime requirement for successfully irrigated agriculture is the development and maintenance of a soil zone in which the moisture-oxygen-salt balance is favorable for plant growth. Plants require both moisture and oxygen to live. When a saline water table rises and remains in the root zone longer than about 48 hours, resulting in an abnormally high saline moisture condition, agricultural production is usually seriously affected.

The presence of oxygen in the interstices of the soil[1] in the root zone is as necessary as water for both seed germination and plant growth. The oxygen content of soil is governed by the rate of diffusion of oxygen through the soil pores. Also, the oxygen content is markedly affected by the moisture content of a soil. Soils with initially low moisture content normally have relatively open pore structures between soil particles, allowing oxygen to freely permeate through the interstices. As the moisture content increases, water displaces the air in the pores, thus forcing the air upward and into the atmosphere. Once the oxygen is expelled, the oxygen content recovery rate is extremely slow in a soil that is in transition from a moist or wet state to a drier state. This slow recovery is caused by the inherently slow rate of diffusion of gases through such soils and the phenomenon of capillary stresses which develop in soils when the water content does not completely fill the voids. The proper balance between soil moisture and oxygen is maintained to a considerable extent by adequate drainage.

A simple but comprehensive definition of adequate drainage is the removal of excess water and salt from the soil at a rate which will permit normal plant growth. Adequate drainage also may be defined as the amount of drainage necessary for successful maintenance and perpetuation of agriculture. This definition does not, however, necessarily imply complete and perfect drainage. Such is generally not feasible because the cost of preventing occasional damage to crops may not be justified solely by the amount of the damage. The aspect of economic justification must then be reconciled. The prime objective of a drainage project should be to

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[1] The term "soil" in this technical manual is loosely used to denote that part of the Earth's mantle above bedrock and includes the materials defined by the soil scientist as soil, subsoil, and substrata.

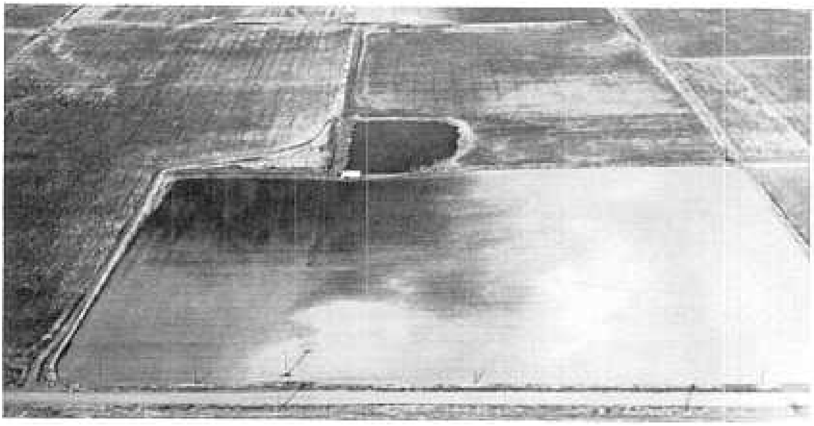
design and construct a drainage system which has optimum integration of soils, crops, irrigation, and drainage.

Drainage can be either natural or artificial. Most lands have some natural surface and subsurface drainage. When natural drainage is inadequate to handle the water reaching the land by either natural or artificial means, manmade or so-called "artificial" drainage is required. Artificial drainage thus fills the gap between that provided by nature and the established need. Artificial drainage usually supplements existing natural systems. For example, natural watercourses can be deepened or, where no suitable ones exist, new watercourses can be constructed. Almost every physical aspect and condition of lands, as well as man's potential agricultural use of them, will affect the ultimate drainage requirement. In humid areas where salt movement into the root zone is not a problem, shallow, closely spaced drains provide a rapid lowering of the water table in the spring, permitting earlier preparation of seedbeds and earlier planting. In arid irrigated areas, the water table is usually lowest in the spring and starts rising as a result of the snowmelt, spring rains, and early irrigations. This rising water table can be saline, and if allowed to permeate into the root zone, will affect both seed germination and plant growth. Drains in arid areas must be designed deep enough and spaced closely enough to provide sufficient head midway between drains to move the ground water to the drains without allowing the ground water to rise into the root zone at any time during the growing season. Capillary rise of salty ground water into the root zone during the growing season usually does not occur under good irrigation practices. Regulated irrigations and the resulting deep percolation are frequent enough to keep the root zone soils leached of salt and also provide sufficient moisture content to prevent appreciable upward capillary movement.

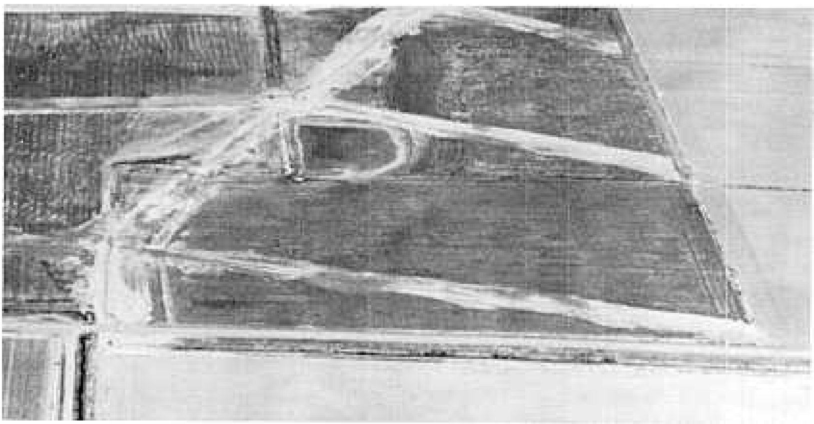
Figure 1-1 shows the land use and conditions of a farm area before, during, and after drain construction. The top photograph on figure 1-1 shows the effects of seepage and salinity on an irrigated area prior to any drainage construction. The dark areas on this photograph are waterlogged soils and the patchy growth areas are a result of salinity. The middle photograph was taken of the same area soon after drain construction. The herringbone pattern of the drainlines is clearly visible. The bottom photograph was taken of the same area 2 years after the drainage system was completed. The land has been completely reclaimed with little evidence of the former problems.

1–2. Scope.—This technical manual:

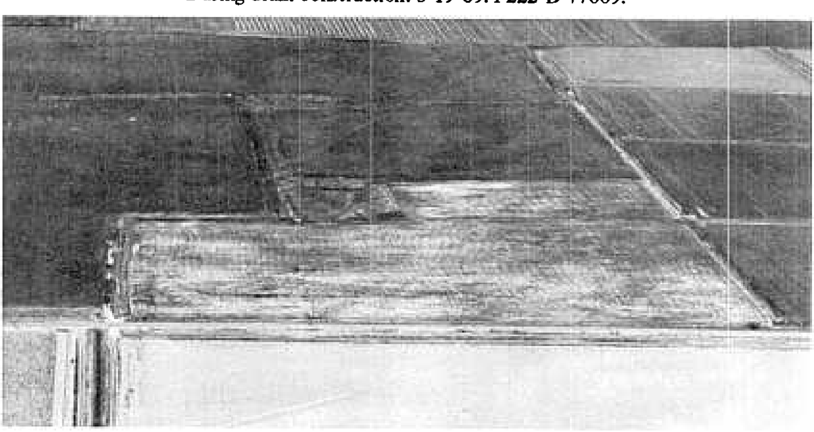
- Reviews the methods and techniques used in solving various phases of drainage problems;
- Suggests pertinent data required;
- Tells where and how to obtain the data; and
- Details how to record, present, analyze, and apply these data.



Before drain construction. 10-27-66. P222-D-77008.



During drain construction. 3-19-69. P222-D-77009.



After drain construction. 10-1-71. P222-D-77010.

Figure 1–1.—Farm conditions before, during, and after drain construction.

Problems of forecasting drainage requirements are discussed and suggestions on drainage design criteria and construction standards are presented. This manual is not intended for use as a theoretical textbook on drainage but, rather, is directed toward field application of engineering knowledge on the subject. The manual does not provide a step-by-step approach which will solve every drainage problem because good judgment, as well as proper procedure, must be used in the solution of drainage problems. An attempt is made to develop guidelines for use in exercising such judgment.

1-3. History.—Drains were constructed and drainage engineering was practiced long before man's recorded history, as evidenced by archeological finds. Some ancient systems were simple, some were elaborate, but very few were entirely successful, and practically none have survived to the present time. Man's drainage problems have been attributed partly to his neglect of drainage systems and partly to his lack of understanding of all the physical and technical problems involved. Man's basic knowledge and understanding of soil physics and hydraulics are now being applied to drainage problems, and drainage engineering is rapidly emerging from the "build it here and see how it works" stage. Drainage engineering is not, however, an exact science and probably never will be, because it remains largely a matter of experience, common sense, and judgment.

1-4. Importance.—The importance of drainage to the irrigation economy of a project, State, or Nation too often has been underestimated. The history of irrigation, as practiced in the United States and the world, universally points out the inescapable conclusion that successful irrigation requires adequate drainage. Only on irrigated lands with the rare combination of adequate natural surface and subsurface drainage will excess surface water and deep percolation from irrigation drain naturally from the land rapidly enough to prevent the rise of ground water to critical levels. Where natural drainage is inadequate and artificial drainage cannot be economically provided, the land cannot be permanently irrigated. Lands having original water tables 5 to 30 meters (20 to 100 feet) below the ground surface, and seemingly favorable natural drainage conditions, have eventually developed excessively high water tables, leading to waterlogging or salinization or both.

Man's knowledge and desires are paradoxical. Few deny that drainage is essential, yet many wishfully hope to get along without it. Canal and distribution systems are essential also, but here the similarity ends. Without these latter features, irrigated agriculture *cannot* exist, but irrigated agriculture—of a sort and for a time—can exist without drainage. Symptoms of high ground water and salt may not develop for some time after the beginning of irrigation, and soil deterioration may take place before the need for drainage is recognized.

1-5. Benefits of Drainage.—Judgments of the benefits of man's acts are always highly subjective. Consequently, some items listed in this section as benefits of drainage are held in disdain by those having different value concepts. In this manual, the subject of benefits will be approached from the viewpoint of establishing and maintaining permanent agriculture. Conditions directly promoting the health and welfare of crops and of the people growing those crops will be considered beneficial. Some of the benefits obviously could be construed as detrimental to other aspects of our ecology—a thought which drainage specialists should constantly keep in mind.

Soil is a porous medium consisting of liquid, gaseous, and solid materials which provide the crops with essential water, oxygen, and nutrients. Unless both the supply of water and oxygen can be maintained, the nutrient intake by crops is reduced. Drainage is essential to maintain the supply of oxygen. Other factors associated with drainage and plant growth are soil temperature, trafficability, resistance to disease and root growth, and chemical and biological conditions favorable to crop growth.

Drainage plays an important part in all of the above factors. Saturated soils directly impede the intake of water and nutrients and curtail root growth. Poor drainage discourages the growth of aerobic bacteria which are needed to provide nitrogen for crops. In saturated soil, lack of oxygen prevents formation of usable forms of nitrogen and sulfur. In addition, toxic organic and inorganic compounds develop in saturated soils.

Subsurface drainage promotes conditions that maintain soil structure, trafficability, and workability. These conditions exist particularly in fine-textured soils containing swelling clays. Efficient farm operations require well-drained soils throughout the season. Poorly drained soils adversely affect preparing, planting, cultivating, irrigating, and harvesting operations.

Saturated soils require as much as three times more heat to raise the soil temperature 1 °C, and they are usually 4 to 8 °C (7 to 14 °F) cooler than similar well-drained soils. Drainage promotes early warming of soils in the spring which, in turn, promotes biological and chemical activity in the soils that is important to seed germination and plant growth. Well-drained soils can be planted from 2 to 3 weeks earlier than similar saturated soils, which is important in areas with short growing seasons and where early harvests bring higher prices.

Most plant root systems will not penetrate deeply into a water table. In an area with a high water table, the root system will be shallow and more susceptible to disease. Cold, wet soils seem to encourage the activities of many disease organisms that attack weak seedlings. In a drained soil, the plant roots can penetrate more deeply, thus enlarging the supply of plant food which produces a healthier, more vigorous growth. Figure 1-2 shows the effects of shallow water tables on plant roots.

Proper control of salinity and alkalinity can be accomplished only in well-drained soils. Leaching water must be able to pass through the soil profile to move excess salts out of the root zone. This movement cannot occur unless free drainage exists. Conversely, a high water table creates a condition wherein capillarity moves salts into the root zone and deposits them there.

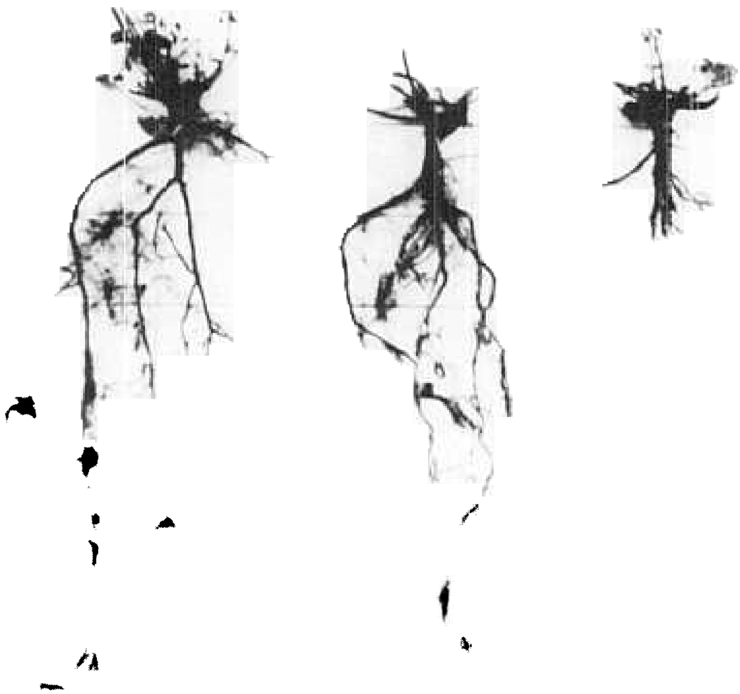


Figure 1–2.—Effects of shallow water table depths on plant roots. These 1-year-old alfalfa plants were grown in different areas over depths to water table of: (left to right) 0.6 meter (2 feet), 0.3 meter (1 foot), and 0.1 meter (4 inches). The most vigorous growth generally occurs when the water table is at least 1 meter below the ground surface. P801-D-77011.

Some of the less tangible benefits resulting from good drainage are:

- The reduction or elimination of mosquito and other insect breeding grounds;
- Control of botulism;
- Improvement of farmlands by elimination of boggy and weed-breeding areas;
- Improvement of public and private roads by elimination of soft spots which results in lower road maintenance costs; and
- A firm, dry land surface to support harvesting machinery.

In summary, the benefits of adequate drainage are:

- A longer growing season;
- Increased soil tilth;
- Early and more vigorous plant growth;
- Larger yields;
- A wider selection of crops;
- Decreased cost of production;
- Vector and weed control; and
- Dry, firm land surfaces.

1-6. Drainage and Environment.—Multipurpose projects require analysis of benefits and costs from a wide range of factors other than agriculture. Unfortunately, many gains and losses to certain aspects of the environment have not been quantified in any generally accepted terms. Dollars and cents dominate economic analyses because actual costs of system construction can be estimated with these terms. However, the net value of eliminating or altering an aspect of the environment and replacing it with another is currently based on the individual values of the people involved. Some irrigators tend to look at wildlife habitat on their land as a troublesome weed patch, while the wildlife specialist sees clean farms as barren wasteland when evaluated as part of the ecology. More and more, drainage engineers must consider all values in planning, constructing, and operating projects. They must share the responsibility with all other disciplines, including soils, geology, ecology, cultural resources, and economics, for identifying the effects of their work on the environment.

Some benefits that cannot be quantified in terms of money can often be realized for little or no cost. For example, fisheries have naturally established themselves in most large drainage systems. With little more than an awareness of what constitutes a favorable fish habitat, the systems possibly could have been planned to develop even better fisheries for little additional cost. All drains require maintenance, however, and the possibility of cleaning them with certain chemicals, such as sulfur dioxide or copper sulfate, should be a prime consideration in planning a drainage system for multiuse.

Establishing wildlife habitats may create insect control problems. Bacteria, viruses, and other pathogens may breed in the habitat, and diseases produced by them may find their way to neighboring communities through carriers such as mosquitoes or domestic and wild animals using the habitat. The benefits and costs associated with maintaining or eliminating such breeding grounds must be weighed along with all other benefits and costs. Consideration of wildlife habitats must include contacts with local health officers.

Water quality has always been a concern of drainage engineers. State and national water quality criteria for surface waters are being upgraded and more precisely defined. These criteria identify total salt load as a concern, and regulations limit allowable quantities of potentially toxic trace elements. These regulations require that the drain system designer know the quality and constituent composition of the drainage system effluent. The applicable quality standards must be met and the required discharge permits obtained before disposal of drainwater to surface waters can take place. In some areas, treatment of drainage waters before final disposal may be required.

A wide variety of considerations could be enumerated, but little in the way of practical guidelines could be offered. The drainage engineer simply must maintain constant awareness of water and land resource uses other than agricultural. Plans must integrate as many positive effects as are practical with the basic objective, and yet the planner must anticipate and remain aware of negative effects upon the environment which must be considered in the overall objective.

1-7. Drainage Nomenclature.—Drainage nomenclature is complex and has been developed from conditions such as the source of water to be moved, when and where the drains are to be built, and their function. Drains may be either surface or subsurface, open or pipe, constructed concurrently with project development or deferred. They sometimes consist of wells (recharge, relief, or pumped) and may fall within various functional classifications:

(a) Surface Drainage.—Surface drainage is the removal of water from the surface of the land. Situations which may produce the need for surface drainage include excess precipitation, water applied in irrigation, losses from conveyance channels and storage facilities, or water which has seeped from ground water at a higher elevation. Control of surface water is normally accomplished by providing channels to facilitate removal.

(b) Subsurface Drainage.—Subsurface drainage is the removal or control of ground water and the removal or control of salts, using water as the vehicle.

Situations which may produce the need for subsurface drainage include percolation from precipitation or irrigation; leakage from canals, drains, or surface water bodies at higher elevations; or leakage from artesian aquifers. Generally, any drain or well which is designed to control or lower the ground water is considered subsurface drainage.

(c) Open and Pipe Drains.—Open drains are channels with an exposed water surface. Pipe drains are buried pipe regardless of material, size, or shape. Generally, all of the nomenclature for other types of drains may be applied to either open or pipe drains. Drain size and purpose, physical condition of the soils, topography, required drain spacing, and annual operation and maintenance costs largely dictate whether drains are to be open or pipe.

(d) Deferred Drainage.—Deferred drainage is that which is provided after project works have been constructed and the irrigation has begun. The deferral of construction of such drains usually is necessary because of the difficulty of locating and designing them accurately before the lands are irrigated and the drainage problem becomes evident. The term "deferred drainage" is more often applied to subsurface drainage because the need for surface drains which are constructed as a part of the initial project works is generally more evident. Bureau of Reclamation policy requires the inclusion of deferred drainage in the project

plan and cost estimate. Only an estimate can be made as to when these expenditures will be required. Experience with past projects shows that about 50 percent of these drains are installed during the first 15 years of project operation. Drainage installations are essentially complete after 30 years unless major changes in water use occur.

(e) Function of Drains.—The nomenclature used for technical aspects of drainage and as used herein is based on the function of the drain. The five types of drains are designated: relief, interceptor, collector, suboutlet, and outlet, see figure 1-3. Relief and interceptor drains have the principal function of controlling ground-water levels. They form the upstream portion of the land drainage system, and the distinction between them is based on the slope of the ground-water body they control. Both relief and interceptor drains may be constructed as either open or pipe drains. They are designed as open drains when they are required to receive irrigation surface waste and excess precipitation from adjacent fields.

(1) *Relief* drains are used to effect a lowering of ground water over relatively large flat areas where percolation from precipitation or irrigation serves as the water source, and where gradients of both the water table and subsurface strata do not permit sufficient lateral movement of the ground water.

(2) *Interceptor* drains are used to cut off or intercept ground water which is moving downslope from some source.

(3) *Collector* drains receive water from subsurface relief or interceptor drains and from farm surface drains carrying irrigation surface waste and storm runoff. Because collector drains control ground water as well as receive flow from tributary subsurface drains, they must be designed with a normal water surface at or below the depth which will provide effective subsurface drainage in adjacent or tributary areas. They may be either open or pipe drains depending on the volume of water to be handled, the available gradient, and whether their tributaries are open or pipe drains.

(4) *Suboutlet* drains have the principal function of conveying water from collector drains to the outlet drain. In general, they are located in topographic lows such as draws and creeks but can also be constructed drains. These drains receive inflows from a number of collector drains and canal and lateral wasteways. Suboutlet drains resemble collector drains in function, except they usually are not required to serve as subsurface drains in the control of ground water to prescribed elevations. They may be located entirely within the project area or they can be the outlet for lands not included in the project. On figure 1-3, the suboutlet drains are shown as the principal creeks of the project area.

(5) *Outlet* drains convey collected water away from the drained area or project. The outlet drain is usually a natural channel in the topographic low for the area to be drained, but where a natural channel does not exist, one can be constructed. Figure 1-3 shows the outlet drain as a river traversing the central portion of the project area.

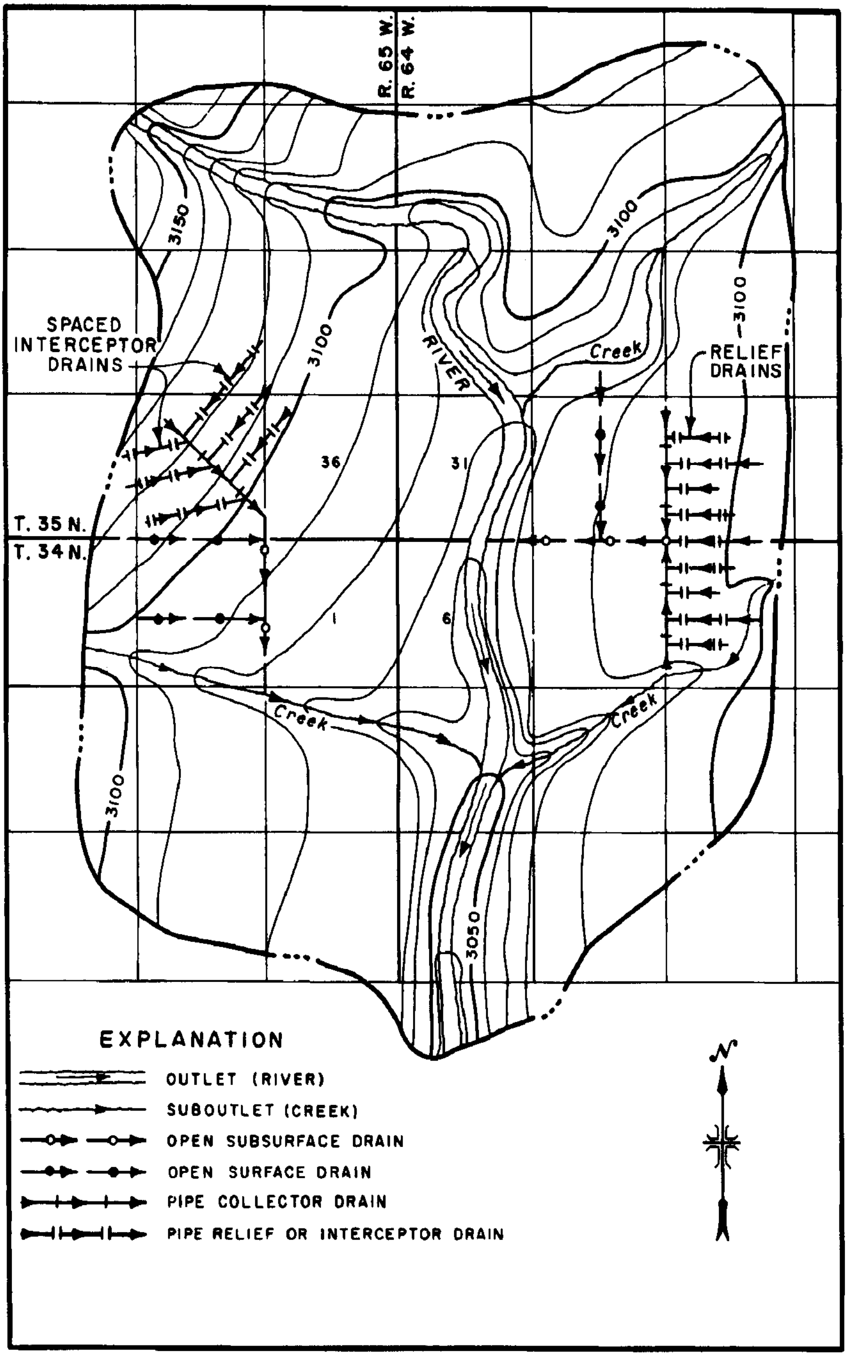


Figure 1–3.—Types of drains. 103-D-1617.

(*f*) *Inverted, Relief, or Pumped Wells.*—These special installations may be used to dispose of surface water, to control ground-water levels, or to relieve hydraulic pressures where local physical conditions can be adapted for their use. An explanation of their use and limitations is discussed in chapter V, part E.

Chapter II

BASIC DATA

2–1. Introduction.—Selection of the optimum drainage plan and the design and construction of adequate and successful drainage facilities depend upon the reliability and adequacy of the basic drainage data. The data requirements for a particular drainage problem vary with the type of problem and the degree of importance of the investigations or report being prepared. The basic data must be sufficiently representative to permit selection of a good drainage plan from which a functionally sound drainage system can be designed and constructed. Cost estimates must be made which are reasonably accurate for the purposes intended. Inadequate or unreliable data introduce serious risks in determining the drainage requirements and cost estimates.

The basic data must provide a knowledge of: (1) capacity of the soils to transmit water; (2) amount, source, movement, and chemical characteristics of the water that must be transmitted; and (3) available hydraulic gradients, both natural and those induced by man. Sufficient data must be gathered to estimate the effects of the drainage plan on both the social and economic environment.

2–2. Topography.—Topography, which is of prime importance in drainage, influences the general plan that must be made and, for most areas, the location of the outlet, suboutlet, and collector drains. Even before reaching the planning and designing stages of drainage, the importance of topographic features can be recognized. Topography can mean the difference between the need for little or no artificial drainage facilities and extensive drainage facilities. Where surface slopes are sufficient, excess precipitation, irrigation water, and canal waste will flow rapidly from the area. Such rapid removal of excess surface water diminishes percolation to the ground-water table. Favorable topography may provide adequate surface drainage and reduce the need for artificial subsurface drainage.

Topographic maps are essential in any detailed drainage investigation. These maps show land slopes, length of slope, location and direction of natural drainage, potential outlets, and other special conditions which affect drainage. In addition, the maps often reveal clues to the type of drainage needed and, to a degree, its practicability. The scale of the maps to be used depends upon the size of the area being studied and the purposes of the investigation. For a reconnaissance study, a scale of 1 inch equals 4,000 feet (1:48,000) is usually adequate, but maps with other scales may be used. For smaller areas or for a more detailed study, a scale of 1 inch equals 2,000 feet (1:24,000) would be advantageous. Detailed studies of special problem areas and the location and design of the constructed drainage system require a scale of 1 inch equals 400 feet (1:4,800). Topographic maps should have contour intervals consistent with the scale used, the size of the area surveyed, and the purpose of the map. For preliminary study of large areas with considerable topographic relief, a 2-meter or 5-foot contour interval is satisfactory provided the natural drainage pattern is adequately shown. A 1-meter or 2-foot interval is usually sufficient for the actual drainage layout, but for large, nearly level areas, a 0.3-meter (1-foot) interval is required. In addition to relief and natural features, topographic maps should show the location of springs, seeps, wells, and cultural features such as roads, railroads, culverts, pipe and utility lines, structures, and land subdivision lines.

In many instances, topographic maps have been prepared for a proposed or existing irrigated or cultivated area, either specifically for the purpose of laying out the irrigation system or for other related purposes. The Soil Conservation Service, Bureau of Reclamation, and other Federal and State agencies are the most probable sources for such maps. The U.S. Geological Survey and the U.S. Coast and Geodetic Survey are usually the best sources of general topographic maps. More detailed information about published geologic maps for individual States is given in the series of geologic map indexes available from the U.S. Geological Survey. Even though the available maps may be inadequate for the study being made, they may contain usable information which may reduce significantly the additional surveying required. If adequate topographic maps are not available, a field survey will have to be made.

Aerial photographs are useful in drainage studies. They supplement topographic maps in presenting an overall picture of natural and artificial drainage ways and particularly of outlet conditions. Additionally, they will often reveal the existence and location of drainage problems, such as seepy areas and saline or alkaline deposits, and may provide clues to the source of excess water. The U.S. Department of Agriculture agencies, such as the Soil Conservation Service and Forest Service, and local county agricultural agencies may have information on the existence of aerial photographs of an area. In addition, the State engineer and the State waterboard, or their equivalents, may have knowledge of the availability of maps or photographs.

Most aerial photographs are of the general-purpose panchromatic type. For small areas, greater use can be made of these photographs when a 2-film filter combination is used. Comparative interpretation of infrared and panchromatic photography, using proper film-filter combinations, yields information on high ground-water areas and also indicates, by contrasting toned areas or patterns, the presence of soluble salts in the root zone. For a more complete discussion on the use of aerial photographs, see *Manual of Photogrammetry* (American Society of Photogrammetry, 1980).

Drainage maps are developed from information taken from topographic maps, aerial photographs, land classification maps, county road maps, and ownership maps. Added to the existing features are drainage design features such as type of proposed or existing drainage systems, observation well locations, depth to barrier, depth to ground-water table, and water table contours. Conventional symbols for drainage maps are shown on figure 2–1.

2–3. Geology.—*(a) General.*—An understanding of geological processes is helpful in appraising and analyzing the occurrence and solution of drainage problems. In some areas, the in-place soil material has been deposited as a result of volcanic eruption. Fine ash material is spread over the land surface in the vicinity of the volcano to depths that sometimes reach many feet. The soil in these areas is fine grained and has adequate hydraulic conductivity near the surface, but becomes less permeable with depth. Near the volcano's cone, the fine ash is usually underlain by volcanic cinders which have very good drainage and stable construction properties.

In other areas, the soil deposition results from glacial action. The textures of these soils, which are called glacial till, vary from clay and fine-grained rock flour to coarse gravels and cobbles. The shape of the grains and the gradation of the formation are a result of the nature and location of the parent material from which they were derived and the glacial phenomena associated with transportation and deposition. Undisturbed glacial till is usually dense and has a very low hydraulic conductivity rate, while till that has been disturbed or reworked is more friable and usually has sufficient hydraulic conductivity to be economically drained. Formations of glacial lakes, and deposition of eskers, moraines, kames, and similar forms are examples of glacial action.

Residual soils formed from disintegration of the underlying parent material are found in many areas. The characteristics of these soils are influenced by the type of parent material, weathering processes, and the reworking action by wind and water. The parent rock material may have been of igneous, sedimentary, or metamorphic origin.

Probably the most widespread soil material in irrigated lands is alluvial in character. These water-deposited materials range in texture from clays to gravels and in all possible combinations thereof. They consist of outwash from mountains, streams, river and lake deposits, and similar formations which result from various geologic processes. As rivers aggrade and degrade over the years, as they meander and entrench themselves, and as mountain streams flow out on the plains, the shape of the land surface is changed. The present topography is the result of these processes over thousands of years. Most alluvial profiles have adequate hydraulic conductivity for economically feasible drainage systems.

Lacustrine deposits consist of materials that have settled out of quiet waters of lakes and are usually recognizable by their flat surfaces surrounded by high ground. Soils can vary from clays to coarse sands in these deposits, and the continuity and structure usually vary throughout the lakebed. Most lacustrine soils can be economically drained.

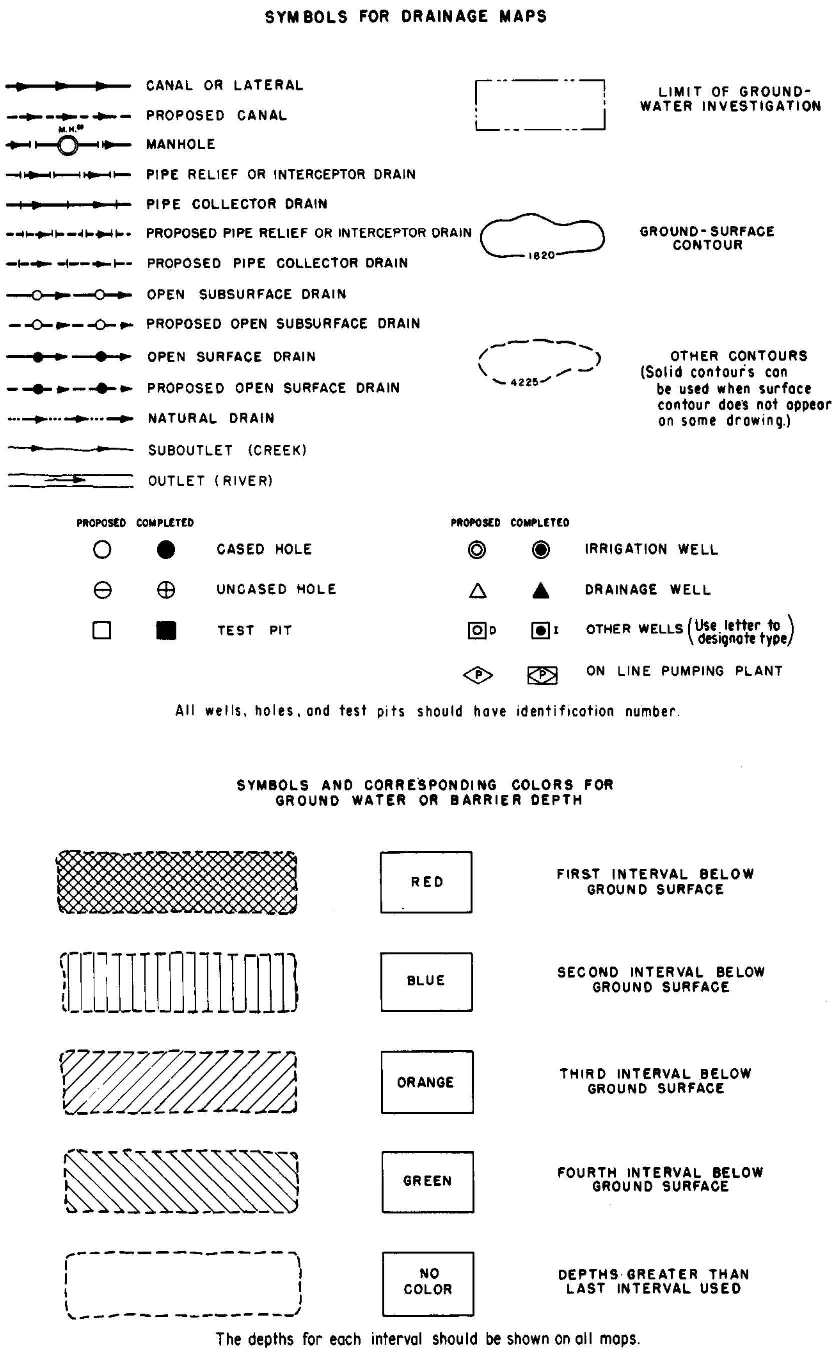


Figure 2-1.—Conventional symbols for drainage maps. 40-D-5063.

Another material which is found in many areas is the eolian or wind-deposited soil. These soil deposits are fine grained due to the limited ability of the wind to carry large grains. Two principal classes of soils formed in this manner are loess and sand dunes, the deposits of which have been found to considerable depths. These soils have adequate drainage characteristics for economically feasible drainage systems.

Because soils are the results of complicated geologic processes, there are many more geologic soil types than mentioned above. Wide varieties of geologic situations have important bearings on drainage investigations and determination of drainage needs. Therefore, in the interest of accuracy, time, and the design of an effective drainage system, an evaluation of the geologic situation by a qualified geologist is desirable.

Positive landform recognition can assist the engineer in determining the types of field investigations needed to solve a drainage problem. Recognition of the landform also plays an important part in evaluating the drainability of lands intended for irrigation development. As an example, the permeability characteristics found at the toe of an alluvial fan may vary greatly from those found in its middle or upper reaches. Likewise, an ancient river channel terrace would exhibit different geohydraulic characteristics from a recent flood plain area.

(b) Barrier.—The barrier is a stratum or layer that restricts the movement of water. Geology is often a key in determining the barrier—also known as the barrier stratum, barrier layer, or barrier zone. These terms are often used in drainage engineering and are related to the relative hydraulic characteristics of various strata.

Since strata in irrigated areas are found in a generally horizontal attitude relative to the ground surface, the barrier zone is usually considered as a barrier to the vertical movement of water. This is not exclusively the condition, however, because in areas of unconformity or folding of geologic strata, a vertical barrier may also restrict the horizontal movement of water.

When water percolating downward under the force of gravity reaches the top of a barrier zone, a saturated condition develops, resulting in differential pressures. Most of the water moves laterally above the barrier zone. Therefore, in ground-water hydraulics, the barrier zone limits the depth of material available for the movement of ground water.

This depth-of-flow zone, together with the material's hydraulic conductivity, greatly influences drainage requirements for a given area. A typical drainage investigation requires a great deal of effort to identify the barrier zone and its depth below the ground surface. This depth-to-barrier data is used to determine the depth-of-flow zone available to a drainage system.

(c) Aquifers.—Geologic identification of artesian aquifers may be important when evaluating drainage requirements and drainage system performance. An artesian aquifer that is under sufficient pressure to cause the piezometric water surface to rise to or near the land surface will contribute to the drainage problem. When this happens, the artesian water, as well as deep percolation from irrigation

and precipitation, must be handled by drainage. This increases the drainage requirements to a quantity such that drainage usually is uneconomical.

2-4. Soil Characteristics.—Of primary concern when evaluating subsurface drainage requirements is determining the capability of the soil (previously defined to include soil, subsoil, substrata, and in some situations the underlying consolidated formation) to transmit water both laterally and vertically. The capability of the soil to transmit water is a function of the hydraulic conductivity, effective depth of the saturated zone, and the hydraulic gradient. All of the soil characteristics of density, porosity, particle size, grain distribution, texture, structure, chemical properties, and water-holding capacity affect the movement of water through soil, as does the chemical composition of the water itself. However, of all the characteristics that affect this movement, the one which integrates the combined effects for a particular water and a particular soil—and the one which is basic in the solution of drainage problems—is the hydraulic conductivity or coefficient of permeability as it is known by most engineers. Studies to establish a relationship between hydraulic conductivity and one or more of the readily determined soil properties have proven to be difficult. In areas where soils were derived from the same source, deposited in the same manner, affected by the same climatic conditions, and, in general, have similar chemical and physical characteristics, a relationship between hydraulic conductivity and these properties can be determined. By using this relationship, the number of hydraulic conductivity tests can be reduced by assigning correlated hydraulic conductivities to similar soils.

(a) Hydraulic Conductivity.—The facility with which water moves in a soil is a measurable property of the soil called hydraulic conductivity. An understanding of and a means of determining this property is essential to understanding and correcting most subsurface drainage problems. Hydraulic conductivity has been defined in various ways. As used herein, it refers to movement of a particular water in a particular soil under specified conditions. It is expressed as the constant K in Darcy's Law: $K = \frac{v}{i}$, where v is velocity of flow and i is the hydraulic gradient.

(1) *Dimensions.*—Physical dimensions for hydraulic conductivity depend on those used to express the velocity. For laboratory-type testing cubic centimeters per square centimeter per second is commonly used; however, this results in extremely small numbers. For field applications cubic meters per square meter per day results in more reasonable size numbers. These units are commonly shortened to centimeters per second and meters per day and are referred to as rates. In the U.S. customary system, cubic feet per square foot per day (feet per day) and cubic inches per square inch per hour (inches per hour) are commonly used. Cubic feet per square foot per year is also used. Table 2–1 presents conversion factors for various hydraulic conductivity units.

(2) *Weighted average hydraulic conductivity.*—This refinement on hydraulic conductivity is often used in the determination of subsurface drainage requirements, and is simply the weighted average hydraulic conductivity of all soils between the maximum allowable water table height and the barrier. The value is obtained by averaging the results from in-place hydraulic conductivity tests at different locations in the area to be drained.

Table 2–1.—*Conversion factors.*

... Example (1) Example (2)

①	②	③	④	⑤	⑥	⑦	⑧
$ft^3/ft^2/yr$	$ft^3/ft^2/d$	$ft^3/ft^2/s$	$in^3/in^2/h$	$gal/ft^2/d$	$m^3/m^2/d$	$cm^3/cm^2/h$	$cm^3/cm^2/s$
	①/365	②/86,400	② × 0.50	② × 7.4805	② × 0.3048	④ × 2.540	③ × 30.48
1	0.00274	3.17×10^{-8}	0.00137	0.0205	0.000835	0.00348	9.67×10^{-7}
365	1	1.16×10^{-5}	0.500	7.4805	0.3048	1.270	0.0003528
31,536,000	86,400	1	43,200	646,317	26 335	109 728	30.48
730	2.0	2.31×10^{-5}	1	14.96	0.6096	2.540	0.0007056
48.79	0.1337	1.55×10^{-6}	0.0668	1	0.0407	0.1698	4.72×10^{-5}
1,197.5	3.2808	3.80×10^{-5}	1.6404	24.54	1	4.167	0.001157
287.4	0.7874	9.11×10^{-6}	0.3937	5.8902	0.240	1	0.000278
1,034,646	2,834.6	0.032808	1,417.3	21,205	864	3 600	1

EXAMPLES:

(1) The hydraulic conductivity of a soil has been determined to be 15.2 $gal/ft^2/d$. To convert to $m^3/m^2/d$–Find value of 1 in Col. ⑤ and move horizontally to value for $m^3/m^2/d$ in Col. ⑥. Multiply 15.2 by value in Col. ⑥ (0.0407) = 0.619 $m^3/m^2/d$.

(2) The hydraulic conductivity of a soil has been determined to be 0.00393 $cm^3/cm^2/s$. To convert to $ft^3/ft^2/d$–Find value of 1 in Col. ⑧ and move horizontally to value for $ft^3/ft^2/d$ in Col. ②. Multiply 0.00393 by value in Col. ② (2,834.6) = 11.14 $ft^3/ft^2/d$.

The weighted hydraulic conductivity for lateral movement through soils may be obtained by the following method:

$$\frac{D_1 K_1 + D_2 K_2 + \ldots\ldots\ldots + D_n K_n}{\text{Total } D}$$

where:

D_1, D_2, and D_n = thickness of first, second, ..., and *n*th. soil strata,
K_1, K_2, and K_n = hydraulic conductivity of first, second, ..., and *n*th. soil strata, and
D = total thickness of soil profile tested.

The weighted hydraulic conductivity for the vertical component may be obtained using:

$$\frac{\text{Total } D}{\frac{D_1}{K_1} + \frac{D_2}{K_2} + \ldots\ldots\ldots + \frac{D_n}{K_n}}$$

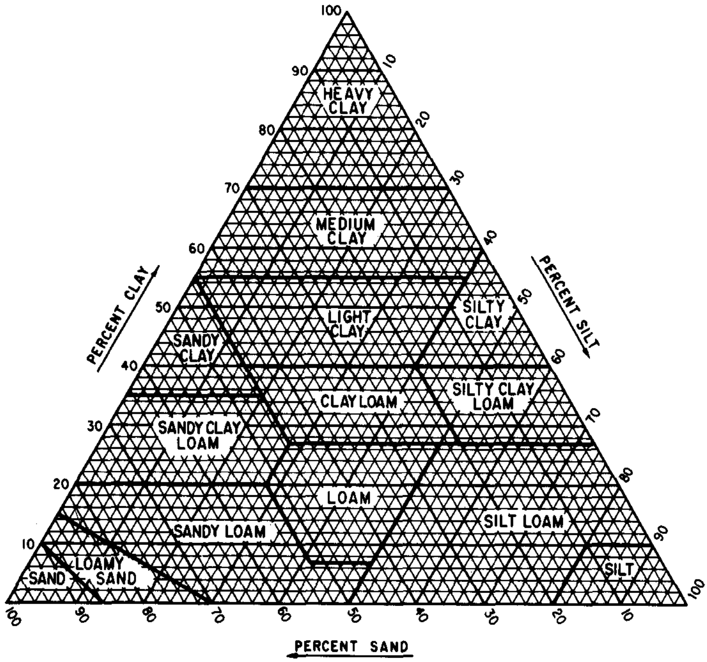
Soils are usually heterogeneous and anisotropic (having unequal physical properties along different axes). This results in nonuniform field conditions for obtaining hydraulic conductivities over an area of appreciable size. High-degree precision in hydraulic conductivity values is therefore not obtainable; however, every effort should be made to get the best accuracy possible. Procedures for the various methods on obtaining hydraulic conductivities are discussed in chapter III.

(*b*) *Texture.*—The term "texture" relates to the proportion of the various sizes of particles in a soil sample. Texture is important in subsurface drainage because it is a soil characteristic which has a general relationship with hydraulic conductivity and water retention. In general, the coarse-textured soils have higher hydraulic conductivities and lower water retention than fine-textured soils. Texture is readily measurable by performing a gradation analysis to separate the size groups. The particle size classification shown in table 2–2 was developed by the U.S. Department of Agriculture. This table is used by the Bureau of Reclamation in land classification and drainage work because it relates to the agricultural properties of the soil and allows better correlation with hydraulic conductivity than do the Casagrande or Unified Soil Classification systems.

Table 2–2.—*Particle size classification.*

Material	Diameter
Stones	Greater than 250 millimeters (mm)
Cobbles	250 to 80 mm
Coarse gravel	80 to 12.5 mm
Fine gravel	12.5 mm
Very coarse sand	2.0 to 1.0 mm
Coarse sand	1.0 to 0.5 mm
Medium sand	0.5 to 0.25 mm
Fine sand	0.25 to 0.10 mm
Very fine sand	0.10 to 0.05 mm
Silt	0.05 to 0.002 mm
Clay	Less than 0.002 mm

Textural classes are arbitrary groupings based on the relative proportion of the various-size particles in the soil mass. The soil texture triangle, figure 2–2, is used to convert quantitative data from detailed gradation analyses of the separates less than 2 millimeters in diameter to textural class names of soils. Textural class names of material larger than 2 millimeters in diameter are as shown in table 2–2.



TEXTURAL CLASSES

TEXTURE		SAND %	SILT %	CLAY %
SAND	(S)	85 to 100	0 to 15	0 to 10
LOAMY SAND	(LS)	70 to 90	0 to 20	0 to 15
SANDY LOAM	(SL)	43 to 85	0 to 50	0 to 20
LOAM	(L)	23 to 52	28 to 50	7 to 27
SILT LOAM	(SiL)	0 to 50	50 to 100	0 to 27
SANDY CLAY LOAM	(SCL)	45 to 80	0 to 28	20 to 35
CLAY LOAM	(CL)	20 to 45	15 to 53	27 to 40
SILTY CLAY LOAM	(SiCL)	0 to 20	40 to 73	27 to 40
SANDY CLAY	(SC)	45 to 65	0 to 20	35 to 55
SILT	(Si)	0 to 20	80 to 100	0 to 12
SILTY CLAY	(SiC)	0 to 20	40 to 60	40 to 60
CLAY	(C)	0 to 46	0 to 40	40 to 100

BASIC TEXTURAL CLASS MODIFYING TERMS

SAND

Diameter, millimeter	U.S. Standard sieve numbers	Term	
0.05 to 0.10	300 to 140	Very fine sand	(VFS)
0.10 to 0.25	140 to 60	Fine sand	(FS)
0.25 to 0.50	60 to 35	Medium sand	(S)
0.50 to 1.00	35 to 18	Coarse sand	(CsS)
1.00 to 2.00	18 to 10	Very coarse sand	(VCsS)

GRAVEL

Content, Percent	Term	
20 to 50	Gravelly	(Gr)
50 to 90	Very Gravelly	(VGr)

Coarse sand: 25% or more VCsS and less than 50% of any other grade of sand.
Sand : 25% or more VCsS, CsS, and S, and less than 50% of F or VFS.
Fine sand : 50% or more FS and less than 25% of VCsS, CsS, and S and less than 50% of VFS.
Very fine sand: 50% or more VFS.

Figure 2-2.—Soil triangle of the basic soil textural classes. 103-D-1618.

(c) Color.—Color is an important soil characteristic that permits quick and easy identification and comparison of soils. In itself, color has no direct influence on the hydraulic conductivity, but when combined with texture and structure, color helps identify similar soils. Results of hydraulic conductivity tests can then be projected for these similar soils.

Soil color can best be described by comparison with the Munsell color chips for hue, value, and chroma. The hue indicates the color's relation to red, yellow, green, blue, and purple; the value indicates the shade from white to black; and the chroma indicates its departure from a neutral of the same lightness.

Nearly every soil profile has many horizons differing in color. A single horizon may be of one color, mottled, or marked with spots or streaks of other colors. Certain combinations of mottled colors are indicative of poor hydraulic conductivity. However, some mottled patterns occur that are not associated with poor drainage, especially in parent materials that are not completely weathered.

A complete discussion on the origin of different soil colors can be found in Agriculture Handbook No. 18, *Soil Survey Manual* (U.S. Dept. of Agriculture, 1962).

(d) Structure.—Soil structure is a characteristic that is very useful in evaluating and correlating the hydraulic conductivities of soils with similar textures. Structure refers to the aggregation of primary soil particles into compound particles which are separated from adjoining aggregates by surfaces of weakness, see figure 2–3. The size, shape, and arrangement of the aggregates and the shape

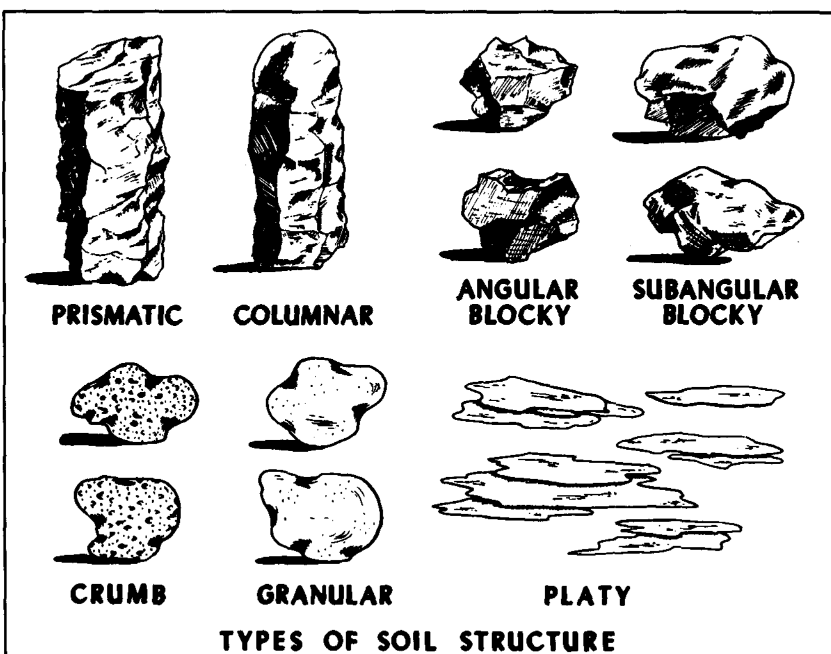


Figure 2–3.—Types of soil structure. 103-D-1619.

and size of the pore spaces give the soil its structure. The shape and arrangement of the aggregates are designated as the *type* of soil structure; size of the aggregates is termed *class* of soil structure; and the degree of distinctness (weak, moderate, or strong) is termed *grade* of soil structure. The principal types of soil structure with which the drainage engineer will be working and the classes and grades of each type are described below.

(1) *Platy.*—In this type of structure, the aggregates are arranged in horizontal sheets. The hydraulic conductivity rate varies with the class of structure and is usually at its highest for medium platy material. The classes of this type of structure are:

Structure class	*Plate thickness, millimeters*
Very thin platy	Less than 1.0
Thin platy	1.0 to 2.0
Medium platy	2.0 to 5.0
Thick platy	5.0 to 10.0
Very thick platy	Greater than 10

Platy material is usually very durable and considered to be of strong grade.

(2) *Prismatic or columnar.*—These structure types are usually found in the upper horizons of a soil profile. In these types, the aggregates form prisms that have longer vertical than horizontal axes. The prism shape can be approximately square, pentagonal, or hexagonal. The aggregates may break horizontally along secondary cleavage planes into blocky or very thick plates, but even these broken sections will have relatively well defined vertical faces. In prismatic structure, the aggregates form flat-topped prisms, while in columnar structure they form round-topped, biscuit-type prisms.

These types of structure are associated with solonetz soils. They appear to have a good angular to subangular blocky structure when dry, but swell together when wet, which results in a very low hydraulic conductivity in both the vertical and horizontal directions.

The classes of these structure types are:

Structure class	*Macroprism width, millimeters*
Very fine prismatic or columnar	Less than 10.0
Fine prismatic or columnar	10.0 to 20.0
Medium prismatic or columnar	20.0 to 50.0
Coarse prismatic or columnar	50.0 to 100.0
Very coarse prismatic or columnar	Greater than 100.0

Prismatic and columnar structures are considered to be strong grades of soil structure.

(3) *Angular blocky.*—When the term, blocky, is used alone as a type of structure, it means angular blocky if the aggregates are in dense blocks bounded by planes intersecting at relatively sharp angles. A soil with this structure usually has good hydraulic conductivity in both horizontal and vertical directions, and the rate is influenced by both the class and grade. For example, very coarse, angular blocky clay-loam soils with strong structural grade (which usually means very distinct cleavage planes between peds[1]) can have in-place hydraulic conductivities as high as 30 meters per day (about 50 inches per hour). At the other extreme, very fine, angular blocky clay-loam soils with a weak structural grade can have in-place hydraulic conductivities less than 0.3 meter per day (about 0.5 inch per hour). The classes are:

Structure class	*Block dimension on any side, millimeters*
Very fine, angular blocky	Less than 5.0
Fine, angular blocky	5.0 to 10.0
Medium, angular blocky	10.0 to 20.0
Coarse, angular blocky	20.0 to 50.O
Very coarse, angular blocky	Greater than 50.0

The grade is weak if the disturbed soil material breaks into a mixture of a few complete peds, many broken peds, and much unaggregated material. The grade is moderate if the disturbed soil material breaks down into many distinct complete peds, some broken peds, and little unaggregated material. The grade is strong if the disturbed soil material consists mostly of complete peds, few broken peds, and little or no unaggregated material.

(4) *Subangular blocky.*—In this type of structure, the aggregates are in dense blocks having mixed rounded and plane faces with vertices mostly rounded. As far as hydraulic conductivity is concerned, there appears to be little difference between the angular and subangular blocky structure. The classes are described as subangular blocky but have the same description and sizes as the blocky structure. The grades have the same designation as blocky structures.

(5) *Granular.*—The granular type of structure is formed of uniformly sized relatively nonporous aggregates, spheroidal or polyhedral in shape, and having plane or cured surfaces which have slight or no conformity with the faces of the surrounding aggregates. Soils with this type of structure usually have good hydraulic conductivities both vertically and horizontally. The hydraulic conductivity rate depends upon the class and grade; the medium granular class has the higher in-place hydraulic conductivity. The classes are:

---

[1] A *ped* can be defined as an individual natural soil aggregate, and should not be confused with a fragment, which is caused by rupture across natural surfaces of weakness.

Structure class	*Aggregate thickness on any side, millimeters*
Very fine granular	Less than 1.0
Fine granular	1.0 to 2.0
Medium granular	2.0 to 5.0
Coarse granular	5.0 to 10.0
Very coarse granular	Greater than 10.0

The grade can vary from weak to strong, but is usually more on the strong side with each ped appearing as a single-grained structure.

(6) *Crumb.*—This type of structure is the same as granular except aggregates appear very porous. It has good hydraulic conductivity rates in both vertical and horizontal directions, with the rates dependent on class and grade. Classes are the same as for granular except there are no coarse or very coarse crumb structures. A crumb-type structure can be of weak, medium, or strong grade.

(7) *Massive.*—Structure type is massive when the soil is coherent and there is no observable aggregation or definite orderly arrangement of natural lines of weakness. A soil with massive structure has neither class nor grade and negligible hydraulic conductivity.

(8) *Single grain.*—Single-grain structure is a noncoherent soil with no observable aggregation, such as sand. Usually, soil with single-grain structure has good vertical and horizontal hydraulic conductivity. A single-grain soil has neither structural class nor grade.

(9) *Structureless.*—This is not a recognized soil structure but in drainage engineering serves to identify in-place sandy materials. A very fine sandy loam identified as being structureless means there is no observable structure but it has none of the unsatisfactory drainage characteristics associated with massive structure. A structureless sandy soil can, and usually does, have good hydraulic conductivity rates.

(*e*) *Specific Yield.*—Specific yield may be defined as the volume of water released from a known volume of saturated soil under the force of gravity and the inherent soil tensions. It is expressed as a percentage of the total volume of saturated soil:

$$\text{Specific yield, } S = \frac{\text{volume of water drained}}{\text{total volume of saturated soil}} \times 100$$

The optimum percent of specific yield in the 1- to 3-meter (4- to 10-foot) zone should be about 6 to 10 percent. A soil in this percent range would have sufficient aeration, hydraulic conductivity, and water-holding properties for optimum crop growth. When the specific yield is less than 3 percent, drainage becomes difficult and expensive. For specific yields greater than 16 to 18 percent, aeration and hydraulic conductivity are good, but the soil moisture-holding capacity is low.

Specific yield values can be determined using undisturbed soil samples of known volume or by field tests. To obtain reliable data, undisturbed samples should be carefully packed in an airtight container as soon as they are taken to prevent them from drying out and cracking. They should also be suspended in a shockproof box when being transported from the sampling site to the laboratory to prevent them from cracking or being disturbed by vibration or sudden impact. Tension tables and pressure cookers capable of holding constant tensions from 0 to 160 centimeters of water are required in the laboratory. Tension tables are easier to use for soils containing little or no swelling clays. For soils that are high in swelling clays, the pressure cooker must be used to prevent excessive cracking.

In field tests, mercury manometers are required at each texture change from 0 to 3 meters (0 to 10 feet) to determine when the tension has stabilized so that final moisture samples can be taken to compare with the initial saturated moisture content. Results from years of field testing a variety of western soils indicate that inherent soil tensions tend to stabilize within the range of 30 to 150 centimeters of water in a free-draining soil. The stabilized tension will vary with texture, organic matter, and depth.

Both laboratory and field determinations of specific yield are expensive and time consuming. Also, a large number of tests must be conducted to obtain the average specific yield for the area to be drained. Conducting only one or two tests per area to be drained could result in erroneous data being used in determining the drainage requirements. Many field offices are not equipped to conduct these tests and, because all drainage requirements are based upon hydraulic conductivity, a correlation study was made between specific yield and undisturbed or in-place hydraulic conductivity.

As a result of this study, a curve showing specific yield versus hydraulic conductivity was prepared, figure 2–4. The curve is based on approximately 2,000 laboratory tests on undisturbed samples of all types of soils. Data used in the development of this curve also include approximately 100 in-place hydraulic conductivity tests versus laboratory specific yield data on undisturbed cores that were taken from the same test holes and zones as the laboratory tests. Both specific yield and hydraulic conductivity determinations were made on each undisturbed sample, and the results are within 10 percent of best obtainable values. A value for specific yield within 10 percent is considered well within the limits of accuracy for all the other factors which must be evaluated in drainage work. Therefore, when the hydraulic conductivity is known, the use of figure 2–4 to obtain values for specific yield is recommended.

The specific yield value used in drainage calculations should relate only to the volume of soil that is unwatered by the drain. The hydraulic conductivity value for entering the curve on figure 2–4 should be the average value for the saturated profile above the drains.

(*f*) *Capillary Fringe*.—The soil zone just above the water table is not at field capacity as assumed in the drain-spacing computations. This zone, sometimes defined as the capillary fringe, varies in thickness according to the soil texture

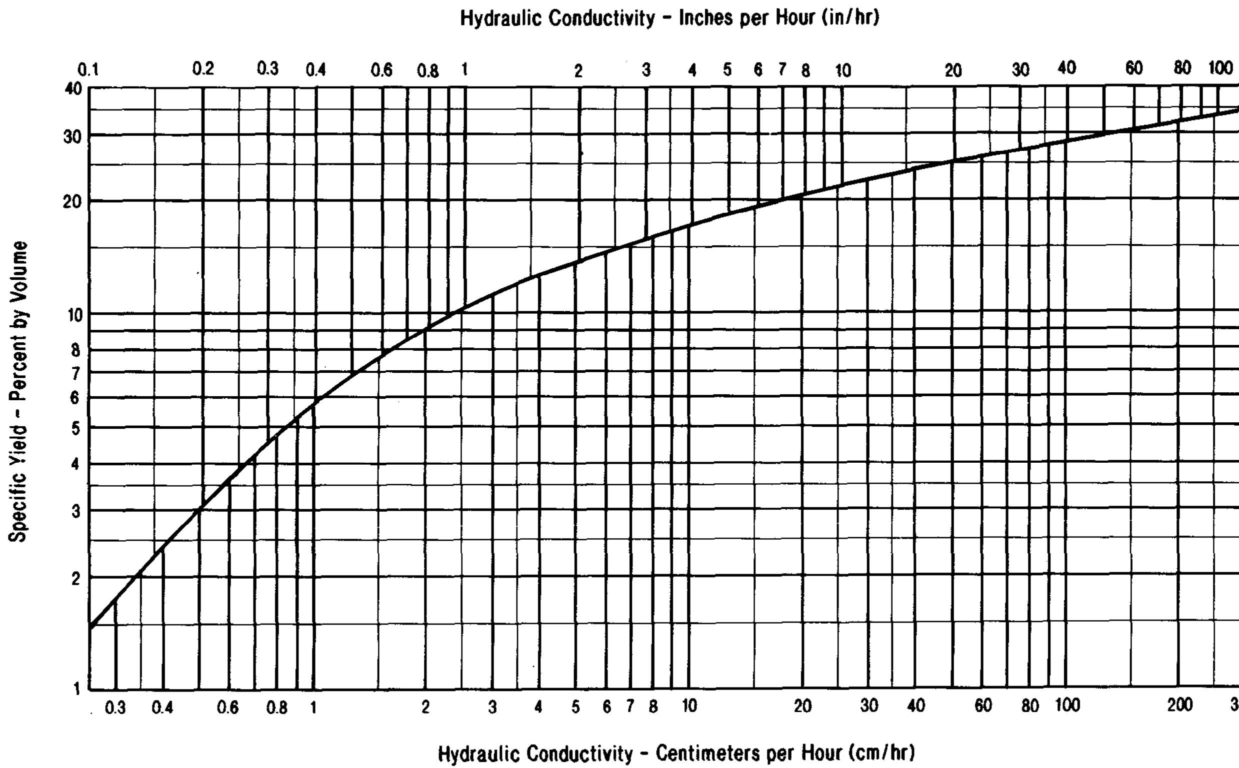


Figure 2–4.—Curve showing general relationship between specific yield and hydraulic conductivity. 103-D-693.

and varies in moisture content from nearly saturated to field capacity. The thickness of this zone is usually small and should not be confused with the total height to which capillary water will rise in a dry soil. From a practical standpoint, the capillary fringe can be ignored when determining the unsaturated root zone depth. With a well-designed subsurface drainage system, the capillary fringe will extend into and remain in the root zone only a short time toward the end of the irrigation season, and production should not be measurably affected.

The question may arise as to what effect the capillary fringe has on the buildup and drawdown of the water table as calculated in the drain-spacing computations. Field studies show that water tables fluctuate between drains as predicted by transient flow drain-spacing computations. The capillary fringe fluctuates with and parallel to the water table, except with a lag in time, and has no measurable effect on the discharge from the drain. Experiments using a small tank filled with sand have shown that the capillary fringe affects or influences the discharge when the depth of saturated flow is of the same order of magnitude as the thickness of the capillary fringe. However, field studies for shallow drains, spaced from 10 to 40 meters (30 to 120 feet) and placed on a barrier, indicate the capillary fringe contributes no measurable water to the discharge. These studies further indicate that when the water table midway between drains drops to approximately 0.15 meter (0.5 foot) above the pipe drain invert, the discharge drops to zero even though the capillary fringe can be at least 0.15 meter (0.5 foot) above the water table. Based on the above findings, the capillary fringe is not used in determining the drainage requirements or in the design of the system. Also, there is no easy, reliable method for measuring this parameter in the field.

2–5. Salinity and Alkalinity.—*(a) General.*—Many factors contribute to the development of saline soil conditions. However, most soils become saline through consumptive use of capillary ground water and irrigation water containing salts. Salt concentrations in soil vary widely both vertically and horizontally depending on such conditions as variations in texture, plant growth, and hydraulic conductivity. This variation shows up strikingly as patchy growths of vegetation in saline soils. The extent of salinization is governed by the rate of evapotranspiration of saline water and the counteraction of leaching water from precipitation and irrigation. Although salts affect plant growth in many ways, the three most important effects are:

(1) Salts cause a reduction in the rate and amount of water that can be withdrawn from the soil by plant roots because of increased osmotic pressure. Plant growth is retarded almost linearly with increases in osmotic pressure (Hayward and Wadleigh, 1949).

(2) Common salts such as sodium, bicarbonate, and chloride are toxic to some plants when present in higher than normal concentrations. The toxic effect is usually critical during the germination period in the 50- or 80-millimeter surface soil zone.

(3) Certain salts, sodium being the best known, when present in high concentrations, can affect the physical condition of the soil. Soils with excess sodium tend to puddle, have poor structure, and develop poor infiltration and hydraulic conductivity rates. Before these soils can be farmed successfully, the salt must be changed chemically by replacing the excessive sodium with calcium and installing a drainage system to facilitate leaching out the replaced sodium salts.

Soil structure depends on the attraction between clay particles in the soil. Calcium, magnesium, and aluminum cations are strongly attracted to clay particles. Soils containing these cations generally form stable soil structures. These cations must be present in waters used to reclaim soils containing sodium and potassium cations (alkaline soils).

Low salt concentrations dominated by sodium cations cause dispersion of clay particles in soils. If sodium is leached without replacing it with calcium, magnesium, or aluminum, the soil remains dispersed after leaching. This destroys soil structure and affects the hydraulic conductivity. In some cases, the clay particles will move downward and form impervious layers in the soil profile.

(*b*) *Leaching Requirement and Salt Balance.*—For soils in arid regions and when there is a presence of salt in the irrigation water, leaching is required to maintain a favorable salt balance in the root zone. This requires that an equal or greater amount of salt must be leached from the soil by the drainage water than is introduced into the soil by irrigation water. It further requires that the drainage system design consider the removal of the leaching water from the substrata. In most cases, the deep percolation inherent with standard irrigation practices will maintain a favorable salt balance and an acceptable concentration in the soil-water solution in the root zone. Water resource agency studies of recent local irrigation practices should be considered in determining expected deep percolation. Should investigations show that the leaching requirement is in excess of the leaching obtained with deep percolation associated with normal irrigation practices, the drainage system requirements and costs should be increased accordingly.

The continuing leaching requirement is not the same as the initial leaching requirement. The permanent deep drainage system for irrigated lands cannot be economically designed, from a drain-spacing standpoint, to take care of the initial leaching requirement. Usually, multilevel drains could be used with the shallower drains installed between the permanent deeper drains. The shallow drains are installed using minimum size pipe and at minimum cost because they will no longer function after the initial leaching has been accomplished. In practical application, the drains are usually designed to satisfy the long-term leaching requirement and the soils will reach acceptable salinity levels after only a few irrigation seasons.

The leaching requirement may be defined as the percentage of infiltrated irrigation water and precipitation that must pass through the root zone to control salts at a specified level. For planning purposes, the leaching requirement may be determined from the equation:

$$\mathrm{LR} = \frac{EC_{iw}}{EC_{dw}} \times 100 \qquad (1)$$

or

$$\mathrm{LR} = \frac{D_{dw}}{D_{iw}} \times 100 \qquad (2)$$

where:

LR = leaching requirement in percent,
EC_{iw} = electrical conductivity of irrigation water including effective precipitation in millimhos per centimeter (mmho/cm),
EC_{dw} = electrical conductivity of drainage water in mmho/cm,
D_{dw} = depth of drainage water in meters, and
D_{iw} = depth of irrigation water in meters including effective precipitation.

The value for EC_{dw} is determined from the relative salt tolerance of the least salt-tolerant crop to be grown in the area. Figure 2–5 shows the salt tolerance for field, vegetable, and forage crops. Except for some specialty crops, a 25-percent yield reduction for the least salt-tolerant principal crop can be used.

To illustrate the process for estimating the leaching requirement, assume that the principal crops for an area are alfalfa ($EC \times 10^3 = 5$, sugar beets ($EC \times 10^3 = 13$), and potatoes ($EC \times 10^3 = 4$). The values in parentheses indicate electrical conductivities in mmho/cm at 25 °C associated with 25-percent reductions in yields. The reader should note that soil water is diluted to near saturation extract concentration just before entering the drain. The salt content of the irrigation water may be expressed in milligrams per liter (mg/L), which can be converted with reasonable accuracy to mmho/cm by dividing the parts per million by 640.

Example calculation:

Given: Total salts in irrigation water = 1000 mg/L. Least salt-tolerant crop is potatoes, with an electrical conductivity of the saturated extract not to exceed 4 mmho/cm at 25 °C.

Then:

$$\mathrm{LR} = \frac{EC_{iw}}{EC_{dw}} \times 100 = \frac{(1{,}000\ /640)}{4} \times 100 = 39 \text{ percent}$$

Figure 2–6 can be used to quickly estimate the leaching requirement and minimum infiltration rate needed to obtain proper leaching under normal irrigation practices.

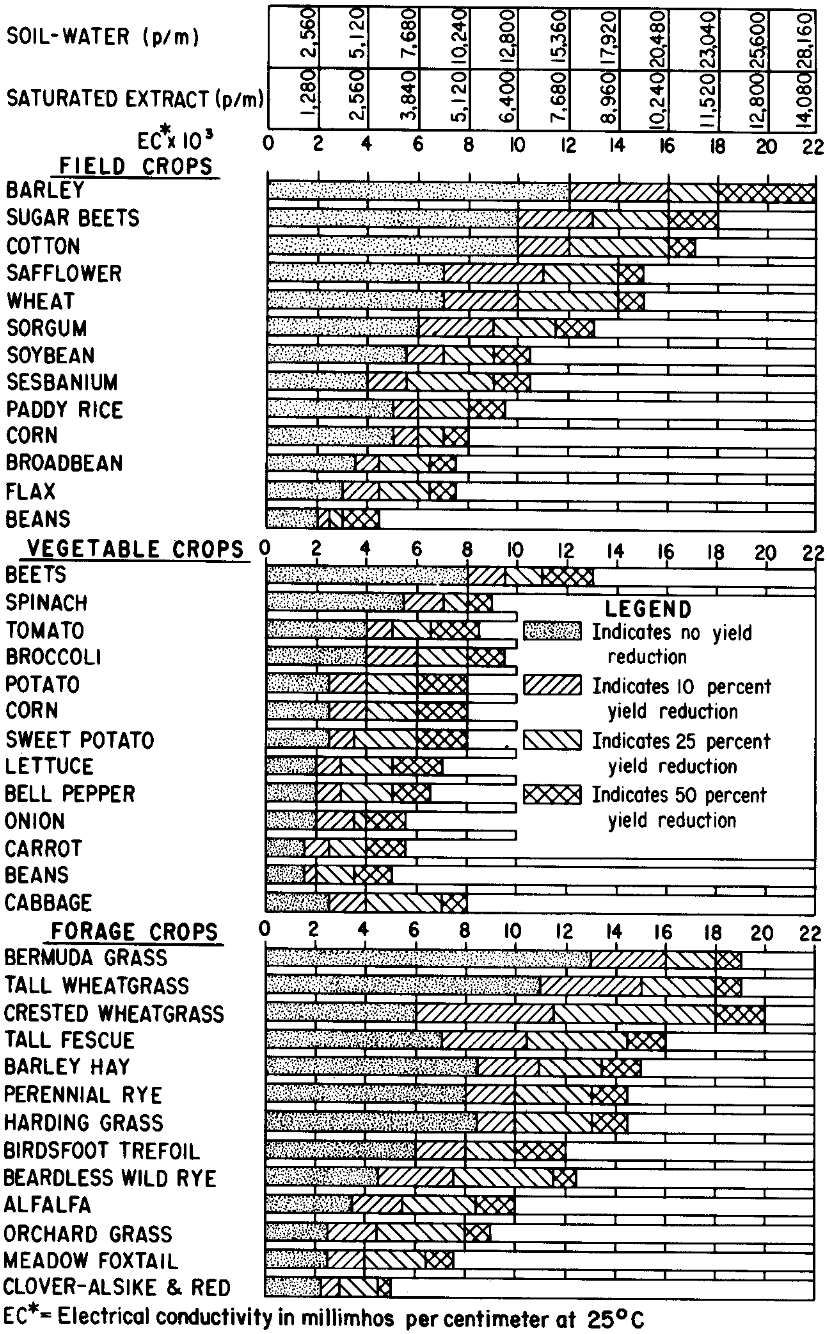


Figure 2–5.—Salt tolerance for field, vegetable, and forage crops. 103-D-1626.

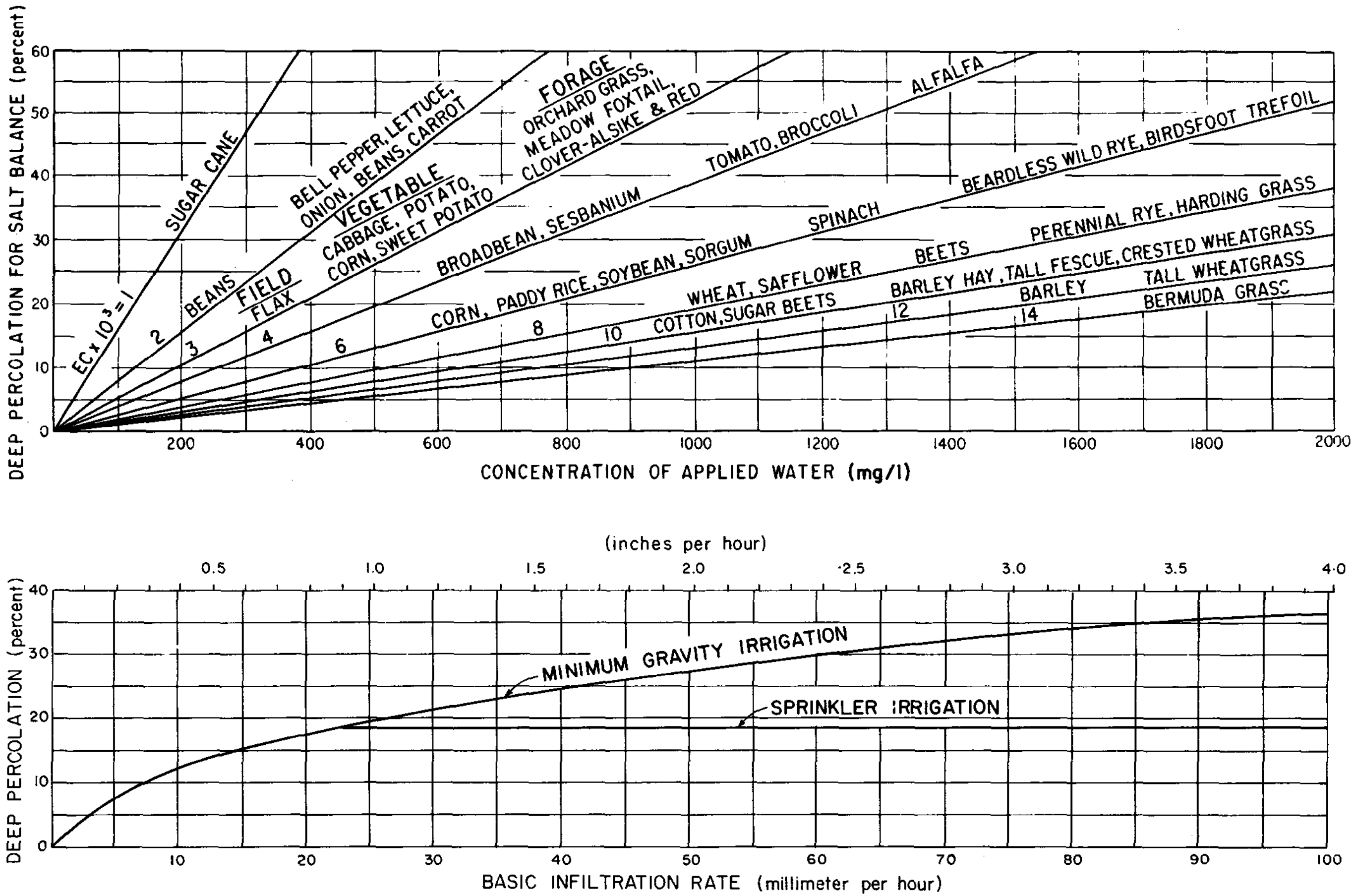


Figure 2–6.—Curves for estimating leaching requirement and minimum infiltration rate. 103-D-1620.

The total infiltration (INF) from an irrigation application is the sum of the total readily available moisture (TRAM) and the deep percolation (DP). TRAM is explained in greater detail in section 2–6(d).

$$\text{INV} = \text{TRAM} + \text{DP}$$

Since the deep percolation is the product of the leaching requirement (LR) and the infiltration, then:

$$\text{INF} = \text{TRAM} + \text{LR} \times \text{INF}$$

and

$$\text{INF} = \frac{\text{TRAM}}{1 - \text{LR}}\text{, LR expressed as a decimal fraction.}$$

In the previous example, if the TRAM in the root zone is 80 millimeters, the infiltration would be:

$$\text{INF} = \frac{80}{1 - 0.39} = 131 \text{ millimeters}$$

and the deep percolation for salt balance would be:

$$\text{DP} = \text{INF} - \text{TRAM} = 131 - 80 = 51 \text{ millimeters}$$

A number of refinements can be considered when calculating leaching requirements, but the majority of these can generally be left out without significantly affecting the results. The most significant exclusions from the preceding example are leaching efficiency of soil types and removal of salt in harvested plants. Sample calculations considering leaching efficiencies are not included here because of the lack of information available on this refinement. For more information on this subject, see Bouwer, 1969.

Significant salt reduction in the soil by removal of all mature crops and residue from the land is feasible only for crops with a large amount of foliage. Sugarcane is used in the following example to determine the volume of salt removed by this method.

Example calculation:

Sugarcane can tolerate the salinity associated with electrical conductivities of about 1 mmho/cm. Assuming an average conductivity of 0.24 mmho/cm for the irrigation and rainwater entering the soil, the leaching requirement is:

$$\text{LR} = \frac{0.24}{1.0} \times 100 = 24 \text{ percent using equation (1).}$$

For a consumptive use of 80 millimeters between irrigations, the total infiltration will be:

$$\mathrm{INF} = \frac{80}{1 - 0.24} = 105 \text{ millimeters (rounded).}$$

Therefore, deep percolation per irrigation = 105 - 80 = 25 millimeters.

The 24 percent leaching requirement is higher than necessary, however, because it does not account for salts removed with crop removal. To adjust the leaching requirement for these salts, the following factors must be known or assumed:

(1) Total yield of sugarcane (green weight) = 165 metric tons per hectare.

(2) Net yield of sugarcane (green weight) = 60 percent of the total yield = 99 metric tons per hectare.

(3) Waste (green weight) = 165 - 99 = 66 metric tons per hectare.

(4) Dry weight of cane is about 40 percent of the green weight; therefore, there are 40 metric tons per hectare of millable cane and 26 metric tons per hectare of waste.

(5) Mineral content (total salts).

Analyses of cane residue show:

Millable cane = 2.2 to 4 percent of dry weight.

Leaves and unusable stalk = 8.1 to 12.1 percent.

(6) Silicate (SiO_2) content of ash.

Millable cane = 40 percent of ash.

Leaves and unusable stalks = 58 percent of ash.

Using the above values:

Total mineral content of millable cane = (0.022)(40) = 0.880 metric ton per hectare.

Total mineral content less SiO_2, = (0.022)(1 - 0.40)(40) = 0.528 metric ton per hectare.

Total mineral content of waste = (0.081)(26) = 2.106 metric tons per hectare.

Total mineral content of waste less SiO_2, = (0.081)(1 - 0.58)(26) = 0.885 metric ton per hectare.

Total salt removed at harvest = 0.528 + 0.885 = 1.41 metric tons.

Cane is harvested three times every 4 years, so the annual salt removal is:

Salt removed = (3/4)(1.41) = 1.06 metric tons per hectare per year.

To adjust the leaching requirements, the following approach can be used:

Known or calculated:

EC_{dw} = 1 mmho/cm = 640 mg/L.

C_{dw} = $0.0006399\ \frac{\text{metric ton}}{\text{m}^3}\left(0.87\ \frac{\text{ton}}{\text{acre}}\right)$ = Amount of salt in drain water.

EC_{iw} = 0.39 mmho/cm = 250 mg/L.

C_{iw} = $0.0002501\ \frac{\text{metric ton}}{\text{m}^3}\left(0.34\ \frac{\text{ton}}{\text{acre}}\right)$ = Amount of salt in irrigation water.

EC_{rw} = 0.023 mmho/cm = 15 mg/L = Measure of salt concentration in rainwater.

C_{rw} = $0.0000147\ \frac{\text{metric ton}}{\text{m}^3}\left(0.02\ \frac{\text{ton}}{\text{acre}}\right)$ = Amount of salt in rainwater.

D_{cu} = $8839\ \frac{\text{m}^3}{\text{hectare}}$ (2.9 acre feet per acre) = Consumptive use of irrigation water.

D_{rw} = $6096\ \frac{\text{m}^3}{\text{hectare}}$ (2.0 acre feet per acre) = Depth of effective precipitation.

T_c = $1.0984115\ \frac{\text{metric tons}}{\text{hectare} \bullet \text{meter}}\left(0.49\ \frac{\text{ton}}{\text{acre} \bullet \text{ft}}\right)$

For salt balance:

$$\text{Salt out} = \text{Salt in}$$

$$C_{dw}D_{dw} + T_c = C_{iw}D_{iw} + C_{rw}D_{rw}$$

Since

$$D_{iw} = D_{cu} + D_{dw}$$

Then,

$$D_{dw} = \frac{C_{iw}D_{cu} + C_{rw}D_{rw} - T_c}{C_{dw} - C_{iw}}$$

$$D_{dw} = \frac{(0.0002501)(8839) + (0.0000147)(6096) - 1.0984115}{(0.0006399 - 0.0002501)}$$

$$= 3083.2\ \frac{\text{m}^3}{\text{hectare}} = 0.308\ \text{m (1.01 acre–ft per acre)}$$

Using equation (2):

$$\mathrm{LR} = \frac{D_{dw}}{D_{iw} + D_{rw}} \times 100$$

$$\mathrm{LR} = \frac{3083\ (100)}{(8839 + 3083) + 165} = 17.1 \text{ percent.}$$

The leaching requirement was reduced from 24 to 17 percent by taking into account the salts removed by crop removal.

Maintenance of a favorable salt balance is a continuous requirement for sustained agricultural production. However, some soils have such high concentrations of salts prior to irrigation that an initial leaching is required before agricultural production can begin. To be practical, the drainage facilities provided should not provide more capacity than the land will require for normal salt balance under irrigation after the initial leaching. This limitation means that during initial leaching, the water table will rise higher than the normal design level between drains.

High exchangeable sodium can cause soil particles to deflocculate. Normally, the hydraulic conductivity of soil materials decreases with an increase in exchangeable sodium and drainage requirements increase accordingly. There are exceptions to this general statement, but the drainage requirement should be based on the in-place hydraulic conductivity without regard to the chemical conditions in the soil that cause this hydraulic conductivity, providing the in-place testing procedures and computations are correct. The substrata hydraulic conductivity of adequately drained land is not expected to decrease but can improve if the irrigation water and soil in the root zone are satisfactory for irrigated agriculture.

(c) *Construction in Sodic Soils.*—Sodic soils are generally unstable and, therefore, difficult to work with using ordinary drain construction methods. Unstable material may prevent an open drain from being excavated to grade because the sides continually cave in. Staged construction may be used to overcome this condition even though considerable time may be required to bring the drain to grade. It is particularly difficult to maintain line and grade in sodic soils for pipe drains, and close inspection is required to assure an acceptable installation. One possible solution is to place stabilizing gravel in the trench until it will support the pipe. In some cases, a specialized trenching machine may be required. The above condition is not exclusively a sodium problem, since it sometimes occurs in a saturated fine sand or silt, but it is intensified if excessive sodium is present.

Another problem is that excavation of sodic soils usually causes them to puddle which further decreases the hydraulic conductivity. There are instances when the water stands over a pipe drain as a result of this condition. Every effort should be made to avoid this problem if possible or to reduce the effects of puddling if the problem is unavoidable. Again, the use of specialized trenching machines and placement of the gravel envelope in direct contact with excavated surfaces will minimize this problem.

For best drainage, sodic soils should be unwatered, usually by well points, and the drains installed in the dry state. However, many times the sodium condition occurs in localized areas rather than covering the entire field. In this event, it may be possible to locate the drain at the edge of the sodic area rather than crossing it. The location of the drain will depend on topography, the location of the sodic area within the field, hydraulic conductivity of soils adjacent to the area, protection required in the field, and other related factors. The drain may be located upslope to intercept ground water before it reaches the sodic area and deep enough to provide some drainage for the area itself. If it is necessary to cross a sodic area, the soil should be disturbed as little as possible, and the trench should be backfilled to normal ground surface with a permeable gravel to minimize the puddling effects.

(*d*) *Classification of Saline and Sodic Soils*.—The following tabulation gives the chemical limits generally acceptable for classification of saline and sodic soils. These limits are of interest to the drainage engineer since they may indicate potential construction problems. Problems in drainage associated with salinity and alkalinity usually differ widely with the type of clay mineral content. The actual excavation conditions must be correlated with chemical and physical properties of the soil to provide a basis for conclusions regarding proper approaches to drainage and drain construction.

Soil	$EC \times 10^3$	Exchangeable sodium percentage (ESP)	pH
Saline	>4	<15	<8.5
Saline-sodic	>4	>15	±8.5
Nonsaline-sodic	<4	>15	8.5 to 10

2–6. Surface Runoff.—Surface flow must be considered in drainage analysis because this water must be carried away from agricultural lands. Since all water moves toward the topographic low points, both surface and subsurface waters normally flow in the same disposal channel. Design considerations must include the total capacity of both sources.

Surface flow originates from precipitation and from irrigation waste, and estimates of these flows are usually available to the drainage engineer from project hydrologists or irrigation district records. When such estimates are not available, the following simplified methods can be used to obtain design estimates for these flows.

(*a*) *Precipitation*.—Precipitation records seldom have to be collected or compiled primarily for drainage investigations. Usually, they will be available from the project hydrologist or from local rain gauge stations, but if not, precipitation data can be obtained from records of the National Weather Service.

(b) Stormflow.—Stormflow depends on topography, soils, vegetative cover, land use, and the climatic characteristics of the area. Surface drains should be designed to handle flows from 5- to 15-year storm frequencies. Where relatively expensive structures are involved or where damage to the structures may dictate the need for a more conservative design, the 25-year storm frequency should be used. As the consequences of inadequate channel capacity usually are not too severe, refinement of capacity estimates is not warranted.

Many formulas and analytical methods are available for estimating storm runoff. The most practical way of estimating surface drainage requirements for storm runoff is by studying existing channels and culverts. Flood capacity or degree of protection used for farm and county roads and irrigation laterals is about the same as for surface drains. If existing facilities are not adequate for a 5-year storm, they will show signs of flooding.

While there are too few existing culverts or drainage channels to permit comparison, some type of analytical method must be used. The McMath formula (Urquhart, 1959) gives results which are considered fairly reliable for planning purposes:

$$\text{McMath formula: } Q = CiS^{1/5}A^{4/5} \tag{3}$$

where:

- Q = flood discharge in cubic feet per second,
- C = coefficient representing the basin characteristics,
- i = rate of rainfall in inches per hour for the time of concentration and frequency,
- S = slope of main channel in units per 1,000 units between the farthest contributing point and the point of concentration, and
- A = area of basin in acres.

Values of C will range from 0.20 for low runoff conditions to 0.75 for high runoff conditions, depending principally on vegetation, soils, and topography. The C value increases as the vegetative cover becomes less dense, as the soil becomes heavier, and as the slope of the ground increases. Of these three basic factors, vegetation and soil have the greater effect on C. A single characteristic, such as a rock surface, may determine the value of C. Usually, no one characteristic will predominate, and all three factors must be considered before selecting a value for C. Arbitrarily weighing their relative importance, with vegetation at 40 percent, soils 40 percent, and topography at 20 percent, will allow selection of appropriate factors for each, which can then be added together to obtain a value for C. Table 2–3 shows drainage basin factors for determining C.

Table 2–3.—*Weighted drainage basin factors for determining C.*

Runoff conditions	Vegetation	Soils	Topography
Low	0.08 (well grassed)	0.08 (sandy)	0.04 (flat)
Moderate	.12 (good coverage)	.12 (light)	.06 (gently sloping)
Average	.16 (good to fair)	.16 (medium)	.08 (sloping to hilly)
High	.22 (fair to sparse)	.22 (heavy)	.11 (hilly to steep)
Extreme	.30 (sparse to bare)	.30 (heavy to rock)	.15 (steep)

Example: For a flat area with heavy soils and good vegetative cover, $C = 0.04 + 0.22 + 0.12 = 0.38$.

The intensity and duration of storm rainfall vary widely in the Western United States. Significant quantities of data are available and elaborate methods have been developed for very refined runoff studies. However, estimating storm runoff for a farm surface drainage study does not require such refined procedures. The National Weather Service has prepared rainfall intensity-frequency data which can be used to advantage (U.S. Dept. of Commerce, 1961). Figure 2–7, which was taken from this reference, shows a 5-year, 1-hour rainfall intensity map. Variations due to topographic influences in mountainous regions are reflected only in a general sense on this map. For a more detailed consideration of topography in the 17 Western States, see reference (U.S. Dept. of Commerce, 1973).

For small areas, where the storm is assumed to cover the whole contributing area, maximum runoff occurs when flow from the farthest part of the area reaches the lower end. This is called the time of concentration for the particular area, and the rainfall intensity corresponding to this period of time is used for runoff estimates. The time of concentration for a particular area depends principally on the length and slope of its main channel. Time of flood concentration can be estimated with sufficient accuracy using the nomograph shown on figure 2–8.

The procedure for estimating flood runoff from a small area is as follows:

(1) Find the value of C for physical conditions of the area from table 2–3.

(2) Estimate the time of concentration from figure 2–8.

(3) Select a value for 5-year, 1-hour rainfall from figure 2–8 for the area under study.

(4) Convert 5-year, 1-hour rainfall value to 5-year, any-hour depth by one of the following equations:

For time of concentration of 1 hour or greater,

$$y = b + \frac{X}{10}$$

For time of concentration less than 1 hour.

$$y = 0.80\, b$$

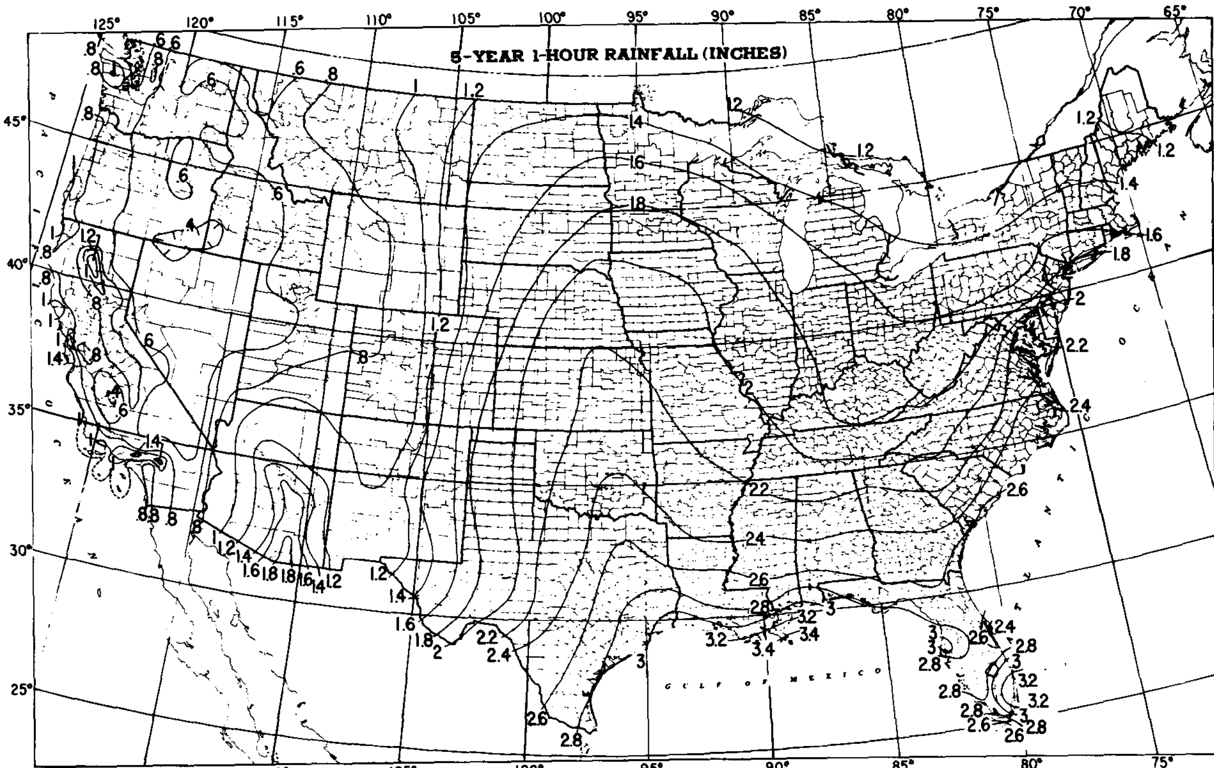


Figure 2–7.—Rainfall intensity map. 103-D-1621.

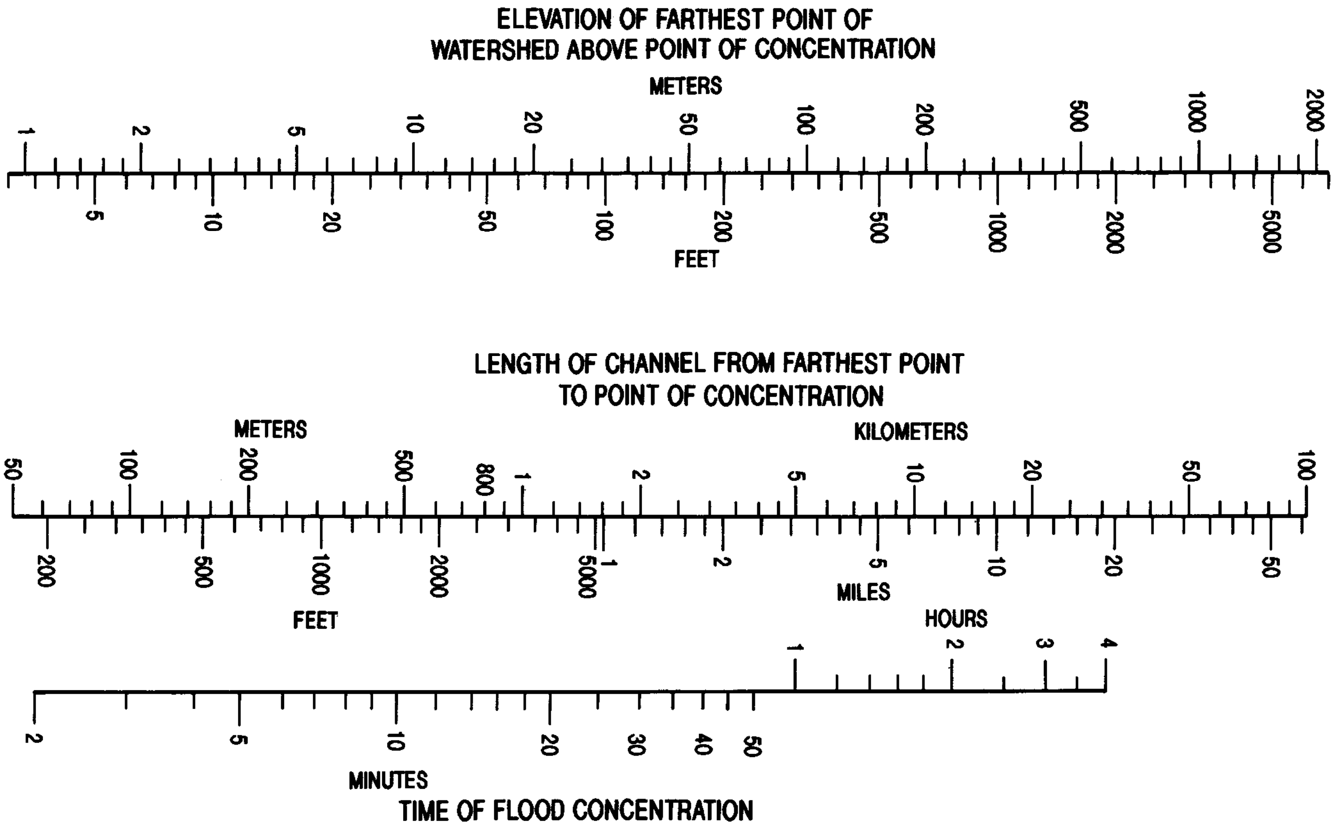


Figure 2–8.—Nomograph for estimating time of flood concentration. 103-D-692.

where:

y = 5-year, any-hour rainfall depth in millimeters,
b = 5-year, 1-hour rainfall depth in millimeters, and
X = required rainfall duration (time of concentration) in hours. X must be greater than 1 hour.

(5) Convert the y value found in (4) above to the required frequency:

Frequency, years	Factor by which to multiply y
10	1.2
15	1.3
25	1.4

(6) The rate of rainfall, i, is: $i = \frac{y}{X}$

(7) Solve for the estimated flood runoff, Q, using equation (3).

Figure 2–9 gives the one-fifth and four-fifths powers of numbers needed in this equation.

The McMath method discussed in the foregoing paragraphs gives satisfactory results when estimating storm runoff in the planning stages of a drainage project.

(c) Estimating Total Runoff from Soil and Cover Conditions.—The following method has been adapted from procedures developed by the Soil Conservation Service (SCS) and is adequate for reconnaissance and feasibility studies. For design, the more detailed procedures in the SCS National Engineering Handbook, Section 4, 1972, should be referred to. Their procedures are based on observations of runoff from watersheds up to approximately 800 hectares (2,000 acres) in size.

This manual presents a highly simplified approach for estimating runoff and should be used with judgment. The primary need for field data in this method is to obtain a measure of infiltration rates. Basic infiltration rates largely determine the runoff from a storm and the curve numbers on figure 2–10. Infiltration rates and curve numbers are affected by conditions on the watershed—primarily by land use and moisture content in the first foot of soil (antecedent moisture) at the time of a storm. Figure 2–10 accounts for these important factors.

Figure 2–10 can be used knowing only the soil textures in the top foot of soil or the SCS hydrologic soil group. However, the engineer must exercise careful judgment to estimate hydrologic conditions on the watershed and enter the figure accordingly. After the curve number has been determined, figure 2–11 can be used to find the direct runoff.

The method of using figures 2–10 and 2–11 is best explained by the following example:

Known:

(1) Area of watershed is 400 hectares (approximately 1,000 acres).

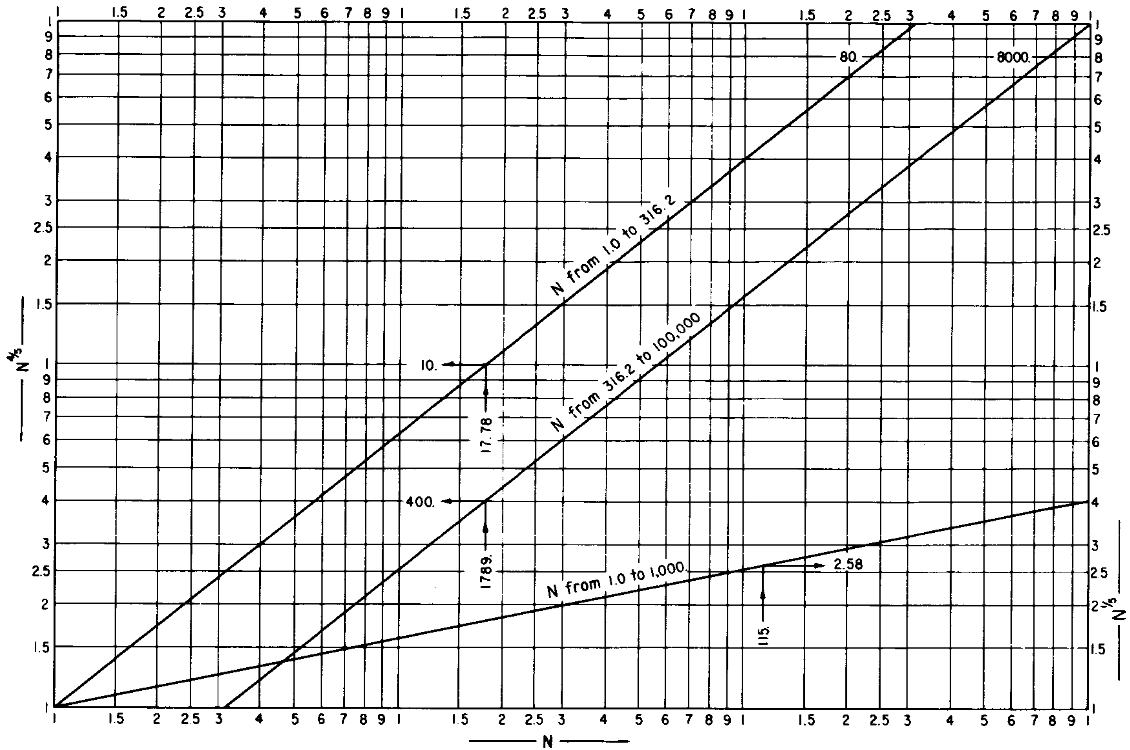


Figure 2–9.—Chart for determning the one-fifth and four-fifths powers of numbers. 103-D-1622.

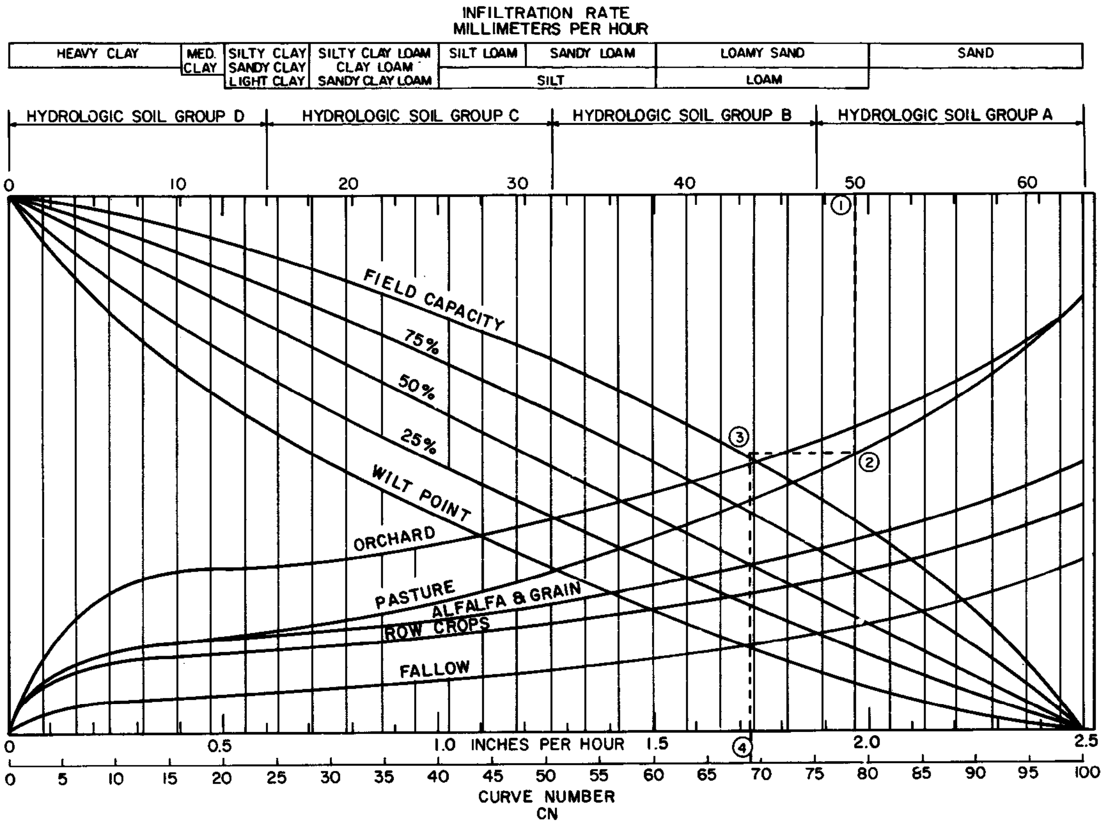


Figure 2-10.—Curve numbers for determining surface runoff. 103-D-1623.

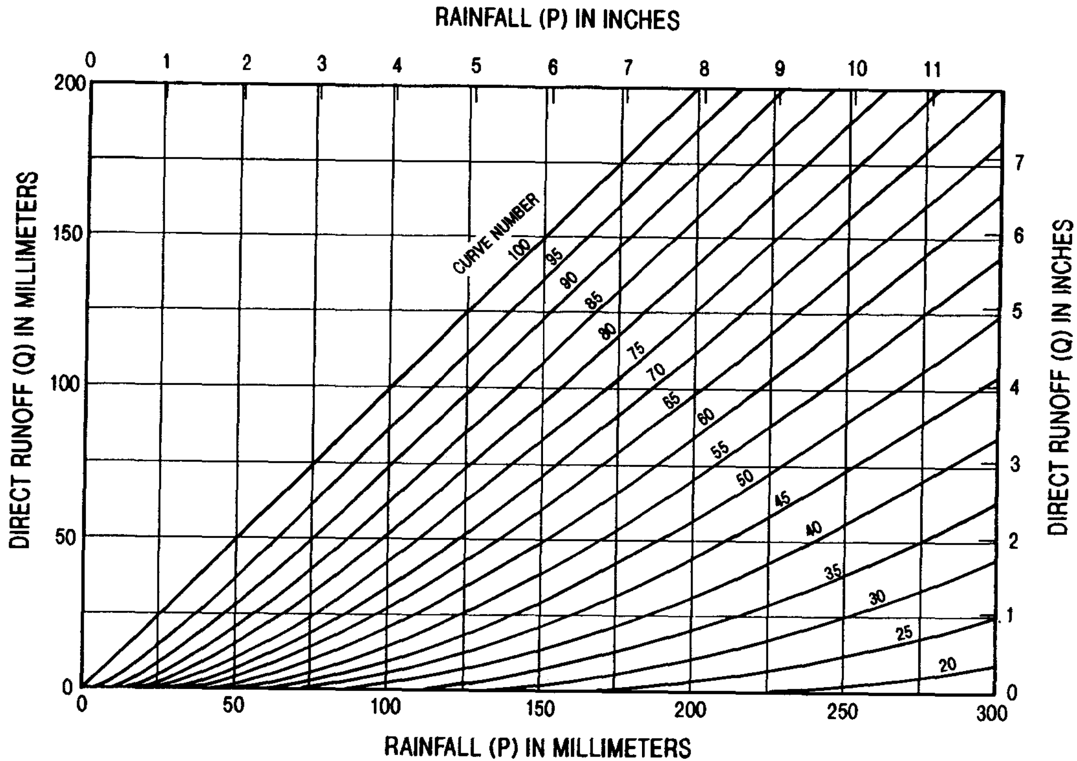


Figure 2–11.—Direct runoff based on curve number and rainfall. 103-DS-1624.

(2) Soil in top foot of the profile is a coarse loamy sand with 50 millimeters (2 inches) per hour infiltration rate.

(3) The watershed is used for pasture.

(4) At the time of a 75-millimeter (3-inch) storm, the soil is at field capacity.

Procedure:

Enter figure 2–10 at the given infiltration rate of 50 millimeters (2 inches) per hour (point 1) and read down the chart to the curve for land use of pasture (point 2). Read across the chart to the curve for soil at field capacity (point 3). Then read down to the bottom edge of the chart to obtain the curve number (*CN*) which is 70 (point 4).

Using a *CN* of 70 and the measured precipitation of 75 millimeters (3 inches), the direct runoff from the storm can be read from figure 2–11. In this example, the runoff is 18 millimeters (0.71 inch) per hectare. For the 400-hectare (1,000-acre) watershed, total runoff would be 72 000 cubic meters (about 54 acre-feet).

This method can be applied to large basins with varying soils, crops, and antecedent moisture conditions. The distribution of the various conditions must be known to estimate the weighted average and total runoff from a basin.

Moisture in the top foot of a soil profile can be estimated adequately by irrigation scheduling techniques explained in subsection 2–6(d).

Figure 2–12 can be used to determine the amount of rainfall that infiltrates the ground surface from a storm. The curve number needed for using this figure is determined as in the previous example for direct runoff.

(*d*) *Estimating Irrigation and Deep Percolation Schedules.*—To adequately analyze a drainage problem in an irrigated area, the engineer must have a working knowledge of plant, soil, and moisture relationships. The ability to estimate the timing of irrigations and estimate root zone moisture levels over a period of time is essential. The methods discussed in this section have been successfully used in Bureau of Reclamation work since the 1950's.

Moisture-holding capacity is the physical property of the soil that determines the maximum amount of water held in the root zone under free-drainage conditions. However, only a portion of this capacity can be used by plants, and this portion is called the available moisture (AM). This available moisture is the amount of water held in the soil between field capacity and the wilting point and is usually expressed in millimeters per meter (inches per foot) of soil.

The total available moisture (TAM) in a root zone is not readily available to plants because of root distribution and the pattern of water use from the root zone. The water that is readily available in a given root zone is called total readily available moisture (TRAM). This is the amount of water available for rapid plant growth. It is a physical characteristic of a given soil profile limited in depth to a specific crop root zone and moisture extraction pattern. With good irrigation

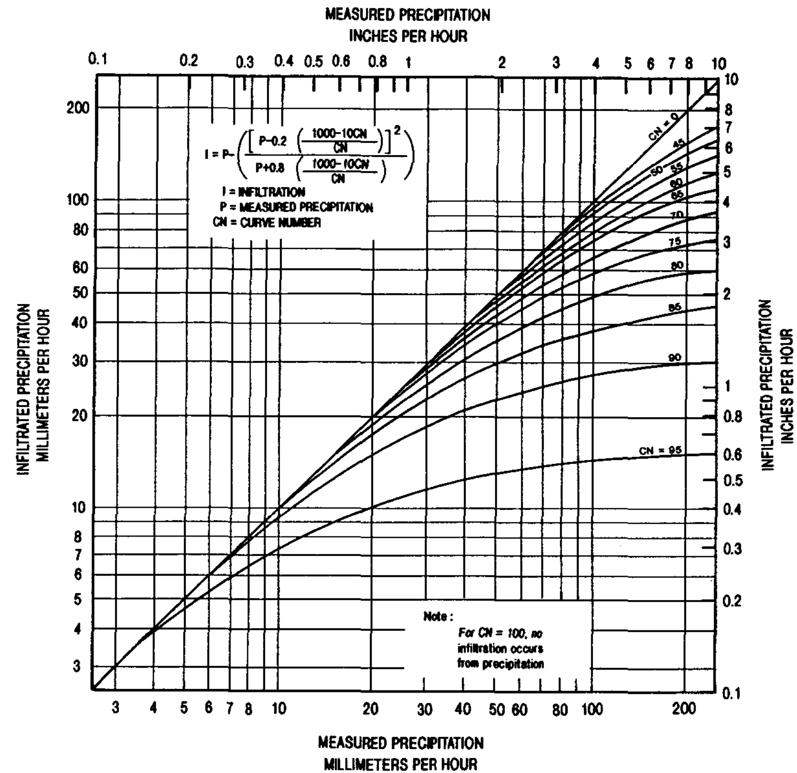


Figure 2–12.—Curves for estimating infiltration based on precipitation. 103-D-1625.

practice and normal root development, the moisture extraction pattern will be about 40 percent for the first quarter of the crop root zone, 30 percent for the second quarter, 20 percent for the third quarter, and 10 percent for the fourth quarter. A water table near the bottom of the normal root zone may alter the moisture extraction pattern which, in turn, may alter the deep percolation and drainage requirements. Unless additional information is available on root growth and moisture extraction near a water table, the above extraction pattern can be used.

The crop root zone varies with different crops, ranging from 0.6 meter (2 feet) for the shallow-rooted crops such as potatoes and vegetables, to 1.8 meters (6 feet) for peach, walnut, and avocado trees. For most irrigated crops, a 0.9 or 1.2-meter (3- or 4-foot) root zone can be used for computing the TRAM.

When the available moisture in the critical quarter is completely exhausted, the plant will be unable to extract sufficient moisture from the remaining quarters to maintain rapid crop growth. For most irrigated crops, the critical quarter should not be permitted to use more than about 75 percent of the available moisture between irrigations. Some potato growers recommend this percentage be held to 50 percent or less.

The first quarter will be the critical one for most soil profiles because of its high (40 percent) extraction rate. However, the critical quarter may change where finer textured soils are underlain by loamy sands or sands in the second or third quarter. The following examples show the procedure for determining TRAM in two different soil profiles of known texture and available moisture:

Example 1:

Soil profile			
Quarter	Texture	AM, millimeters (inches)	TRAM, millimeters (inches)
First	CL	63.50 (2.5)	$(63.50 \times 0.75)/0.40 = 119.06$ (4.69)
Second	CL	50.80 (2.0)	$(50.80 \times 0.75)/0.30 = 127.00$ (5.00)
Third	SiL	55.88 (2.2)	$(55.88 \times 0.75)/0.20 = 209.55$ (8.15)
Fourth	S	25.40 (1.0)	$(25.40 \times 0.75)/0.10 = 190.50$ (7.50)

The first quarter has the lowest TRAM so it is the critical quarter. When the daily consumptive use is a maximum of 6.35 millimeters (0.25 inch) per day, an irrigation would be required about every 18 days for continued rapid plant growth. Using 18 days, the moisture used would be $18 \times 6.35 = 114.30$ millimeters instead of 119.06 millimeters, and the irrigation schedule should be developed using the 114.30 millimeters.

Example 2:

Soil profile			TRAM,
Quarter	Texture	AM, millimeters (inches)	millimeters (inches)
First	CL	63.50 (2.5)	$(63.50 \times 0.75)/0.40 = 119.06$ (4.69)
Second	CL	50.80 (2.0)	$(50.80 \times 0.75)/0.30 = 127.00$ (5.00)
Third	S	25.40 (1.0)	$(25.40 \times 0.75)/0.20 = 95.25$ (3.75)
Fourth	SiL	55.88 (2.2)	$(55.88 \times 0.75)/0.10 = 419.10$ (16.50)

In this example, the third quarter is the critical one because it has a TRAM of only 95.25 millimeters (3.75 inches). When the daily consumptive use is a maximum of 6.35 millimeters (0.25 inch) per day, an irrigation will be required every 15 days for rapid plant growth.

Local farm organizations sometimes recommend that the total available moisture (TAM) be depleted by only a certain percent between irrigations. If so, the 75-percent factor in the previous examples should be adjusted. The TAM is the sum of the AM values for each quarter of the root zone expressed in millimeters or inches.

For example, an association of local potato growers might recommend that the root zone should not be depleted of more than 35 to 40 percent of the TAM between irrigations. In example 1, there would be 195.58 millimeters (7.7 inches) of TAM in the root zone. If 40 percent of this amount were used between irrigations, the TRAM would be $195.58 \times 0.40 = 78.23$ millimeters (3.08 inches), and an irrigation would be required every 12 days. Assuming the normal moisture extraction pattern, the first quarter would supply $78.74 \times 0.40 = 31.50$ millimeters (1.2 inches), the second quarter $78.74 \times 0.30 = 23.62$ millimeters (0.9 inch), the third quarter $78.74 \times 0.20 = 15.75$ millimeters (0.6 inch), and the fourth quarter $78.74 \times 0.10 = 7.87$ (0.3 inch).

If the recommendation had been that the available moisture in the critical quarter should not be depleted more than about 50 percent, the result would have been about the same as in the above recommendation. In example 1, the first quarter was the critical quarter, so:

$$(63.50 \times 0.50)/0.40 = 79.38 \text{ millimeters (3.12 inches) TRAM}$$

This is approximately the same as the 78.23 millimeters (3.08 inches) computed using TAM, so the depletion limits could have been recommended either way.

Available moisture estimates may be available from previous soil classification studies made in the area. Also, agricultural bulletins published by Federal or State agencies or local colleges and universities often have this information.

Available moisture may be measured by the methods described in Reclamation Instructions Series 510, Land Classification Techniques and Standards.

Annual irrigation schedules for any area will vary from year to year because of variations in crops, acreages, rainfall, solar radiation, and time of planting. Once the total readily available moisture, root zone depth, and crops have been selected for study, the scheduling process is a simple bookkeeping exercise. Normally, the schedule can be based on the TRAM of the entire root zone; however, there are occasions when the moisture content in each quarter of the root zone will be of interest to the engineer. For these occasions, the same techniques that follow can be used, but the procedure must be applied to each quarter of the root zone.

Usually the effects of rainfall can be ignored when annual precipitation is less than 254 millimeters (10 inches). In areas with significant rainfall, the amount that infiltrates the soil surface can be estimated from figure 5–7 in chapter V or using the techniques outlined in section 2–6(c).

The consumptive use of water by plants can be estimated many different ways. In some areas, measured data are available through colleges, extension agents, or Government agencies. In drainage design, the Blaney-Criddle method provides reasonable estimates of irrigation timing (Blaney and Criddle, 1962). Monthly consumptive use values should be determined and daily use values estimated by simply dividing the monthly use by the number of growing days in the month. A more refined estimate using the Blaney-Criddle method is to estimate the consumptive use for various crop growth stages from planting time through harvest (U.S. Dept. of Agriculture, 1967).

For the calculations that follow, assume that the crop of interest is alfalfa and that the growing season begins on May 14 and ends on September 21. Also, assume the area has negligible rainfall. The monthly and daily consumptive uses are:

Sample consumptive use values for alfalfa, in millimeters (inches)

	May	*June*	*July*	*August*	*September*	*Total*
Monthly	61.21	138.93	157.48	139.95	72.64	570.21
	(2.41)	(5.47)	(6.20)	(5.51)	(2.86)	(22.45)
Daily	3.81	4.57	5.08	4.57	3.56	
	(0.15)	(0.18)	(0.20)	(0.18)	(0.14)	

From the previous example 1 for estimating the TRAM, the moisture used between irrigations was 119.13 millimeters (4.69 inches). The total amount of water that infiltrates the soil surface upon each irrigation will be equal to the TRAM plus any water that deep percolates because of inefficiencies and leaching requirements (see secs. 2–5, 4–16, 4–17, and fig. 2–6 in sec. 2–5). The drains must be designed for the greater of the two estimates for deep percolating water: (1) leaching requirement, or (2) normal deep percolation from irrigation

Table 2–4a.—*Irrigation and deep percolation schedule for alfalfa (metric units).*

Date	Time period, days	Daily consumptive use, millimeters	Consumptive use for period, millimeters	Remaining TRAM, millimeters	Infiltraction, millimeters	Total moisture, millimeters	Ending TRAM, millimeters	Deep percolation, millimeters
5-14	–	3.81	–	0	Snowmelt[1] (157.23)	157.23	119.13	[2]38.10
5-31	17	3.81	64.77	54.36	0	54.36	54.36	–
6-11	[3]11	4.57	50.29	4.06	157.48	161.54	119.13	42.42
6-30	19	4.57	86.87	32.26	0	32.26	32.26	–
7-6	6	5.08	30.48	1.78	157.48	159.26	119.13	40.13
7-29	23	5.08	116.84	2.29	157.48	159.76	119.13	40.64
7-31	2	5.08	10.16	108.97	0	108.97	108.97	–
8-23	23	4.57	105.16	3.81	157.48	161.29	119.13	42.16
8-31	8	4.57	36.58	82.55	0	82.55	82.55	–
9-21	21	3.56	74.68	7.87	0	7.87	7.87	–
					787.15			203.45

[1] Assumed 196.34 millimeters of snowmelt of which 20 percent runs off.
[2] Assumed.
[3] Rounded down to a whole day.

Table 2–4b.—*Irrigation and deep percolation schedule for alfalfa (U.S. customary units).*

Date	Time period, days	Daily consumptive use, inches	Consumptive use for period, inches	Remaining TRAM, inches	Infiltration, inches	Total moisture, inches	Ending TRAM, inches	Deep percolation, inches
5-14	–	0.15	–	0	Snowmelt[1] (6.19)	6.19	4.69	1.50
5-31	17	.15	2.55	2.14	0	2.14	2.14	–
6-11	[3]11	.18	1.98	0.16	6.20	6.36	4.69	1.67
6-30	19	.18	3.42	1.27	0	1.27	1.27	–
7-6	6	.20	1.20	0.07	6.20	6.27	4.69	1.58
7-29	23	.20	4.60	0.09	6.20	6.29	4.69	1.60
7-31	2	.20	0.40	4.29	0	4.29	4.29	–
8-23	23	.18	4.14	0.15	6.20	6.35	4.69	1.66
8-31	8	.18	1.44	3.25	0	3.25	3.25	–
9-21	21	.14	2.94	0.31	0	0.31	0.31	–
					30.99			8.0

[1] Assumed 7.73 inches of snowmelt of which 20 percent runs off.
[2] Assumed.
[3] Rounded down to a whole day.

inefficiency. In this example, assume that the overall farm efficiency is 60 percent and about 20 percent of the delivery runs off as surface waste. Then:

Farm delivery	= 119.13/0.60	= 198.55 millimeters (7.8 inches) per irrigation
Runoff	= 0.20 (198.55)	= 39.71 millimeters (1.6 inches)
Infiltration	= 198.55 - 39.71	= 158.84 millimeters (6.2 inches)
Deep percolation	= 158.84 - 119.13	= 39.71 millimeters (1.5 inches) per irrigation

The process for calculating the irrigation schedule is shown in table 2–4. Table 2–4 shows a convenient form for keeping records of soil moisture and deep percolation. In calculating the schedule, fractions of a day are truncated when determining days of moisture left in the soil.

In areas where rainfall must be considered, the infiltrated rainfall is simply added to the bookkeeping as shown in the following example:

Assume a typical rainfall pattern in the area as follows and that the infiltrated rainfall has been estimated using figure 5–7. Procedures outlined in section 2–6(c) could also be used to estimate infiltrated rainfall.

Measured and infiltrated rainfall pattern for sample problem

Date	*Measured* millimeters (inches)	*Infiltrated* millimeters (inches)
5-20	13.46 (0.53)	12.70 (0.50)
5-30	11.68 (0.46)	10.92 (0.43)
6-12	6.35 (0.25)	5.08 (0.20)
6-22	29.46 (1.16)	25.40 (1.00)

Table 2–5 shows how this rainfall pattern would affect the results shown in table 2–4.

Section 5–5 of this manual shows an example of how ground-water buildup is determined from deep percolation and how an irrigation schedule is used in transient state drainage analysis.

(*e*) *Farm Waste*.—Farm-surface waste from irrigation varies with many factors, including soil texture, type of irrigation system, land slope, length of irrigation run, and irrigation efficiency. With good management, it is possible to irrigate without any wastewater leaving the irrigated area, but irrigation without surface waste is the exception rather than the rule. A deep sandy soil with flat slopes and short runs is the most easily managed condition for having negligible wastewater, whereas a fine-textured soil on steep slopes with long runs is very difficult to manage without having waste. In practice, a drainage system must be designed with an allowance for farm waste unless prior irrigation operations in the area have clearly shown this allowance to be unnecessary.

Table 2–5a.—*Irrigation and deep percolation schedule for alfalfa including rainfall (metric units).*

Date	Time period, days	Daily consumptive use, millimeters	Consumptive use for period, millimeters	Remaining TRAM, millimeters	Infiltraction, millimeters	Total moisture, millimeters	Ending TRAM, millimeters	Deep percolation, millimeters
5-14	–	3.81	–	0	Snowmelt (157.23)	157.23	119.13	38.10
5-20	6	3.81	22.86	96.27	12.70	108.97	108.97	–
5-30	10	3.81	38.10	70.87	10.92	81.79	81.79	–
5-31	1	3.81	3.81	77.98	0	77.98	77.98	–
6-12	12	4.57	54.86	23.11	5.08	28.19	28.16	–
6-18	6	4.57	27.43	0.76	157.48	158.24	119.13	39.12
6-22	4	4.57	18.29	108.84	25.40	126.23	119.13	7.11
6-30	8	4.57	36.58	82.55	0	82.55	82.55	–
7-16	16	5.08	81.28	1.27	157.48	158.75	119.13	39.62
7-31	15	5.08	76.20	42.93	0	42.93	42.93	–
8-9	9	4.57	41.15	1.78	157.48	159.26	119.13	40.13
8-31	22	4.57	100.58	18.54	0	18.54	18.54	–
9-5	5	3.56	17.78	0.76	157.48	158.24	119.13	39.12
9-21	16	3.56	56.90	62.23	0	62.23	62.23	–
					841.25			203.70

Table 2–5b.—*Irrigation and deep percolation schedule for alfalfa including rainfall (U.S. customary units).*

Date	Time period, days	Daily consumptive use, inches	Consumptive use for period, inches	Remaining TRAM, inches	Infiltraction, inches	Total moisture, inches	Ending TRAM, inches	Deep percolation, inches
5-14	–	0.15	–	0	Snowmelt (6.19)	6.19	4.69	1.50
5-20	6	.15	0.90	3.79	0.50	4.29	4.29	–
5-30	10	.15	1.50	2.79	0.43	3.22	3.22	–
5-31	1	.15	0.15	3.07	0	3.07	3.07	–
6-12	12	.18	2.16	0.91	0.20	1.11	1.11	–
6-18	6	.18	1.08	0.03	6.20	6.23	4.69	1.54
6-22	4	.18	0.72	3.97	1.00	4.97	4.69	0.28
6-30	8	.18	1.44	3.25	0	3.25	3.25	–
7-16	16	.20	3.20	0.05	6.20	6.25	4.69	1.56
7-31	15	.20	3.00	1.69	0	1.69	1.69	–
8-9	9	.18	1.62	0.07	6.20	6.27	4.69	1.58
8-31	22	.18	3.96	0.73	0	0.73	0.73	–
9-5	5	.14	0.70	0.03	6.20	6.23	4.69	1.54
9-21	16	.14	2.24	2.45	0	2.45	2.45	–
					31.12			8.0

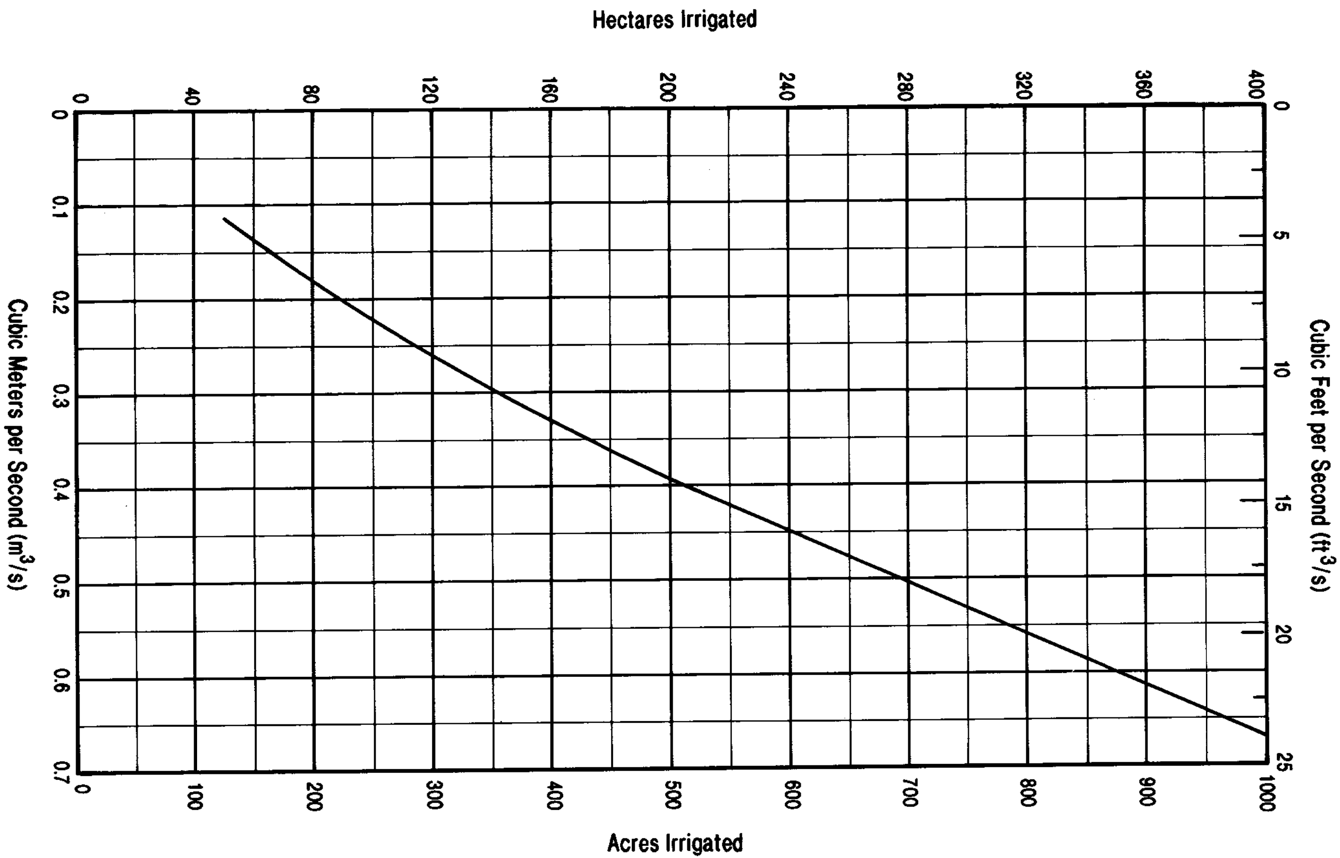


Figure 2–13.—Typical canal and lateral capacity curve for units less than 400 hectares (1,000 acres). 103-D-648.

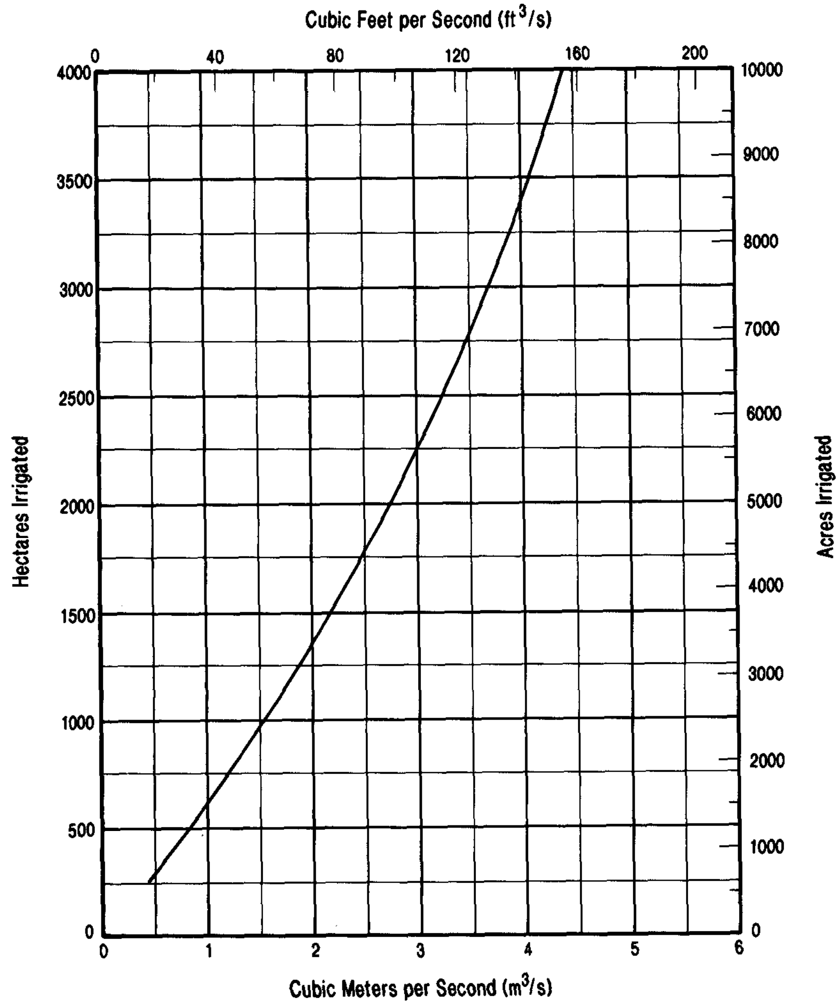


Figure 2–14.—Typical canal and lateral capacity curve for units greater than 400 hectares (1,000 acres). 103-D-649.

Farm waste may amount to as much as 50 percent of the water applied to any farm unit. The total amount of farm waste that must be carried at a particular time at any one point in a drain depends on the amount that is wasted from any single farm unit and on the number of farm units that are being irrigated at the same time above any design point. The number of farm units that can be irrigated simultaneously is considered in the design of the project irrigation system. The same criteria should be used to determine an allowance for farm waste. Canal and lateral capacity curves similar to those shown on figures 2–13 and 2–14 can be prepared for each particular situation from the criteria. These curves are based on the soil, climate, cropping pattern, and similar factors for the particular project and take into consideration the rotation of irrigation water among farm units. These same factors can be used in establishing farm waste capacity in drains unless better information, such as actual measurements of farm waste on an operating project, is available.

For any point on the drain, a topographic map on which the irrigated land and the drain are located will permit determination of the total irrigated acreage whose farm waste must pass through that point on the drain. The lateral capacity for that acreage can then be taken from a curve similar to the one shown on figure 2–13 or 2–14. By applying a factor to that capacity, a factor which will vary somewhat with project characteristics, the drain capacity allowance for farm waste can be obtained. For most irrigation projects, this factor ranges from 15 to 25 percent.

For example, assume that the topographic map shows there are 350 irrigable hectares (approximately 865 irrigable acres) which slope toward the point on the drain in question. From figure 2–13, a lateral capacity of 0.60 cubic meter (21 cubic feet) per second is found for 350 hectares (800 acres). The drain capacity for farm waste would then be 15 percent of this value, or 0.09 cubic meter (approximately 3.2 cubic feet) per second at that point on the drain.

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⟨⟨Chapter III

FIELD AND LABORATORY PROCEDURES

A. In-Place Hydraulic Conductivity Tests Below a Water Table

3–1. Objective.—A number of tests for determining the in-place hydraulic conductivity below a water table have been developed. Two tests that have been found to be the most adaptable use the auger-hole and piezometer test procedures. Both procedures measure the rate of change of the water level in a hole or the difference of water-level elevation with time. Any procedure that can accurately measure water-level change with time is satisfactory.

For aquifers, the well pumping method is used to determine the hydraulic conductivity and transmissivity of gravels and gravelly materials below a water table where the coarse materials interfere with conduction preparations for auger-hole test. Test procedures and data analyses for the classic well pumping method are described in the Bureau of Reclamation's *Ground Water Manual* (1977). The well pumping method, an expensive test both in time and materials, is used mainly for determining the suitability of an area to be drained by pumping rather than by horizontal drains.

3–2. Auger-Hole Test for Hydraulic Conductivity.—*(a) Introduction.*— The auger-hole test measures the average horizontal hydraulic conductivity of the soil profile from the static water table to the bottom of the hole. This test can be run in the presence of a barrier either at or below the bottom of the hole.

This manual describes the equipment, procedures, and calculations used in making this test. The development of the analytical details of the auger-hole test are given in a paper by Maasland and Haskew (1958).

(b) Equipment.—Equipment requirements for the auger-hole test are flexible, but the following items have been used successfully:

(1) *An 80-millimeter (nominal 3-inch) diameter auger with three 1.5-meter (5-foot) extension handles and a 110-millimeter (nominal 4-inch) diameter auger.*—An 80-millimeter-diameter auger is used initially for the auger-hole test. In the finer textured soils, the pressure required for the initial augering causes a thin, dense seal to form on the sides of the hole. This seal is hard to remove even with a hole scratcher.

However, reaming the 80-millimeter hole with a 110-millimeter-diameter auger applies less pressure to the sides of the hole and the resulting seal is very thin and easier to remove. The removal of this thin seal is essential to obtain reliable data from the test. Three 1.5-meter extension handles for the augers are usually sufficient for most test holes.

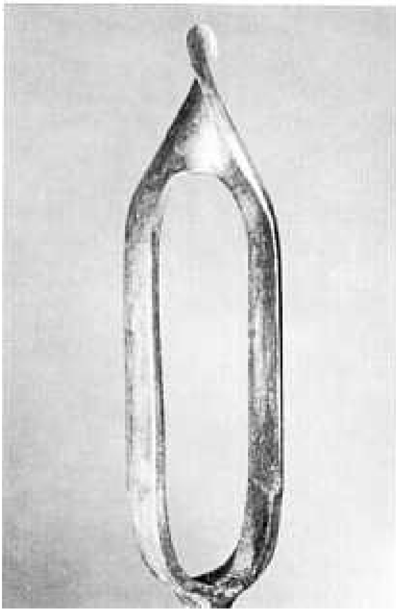
The Durango- and Orchard-type augers are suitable for most soils, but the Dutch-type auger is preferable for some of the high clay and cohesive soils. Samples from the Durango-type auger are less disturbed than those from the other two types, thus permitting a more reliable evaluation of soil structure. Figure 3–1 shows photographs of the different types of soil augers generally used in drainage investigations.

(2) *Equipment used to record changes in water table elevation.*—Two types of equipment have been used to record the recovery of the water table. The first type consists of a data logger with a preprogrammed logarithmic sampling schedule connected to a pressure transducer. The second type consists of a recorder board, recording tape, and float apparatus. The data logger setup can record recovery data points beginning at time zero, which is impossible to do using the float and recorder board. This capability allows the test to be conducted in materials with higher hydraulic conductivity rates than can be done with a float apparatus. The high initial costs of a data logger would be difficult to justify if only a limited number of auger-hole tests are to be conducted.

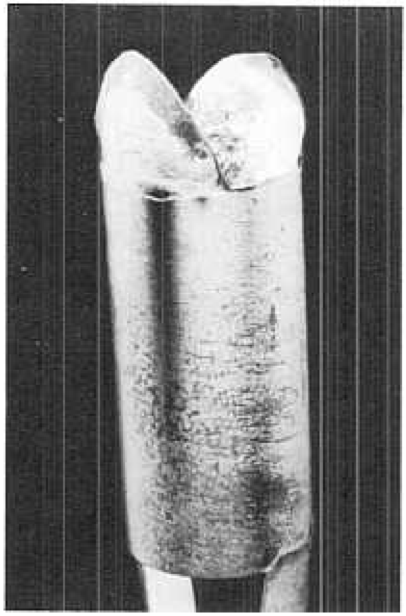
Water table recovery data collected on a data logger can be downloaded directly to a computer. A spreadsheet can then be set up to compute test results.

(3) *Recorder board, recording tape, and float apparatus.*—This equipment is preferable to manual measuring equipment such as an electric sounder because it is less expensive, easier to construct, simpler to operate, and provides a permanent record. The board commonly used is 50 millimeters (2 inches) thick by 100 millimeters (4 inches) wide by 250 millimeters (10 inches) long. A notch 65 millimeters (2-1/2 inches) long and wide enough to hold a nylon roller is made 25 millimeters (1 inch) from one end and 15 millimeters (1/2 inch) from a side. A nylon roller, which can be taken from a regular chair caster, is installed in the notch and fastened in place. A pointer is fastened directly over the roller to act as a reference point during the test. A 50-millimeter (2-inch) diameter recess is drilled near the roller to hold the stopwatch and is located so that the operator can observe the stopwatch and mark on the recording tape without looking up from the stopwatch. A threaded metal plate for attaching a tripod is attached to the underside of the board on the opposite end from the roller and stopwatch.

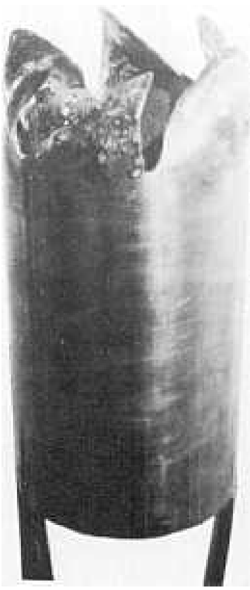
The float should be less than 75 millimeters (3 inches) in diameter and weighted at the bottom. It should also be sufficiently buoyant and counterbalanced to prevent any lag in the rise of the float as the water table rises in the hole. A counterweight that weighs slightly less than the float is used to keep the float string tight. The float should have sloping shoulders so it will be less



Open PX-D-27436



Orchard. PX-



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Figı Types hand soil augers

likely to catch on pebbles or roots on the sides of the open hole or on the joints and perforations in the casing.

Recorder tapes are made from 1.5-meter (5-foot) graph paper strips cut 20 millimeters (3/4 inch) wide and backed with strapping tape. Paper staples are fastened at both ends so the strip can be connected to the float and counterweight. Figure 3–2 shows a schematic of the equipment set up for the auger-hole test.

(4) *Tripod.*—Any rigidly constructed tripod can be used. Planetable tripods furnish a rigid support and allow fast setting up and leveling of the recording board.

(5) *Measuring rod or tape.*—A measuring rod can be made, or a tape with a weight on the bottom can be used.

(6) *Hole scratcher.*—A hole scratcher can be made in a number of ways. The easiest method uses a wooden cylinder, 85 millimeters (3-1/2 inches) in diameter by 75 millimeters (3 inches) long, with small nails protruding as necessary for the auger being used. The heads of the nails, after they have been driven into the cylinder, are cut off to create sharp edges which will break the seal around the periphery of the hole. A 13-millimeter (1/2-inch) coupler attached to the wooden cylinder allows the scratcher to use the same extension handles as the augers. A more efficient hole scratcher can be made from a 85-millimeter (3-1/2-inch) outside-diameter black iron pipe cut 125 millimeters (5 inches) long. A 13-millimeter (1/2-inch) coupling is then welded to a 85-millimeter (3-1/2-inch) diameter by 7-millimeter (1/4-inch) thick plate which, in turn, is welded to one end of the pipe. Holes 3 millimeters (1/8 inch) in diameter are then drilled into the pipe in a staggered pattern. Concrete nails are then inserted through each drilled hole from the inside of the pipe. The length of the nails used depends on the diameter of the auger to be used. A wooden block, 80 millimeters (3-1/4 inches) in diameter and 125 millimeters (5 inches) long, is then placed inside the pipe to hold the nails in place. The block can be held in position by drilling a few holes at the pipe ends for holding screws. As different auger-hole diameters are required, longer or shorter nails can be placed in the scratcher. A typical hole scratcher is shown on figure 3–3.

(7) *Bailer or pump.*—A bailer can be made from a 1-meter length of 90-millimeter (nominal 3-1/2-inch) diameter, thin-walled conduit with a rubber or metal foot valve at one end and a handle at the other end. Bailers longer than 1 meter are difficult to insert and remove from the auger hole. The hole in the foot valve should be large enough to allow water to enter as rapidly as possible. The bailer should be weighted at the bottom to increase its ability to submerge. Present-day requirements for water quality sampling have made many types of commercial bailers available. They are manufactured from a variety of materials which range from teflon to stainless steel. We have found that a bailer of the appropriate diameter made from schedule 40 PVC is adequate. A lightweight stirrup pump, similar to the one shown on figure 3–3, capable of pumping about 1.5 liters per second (about 20 gallons per minute), is preferable to the bailer.

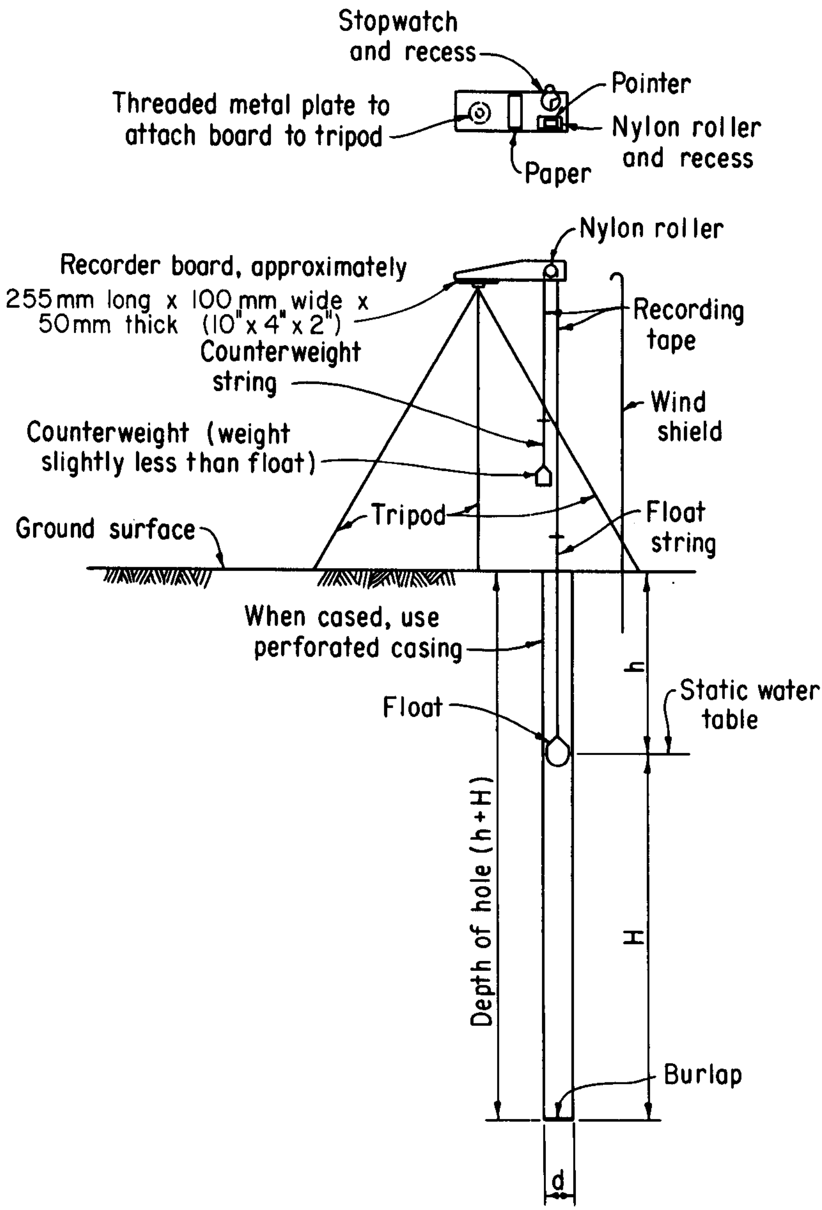


Figure 3-2.—Equipment setup for the auger-hole or piezometer test. 103-D-651.

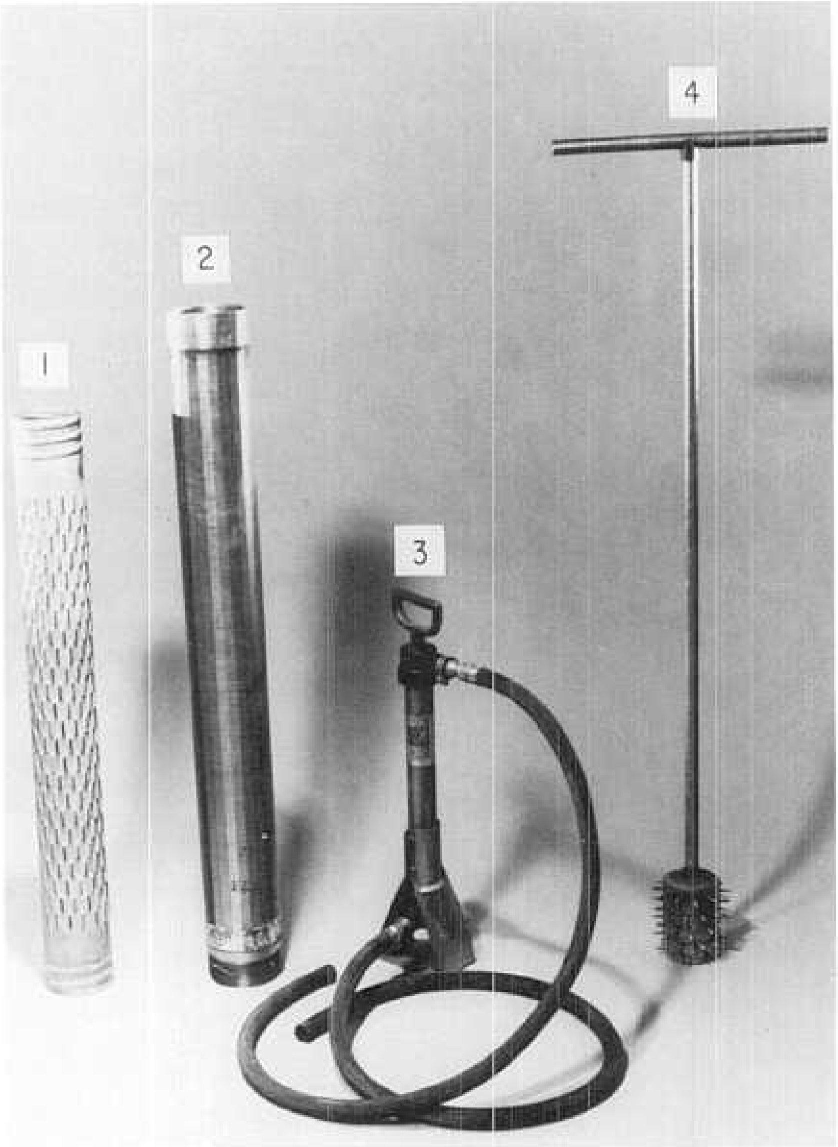


Figure 3–3.—Equipment for auger-hole test. Item (1) perforated casing, (2) wire-wound well screen, (3) stirrup pump, and (4) hole scratcher. P801-D-77012.

(8) *Stopwatch.*—Any standard stopwatch or digital watch with seconds registered is satisfactory when using the float apparatus. All readings should be made from a single reference time which is the beginning of bailing, and all time during the test should be accounted for.

(9) *Inside calipers.*—An ordinary pair of inside calipers can be used to determine the diameter of the hole. To prevent the points of the caliper legs from gouging the walls of the auger hole, small flat plates should be welded to the legs. An extension rod screwed into the top of the calipers is used to measure the hole diameter at various depths. The average hole diameter is used in the calculations. The diameter is difficult to measure below the water table with ordinary inside calipers because the water surface reflects light and prevents visual determination of the contact of the calipers with the sides of the hole. For this reason, it is satisfactory to determine the average hole diameter by the measurements made about 0.3 meter (1 foot) below the ground surface and just above the water table.

(10) *Burlap.*—Burlap or a similar permeable material will prevent soils from entering at the bottom of the hole. Each hole requires a piece measuring about 0.6 meter (2 feet) square.

(11) *Perforated casing or wire-wound well screen.*—This protection is necessary for auger holes in unstable soils. The casing or screen should have the same or a slightly larger outside diameter than the hand auger. As the screen or casing is pushed into the ground, the casing and the periphery of the hole make definite contact. Commercial well screen with at least a 10-percent perforated area is the most desirable; however, if this is not available, a thin-walled downspout casing with 4- to 5-percent perforations is satisfactory. In most agricultural soils, about two hundred 5- by 25-millimeter hacksaw perforations per meter will give 4- to 5-percent perforations. Commercially available slotted PVC casing has also proven adequate for conducting auger-hole tests. Figure 3–3 shows a typical perforated casing and wire-wound well screen.

(12) *Mirror or strong flashlight.*—Either one of these items can be used to examine the sides of the auger hole and facilitate measurements with the calipers.

(13) *Windshield.*—When wind protection is required, a windshield such as a 1- by 1-meter sheet of plywood has been used satisfactorily.

(c) Procedure.—The most efficient team for performing the auger-hole field test for hydraulic conductivity consists of two people. One operates the recorder board, puts the float in the hole, and operates the stopwatch, and the other operates the bailer or pump. After the water level in the hole has stabilized, an experienced team can perform the entire test in 10 to 15 minutes in most soils.

At sites where detailed soil profile data do not exist, a pilot hole will have to be drilled and logged, and test zones selected.

The hole should be augered vertically and as straight as possible to the required depth. If the soil is homogeneous throughout the profile, the hole can be excavated to the total depth to be tested. When the soil is heterogeneous, tests should be made for each change in texture, structure, and color. If the material is highly permeable throughout the profile to be tested, it is best to stop the hole about 0.6 or 1.0 meter (2 to 3 feet) below the water table so that one bailing will draw the water down to about the bottom of the hole. Upon completion of the augering, the sides of the hole should be scratched to break up any sealing effect caused by the auger. Scratching is not necessary in the coarser textured soils. Burlap is then forced to the bottom of the hole and tamped lightly to prevent any soils from entering the bottom. The sealing effect can be overcome by allowing the water table to rise to the static water level, and then gently pumping or bailing the water out to develop the best flow characteristic. Afterward, time must be allowed for the water table to reach static level before running the test. Prior to starting the test, the depth to the static water table from the ground surface, the total depth of the hole, and the distance from the static water table to the bottom of the hole should be measured carefully. Figure 3–4 shows a sample data and computation sheet for the test.

To begin the test, the tripod with the recorder board, recording tapes, and float apparatus is placed near the hole so the float can be centered over the hole and moved freely into it. The float is then lowered into the hole until it floats on the static water table level. After a short time period, to allow the water to return to static level, a zero mark is made on the tape, and the counterweight positioned so the full change of water table level can be recorded. This positioning may require that the counterweight hang inside the casing. The float is then removed, and the water is bailed or pumped from the hole as quickly as possible to minimize the amount of water which returns before the readings are started. For best results, sufficient water should be bailed or pumped from the hole so all readings can be completed before the water level rises to half its original height, or 0.5 H. One or two passes with the bailer are usually sufficient for most agricultural soils. As the last bail is withdrawn from the hole, or the pump starts drawing air, the float should be placed in the hole as quickly as possible. When the water level rises rapidly, the float can be left in the hole and below the bailer or foot valve, which will minimize the amount of water returning into the hole before the first reading can be made. The stopwatch is started at the moment the first bailer is withdrawn, or when pumping begins, and should run continuously until completion of the test.

When using the recorder board and float mechanism, using equal time intervals is convenient, starting from the initial tick mark on the recorder tape. As equal time intervals are read on the stopwatch, the operator marks the tape opposite the pointer. Measurements are continued until recovery of water in the hole equals about 0.2 of the depth initially bailed out or, stated another way, until a reading on the measuring tape of $0.8Y_o$ has been reached (Y_o is the distance the water in the hole was lowered by bailing). Upon completion of the test, the final time is recorded at the last tick mark on the recorder tape. Any irregularities in the record can be quickly observed on the recorder tape, and if all readings are highly irregular, the test should be rerun after the static water table has been

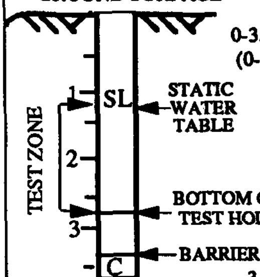
HOLE NO. E - 4 **LOCATION** SAMPLE FARM

OBSERVER D.M.S. **DATE** AUGUST 6, 1992

HOLE: CASED ☒ **UNCASED** ☐

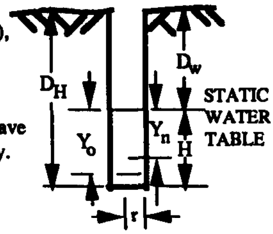
HOLE DIAMETER 102 **MILLIMETERS** (4 inches)

LOG **DESCRIPTION**



0-3.35 m: (0-11ft) Light brown sandy loam(SL), friable, nonsticky, granular. Wet at 1.52 m. (5ft) Slightly compacted below 1.83 m (6ft). Appears to have good hydraulic conductivity.

3.35-3.66 m: (11-12ft) Blue gray clay (C), Sticky, structureless. Appears to be impermeable.



r = 0.051 m (0.167 ft)

D_H = 2.74 m (9.0 ft)

D_w = 1.46 m (4.8 ft)

H = 1.28 m (4.2 ft)

Y_o = 0.96 m (3.15 ft)

0.8 Y_o = 0.77 m (2.52 ft)

	TIME SECONDS	Δt	Y_n meters	(feet)	ΔY meters	(feet)
	0	--	0		--	
	13	13	0.960	(3.15)	Y_o	
	23	10	0.927	(3.04)	0.033	(0.11)
	33	10	0.893	(2.93)	0.034	(0.11)
	43	10	0.860	(2.82)	0.033	(0.11)
	53	10	0.823	(2.70)	0.037	(0.12)
	63	10	0.789	(2.59)	0.034	(0.11)
0.8 Y_o	73	10	0.759	(2.49)	0.030	(0.10)
	83	10	0.732	(2.40)	0.027	(0.09)
	93	10	0.704	(2.31)	0.028	(0.09)

$\bar{Y}_n = \frac{0.960 + 0.759}{2} = 0.860$ meter (2.82 feet)

$\Delta Y = 0.0335$ meter (0.11 ft)

$\Delta t = 10$ seconds

$\frac{H}{r} = \frac{1.28}{0.051} = 25.10$

$\frac{\bar{Y}_n}{r} = \frac{0.860}{0.051} = 16.86$

C = 390 (from chart)

$K = C \frac{\Delta Y}{\Delta t} = 1.31$ meters (4.3 feet) per day

or 5.45 centimeters (2.151 inches) per hour

Figure 3–4.—Data and computation sheet on auger-hole test for hydraulic conductivity. 103–D–650.

reestablished. Only the period covering the regularly spaced tick marks below $0.8Y_o$ is used in the computations. One irregular spacing usually occurs at the beginning of the test while the float is steadying. As the water rises above $0.8Y_o$, the marks will no longer be equally spaced, but will become closer together with each successive reading. The beginning of the shorter spacings usually will occur around $0.8Y_o$, but two or three extra readings are recommended to show that the spacings are definitely getting closer together.

The use of a pressure transducer and a data logger eliminates or greatly reduces many of the problems related to recording water table recovery discussed in the above paragraphs. With this equipment, the pressure transducer is placed near the bottom of the hole and calibrated to the static water level. The data logger is started just prior to removing the bailer from the hole. Running the data logger until 50 percent recovery has occurred will provide adequate data for computation of the hydraulic conductivity rate.

(*d*) *Calculations.*—Upon completion of the auger-hole field test for hydraulic conductivity, the time intervals and the corresponding distances between tick marks on the recorder tape are transferred to the computation sheet. Sample computations are shown on figure 3–4. The initial Y_n for time zero can be computed or extrapolated from a Y_n versus time curve if the time from start of pumping to the first tick mark is less than 10 seconds.

Determining the initial Y_n is necessary only when the time interval between the starting time and the first measurement is longer than about 5 seconds and the water level recovery rate is very fast. Extrapolating the data to determine Y_o, or the initial Y_n, is not always reliable. Every effort should be made to keep the time interval between the start of pumping and the first tick mark as short as possible. This short time interval is particularly important in sands and gravels with rapid recovery rates.

Care should be taken in selecting consistent, consecutive time intervals and water table rises to be used in determining the average distance from static water table to the water surface in the hole during the test period, $\overline{Yn}$; the average incremental rise during incremental time intervals, ΔY; and the average incremental time interval between ticks, Δt.

Water table recovery data collected by a data logger using a properly programmed logarithmic sampling schedule will provide data points beginning at time zero. This early time data greatly reduces, if not eliminates, the concerns discussed in the preceding paragraphs. As it is difficult to start the data logger at the exact time water table recovery begins, the early time data should be plotted to determine the point when computations should begin.

The C value needed in the computations shown on figure 3–4 is determined from the graphs of figure 3–5 or 3–6, which are intended for use where the barrier is considered to be at infinity or at zero distance below the bottom of the hole. The C values plotted against the dimensionless parameter $\overline{Yn}/r$ simplify the determination of C for a wide range of values of H/r and $\overline{Yn}/r$. For the usual case where no barrier is present, or the barrier is equal to or greater than H below the

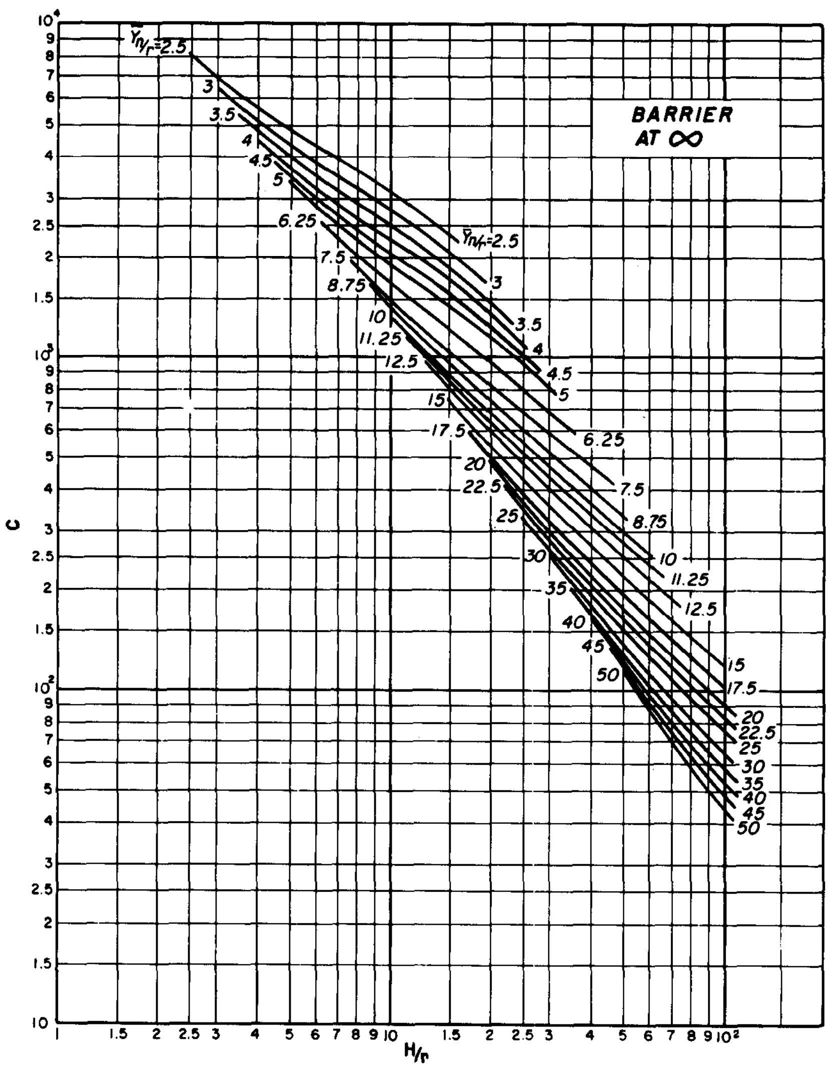


Figure 3–5.—Values of C when barrier is below bottom of hole during auger-hole test (Maasland and Haskew, 1958). 103–D-653.

bottom of the hole, figure 3–5 should be used to determine C. If the hole has been terminated on a slowly permeable zone, figure 3–6 should be used. If the hole penetrates into a slowly permeable zone below a permeable zone, figure 3–6 should be used with H as the distance from the level of the static water table to the slowly permeable layer instead of to the bottom of the hole, as is the usual case. The hydraulic conductivity can then be determined by multiplying the C factor by $\Delta Y/\Delta t$. The resulting hydraulic conductivity has units of meters per day (feet per day) or centimeters per second (inches per hour).

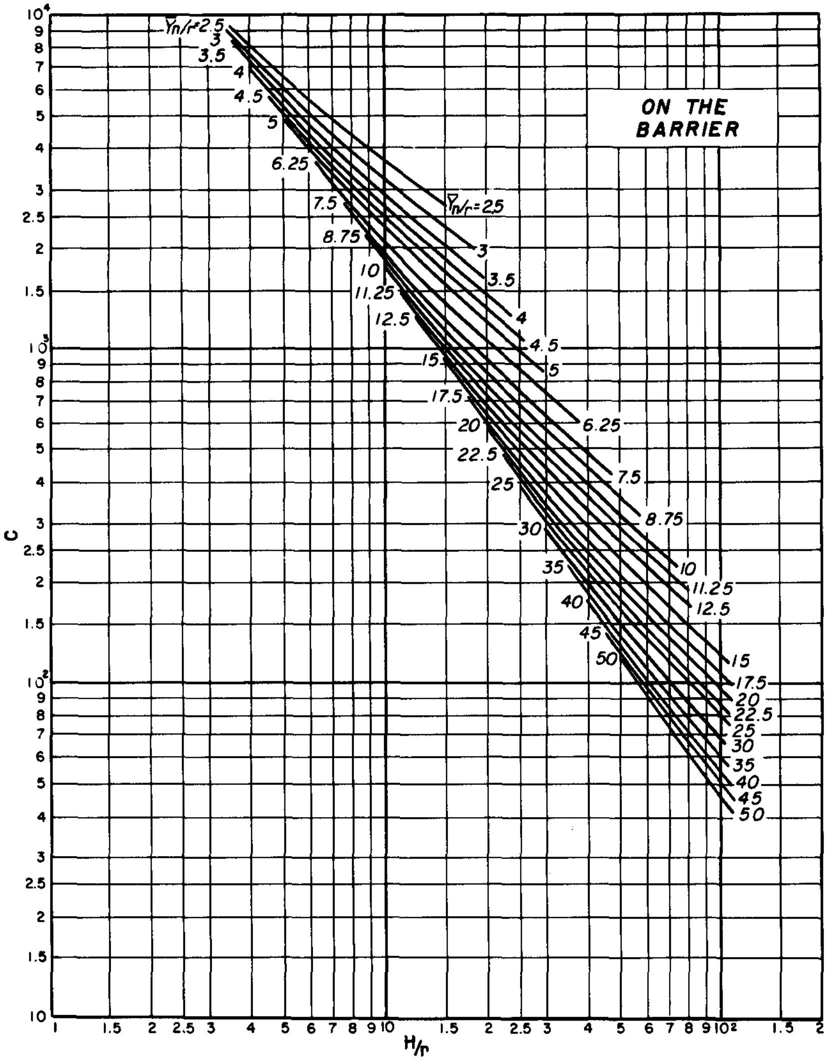


Figure 3–6.—Values of C when barrier is at bottom of hole during auger-hole test (Maasland and Haskew, 1958). 103–D-652.

(*e*) *Limitations.*—The auger-hole test furnishes reliable hydraulic conductivity data for most conditions; however, the results are entirely unreliable when the hole penetrates into a zone under piezometric pressure. Small sand lenses occurring between less permeable layers make the test more difficult to perform and may yield unreliable data. Water flowing into the hole through the lenses falls on the float apparatus and causes erratic readings. The auger-hole test also cannot be used when the water table is at or above the ground surface because surface water or water running through permeable surface layers will cause erroneous readings. A depth of more than 5 meters (20 feet) to water table, although not a limitation as far as obtaining valid data is concerned, makes obtaining reliable data extremely difficult.

Comparatively high hydraulic conductivity rates, in the magnitude of 6 meters per day (10 inches per hour) or more, make the auger-hole test difficult to perform because the bailer cannot remove the water as fast as it enters. A pump will remove the water from the hole rapidly, but in very permeable soils only one or two readings can be obtained before recovery exceeds 0.2 of the initial drawdown. A hydraulic conductivity can be calculated from only one or two readings, but the results could be erroneous. The use of a data logger to collect water table recovery data will solve this problem, which occurs when using float-activated equipment. Tests have been successfully run in alluvial materials having hydraulic conductivity rates of over 30 meters per day (50 inches per hour) using a data logger.

At the other extreme, auger-hole tests in soils with hydraulic conductivity rates in the range of 0.0006 to 0.006 meter per day (0.001 to 0.01 inch per hour) usually give such erratic readings that accurate values cannot be obtained. However, the results can be important in determination of drainage requirements even though exact values are not obtained. The knowledge that hydraulic conductivities are very high or very low can be quite useful from a practical standpoint.

The difficulty usually encountered in augering or digging a hole of uniform size through rocky or coarse-gravel material can prevent the performance of an auger-hole test. Casing can sometimes be used to stabilize the walls of the hole if a test is needed in these materials. Generally, however, most agricultural soils being investigated for subsurface drainage systems can be tested by the auger-hole method if a water table exists close enough to the ground surface.

(*f*) *Step Tests in Layered Soils.*—Step tests are used to determine the hydraulic conductivity of layered soils. Step tests are simply a series of auger-hole tests in or near the same hole location but at different depths. The hole is initially augered to within 75 to 100 millimeters (3 or 4 inches) of the bottom of the first texture change below the water table, and then the first auger-hole test is run and the hydraulic conductivity computed. The hole is then augered to within 75 to 100 millimeters of the bottom of the next texture change, the second test is run, and the average hydraulic conductivity for both layers can then be determined. The procedure continues until the last layer to be tested has been reached. The hydraulic conductivity value calculated for each step will be the average value from the water table to the depth of the hole. The hydraulic conductivity for the individual texture is found from the formula:

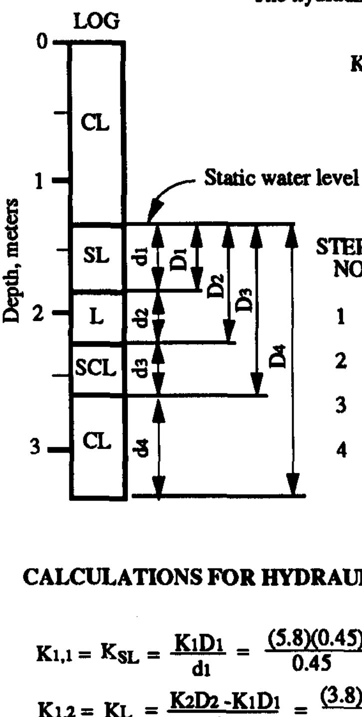
$$K_{n,x} = \frac{K_n D_n - K_{n-1} D_{n-1}}{d_n} \tag{1}$$

where:

$K_{n,x}$ = hydraulic conductivity to be determined
K_n = hydraulic conductivity obtained in the nth step of test,
$K_{n\text{-}1}$ = hydraulic conductivity obtained in the (n–1) step,

d_n = thickness of the nth stratum $(Dn - D_{n\text{-}1})$,
D_n = total thickness of the nth step from the static water level,
$D_{n\text{-}1}$ = total thickness from the static water level for the (n–1) step,
n = number of the test, and
x = step number.

Test errors may produce negative results, and the test should be rerun. If the results are still negative after a rerun, the piezometer test described in section 3–3 should be used. A sample calculation sheet for the step test is shown on figure 3–7.



The hydraulic conductivity for a specific layer is given by:

$$K_{n,x} = \frac{K_n D_n - k_{n-1} D_{n-1}}{d_n}$$

$$d_n = D_n - D_{n-1}$$

FIELD TEST DATA

STEP NO.	K_n, cm/h	d_n, meters	D_n, meters
1	$K_1 = 5.8$	d_1=1.76-1.31=0.45	D_1=1.76-1.31=0.45
2	$K_2 = 3.8$	d_2=2.19-1.76=0.43	D_2=2.19-1.31=0.88
3	$K_3 = 3.0$	d_3=2.59-2.19=0.40	D_3=2.59-1.31=1.28
4	$K_4 = 2.0$	d_4=3.38-2.59=0.79	D_4=3.38-1.31=2.07

CALCULATIONS FOR HYDRAULIC CONDUCTIVITY OF SPECIFIC LAYERS

$K_{1,1} = K_{SL} = \frac{K_1 D_1}{d_1} = \frac{(5.8)(0.45)}{0.45}$ = 5.8 centimeters per hour (2.3 inches per hour)

$K_{1,2} = K_L = \frac{K_2 D_2 - K_1 D_1}{d_2} = \frac{(3.8)(0.88) - (5.8)(0.45)}{0.43}$ = 1.71 centimeters per hour (0.64 inch per hour)

$K_{1,3} = K_{SCL} = \frac{K_3 D_3 - K_2 D_2}{d_3} = \frac{(3.0)(1.28) - (3.8)(0.88)}{0.40}$ = 1.24 centimeters per hour (0.53 inch per hour)

$K_{1,4} = K_{CL} = \frac{K_4 D_4 - K_3 D_3}{d_4} = \frac{(2.0)(2.07) - (3.0)(1.28)}{0.79}$ = 0.38 centimeters per hour (0.15 inch per hour)

Figure 3–7.—Data and computation sheet on step test for hydraulic conductivity. 103–D-1627.

3-3. Piezometer Test for Hydraulic Conductivity.—*(a) Introduction.*— The piezometer test measures the horizontal hydraulic conductivity of individual soil layers below a water table. This test is preferred over the auger-hole test when the soil layers to be tested are less than 18 inches thick and when individual layers below the water table are to be tested. In subsurface drainage investigations, an important application of this test is to provide data for determining which layer below a proposed drain depth functions as the effective barrier layer. This test also provides reliable hydraulic conductivity data for any soil layer below the water table.

(b) Equipment.—Suggested equipment required for the piezometer test is:

(1) Casing of minimum 25-millimeter (1-inch) i.d. (inside diameter) 40- to 50-millimeter i.d. recommended) consisting of a thin-walled electrical conduit for depths to 4 meters and black iron pipe with smooth inside walls for depths greater than 4 meters.

(2) Ship auger which fits inside the casing.

(3) Pipe-driving hammer, consisting of a piece of 50-millimeter (2-inch) iron pipe which fits over the casing with a 5-kilogram (10-pound) weight fixed to the pipe. A small sledge hammer can be used in place of the 5-kilogram (10-pound) weight.

(4) Hand-operated pitcher pump with hose and foot valve, or a bailer which will fit inside the casing.

(5) Recorder board, recording tapes, and float apparatus or an electrical sounder. The float resembles the float made for the auger-hole test, but is of smaller size to fit into the smaller diameter casing. The counterweight must be adjusted accordingly.

(6) Computation sheets, clipboard, stopwatch, measuring tape or rod, windshield, and casing puller.

(7) Bottle or vegetable brush for cleaning soil film from inside of test pipe. The brush should be fitted with a coupler that attaches to the auger handle.

(c) Procedure.—A two-man team is desirable in performing the piezometer field test for hydraulic conductivity. The test layer should be at least 300 millimeters (12 inches) thick so that a 100-millimeter (4-inch) length of uncased hole, or cavity, can be placed in the middle of it. This placement is especially important if a marked difference in the texture, structure, or density of the layers exists above and below the test layer. After the test layer has been selected, the topsoil is removed from the ground surface, and a hole is augered to within 0.5 meter (2 feet) of the test layer. Some operators prefer to auger 150 to 300 millimeters (6 to 12 inches), then drive the casing and repeat this process for the entire depth of the hole. However, this method is slow, and experience shows its use is generally not warranted. Other operators jet the casing to within 0.5 to 0.75 meter (2 to 3 feet) of the test layer and then auger and drive the casing the remaining distance. This procedure requires additional equipment that usually cannot be moved in to a waterlogged field. The augering and driving procedure is always used for the last 0.5 meter (2 feet) to assure a good seal and also to minimize soil disturbance. The casing is stopped at the depth selected for the top of the 100-millimeter (4-inch) long cavity, and the cavity is then augered below the

casing. After some recovery has occurred, the pipe should be cleaned with a bottle brush to remove the soil film that the float may cling to.

The size and shape of the cavity are important in the test, so care should be taken to assure that it is the predetermined length and diameter. If the soil in the test layer is so unstable that the cavity will not remain open during the test, screens should be made that can be pushed down inside the casing. For a 25-millimeter (1-inch) i.d. casing and a 100-millimeter (4-inch) cavity, the screen should be 125 millimeters (5 inches) long and have a 24-millimeter (15/16-inch) o.d. (outside diameter). A rigid point should be welded on the bottom of the screen to facilitate pushing it down inside the casing. A pole about 20 millimeters (3/4 inch) in diameter can be used to push the screen to the bottom of the cavity. A small bent nail or hook placed on the opposite end of the pole will allow the screen to be reclaimed at the end of the test by hooking the nail into the screen and pulling it out. The cavity is cleaned by gently pumping or bailing water and sediment out of the hole until the discharge is clear.

After the water table has returned to equilibrium, the recorder board and float apparatus are set up and the float dropped down the casing. Figure 3–2 shows the equipment setup. When the float comes to rest, the pointer is set at zero on the recorder tape, the float is removed from the hole, and the water is pumped or bailed out. A small foot valve for the suction line can be made similar to larger commercial types, or a bailer similar to that used in the auger-hole test can be made. After pumping or bailing the water, the float is immediately dropped down the casing. When the float starts to rise, a tick mark is made on the recorder tape and at the same time the stopwatch is started. Select a convenient time interval between observations and make corresponding tick marks on the recorder tape. Removal of all of the water from the piezometer is not essential because measurements can be obtained and used anywhere between the static water table level and the initial bailed-out level. Obtaining three or four readings during the first half of the water rise will give consistent results.

(d) Calculations.—After completion of the piezometer test, the hydraulic conductivity is calculated from the equation developed by Kirkham (1945):

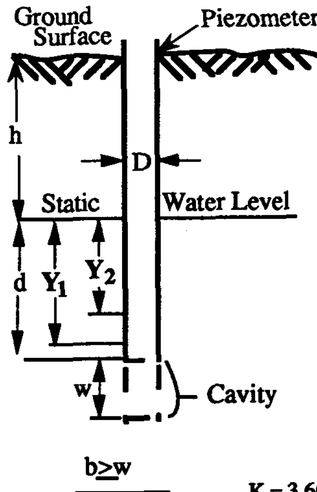
$$K = \frac{3{,}600\pi\left(\frac{D}{2}\right)^2 \log_e\left(\frac{Y_1}{Y_2}\right)}{A(t_2-t_1)} \tag{2}$$

where:

K	=	hydraulic conductivity in centimeters per hour (inches per hour),
Y_1 and Y_2	=	distance from static water level to water level at times t_1 and t_2 in centimeters (inches),
D	=	diameter of casing in centimeters (inches),
t_2–t_1	=	time for water level to change from Y_1 to Y_2 (seconds), and
A	=	a constant for a given flow geometry in centimeters (inches).

A sample calculation using this equation is shown on figure 3–8.

Location: Hole C-2 -- Sample Farm
Observer: A.P.B. Date: October 9, 1974



h = 218.44 centimeters (86.00 inches)
Ground Surface to static water level

D = 2.54 centimeters (1.00 inch)
Inside diameter of piezometer and cavity

d = 237.74 centimeters (93.60 inches)
Static water level to bottom of piezometer

w = 10.16 centimeters (4 inches)
Length of cavity

A = 33.27 centimeters (13.1 inches)
Constant for a given flow geometry taken from curve.

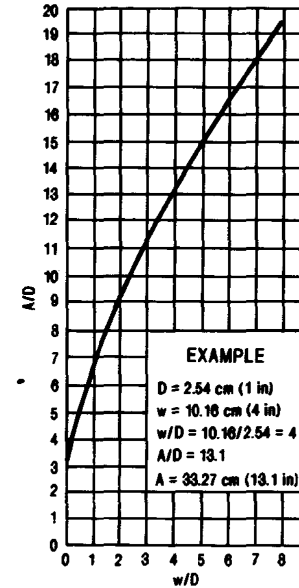
K = Hydraulic conductivity, centimeters per hour (inches per hour)

b = depth to texture change

Y_1, Y_2 = Distance from static water level at time t_1 and t_2 in centimeters (inches).

$(t_2 - t_1)$ = Time for water to change from Y_1 to Y_2 (seconds)

$$K = \frac{3{,}600\,\pi\,(D/2)^2 \log_e (Y_1/Y_2)}{A(t_2 - t_1)}$$, centimeters per hour (inches per hour)



A as a function of D and w.
Redrawn from LUTHIN & KIRKHAM (1949).
Revised by USBR (Mantel, 1972)

Time (seconds)		Y, centimeters (inches)		A, cent. (inches)	t_2-t_1	Y_1/Y_2	$\log_e Y_1/Y_2$	$3{,}600\,\pi\,(D/2)^2$ cm^2sec/hr (in^2sec/hr)	K cm/hr (in./hr)
Initial (t_1)	Final (t_2)	Initial(Y_1)	Final (Y_2)						
0	30	218.44 (86.00)	197.87 (77.90)	33.27 (13.1)	30	1.104	0.099	18241.47 (2827.44)	1.80 (0.71)
30	60	197.87 (77.90)	178.44 (70.25)	33.27 (13.1)	30	1.109	0.103	18247.47 (2827.44)	1.88 (0.74)
60	90	178.44 (70.25)	160.02 (63.00)	33.27 (13.1)	30	1.115	0.109	18241.47 (2827.44)	1.99 (0.78)
90	120	160.02 (63.00)	145.47 (57.27)	33.27 (13.1)	30	1.100	0.095	18241.47 (2827.44)	1.74 (0.68)
120	150	145.47 (57.27)	131.17 (51.64)	33.27 (13.1)	30	1.109	0.103	18241.47 (2827.44)	1.88 (0.74)

Average for 5 readings = 1.86 (0.73)

Figure 3–8.—Data and computation sheet on piezometer test for hydraulic conductivity. 103–D–680.

The constant A may be taken from the curves shown on figures 3–8 or 3–9. The curve on figure 3–8 is valid when d and b are both large compared to w (d = distance from the static water level to bottom of piezometer; b = distance below bottom of cavity to top of the next zone; and W = length of cavity.) According to Luthin and Kirkham (1949), when $b = 0$ and d is much greater than w, the curve will give an A factor for $W = 4$ and $D = 1$, which will be approximately 25 percent too large.

The chart on figure 3–9 is used for determining A when piezometric pressures exist in the test zone. When pressures are present, additional piezometers must be installed. The tip of the second piezometer should be placed just below the contact between layers in a layered soil, see figure 3–10. In deep uniform soils, the second piezometer tip should be placed an arbitrary distance below the test cavity.

After installing the piezometers, the following measurements should be made:

(1) Distance H, in meters (feet), between piezometer tips,

(2) Difference Δ in meters (feet), between water levels in the piezometer at static conditions, and

(3) Distance d', in meters (feet), between center of the lower piezometer cavity and the contact between soil layers in layered soils.

The A value from figure 3–9 is used in equation (2) to determine the hydraulic conductivity.

(e) Limitations.—Installation and sealing difficulties encountered in gravel or coarse sand material comprise one of the principal limitations of the piezometer test for hydraulic conductivity. Even when the hole can be augered in these materials, rocks on the sides of the hole often dent or rip the casing. Also, when the casing bottoms in coarse gravel, a satisfactory cavity cannot be obtained.

Six meters (20 feet) is about the practical limit of hole depth, both for installation and water removal with a stirrup pump. Duplicate tests in soils of very low hydraulic conductivity (0.0025 to 0.025 centimeter per hour) are always in the low range, but can vary as much as 100 percent. However, this much variation has little consequence in this low range. Test layers less than about 25 to 30 centimeters (10 to 12 inches) thick and lying between more permeable materials will not give reliable results because of the influence of the more permeable materials. The size of the casing is a matter of preference, as long as it is 25 millimeters (1 inch) or more in diameter. Field experience has shown that 38-millimeter (1-1/2-inch) i.d. piezometers provide adequate open area for float operation. Pipe diameters greater than 50 millimeters (2 inches) are difficult to install properly.

3–4. Pomona Well Point Method.—This method resembles the piezometer test discussed in the preceding paragraphs, except that this method measures discharge for a fixed draw-down rather than the water table recovery rate. These differences allow data collection in unstable materials where an open cavity is

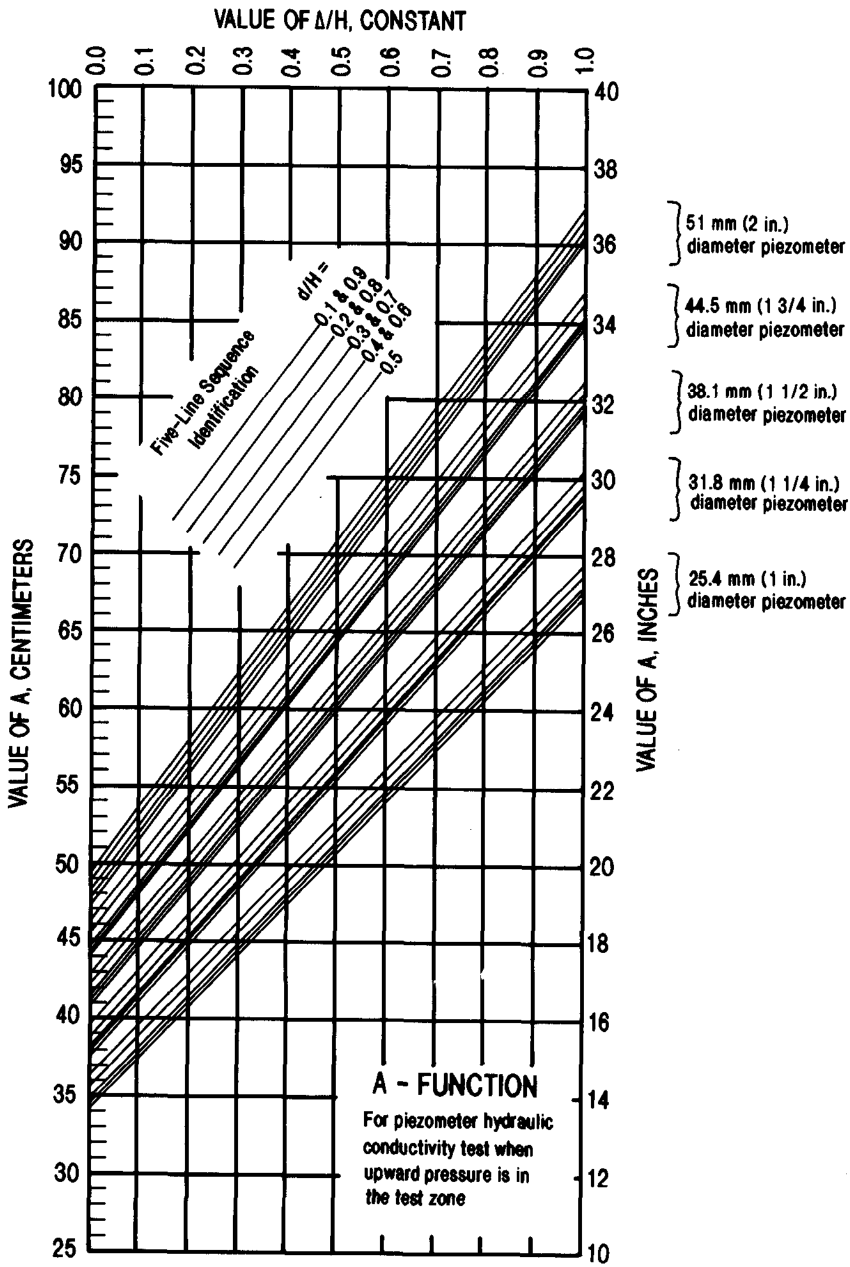
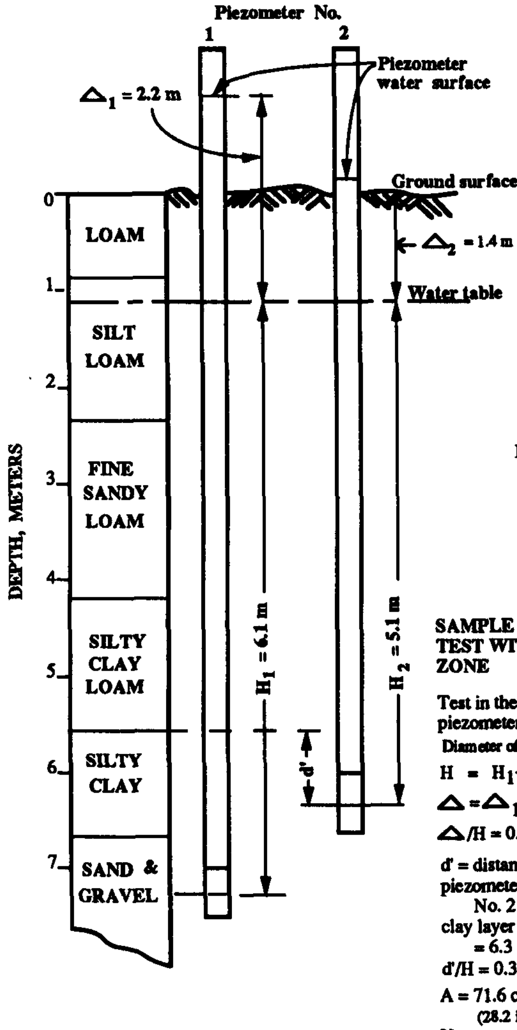


Figure 3–9.—Chart for determining *A*-function on piezometer test for hydraulic conductivity when there is upward pressure in the test zone. 103–D–1628.



$$K = \frac{3{,}600\pi (D/2)^2 \log_e (Y_1/Y_2)}{A(t_2 - t_1)}$$

Notes:
d' = Distance from top of test layer to center of test cavity.
H = Distance from water table to center of test cavity.

SAMPLE CALCULATION FOR PIEZOMETER TEST WITH UPWARD PRESSURE IN TEST ZONE

Test in the silty clay - - Find A - Function using piezometers 1 and 2

Diameter of piezometer is 3.8 centimeters.

$H = H_1 - H_2 = 6.1 - 5.1 = 1$ meter (3.3 feet)

$\triangle = \triangle_1 - \triangle_2 = 2.2 - 1.4 = 0.8$ meter (2.6 feet)

$\triangle/H = 0.8/1.0 = 0.8$

d' = distance from ground surface to center of test cavity in piezometer
No. 2 minus the distance from ground to top of silty clay layer
= 6.3 - 6.0 = 0.3 meter (1.0 foot)

d'/H = 0.3/1 = 0.3

A = 71.6 centimeters (from A- function chart)
(28.2 inches)

Use recovery data from piezometer No. 2 to determine K value for the silty clay layer.

Figure 3–10.—Sample calculation for piezometer test with upward pressure in the test zone. 103–D–1629.

difficult to maintain. This test method can also be used in materials where the water recovery rate is very rapid.

The setup may be identical to the piezometer test or it may employ a driven well point.

After installation is complete and the well has been developed, the test is conducted by pumping at a rate to maintain a fixed drawdown. The discharge is measured for 1 out of every 5 minutes until a steady rate is obtained. When the system reaches equilibrium, the discharge rate is measured. The hydraulic conductivity rate is determined by:

$$k = Q/Ah$$

where:

K = Hydraulic conductivity
Q = Discharge rate
A = A constant for a given flow geometry (see figs. 3–8, 3–9)
h = Head difference

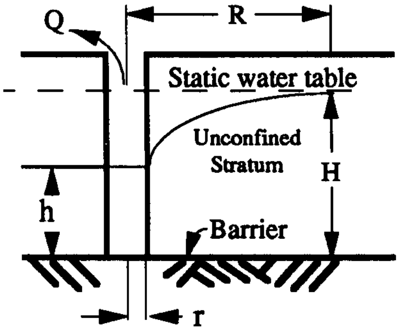
Layered soils can easily be investigated, and the soil need not support a cavity if a screened well point is used. Even when the cavity is unsupported, as in the piezometer setup, there is substantially less hydrostatic pressure on the cavity than in the piezometer test. The primary limitations are the time required to conduct the test and the impracticality of measuring low permeabilities.

3–5. Single Well Drawdown Test for Hydraulic Conductivity.—Coarse sands and gravels usually make the auger-hole (pump-out) and piezometer tests difficult to run. An alternative pump-out test can be made to obtain a rough estimate of hydraulic conductivities in these materials. The test is a small-scale version of a regular pump test for large wells.

Equipment for the test is the same as that used for the auger-hole test except the recorder board and tripod are not used. A gasoline-driven pump with a valved discharge should be used. A calibrated bucket and a stopwatch should be used to determine flow rate.

Hole preparation is much the same as for the auger-hole test; however, hand augering is usually too difficult. Once the hole is prepared and the static water level is measured, water is pumped from the hole at a constant rate. After some time, the water level in the hole will reach a steady-state level. Steady state can be assumed to exist when the water level in the hole drops less than 30 millimeters (0.1 foot) in 2 hours. When steady-state conditions exist, the flow rate and depth of water in the hole are recorded. These data, along with the distance from the static water level to the bottom of the hole, are used in one of the equations shown on figure 3–11. Use the equation that most nearly approaches the test conditions.

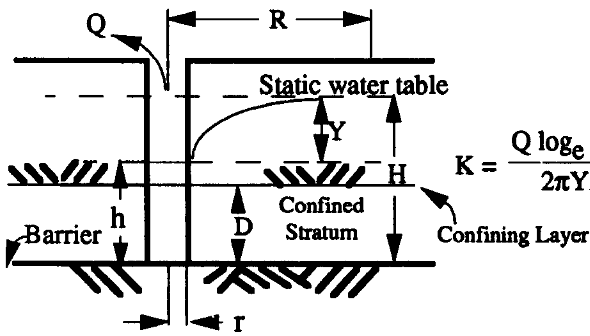
This method should be used only in highly permeable sands and gravels to obtain an estimate of hydraulic conductivity when the auger-hole or piezometer tests fail to give satisfactory results.



$$K = \frac{Q \log_e (R/r)}{\pi (H^2 - h^2)}$$

Assume R = 500 x r for most cases.

(a) Pumping from a uniform unconfined stratum, water table in stratum being pumped.



$$K = \frac{Q \log_e (R/r)}{2\pi YD}$$

(b) Pumping from a confined stratum, water table above stratum being pumped.

K = Hydraulic conductivity, m^3/m^2/day (ft^3/ft^2/day)
Q = Flow rate at steady state conditions, m^3/day (ft^3/day)
Y = Drawdown from static water surface = H-h, m (ft)
H = Height of static water table above bottom of hole, m (ft)
h = Depth of water in hole at steady state pumping conditions, m (ft)
D= Flow thickness of strata between bottom of the hole and overlying (confining) stratum, m (ft)
R = Distance from centerline of well to point of zero drawdowm, m (ft)
r = Effective radius of well, m (ft)

Figure 3–11.—Determination of hydraulic conductivity by pumping from a uniform or confined stratum. 103–D–1630.

B. In-Place Hydraulic Conductivity Tests Above a Water Table

3–6. Objective.—The two methods that have been adapted for use in drainage investigations are the shallow well pump-in test and the ring permeameter test. These tests are used to determine the hydraulic conductivity rates of soils above a water table, and these rates are then used to predict the subsurface drainage requirements. To minimize extraneous effects on hydraulic conductivity, the water used in the tests must be free of sediment and should be warmer than the soil.

3–7. Shallow Well Pump-in Test for Hydraulic Conductivity.—*(a) Introduction.*—The shallow well pump-in test for hydraulic conductivity, also known as the well permeameter test, is used when the water table is below the zone to be tested. Essentially, this test consists of measuring the volume of water flowing laterally from a well in which a constant head of water is maintained. The lateral hydraulic conductivity determined by this test is a composite rate for the full depth of the tested hole.

(b) Equipment.—Equipment requirements for the shallow well pump-in test include the following items previously described for the auger-hole test in section 3–2: 75- and 100-millimeter (3- and 4-inch nominal) diameter soil augers, hole scratcher, perforated casing, burlap, and wristwatch with a second hand. Additional equipment items are:

(1) Water-supply tank truck of at least 1,200-liter (350-gallon) capacity with gasoline-powered water pump.

(2) Calibrated head tank, 200-liter (50-gallon) minimum. This tank should have fittings so that two or more tanks can be connected when required.

(3) Eight meters (25 feet) of 25- to 50-millimeter (1- to 2-inch), heavy-walled hose for rapid filling of head tank from supply tank.

(4) Wooden platform to keep head tank off the ground and to prevent rusting.

(5) A 25-millimeter (1-inch) diameter pipe 1 meter long to be driven into the ground and wired to head tank to keep tank in position.

(6) Constant-level float valve (carburetor) which must fit inside the casing.

(7) A rod threaded to fit the threads on top of the carburetor, used to regulate the depth that the float valve is lowered into the hole.

(8) Sufficient 10- or 12.5-millimeter (3/8- or 1/2-inch) i.d. flexible rubber tubing to connect tank to carburetor.

(9) Plexiglass cover, 300 by 300 millimeters (12 by 12 inches) by 3 millimeters (1/8 inch) thick, with hole in center for carburetor rod, and two other holes, one for rubber tubing and one for measuring water level and temperature of water in the hole.

(10) Filter tank and filter material.

(11) Steel fenceposts with post driver, four required per site. Approximately 25 meters of fencing wire (needed only when site must be fenced).

(12) Thermometer which can be lowered into hole, Celsius scale preferred.

(13) Three-meter (10-foot) steel tape, clipboard, computation sheet, and a 40-centimeter (16-inch) tiling spade.

Figure 3–12 shows a schematic of the equipment set up for this test.

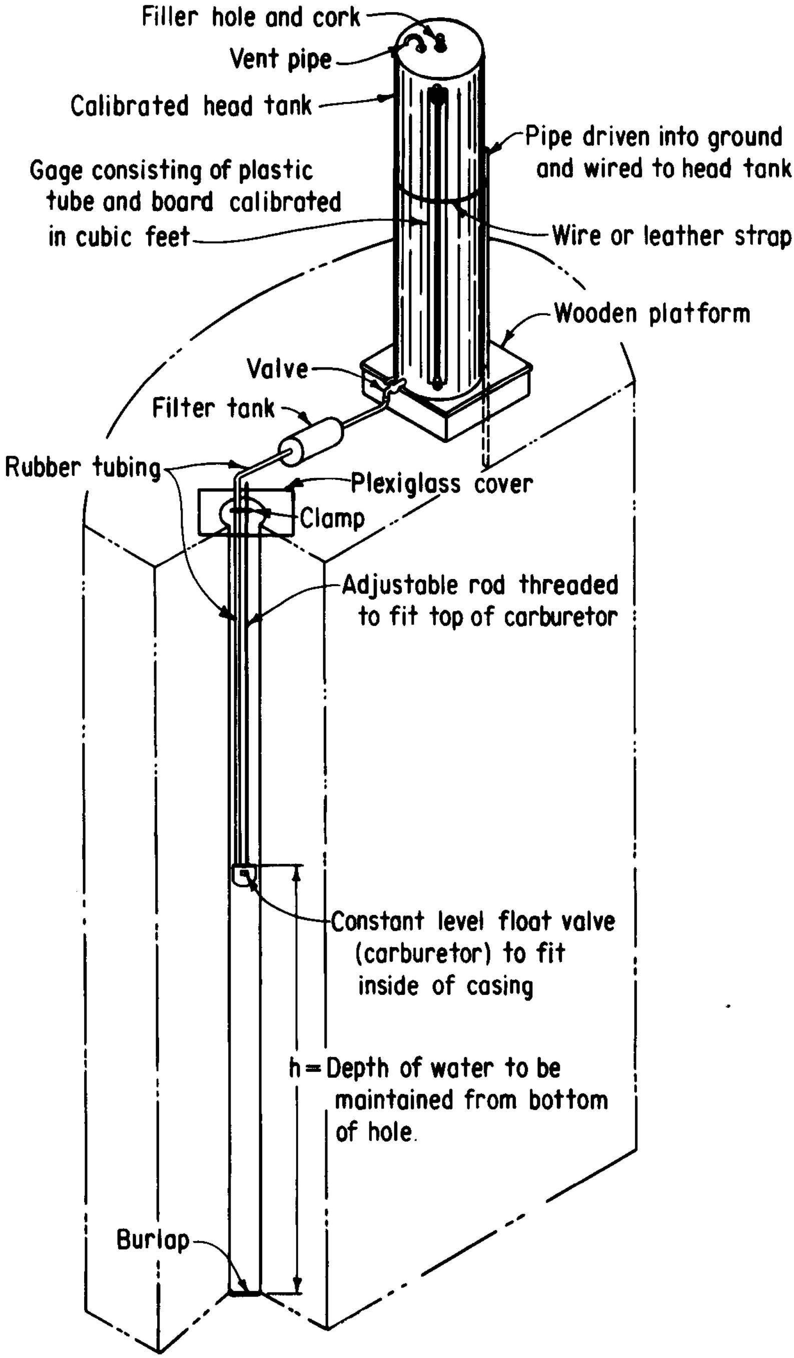


Figure 3–12.—Equipment setup for a shallow well pump-in test. 103–D–655.

The constant-level float valve (carburetor) suggested for use in this test and in the ring permeameter test, described later, can be constructed out of various materials and can be made in different shapes. The only requirements are that it must fit inside a 100-millimeter (4-inch) diameter hole, have adequate capacity, cause minimum aeration of water, and control the water level within plus or minus 15 millimeters. Material to construct a carburetor that has proven satisfactory consists of the following:

(1) One-half meter (20 inches) of 20- by 3-millimeter (3/4- by 1/8-inch) metal strap,

(2) One large tractor carburetor, needle valve, a needle valve seat at least 3 millimeters (1/8 inch) in diameter, a float made of styrofoam,

(3) Two 20- by 6-millimeter (3/4- by 1/4-inch) bushings, and

(4) One 20-millimeter (3/4-inch) coupling.

A photograph of a typical carburetor is shown on figure 3–13.

(c) Procedure.—A two-man team can efficiently install the equipment and conduct the shallow well pump-in test. The hole for the test should first be hand augered with a 75-millimeter (3-inch nominal) diameter auger and then reamed with the 100-millimeter (4-inch nominal) diameter auger. A complete log, including texture, structure, mottling, and color, should be obtained for use in interpreting and projecting results. The hole should be carefully scratched after completion to the desired depth to break up any compaction caused by the 100-millimeter auger and to remove any loose material on the sides. In unstable soils, a thin-walled perforated casing should be installed, with perforations extending from the bottom of the hole up to the predetermined controlled water level. A commercial well screen or slotted-PVC casing should be used, but when not available, a 100-millimeter (4-inch nominal) diameter, thin-walled casing with about 180 uniformly spaced, hand-cut perforations per meter, 3 millimeters wide by 25 millimeters long (1/8 inch wide by 1 inch long), will be satisfactory for most soils.

The constant-level float valve should be installed and approximately positioned. The float valve is then connected with tubing to the head tank, which is on an anchored platform beside the hole. The 10- or 12.5-millimeter (3/8- or 1/2-inch) tubing will allow sufficient water to flow into the carburetor when testing moderately permeable soils. The hole should then be filled with water to approximately the bottom of the carburetor. The valve on the head tank is then opened, and the height of the carburetor is carefully adjusted to maintain the desired water level. The plexiglass cover will keep small animals and debris out of the hole, hold the carburetor float adjusting rod, and allow observation of the carburetor during the test. The time and the reading on the tank gauge are recorded after everything is operating satisfactorily. The tank should be refilled when necessary. Each time the test site is visited, a record should be kept of the time,

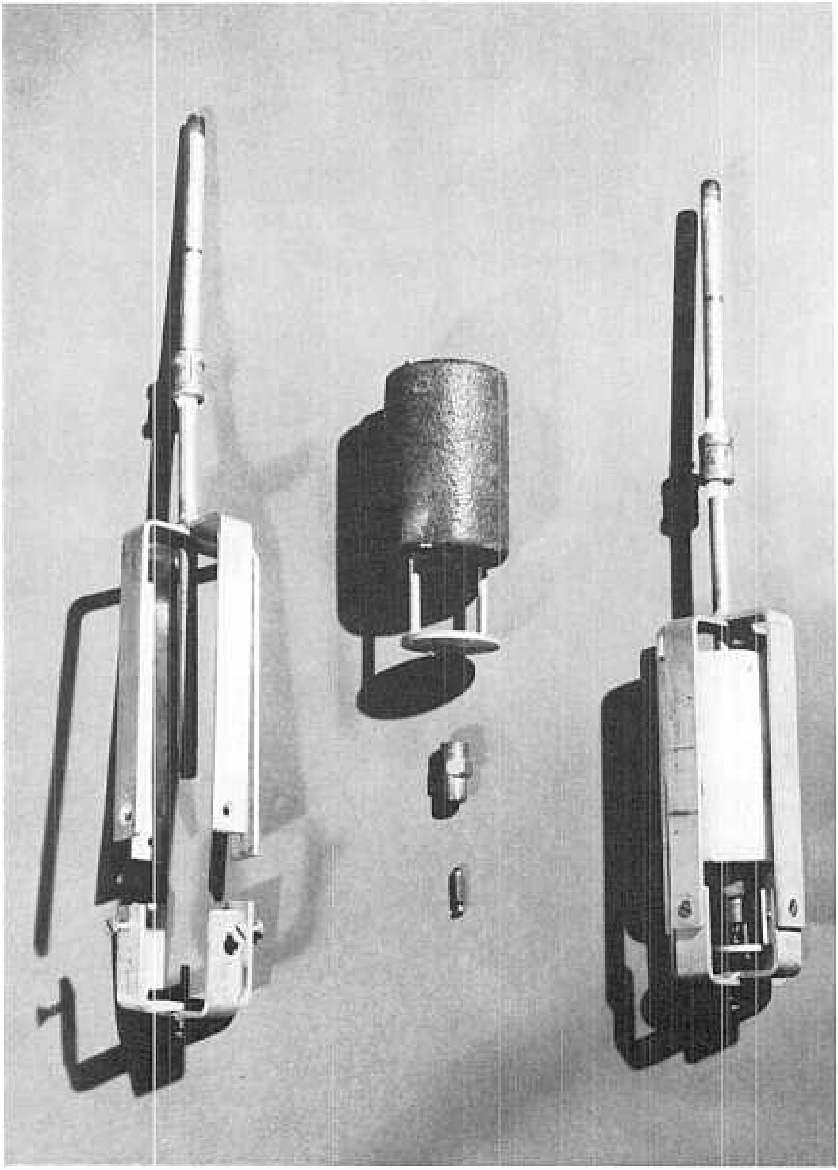


Figure 3–13.—Typical constant-level float valve used in hydraulic conductivity tests. Fully assembled float valve is shown on the right. P801–D–77013.

tank gauge readings, and volume of water added. Reading times are determined by the type of material being tested and will range from 15 minutes to 2 hours. Although not a necessity, the use of automatic recorders is desirable so that a complete record may be kept of water movement into the hole. When water temperature fluctuations exceed 2 °C, viscosity corrections should be applied.

If the test water contains suspended material, a filter tank should be installed between the head tank and the carburetor. Polyurethane foam is a satisfactory filter material. In-line milk filter socks have also been used successfully. Figure 3–14 shows a typical filter tank and material.

The nomographs shown on figures 3–15a and 3–15b are used to estimate the minimum and maximum volume of water to be discharged during a pump-in hydraulic conductivity test. These nomographs provide an excellent guide to determine the amount of water that should be discharged into the hole before the readings become unreliable. The nomographs are especially useful in sands because the minimum amount of water will be discharged into the hole in a very short time. Readings should be taken as soon as the minimum is reached. To use the nomographs, the specific yield must be estimated from the hydraulic conductivity, texture, and structure of the soil. Knowing the depth of water maintained from the bottom of the hole, h, and the radius of the hole, r, the minimum and maximum amounts of water needed to meet the conditions set up in the mathematical model can be determined. When the minimum amount has been discharged into the soil, the hydraulic conductivity should be computed following

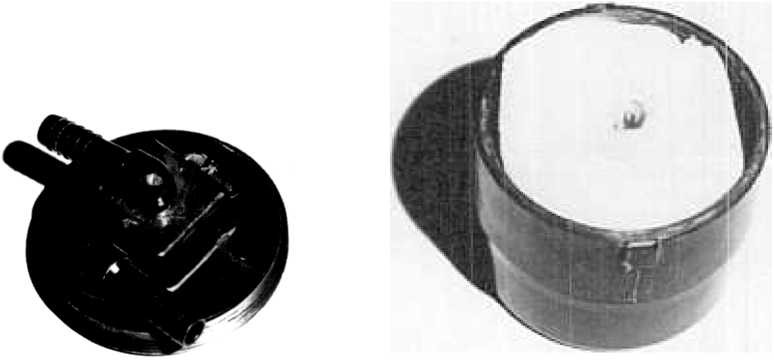
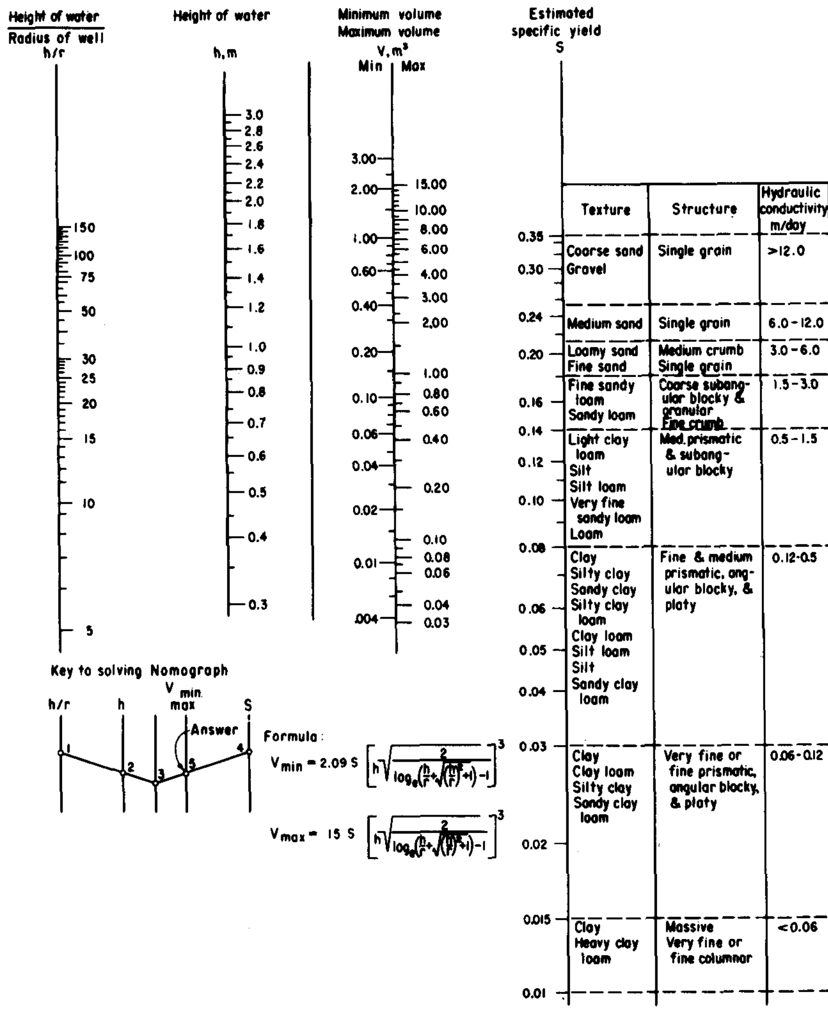


Figure 3–14.—Typical filter tank and filter material. P801–D–77014.



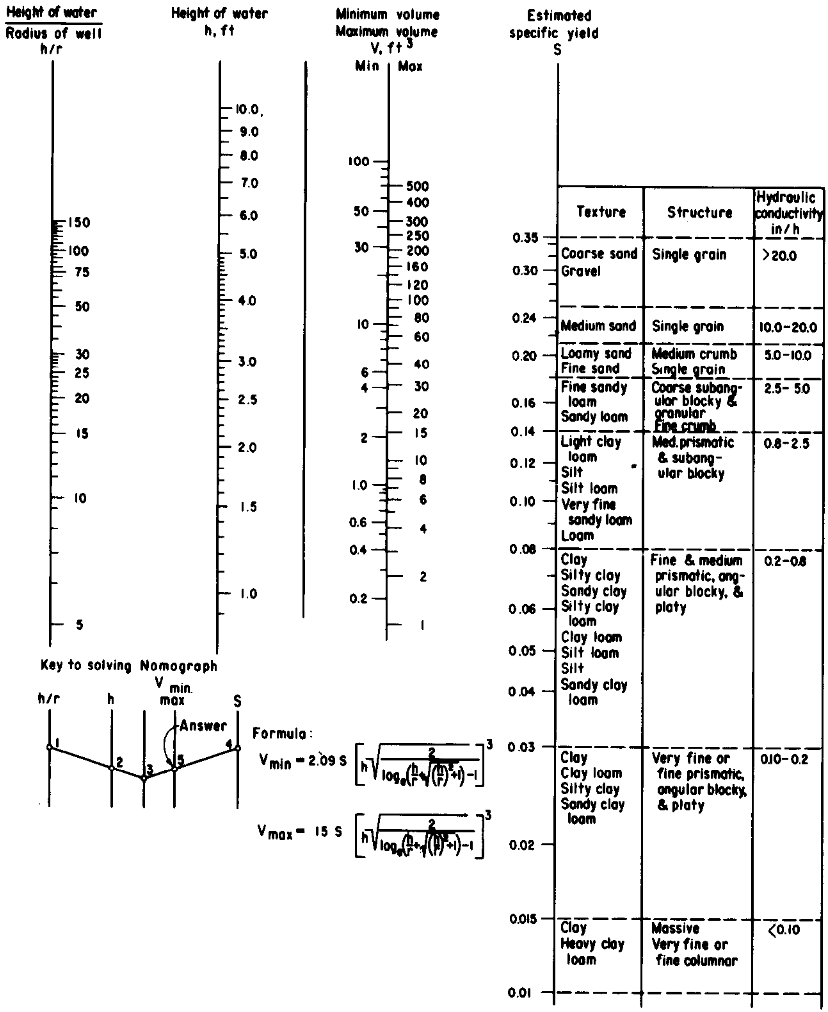
Texture	Structure	Hydraulic conductivity m/day
Coarse sand, Gravel	Single grain	>12.0
Medium sand	Single grain	6.0 - 12.0
Loamy sand, Fine sand	Medium crumb, Single grain	3.0 - 6.0
Fine sandy loam, Sandy loam	Coarse subangular blocky & granular, Fine crumb	1.5 - 3.0
Light clay loam, Silt, Silt loam, Very fine sandy loam, Loam	Med. prismatic & subangular blocky	0.5 - 1.5
Clay, Silty clay, Sandy clay, Silty clay loam, Clay loam, Silt loam, Silt, Sandy clay loam	Fine & medium prismatic, angular blocky, & platy	0.12-0.5
Clay, Clay loam, Silty clay, Sandy clay loam	Very fine or fine prismatic, angular blocky, & platy	0.06-0.12
Clay, Heavy clay loam	Massive, Very fine or fine columnar	<0.06

Formula:

$$V_{min} = 2.09\,S\left[h\sqrt{\frac{2}{\log_e\left(\frac{h}{r}+\sqrt{\left(\frac{h}{r}\right)^2+1}\right)-1}}\right]^3$$

$$V_{max} = 15\,S\left[h\sqrt{\frac{2}{\log_e\left(\frac{h}{r}+\sqrt{\left(\frac{h}{r}\right)^2+1}\right)-1}}\right]^3$$

Figure 3–15a.—Nomograph for estimating the minimum and maximum volume of water to be discharged during a pump-in hydraulic conductivity test (metric units). 103–D–1193.



Formula:

$$V_{min} = 2.09\,S\left[h\sqrt{\frac{2}{\log_e\left(\frac{h}{r}+\sqrt{\left(\frac{h}{r}\right)^2+1}\right)-1}}\right]^3$$

$$V_{max} = 15\,S\left[h\sqrt{\frac{2}{\log_e\left(\frac{h}{r}+\sqrt{\left(\frac{h}{r}\right)^2+1}\right)-1}}\right]^3$$

Texture	Structure	Hydraulic conductivity in/h
Coarse sand Gravel	Single grain	>20.0
Medium sand	Single grain	10.0–20.0
Loamy sand Fine sand	Medium crumb Single grain	5.0–10.0
Fine sandy loam Sandy loam	Coarse subangular blocky & granular Fine crumb	2.5–5.0
Light clay loam Silt Silt loam Very fine sandy loam Loam	Med. prismatic & subangular blocky	0.8–2.5
Clay Silty clay Sandy clay Silty clay loam Clay loam Silt loam Silt Sandy clay loam	Fine & medium prismatic, angular blocky, & platy	0.2–0.8
Clay Clay loam Silty clay Sandy clay loam	Very fine or fine prismatic, angular blocky, & platy	0.10–0.2
Clay Heavy clay loam	Massive Very fine or fine columnar	<0.10

Figure 3–15b.—Nomograph for estimating the minimum and maximum volume of water to be discharged during a pump-in hydraulic conductivity test (U.S. customary units). 103–D–1631.

each reading. The test can be terminated when a relatively constant hydraulic conductivity value has been reached, and the total volume discharged into the soil is not greater than the maximum value taken from the nomograph.

(d) Calculations.—A sample computation sheet for the shallow well pumping test is shown on figure 3–16. Figures 3–17a, 3–17b, 3–18a, and 3–18b show equations and nomographs used in the computations. The use of these figures depends upon the depth of water maintained from the bottom of the hole, h, and the depth of the water table or depth to an impervious strata from the surface of water maintained, T_u. The h value can be determined accurately, but the depth to an impervious or restrictive zone, T_u, requires a deep pilot hole near the test site. Any zone which appears, from visual inspection, to have a much lower hydraulic

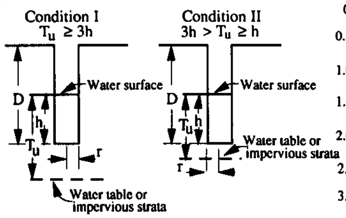
Locaton: Hole C-3--Sample Farm
Obserfer: A.P.B. Date: October 8, 1974

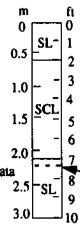
D = 1.83 meters (6.0 feet)
r = 0.051 meters (0.167feet)

Water table or impervious strata = 2.13 meters (7.0 feet) below ground surface

T_u= 1.37 meters (4.5 feet) Depth of water table or impervious strata from surface of water maintained

h = 1.07 meters (3.5 feet) Depth of water maintained from bottom of hole



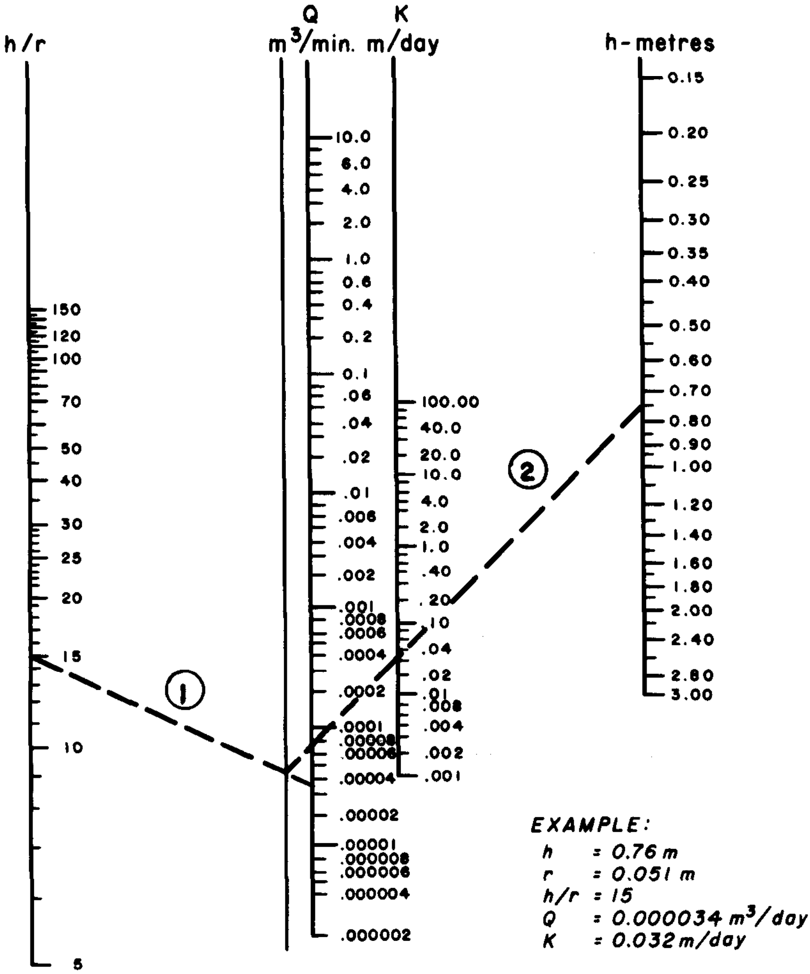


0.0 to 0.6m (0 to 2 ft) - Light Sandy Loam, friable, non-sticky
0.6 to 2.1m (2 to 7 ft) - Light grayish brown Sandy Clay Loam, friable, slight stickiness, damp at about 2.1m (7 ft) Fair hydrualic conductivity.
Slight compaction at 1.8 to 2.1m (6 to 7 ft)
Water table 2.19m (7.2 ft)
2.1 to 3.0m (7 to 10 ft) - Light Brown Sandy Loam, friable, good hydraulic conductivity, nonsticky.

Initial		Final		Time	Tank Reading				Q,		Temp. of water, C	Viscosity of water, Centipoise	Adjusted Q,		K,	
					Initial		Final									
Date	Time	Date	Time	min	m^3	ft^3	m^3	ft^3	m^3/min	ft^3/min			m^3/min	ft^3/min	m/d	in /hr
10 - 8	0800	10 - 8	1100	180	0	0	0.173	6.12	0.000963	0.034	16	1.1111	0.000708	0.025	0.73	1.20
10 - 8	1100	10 - 8	1400	180	0	0	0.169	5.97	0.000939	0.033	18	1.0559	0.000481	0.017	0.49	0.80
10 - 8	1400	10 - 8	1800	240	0	0	0.170	6.00	0.000708	0.025	Note: Connected two barrels					
10 - 8	1800	10 - 9	0530	690	0	0	0.351	12.41	0.000509	0.018	for greater capacity.					
10 - 9	0530	10 - 9	1130	360	0	0	0.193	6.82	0.000536	0.019	16	1.1111	0.000536	0.019	0.52	0.85
10 - 9	1130	10 - 9	1800	390	0	0	0.217	7.65	0.000556	0.020	19	1.0299	0.000515	0.018	0.50	0.82
10 - 9	1800	10-10	0530	690	0	0	0.343	12.10	0.000497	0.018	13	1.2028	0.000538	0.019	0.52	0.85
10-10	0530	10-10	1130	360	0	0	0.188	6.63	0.000522	0.018	15	1.1404	0.000535	0.019	0.52	0.85

Adjusted to average tank water temperature. -- see Figure 3 - 20 for method.
Remarks: No trouble with apparatus, assumed test satisfactory and results reliable.
Calculation: h/r = 1.07/0.051 = 20.96 h/T_u = 1.07/1.37 = 0.78
Q (average after stabilization) = 0.000536 cubic meter (0.019 cubic feet) per minute
3h (or 3 x 1.07m) > T_u (1.37m) > h(1.07m), so use Conditon II.
From nomograph (Fig. 3 - 18a&b) : K = 0.52 meter per day (0.85 in per hour)

Figure 3–16.—Data and computation sheet on shallow well pump-in test for hydraulic conductivity. 103–D–467.

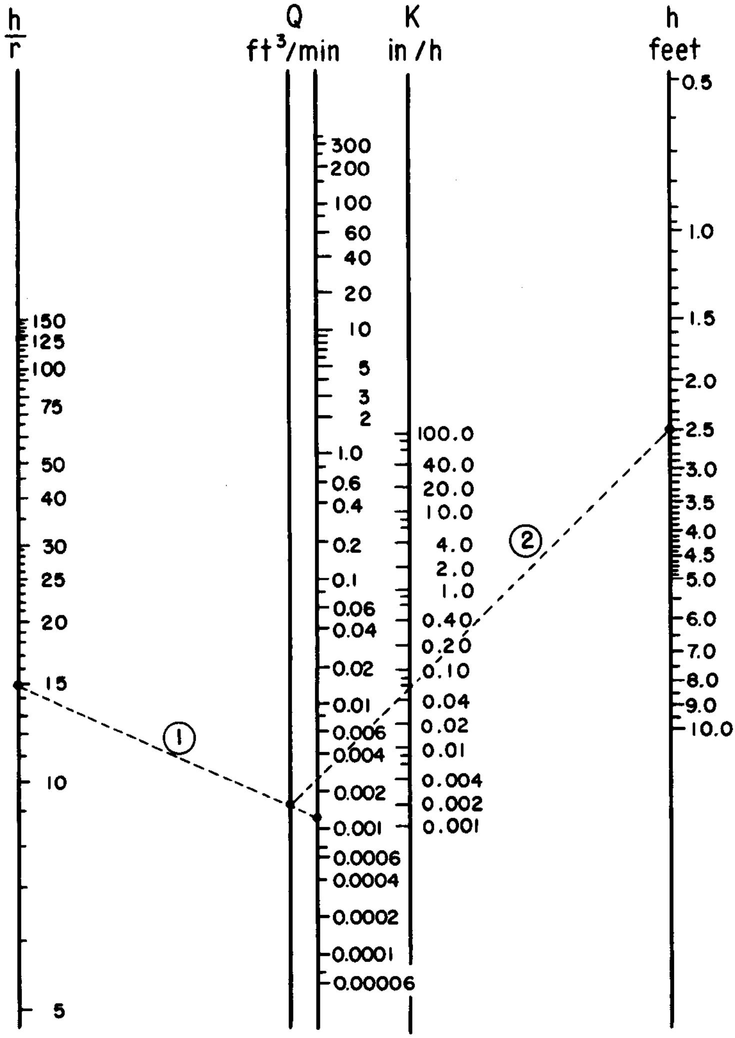


CONDITION I

$$T_u \geq 3h$$

$$K\ (\mathrm{m/day}) = 1440\ \frac{\left[\log_e\left(\frac{h}{r}+\sqrt{\left(\frac{h}{r}\right)^2+1}\right)-1\right]Q}{2\pi h^2}$$

Figure 3-17a.—Nomograph for determining hydraulic conductivity from shallow well pump-in test data for condition I (metric units). 103-D-1191.



Example:

$h = 2.5$ ft

$r = 0.167$ ft

$h/r = 15$

$Q = 0.0012$ ft^3/min

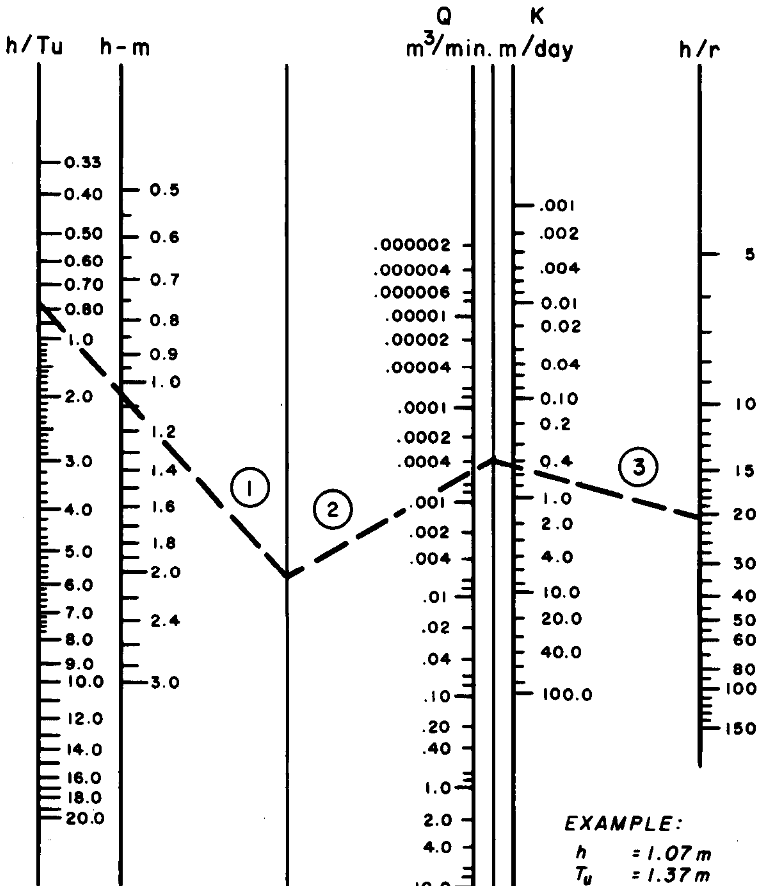
$K = 0.06$ in/h

CONDITION I

$T_u \geq 3h$

$$K = \frac{720\left[\log_e\left(\frac{h}{r} + \sqrt{\left(\frac{h}{r}\right)^2 + 1}\right) - 1\right]Q}{2\pi h^2}$$

Figure 3–17b.—Nomograph for determining hydraulic conductivity from shallow well pump-in test data for condition I (U.S. customary units). 103–D–657.



EXAMPLE:

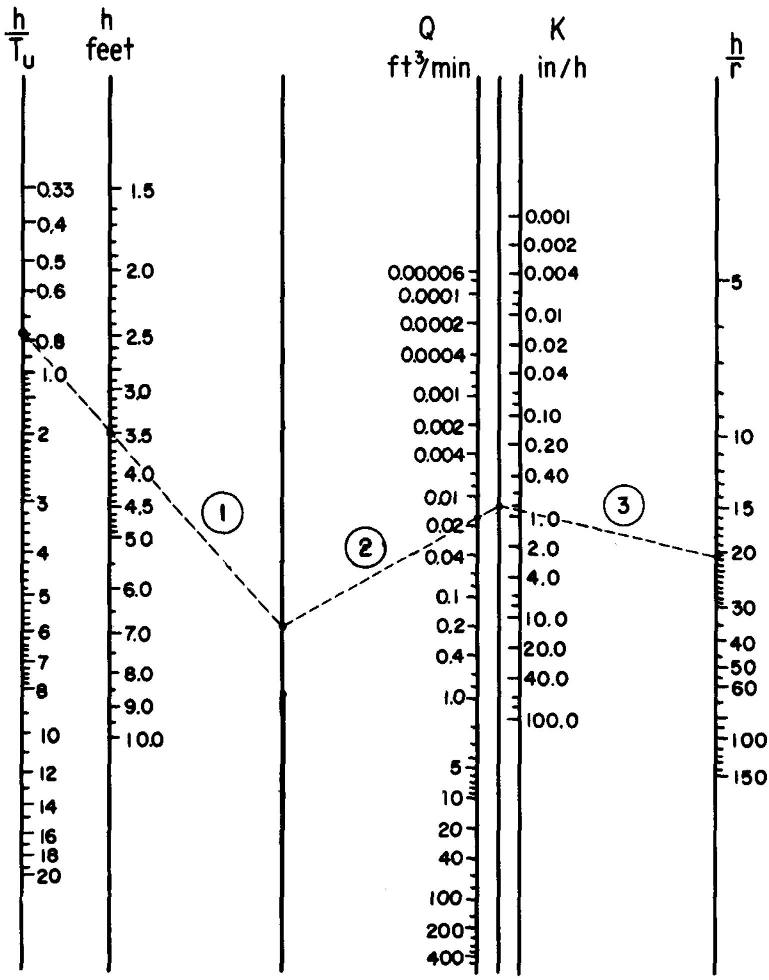
h = 1.07 m
T_u = 1.37 m
h/T_u = 0.78
Q = 0.00054 m³/min.
r = 0.051 m
h/r = 20.96
K = 0.52 m/day

CONDITION II

$3h \geq T_u \geq h$

$$K = 1440 \left[\frac{3 \log_e \frac{h}{r}}{\pi h (h + 2T_u)} \right] Q$$

Figure 3–18a.—Nomograph for determining hydraulic conductivity from shallow well pump-in test data for condition II (metric units). 103–D–1192.



Example:
$h = 3.5$ ft
$T_U = 4.5$ ft
$h/T_U = 0.78$
$Q = 0.019\ ft^3/min$
$r = 0.167$ ft
$h/r = 20.96$
$K = 0.80$ in/h

CONDITION II

$3h \geq T_U \geq h$

$$K = 720\left[\frac{3 \log_e \frac{h}{r}}{\pi h(h+2T_U)}\right] Q$$

Figure 3–18b.—Nomograph for determining hydraulic conductivity from shallow well pump-in test data for condition II (U.S. customary units). 103–D–657.

conductivity than the zone above should be considered as a restrictive zone for determining T_u. A water table should also be considered a barrier when estimating T_u. If an in-place hydraulic conductivity test in this zone indicates the zone is not restrictive, the hydraulic conductivity can be recomputed using a larger T_u value and the appropriate equation or nomograph.

(e) Limitations.—The time required to set up the equipment and complete the test constitutes the principal limitation of this test. Also, a relatively large amount of water is required, especially if the material has a hydraulic conductivity over 4 to 6 centimeters per hour. In soils high in sodium, the water used should contain 1,500 to 2,000 milligrams per liter of salts, preferably calcium. Rocky material or coarse gravels may prevent augering the hole to accurate dimensions. Also, comparisons of electric analog test results with values from the auger-hole test show that the h/r ratio must be equal to or greater than 10.

Water moving outward from the hole sometimes causes the fines near the surface to form a seal before a constant hydraulic conductivity rate has been reached. If a constant rate cannot be obtained by the time the estimated maximum flow has occurred, the fines can be flushed back into the hole by removing the equipment and bailing all water out of the hole or by gently surging the hole with a solid surge block and then pumping the water out. This procedure is not always successful, but should be tried before abandoning the test site. Use of a filter on the supply line will generally prevent this problem.

3-8. Ring Permeameter Test.—*(a) Introduction.*—In drainage studies, the lateral hydraulic conductivity of the soil must be known to determine drain spacing. Usually the vertical hydraulic conductivity is assumed to be sufficient to permit deep percolation from irrigation and rainfall to reach the saturated zone in which it moves horizontally. However, slowly permeable layers interfere with percolation and cause temporary perched water tables in the root zone. Thus, a means of determining the vertical hydraulic conductivity of such a tight layer is desirable.

The ring permeameter test is a specialized in-place method of obtaining vertical hydraulic conductivity of a critical zone. The test is based on Darcy's law for movement of liquids through saturated material. The test is time consuming when compared with the auger-hole test, but the results are uniformly dependable. Tensiometers and piezometers are used to confirm existence of saturated conditions, absence of a perched water table, and fulfillment of the requirements of Darcy's law.

(b) Equipment.—Equipment required for the ring permeameter method is as follows:

(1) A 14-gauge-steel, welded-seam cylinder, 457-millimeter (18-inch) i.d. by 508 millimeters (20 inches) high, with a reinforcing band on top and sharpened bottom edge (seam weld must be ground flush).

(2) A 508-millimeter (20-inch) diameter by 12.7-millimeter (1/2-inch) thick driving disk with a 450-millimeter (17-3/4-inch) diameter by 12.7-millimeter (1/2-inch) thick center ring. This disk fits inside the 457-millimeter

cylinder and has a 0.6-meter (2-foot) length of 25-millimeter (1-inch) pipe welded in the center for a hammer guide.

(3) A 25- to 35-kilogram (50- to 75-pound) driving hammer (heavy steel cylinder with hole in the center and pipe welded to center which fits over the 25-millimeter (1-inch) pipe on driving disk).

(4) A water-supply tank truck of at least 1,250-liter (350-gallon) capacity and a gasoline-powered water pump to fill the tank truck. Also, about 7 meters (25 feet) of 25- to 38-millimeter (1- or 1-1/2-inch), heavy-walled hose are needed to fill the tank from the water truck.

(5) Two calibrated 200-liter (50-gallon) head tanks.

(6) Two wooden platforms to keep head tanks from rusting.

(7) Two 25-millimeter (1-inch) diameter pipes 1 meter (4 feet) long, driven into the ground to keep tanks upright.

(8) Sufficient 10-millimeter (3/8-inch) i.d. rubber tubing to connect tanks to constant-level float valves (carburetors).

(9) Two constant-level float valves (carburetors).

(10) Adjustable rods to hold the carburetors at the desired elevation and threaded bolts which fasten to the steel cylinder and support the adjustable rods.

(11) Two 13-millimeter (1/2-inch) i.d. piezometers, 450 millimeters (18 inches) long, rigid copper tubing, and a small driving hammer to fit over the 13-millimeter tubing.

(12) An 11-millimeter (7/16-inch) wood auger for cleaning out piezometers and clean sand to fill cavities in piezometers.

(13) Bentonite to seal tensiometers and piezometers.

(14) Two mercury manometer-type tensiometers and mercury for them.

(15) Distilled water to fill tensiometers initially. (Distilled water is desirable but unnecessary after initial filling.)

(16) Small air syringe to fill tensiometers and expel air after filling.

(17) A 25-millimeter (1-inch) wood auger for installing tensiometers.

(18) Thermometer, Celsius preferred.

(19) Filter tank and filter material.

(20) Tiling spade to clean the hole, and a rope bucket for removing soil from hole.

(21) A 3-meter ladder (needed only for deep layer testing).

(22) Washed sand of uniform size, passing the No. 14 sieve and retained on the No. 28 sieve.

(23) Cover for the 457-millimeter (18-inch) cylinder to reduce evaporation and keep out debris.

(24) Steel fenceposts with post driver (four required per site and needed only when site must be fenced). Wire for fencing site, about 25meters.

(25) A 3-meter (10-foot) steel tape, carpenter's level, white chalk, clawhammer, wire-cutting pliers, clipboard, and reference sheets.

Figure 3–19 shows the equipment set up for this test.

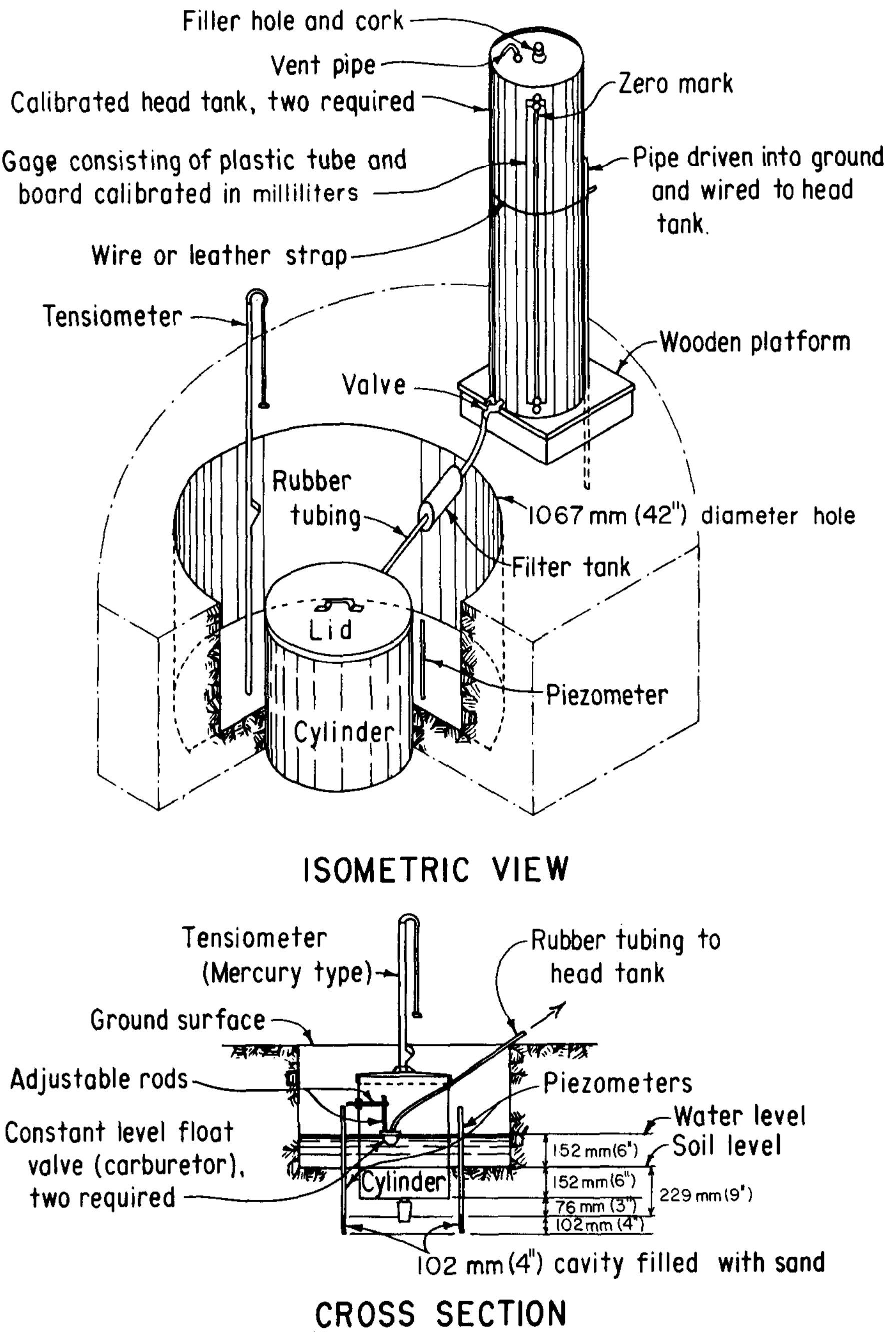


Figure 3–19.—Equipment setup for the ring permeameter hydraulic conductivity test. 103–D–658.

(c) Procedure.—A two-man team can efficiently install the equipment and conduct the ring permeameter test. After the site has been selected and the zone of critical hydraulic conductivity determined, a 1-meter-diameter hole is excavated to within 75 millimeters (3 inches) of the test zone. The last 75 millimeters are excavated when the equipment is ready for installation, taking care not to walk on the area to be tested. The testing area, which will be inside the 18-inch cylinder, is checked with a carpenter's level to assure that it is level before the cylinder is placed. The cylinder is marked with chalk 150 millimeters (6 inches) from the bottom edge and driven 150 millimeters into the soil with the driving disk and hammer. The cylinder should be kept level during driving, and the blows should be as powerful and steady as practicable. After the cylinder has been driven to the desired depth, the soil immediately against its inside and outside wall is tamped lightly to prevent channeling along the sides. About 25 millimeters of clean, uniform, permeable sand is spread over the area inside the cylinder to minimize puddling of the soil surface during the test. The outside periphery of the cylinder is also tamped to keep water from channeling down along the sides and causing erroneous tensiometer readings.

Next, the two 450-millimeter (18-inch) piezometers are marked 230 millimeters (9 inches) from the sharpened bottom and installed on opposite sides of the cylinder and about 75 to 100 millimeters (3 to 4 inches) distant from it. The piezometers are installed by driving them 50 to 75 millimeters into the soil, augering out the core, and continuing this process until the 230-millimeter (9-inch) mark is at ground level. Care should be taken that the piezometers do not turn or come up with the auger during installation. A 100-millimeter (4-inch) long cavity is then augered below each piezometer and filled with clean, fine sand. As an additional means of preventing channeling along the sides, a 1:1 bentonite-soil mixture is tamped around the piezometers. Caution should always be exercised to ensure that no bentonite falls into the piezometers or into the testing ring. The piezometers are filled with water and checked to assure that they are functioning properly. If the water falls in the piezometers, the installation is satisfactory. A small can should be placed over each piezometer to keep out dirt and water during the remainder of the installation. If the water does not fall, the piezometers should be flushed with a stirrup pump and reaugered if flushing does not clear them.

The two calibrated and tested tensiometers are then installed on opposite sides of the cylinder and 75 to 100 millimeters (3 to 4 inches) from it on a line at right angles to that of the piezometers. The calibration and testing should be done in the laboratory. Instructions for calibrating and testing can be obtained from the manufacturer. During the calibration, 100 on the scale should be set at zero tension so that pressures caused by a rising water table can be observed if the water table rises above the tensiometer cup. The holes for the tensiometers are excavated with a 25-millimeter (1-inch) soil auger to a depth of 230 millimeters (9 inches). A small amount of dry soil is then dropped into the hole, followed by a small amount of water. The tensiometer is then placed in the hole, with the glass tubes facing away from the sun, and worked up and down in the mud to obtain good contact between the porous cup, the mud, and the undisturbed soil. The annular space around the tensiometer is filled and tamped with dry soil to within about 25 millimeters (1 inch) of the soil surface. A 1:1 bentonite-soil mixture is then added to prevent channeling. Mercury is placed in the reservoir cup and the tensiometer tubes filled with water. A small air syringe is used to remove air from the tensiometer tube by forcing water through the system.

The carburetor float apparatus is installed and adjusted to hold a constant 150-millimeter (6-inch) head in the cylinder, and the carburetor is connected to the head tank with rubber tubing. If the test water contains suspended material, a filter tank should be installed with the tubing as described in section 3–7. The tank should always be anchored, and the gauge should always face away from the sun. The cylinder is then filled with water to the 150-millimeter (6-inch) mark and the tank valve opened. The hole outside the cylinder should also be filled with water to a depth of 150 millimeters (6 inches) and should be kept to this 150-millimeter (6-inch) depth during the entire test period. The extra tank and carburetor are used for this purpose. When all adjustments have been made and the tensiometers are full, the time and water content of the tank are recorded.

The head tank should be checked at least two or three times a day, depending upon the percolation and hydraulic conductivity rates, and filled as necessary. Each time the site is visited, a record should be made of the time, volume of water in the tank, gauge readings of the tensiometers and piezometers, temperature, and the hydraulic conductivity. When the tensiometer gauges read approximately 100 (zero tension), no water shows in the piezometer, and water is moving through the 150-millimeter (6-inch) test layer at a constant rate, the requirements of Darcy's law may be assumed to have been met and valid test results can be obtained to calculate hydraulic conductivity. Tensiometer readings sometimes fluctuate when the soil is at or near saturation, and it is not always possible to get the 100 reading. Gauges fluctuating between 100 and 105 are probably indicating saturated conditions for that particular soil. Also, it is not necessary for both tensiometers to have the same reading providing they both read in the 100 to 105 range.

If the saturated front should reach a zone less permeable than the test layer before the requirements of Darcy's law are met, a mound of water will build up into the test zone. When this buildup occurs, the hydraulic gradient will be less than unity, and the pressure at the base of the soil column being tested will be greater than atmospheric. Both the piezometers and tensiometers will indicate this condition. When the piezometers show that a mound has reached the bottom of the cylinder, the test will no longer give a true hydraulic conductivity value. When this condition occurs, the test will either have to be stopped or the mound lowered below the bottom of the cylinder. When the material between the bottom of the cylinder and the less permeable zone has a fair rate of hydraulic conductivity, it is sometimes possible to lower the water table mound by augering a number of holes around the outside periphery of the cylinder approximately 250 millimeters (10 inches) from the sides. These holes, when filled with sand, will act as inverted drainage wells and, under most conditions, will lower the mound. If the holes do not provide the necessary drainage, the testing equipment should be lowered to the less permeable zone and the test rerun.

At the close of the test, the soil is excavated from around the outside of the cylinder and cut for a short distance under the cylinder. A chain placed around the cylinder and pulled by a truck will usually break the soil across the bottom to allow examination for root holes, cracks, and possible channeling.

(d) Calculations.—Hydraulic conductivity computations for the ring permeameter test are made using the Darcy flow equation:

$$K = \frac{VL}{tAH} \tag{3}$$

where:

K = Hydraulic conductivity in centimeters (inches) per hour,
V = volume of water passed through the soil in cubic centimeters (inches),
A = cross-sectional area of the test cylinder in square centimeters (inches),
t = time in hours,
L = length of the soil column in centimeters (inches), and
H = height of the water level above the base of the ring in centimeters (inches).

Sample data sheets and computations are shown on figures 3–20a and 3–20b.

When fluctuations in the water temperature exceed 2 °C, viscosity adjustments should be made. This adjustment usually results in more uniform hydraulic conductivity values, and is illustrated on the sample data sheets, figures 3–20a and 3–20b.

(e) Limitations.—The principal limitation in this test is that the material directly below the test zone must have equal or greater hydraulic conductivity than the test zone. Also, it must extend to a sufficient depth below the test zone so that a steady-state flow is reached for at least three consecutive hourly readings before any water mound builds up to the bottom of the cylinder. Another limitation is the presence of progressively tighter soils below the test zone. A steady-state flow is never reached under this condition, and the hydraulic conductivity apparently decreases as the test proceeds.

Unreliable data may result when the test zone is immediately above a thick, very permeable material. A fairly steady-state flow can be obtained, but the tensiometers in the very permeable material will never indicate zero tensions below the test zone and, thus, the requirements of Darcy's law are not met.

This test cannot be used in rocky or coarse gravel materials because the cylinder cannot be driven into such material without allowing channeling along the inside periphery of the ring during the test.

Initial		Final		Time	Tank reading cm^3		V	Q	[1] Temp of Water	[2] Viscosity of Water	[3] Adj Q	K	[4] Tensiometer Readings		Piezometer Readings	
Date	Time	Date	Time	hours	Initial	Final	cm^3	$\frac{cm^3}{hr}$	^0C	Centipoise	$\frac{cm^3}{hr}$	cm/hr	N-side	S-side	E-side	W-side
10-13-74	0800	10-13	1212	4.20	0	5932	5932	1412	17	1.0828	1376		145	153	dry	dry
10-13-74	1212	10-13	1630	4.30	5932	11831	5899	1372	19	1.0229	1272		138	142	dry	dry
10-13-74	1630	10-14	0725	14.92	11831	28546	16715	1121	13	1.2028	1213		135	138	dry	dry
10-14-74	0725	10-14	1235	5.17	28546	34576	6031	1167	16	1.1111	1167		131	133	dry	dry
10-14-74	1235	10-14	1635	4.00	34576	39296	4720	1180	18	1.0559	1121		122	127	dry	dry
10-14-74	1635	10-15	0750	15.25	39296	54798	15505	1016	14	1.1709	1072		117	117	dry	dry
10-15-74	0750	10-15	1215	4.42	0	4605	4605	1042	16	1.1111	1042	0.32	111	113	dry	dry
10-15-74	1215	10-15	1710	4.92	4605	9603	4998	1016	19	1.0299	942	0.29	108	109	dry	dry
10-15-74	1710	10-16	0735	14.42	9603	22663	13060	906	12	1.2363	1008	0.31	103	105	dry	dry
10-16-74	0735	10-16	1210	4.58	22663	27219	4556	995	15	1.1404	1021	0.31	105	104	dry	dry
10-16-74	1210	10-16	1650	4.67	27219	32151	4932	1056	18	1.0559	1004	0.31	102	102	dry	dry
10-16-74	1650	10-17	0820	15.50	32151	46392	14241	919	13	1.2028	995	0.30	104	102	dry	dry

Notes: [1] This is the temperature of the water moving into the test zone and is measured in the test cyliner.

[2] To convert to pascal seconds, divide by 1000.

[3] Adjusted Q = Q times viscosity of water at test temperature divided by viscosity of water at temperature at which the water seemed to stabilize which in this test was 16°C.

(i.e. Adjusted Q (first time increment) = $1.412 \times \frac{1.0828}{1.1111} = 1376$ *(Adjusted to temperature of 16°C)*

Location: Hole D-2--Sample Farm Observer: A.P. Brown

Depth: 107 to 122 centimeters (42 to 48 inches)

Calculations: $K = \frac{VL}{tAH} = \frac{QL}{AH}$ (centimeters per hour)

Q = 1002 cubic centimeters per hour (Adjusted Q, average of last 6 time increments)

$A = \pi r^2 = \pi(0.2286m)^2 = 0.1642\ m^2 = 1,642\ cm^2$

L = 0.1524 meters = 15.24 centimeters

H = 0.3048 meters = 30.48 centimeters

Therefore: $K = \frac{(1002)(15.24)}{(1642)(30.48)}$ = 0.305 centimeters per hour (0.12 inches per hour)

[4] A tensiometer reading of 100 represents zero tension (atmospheric pressure)

Figure 3–20a.—Data and computation sheet on ring permeameter test for hydraulic conductivity (metric units). 103–D–659.

Initial		Final		Time, hours	Tank reading, in³		V, in³	Q, in³/hr	[1]Temp of water, °C	Viscosity of water, Centipoise	Adj Q, in³/hr	K, in/hr	Tensiometers		Piezometers	
Date	Time	Date	Time		Initial	Final							N-side	S-side	E-side	W-side
10-13-74	0800	10-13	1212	4.20	0	362	362	86.2	17	1.0828	84.0		145	153	dry	dry
10-13-74	1212	10-13	1630	4.30	362	722	360	83.8	19	1.0299	77.7		138	142	dry	dry
10-13-74	1630	10-14	0725	14.92	722	1742	1020	68.3	13	1.2028	74.0		135	138	dry	dry
10-14-74	0725	10-14	1235	5.17	1742	2110	368	71.2	16	1.1111	71.2		131	133	dry	dry
10-14-74	1235	10-14	1635	4.00	2110	2398	288	72.0	18	1.0559	68.5		122	127	dry	dry
10-14-74	1635	10-15	0750	15.25	2398	3344	946	62.0	14	1.1709	65.4		117	117	dry	dry
10-15-74	0750	10-15	1215	4.42	0	281	281	63.6	16	1.1111	63.6	0.12	111	113	dry	dry
10-15-74	1215	10-15	1710	4.92	281	586	305	62.0	19	1.0299	[2]57.5	0.11	108	109	dry	dry
10-15-74	1710	10-16	0735	14.42	586	1383	797	55.3	12	1.2363	61.6	0.12	103	105	dry	dry
10-16-74	0735	10-16	1210	4.58	1383	1661	278	60.7	15	1.1404	62.3	0.12	105	104	dry	dry
10-16-74	1210	10-16	1650	4.67	1661	1962	301	64.4	18	1.0559	61.3	0.12	102	102	dry	dry
10-16-74	1650	10-17	0820	15.50	1962	2831	869	56.0	13	1.2028	60.6	0.12	104	102	dry	dry

Notes: [1]This is the temperature of the water moving into the test zone and is measured in the test cylinder.

[2]Adjusted $Q = \frac{1.0299}{1.1111} \times 62.0 = 57.5$ (Adjusted to average tank water temperature of 16 °C which is the first reading after apparent stabilization)

Location: Hole D-2--Sample Farm Observer: A.P. Brown

Depth: 42 to 48 inches

Calculations: $K = \frac{VL}{tAH} = \frac{QL}{AH}$ (inches per hour)

$Q = 61.2$ cubic inches per hour average (Average for 48.5 hours)

$A = \pi r^2 = 3.1416 \times 9^2 = 254.5$ square inches

$L = 6$ inches

$H = 12$ inches

Therefore: $K = Q \times 0.00196 = 61.2 \times 0.001965 = 0.12$ inch per hour

Figure 3–20b.—Data and computation sheet on ring permeameter test for hydraulic conductivity (U.S. customary units). 103–D–659.

3-9. Test Pit Method.—*(a) Introduction.*—There is no exact method for determining the hydraulic conductivity above a water table in soils of coarse gravel and cobbles with matrices of finer materials. The following procedure, equations, and sample computations describe one method which is considered sufficiently accurate to give a reasonable hydraulic conductivity when applied to field problems.

The test pit can be of three different shapes: (1) a circular test pit of diameter a, (2) a square test pit with side dimensions of a, and (3) a rectangular test pit with side dimensions a by $2a$.

The test should be conducted in only one textural classification such as a cobbly, coarse gravelly, or loamy sand. A backhoe, power auger, or hand tools can be used to excavate down to the test zone. The test pit is then carefully excavated to the desired shape and depth by hand. For the different shaped pits, an a value of 0.3 meter (1 foot) should be adequate. Larger sizes can be used, but will require proportionally more water. Small cavities left when cobbles are removed, or a few small cobbles sticking out into the test pit, will cause little difference in the quantity of water entering the test pit, the average diameter of a circular pit, or in the side dimensions of a square or rectangular pit.

Matrices with textures such as fine sands, silts, silt loams, and very fine sands tend to slough into the pit when saturated. For these conditions, the pit should be filled with a clean (washed) fine gravel before water is applied.

(b) Procedure.—After the test pit has been excavated and, if required, backfilled with fine gravel, it is filled to a predetermined depth with clean water. All water entering the pit should be filtered to remove the suspended silts and clays. The depth of water in the hole can be maintained by using bypass hoses and a large carburetor for the finer regulation to keep the water depth reasonably constant. The carburetor can be installed by placing it in a perforated tin can located in the middle of the test pit. This test normally takes only a short time to run, so the water depth in the pit can be maintained by hand if a carburetor is not available. A clear plastic cover should be placed over the pit to keep material from blowing in.

(c) Calculations.—The following equation is used to compute the hydraulic conductivity:

$$K = \frac{1{,}440\,Q}{CaD} \qquad (4)$$

where:

K = hydraulic conductivity in meters (feet) per day,
a = diameter of a circular pit, the side dimension of a square pit, or the a dimension of a rectangular pit that is a by $2a$ all in meters (feet),
Q = quantity of flow per unit of time in cubic meters (feet) per minute,
D = depth of water maintained in the test pit in meters (feet), and
C = conductivity coefficient from the following tabulation:

Conductivity coefficient

$\frac{D}{a}$	Circular test pit of diameter a	Square test pit of dimension $a \times a$	Rectangular test pit of dimensions $a \times 2a$
1	1.50 (4.92)	1.67 (5.49)	2.24 (7.35)
2	2.11 (6.92)	2.34 (7.68)	3.01 (9.89)
3	2.68 (8.78)	2.96 (9.70)	3.71 (12.18)
4	3.25 (10.65)	3.54 (11.63)	4.40 (14.44)
5	3.78 (12.39)	4.13 (13.54)	5.06 (16.59)
6	4.29 (14.09)	4.67 (15.33)	5.68 (18.62)
7	4.84 (15.87)	5.23 (17.15)	6.30 (20.68)
8	5.34 (17.52)	5.78 (18.95)	6.95 (22.81)
9	5.86 (19.22)	6.32 (20.74)	7.57 (24.82)
10	6.32 (20.72)	6.86 (22.51)	8.19 (26.87)

A sample data and computation sheet is shown on figure 3–21. Sufficient time must elapse after filling the test pit and before taking measurements to permit establishment of a relatively steady state of flow. A comparison of values of C obtained by an electric analog study with K values determined analytically showed the analog values to be about 30 percent lower at a ratio of $\frac{D}{a} = 3$ and about 10 percent lower at a ratio of $\frac{D}{a} = 10$ than the analytical study. Whenever possible, the test pit method should be checked against some other method of determining hydraulic conductivity.

3–10. Test for Determining Infiltration Rate.—Although the drainage engineer is mainly concerned with the hydraulic conductivity of the soil, the infiltration rate is also important in determining the deep percolation and runoff that must be carried by the drains. Infiltration is generally considered as the rate at which water enters the soil surface. Hydraulic conductivity is considered as the rate at which water will move through a unit cross section of soil under a unit hydraulic gradient. The two terms need not be and generally are not identical. In fact, they are identical only if all the following conditions are true:

(a) The soil must be homogeneous throughout.
(b) A zero head of water must be maintained at the soil surface.
(c) No lateral movement of the water may occur.
(d) The surface soil may not restrict the water movement.
(e) Atmospheric pressure must exist at all times at the base of the downward advancing waterfront.

These conditions might occur in a sandy soil before the water reaches an impervious layer or a water table. Usually, in an infiltration test the infiltration rate will be greater in the initial stage than the hydraulic conductivity rate. The infiltration rate will be greater because of some lateral movement and because a

Location: ______________________

Observers: ______________________ Date: ________

Texture of test zone: ______ Structure of test zone: ________

Type of pit: circular with diameter a

D = 0.6 meter (2 feet)

a = 0.3 meter (1 foot)

$\frac{D}{a} = 2$

C = 6.92

Time Initial	Time Final	Time, min	Tank reading, m^3 (ft^3) Initial	Tank reading, m^3 (ft^3) Final	Q m^3/min (ft^3/min)	Hydraulic conductiviy, K m/day (ft/day)
0800	0810	10	0 (0)	0.144 (5.10)	0.0144 (0.510)	16.658 (53.5)
0810	0820	10	0.144 (5.10)	0.283 (9.98)	0.0138 (0.488)	15.953 (50.8)
0820	0830	10	0 (0)	0.119 (4.20)	0.0119 (0.420)	13.756 (43.6)
0830	0840	10	0.119 (4.20)	0.237 (8.36)	0.0118 (0.416)	13.619 (43.4)
0840	0850	10	.0237 (8.36)	0.354 (12.51)	0.0117 (0.415)	13.586 (43.2)

Calculations: $K = \frac{1440\,Q}{C\,a\,D}$

$$K = \frac{1440\,Q}{(6.92)(0.3)(0.6)} = 1156\,Q, \text{ m/day} \qquad (104.05\,Q, \text{ ft/day})$$

Figure 3–21.—Data and computation sheet on test pit method for hydraulic conductivity. 103–D–1632.

head of surface water greater than zero must be maintained of necessity. A downward capillary pull, which initially is significant, also exists. As the wetting front moves downward, lateral and vertical capillary movement becomes negligible; the hydraulic gradient will approach unity, and the infiltration rate will approach the hydraulic conductivity rate.

The same equipment can be used for the infiltration test as is used for the ring permeameter test. The site selected for the infiltration test should be representative of conditions that will be encountered when the area is irrigated. If the area is already under cultivation, the 457-millimeter- (18-inch-) diameter cylinder should be set in a level area and driven in about 25 millimeters (1 inch). Care should be taken that the soil within the cylinder has not been compacted or sealed. Infiltration rates for virgin soil will not be indicative of the infiltration rate of a cultivated soil. Therefore, if the area has never been cultivated, the soil in the test site should be turned over to a depth of 200 to 250 millimeters (8 to 10 inches), then leveled, and all large clods broken up and worked into the soil before the cylinder is installed. When the cylinder has been installed, both the inside and outside edges at the soil surface should be carefully tamped to seal possible cracks.

Next, a mound of soil, metal, or plastic, 150 millimeters (6 inches) high and about 1 meter in diameter, should be constructed around the cylinder. A calibrated tank should be set up outside the mound, and the carburetor and connections should be installed as described for the ring permeameter test. Before starting the test, a moisture sample should be taken just outside the cylinder at 50-, 150-, and 250-millimeter (2-, 6-, and 10-inch) depths to determine the moisture content in the top foot. Both the cylinder and mound should be filled with about 75 millimeters (3 inches) of water, the time recorded, and the water withdrawn from the calibrated supply tank. The 75-millimeter depth of water is maintained inside the mound by a second tank and carburetor. A reading on the tank supplying water to the cylinder should be taken every 5 minutes for the first 30 minutes, every 15 minutes for the second 30 minutes, every 30 minutes for the second hour, and at 1-hour intervals for the next 5 hours. The cylinder should be permitted to go dry, and after 24 hours the surface should be scratched to a depth of about 25 millimeters (1 inch) and the test rerun the same as the first day. Before the second test is started, moisture samples should be taken outside the ring at the same depths as on the previous test.

Because infiltration is defined as the volume of water passing into the soil per unit of area per unit of time, the cross-sectional area of the cylinder should be computed: ($\pi r^2 = 3.1416 \times 22.86^2 = 1{,}642$ square centimeters). Therefore, 1,642 cubic centimeters are equal to 1.0 centimeter (0.39 inch) inside the cylinder. If 1,642 cubic centimeters run through the cylinder in 1 hour, the infiltration rate would be 1 centimeter per hour. When recording the rate for a particular site, the textures of both the surface and underlying zone should be shown. For example, if the surface texture is a fine sandy loam underlain by a clay loam, the texture should be shown as FSL 20 centimeters (8 inches)/CL.

First Run

Initial Date	Initial Time	Final Date	Final Time	Time, hours	Tank Reading, cm^3 Initial	Tank Reading, cm^3 Final	Volume cm^3	Q, cm^3/hr	Infiltration, cm/hr

Second Run

Location:

Project:

Party:

Calculations:

Q = ____cubic centimeters per hour

$A = \pi r = \pi \times 22.86$
= 1641.7 square centimeters for a 45.72-centimeter (18-inch) diameter ring.

Infiltration rate = $\frac{Q}{1641.7}$

(centimeters per hour)

NOTES:

Figure 3–22.—Data sheet for determining infiltration rate. 103–D–1633.

The initial readings can be used to estimate the infiltration rate during wetting, and the later readings indicate the steady state infiltration rate. A sample data sheet for determining the infiltration rate is shown on figure 3–22.

C. Laboratory Tests for Hydraulic Conductivity

3–11. Hydraulic Conductivity From Undisturbed Soil Samples.—An undisturbed sample is one taken from the test site with as little disturbance as possible. Several different methods are used for taking undisturbed samples, but all methods attempt to provide for removal of a certain size of earth sample without disturbing the relation of the soil grains to each other with respect to compression, expansion, or lateral displacement. A properly performed test on such a sample should give a hydraulic conductivity value reasonably consistent with the accuracy obtained from an in-place field test. However, there are economic limitations in using this type of sample in an overall drainage study. A properly obtained undisturbed sample is usually about 100 to 150 millimeters (4 to 6 inches) long, but for solution of drainage problems it is necessary to know the hydraulic conductivity through at least a 3-meter (10-foot) depth over the study area. Therefore, in a heterogeneous profile, many samples must be taken in the field and tested in the laboratory to get the desired information. This procedure is usually more costly than obtaining an equal amount of data by in-place testing.

The lateral hydraulic conductivity of many soils is greater than the vertical and may be many times greater. This is a result of the natural deposition of soils in horizontal layers. Although movement of ground water to a drain is a resultant of lateral and vertical components, the movement is primarily lateral.

The hydraulic conductivity value used in the solution of drainage problems is usually the resultant value of lateral and vertical hydraulic conductivities that apply to the particular problem, but in some instances the vertical hydraulic conductivity alone is of critical importance.

Either horizontal or vertical undisturbed soil samples can be taken. Horizontal samples taken at depths greater than a meter are especially costly. Undisturbed samples taken in both directions can be used to analyze drainage requirements, but inplace test results provide more reliable data, particularly for a large volume of material. Methods of taking undisturbed samples and laboratory methods of determining hydraulic conductivity are described in Reclamation Instructions, Series 510, Land Classification Techniques and Standards.

3–12. Hydraulic Conductivity From Disturbed Soil Samples.—A disturbed (or remolded) soil sample is one in which no attempt has been made to maintain the natural relation of the grains to each other and, in fact, the grains are deliberately disturbed. The sample is usually taken from an auger hole and broken up in a machine before the test is run. The hydraulic conductivity values obtained by this procedure have a doubtful relation to the true hydraulic conductivity value of the soil in its natural state and should not be used for determining drainage requirements. However, loose and uncemented sands and gravels have about the same hydraulic conductivity in both the disturbed and undisturbed states. Disturbed hydraulic conductivity, pH, and electrical conductivity can also serve as screening tests to identify possible sodium problems.

D. Observation Holes and Piezometers

3–13. Introduction.—Observation holes and piezometers for drainage studies are needed to furnish information concerning the character of soil materials and to provide a means for periodic observation of the location, fluctuations, and pressures of ground-water bodies. Observations for ground-water information serve three purposes: (1) to measure the static water level, (2) to measure the pressure of the water at a given point in an aquifer, and (3) to sample water quality.

3–14. Location of Observation Holes.—Selection of hole locations should be made in the field where conditions that might affect the general water table can be readily observed. Holes should be located to eliminate the effect of ponds, lakes, road borrow ditches, canals, laterals, rivers, and similar water-holding reservoirs on the general water table. If the hole cannot be located to completely eliminate the effect of surface water, it is important that a notation be made of the presence or recent presence of water on the surface each time the depth to water is measured.

Observation holes should be located on a fence line or near some other reasonably permanent structure to ensure their permanence. When possible, they should be located near an all-weather road so they can be easily reached at regular intervals throughout the year. When installed prior to construction of the irrigation system, the holes should be located in the arable land area where they will be of maximum value after irrigation. Usually they should not be located on high, nonirrigated divides. Holes should always be logged carefully, using agricultural soil classification, and should also be located so cross sections can be drawn both parallel and perpendicular to the surface slopes. At breaks in slopes, holes should be located both above and below the break so that the drawdown in the water table caused by the break can be shown. Occasionally, observation holes can be located on a grid system along a land subdivision. This method of locating observation holes should be used only when the topography is uniform. Generally, observation wells will be located based on landform and local topography. Placement with a legal subdivision is considered the least important parameter.

Piezometers are located where needed to provide information on vertical movement of water. They are always installed in clusters of two or more, each terminating at a different depth, and their logs and location should follow the same criteria as stated for open observation holes.

3–15. Installation of Observation Holes.—Observation holes may be installed by any of several methods, depending on the character of the material, required depth of hole, and the equipment and personnel available. A 50- to 100-millimeter (2- to 4-inch) diameter hole is usually sufficient. If the materials are unconsolidated and the hole is not deep, a hand auger may be used. Generally, a power auger should be used if a large number of holes are required; the material is compacted; sand and gravel are encountered; or the holes are over 3 meters deep.

The hole should be augered to final depth and pumped until the discharge is clear. About 100 millimeters (4 inches) of sand or gravel are then put into the hole before the perforated casing is installed. The annular space around the casing should then be filled with sand (passing the No. 8 sieve and retained on the No. 18 sieve) to the top of the perforations. At this point, a 1:1 bentonite-soil mixture should be tamped around the casing and mounded at the ground surface. This mixture will prevent surface water from flowing directly into the sand and casing. A concrete collar should be placed around the pipe at the ground surface if the installation is to be permanent.

The depth of an observation hole usually should be below the lowest expected water level. Deeper holes may be necessary to locate and identify artesian aquifers or deep barriers. A careful log of each hole should be made showing texture, structure, color, moisture, etc. Sufficient samples of the materials should be taken for mechanical analyses to ensure that accurate texture appraisals are being made.

When a sodic environment is suspected, some samples should also be taken for exchangeable sodium analyses.

3–16. Casing for Observation Holes.—Generally, most observation holes will be in material that will not stay open without casing. Many types of material can be used for the casing, and the type chosen will depend on the cost and availability of the material and the degree of permanence required. The least expensive material is probably thin metal stovepipe or downspout pipe; however, standard pipe or well casing is ordinarily used. With the present emphasis on water quality, observation wells may also serve as sample sites. If used as a sampling site, the casing material should meet EPA standards for the type of samples collected. These various sampling standards have resulted in the manufacture of many different types of slotted pipe. They range from stainless steel to teflon, to PVC, and are available through many suppliers. For most drainage work, slotted PVC casing is adequate. Several states have statutory requirements for the completion of monitoring wells. These are legal requirements that must be met. All casings for observation holes must be perforated and should be large enough in diameter to allow acquisition of water quality samples. A satisfactory method is to perforate at about 150-millimeter (6-inch) vertical intervals, with the perforations alternating on opposite sides of the pipe and extending from the bottom of the pipe to within 1 meter of the ground surface. The perforations should be large enough for water to enter but small enough to prevent soil materials from entering the casing in any quantity. Generally, a slot about 3 millimeters (1/8 inch) wide will be satisfactory. When automatic water table recorders are to be used, the observation hole should be at least 100 millimeters (4 inches) in diameter and cased with an economical commercial well screen.

The casing should be extended 300 to 450 millimeters (12 to 18 inches) above ground surface so that it will be visible from a distance. An additional aid is to paint the extended portion of the pipe either yellow, orange, or some other color that contrasts with the natural surroundings. This not only makes the hole easy to locate for measuring, but also makes it easy for the farmer to see the casing in a cultivated field. When the casing is not protected by a fence or similar permanent structure, a painted 100- by 100-millimeter (4- by 4-inch) by 1-meter (4-foot) wood post or a painted steel post should be installed near the casing. The hole number should be painted or stamped on the post for easy identification.

Another method that can be used if it is considered inadvisable to leave a rigid pipe or post projecting in a field is to attach a rubber hose to the top of the casing. The casing is cut off about 150 millimeters (6 inches) below the ground surface and a rubber hose about 600 millimeters (2 feet) long slipped over the top of the casing. This method results in fewer damaged observation holes and less damage to farm equipment.

The casing should be capped and the cap tightened with a wrench to prevent rocks or sticks from being dropped down the casing to check the water level. A hole should be drilled in the cap or in the pipe just below the cap to prevent pressure or vacuum from building up during fluctuations in the water table.

3–17. Piezometers.—The piezometer is a device which allows measurement of the piezometric water surface at a given point in an aquifer. This device is important because pressure differentials exist in a moving ground-water body. Differential elevations of the water table, as measured in observation holes, give only information on the thickness of unconfined water bodies and the gradient of their phreatic water surfaces. Data from piezometers give information on vertical pressure differentials in confined and unconfined water bodies. Piezometer measurements are frequently used in the study of seepage flow from canals, laterals, or other surface sources to determine ground-water flow patterns and in the determination of upward leakage from a confined aquifer. In such studies, groups of two or more piezometers are used to measure the hydrostatic pressure at specific depths in separate saturated soil strata. Single piezometers do not show the water table except in very permeable material, and should not be used in lieu of an observation well.

3–18. Installation of Piezometers.—The method of installing a piezometer pipe must be such that a tight seal is formed around the outside of the pipe to prevent vertical movement of water between the pipe and wall of the hole. For shallow installations, pipe as small as 10-millimeter (3/8-inch) diameter and up to as large as a 100-millimeter (4-inch) diameter can be used. However, 25- to 50-millimeter (1- to 2-inch) diameter pipe has been found to be the easiest to install.

There are many methods of installing piezometers. For depths less than 1.5 meters (5 feet), alternate augering and driving of the piezometer pipe provides a good seal. For depths more than 1.5 meters, the hole can be augered to within about 0.5 meter (18 inches) of the proposed bottom, the pipe placed in the hole, and the alternate augering and driving method used for the last 0.5 meter (18 inches). A driving head should be used when driving the pipe to prevent splitting or smashing the end. A type of driver which has been used successfully consists of a 0.6-meter (2-foot) length of pipe with an inside diameter slightly larger than the outside diameter of the pipe to be driven. The driving pipe should have an end cap. A 5- to 10-kilogram (10- to 20-pound) weight can be welded to the pipe to give the driver additional weight. A hardwood or plastic plug should be inserted into the cap of the driving pipe to prevent the driver from hitting the piezometer pipe directly. A standard wood auger fitting inside the piezometer can be used as an auger. The auger must be altered by grinding the end to a point to penetrate the soil. A 12-millimeter (1/2-inch) pipe coupling must be welded to the shank to accept a handle and extensions. When using the alternate augering and driving method, the hole is augered about 150 millimeters (6 inches) below the pipe each time, and the pipe is then driven to the bottom of the hole. A cavity about 100 millimeters (4 inches) long and with the same diameter as the inside pipe diameter is augered below the bottom of the pipe to provide an easy access for water entering the pipe. This cavity should be flushed by inserting a hose to the bottom of the cavity and pumping out the water. After flushing, the cavity should be filled with sand to assure that it remains open.

An alternate method of installing deep piezometers and multiple piezometers is to auger to the full depth with a power auger. Before the pipe is installed, about 100 millimeters (4 inches) of coarse sand or fine gravel are poured into the hole. The pipe is then installed on top of the sand and another 25 to 50 millimeters (1 to 2 inches) of sand poured around it. The annular space around the pipe is then sealed with grout or a dry 1:1 bentonite-soil mixture to eliminate vertical water movement around the pipe. This seal should be a minimum of 0.6 meter (2 feet) thick vertically when grout is used and a minimum of 1.5 meters (5 feet) thick when the bentonite-soil mixture is used. When more than one piezometer is installed in the same hole, the above procedure is repeated except that the sealant must fill the annular space between piezometer levels and for a 0.6- to 1.5-meter (2- to 5-foot) distance above the last piezometer. Remaining annular space can be filled with any material available.

In unstable material, an outside casing must be used to keep the hole open. After the pipe has been installed, the casing is removed by pulling as the sealer is placed and the hole is filled.

After a period of 24 hours, the piezometer should be tested to ensure that it is functioning properly. Water is then pumped from or poured into the pipe, and the time is observed for the water level to rise or fall. If there is a definite rise or fall in the water level in the pipe, the piezometer is functioning properly. If the rate of rise or fall is very slow, the pipe might be plugged at the bottom and should be flushed or reaugered. A piezometer installation should not be considered complete until it has been tested and found to function properly. If the piezometer is capped, a perforation must be made in the cap or in the pipe just below the cap to assure atmospheric pressure within the pipe.

3–19. Records of Observation Holes.—A permanent record should be made of all observation holes. This record should include such items as the location and depth of the hole; type, depth, diameter, perforated length, and total length of the casing installed; a log of the hole showing a complete textural description of the material encountered; elevation of natural ground surface at the top of the hole and of the measuring point from which measurements of the depth to water will be made (usually the top of the casing); and the periodic measurements of depth to water. When cooperative programs with the U.S. Geological Survey (USGS) are carried on, it may be preferable to use their forms for recording information on the hole and for recording water level measurements.

3–20. Numbering System for Observation Holes.—A numbering system for observation holes should be established for ready reference in the field and for location on maps. Two systems have proved satisfactory, the coordinate system and a land subdivision system developed by the USGS.

In the coordinate system, the study area is located on a map, and the north-south (N-S) and east-west (E-W) lines, called the zero lines, are established. These lines can be in any location with respect to the area, but it is a little easier and there is less chance for error if the E-W line is chosen to be adjacent to the south of the area and the N-S line adjacent to the west of the area. The area can then be

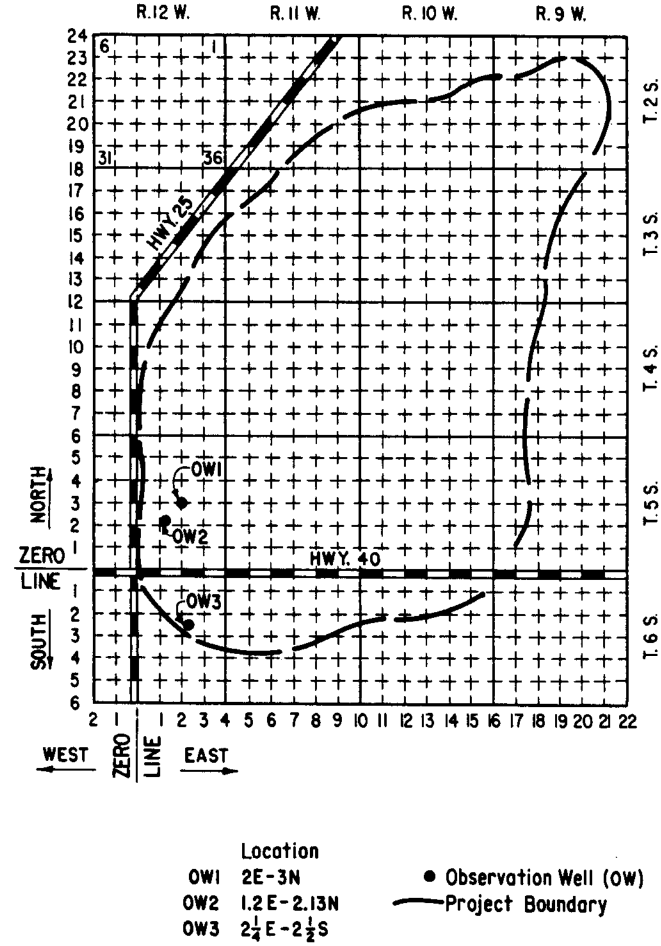


Figure 3–23.—Coordinate system for numbering observation holes. 103–D–1636.

visualized as being in the first quadrant of a rectangular coordinate system. Figure 3–23 shows an example of this system. A well that is 0.6 kilometer (2 miles) east and 0.9 kilometer (3 miles) north of the intersection of the zero lines (point of origin) would be well No. 2E-3N. Wells do not have to be located an even number of miles from the point of origin; they can also be located by decimals (1.2E-2.13N) or by fractional parts of a mile (2-1/4E-2-1/2S). This system not only readily locates the wells on maps and in the field, but also identifies their location with respect to each other. The system operates best in an area which has had a land survey, but this is not essential. Locating the point of origin at the intersection of two highways that traverse the area might be convenient. In this case, wells in all four quadrants could have numbers with combinations of E, W, N, and S.

The USGS method is based on a land subdivision system which uses township, range, section, and four lowercase letters for well locations. The first designation of a well number denotes the township, the second the range, and the third the section. Each township contains 36 sections, and each section is 1 mile square (640 acres). The lowercase letters that follow the section number indicate the position of the well within the section. The first letter indicates the quarter section, the second the quarter-quarter section, and the third, if present, the quarter-quarter-quarter section, or 10-acre tract. The letters *a*, *b*, *c*, and *d* are assigned in a counterclockwise direction, beginning in the northeast quadrant of the section, or quarter-quarter section. If two or more wells are located within the same 10-acre tract, they are distinguished by a numeral following the lowercase letters. Figure 3–24 shows an example of the USGS numbering system.[1]

3–21. Measuring Devices for Depth to Water.—There are several devices for measuring the depth to water in an observation hole. Figure 3–25 shows the most commonly used devices. Probably the most widely used is the weighted, chalked line. An ordinary steel tape with a suitable weight attached to the end is chalked for the first 0.5 to 1.0 meter (2 to 3 feet) with carpenter's chalk or ordinary blackboard chalk. When immersed in water, the chalk will change color, and the point to which the tape penetrates the water surface can easily be read. The tape is lowered into the hole until it reaches the water and then further lowered until an even meter mark is held at the measuring point. The reading on the chalked portion is subtracted from the reading at the measuring point and the difference is the depth to water. This procedure may require more than one try to get the end of the tape properly submerged, but can be done quickly if the approximate depth to water is known.

Another method is to use a steel tape with a "popper" attached to the end of the tape. A popper can be made from a 12-millimeter (1/2-inch) pipe plug. A fastener is welded to the head end of the plug so that it can be fastened to the end of the steel tape. The threaded end of the plug is hollowed out to provide an air pocket. The popper is lowered into the hole, and a distinct "pop" can be heard when the popper meets the water surface. With a little experience, the water surface can be located within 3 millimeters (0.01 foot). The tape is read at the measuring point when the popper is just touching the water, and the distance from the end of the popper to the tape is added to the reading to obtain the depth of the water surface from the measuring point.

A graduated rule or dipstick made of 12-millimeter (1/2-inch) thick by 25-millimeter (1-inch) wide hardwood is useful for measuring water levels within

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[1] U.S. Geological Survey Water-Supply Papers. This system is not used by the USGS in the State of Washington and cannot, of course, be used in States that do not use the rectangular system of the United States public land surveys.

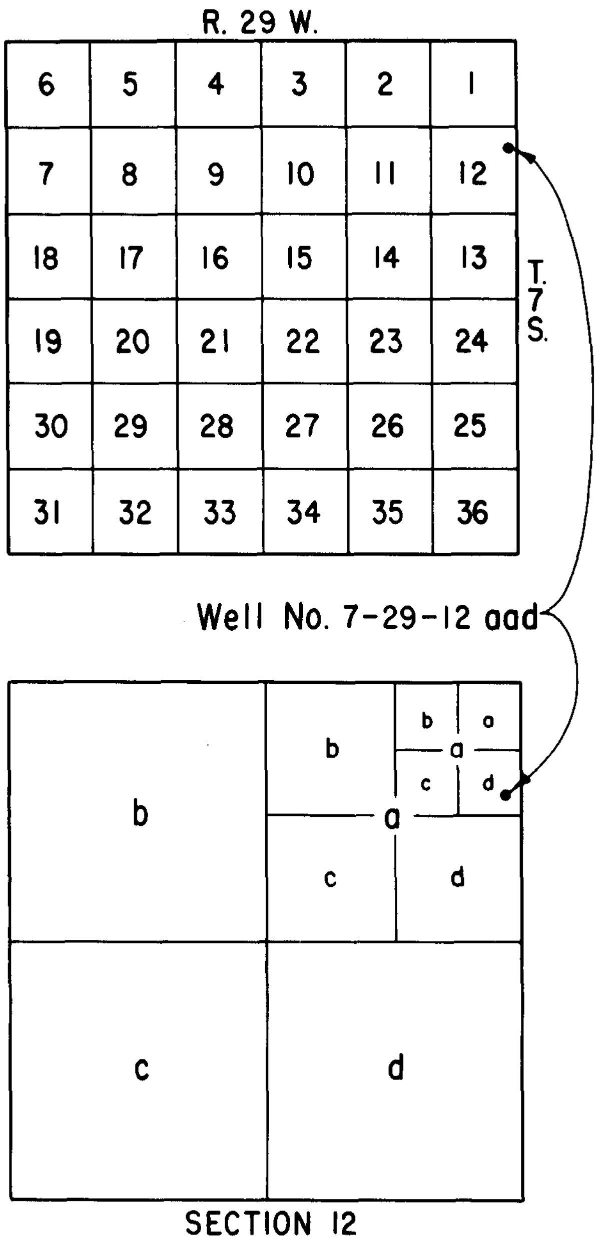
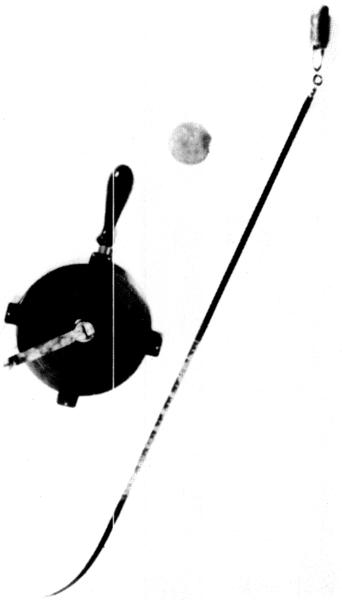
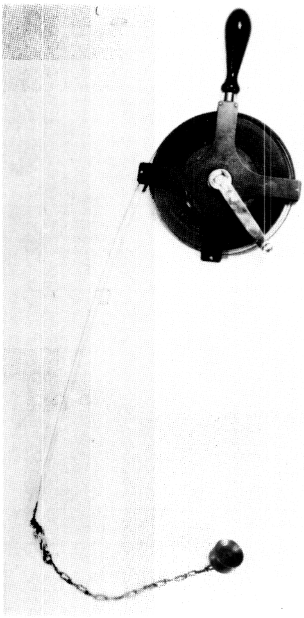


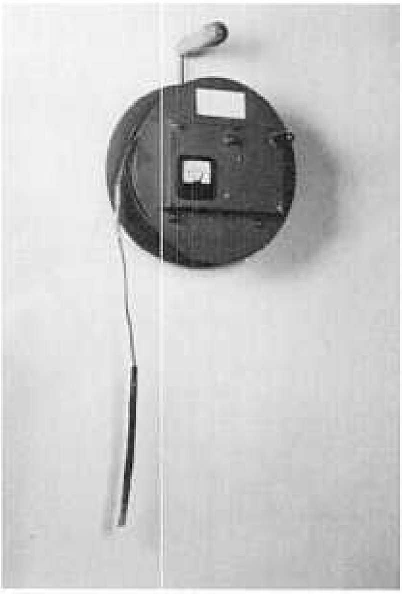
Figure 3-24.—USGS township-range well numbering system. 103-D-696.



Chalked Line. PX–D–25997.



Popper. PX–D–25996.



Electric Sounder. PX–D–25177.

Pressure Transducer and Data Logger.
PX–D–25995.

Figure 3–25.—Devices for measuring depth to water in wells.

2.5 meters (8 feet) of the surface. This device can be jointed like a fishing rod or hinged and folded for convenience. The wood is not painted or treated in any way, which eliminates the need for chalking. With all nonelectric measuring devices except the popper, caution should be exercised to avoid errors in measurement caused by displacement of a sufficient volume of water with the device during the measuring process, particularly when measuring in small diameter pipes.

All permanent pump installations should include an air line and gauge with which to measure drawdown during pumping. The air line usually consists of 6-millimeter (1/4-inch) tubing of sufficient length to extend below the lowest water level to be measured. The vertical distance from the center of the pressure gauge to the bottom of the air line should be measured at the time of installation. A pressure gauge and an ordinary tire valve are placed in the line at the surface so air can be pumped into the line and the pressure measured. To measure the depth of water, pump air into the line until a maximum reading occurs on the gauge. This reading is equal to the pressure exerted by the column of water standing above the bottom of the air line in the well. The depth to water below the pressure gauge is then computed by subtracting the gauge reading from the vertical distance to the bottom of the air line. If the gauge reads in kilopascals, multiply the reading by 0.102 to convert to meters.

Example: If the length of the air line from center of gauge to bottom of air line is 30 meters (100 feet) and the gauge reads 150 kilopascals (21.6 pounds per square inch), the water level in the well is 15 meters (50 feet), $30 - (150 \times 0.102)$ $[(100 - (21.6 \times 2.31)]$, below the center of the gauge. Unless carefully calibrated against taped readings, the air line is accurate only to about plus or minus 0.15 meter (0.5 foot).

Several commercial electrical sounding devices are available for measuring the depth to water in a well or observation hole. Most of these devices are based on completing an electrical circuit through the water in the well. Some use two electrodes and the circuit is completed when they reach the water surface. Others use only one electrode and the well casing serves as the other electrode. These devices usually employ flashlight batteries for power, and contact with water is signaled by a bell, buzzer, light, or movement of an ammeter indicator. The electrodes are attached to insulated wire which is marked in increments of length. Devices are also available which measure various water-quality parameters as well as depth. Parameters most likely to require measurement during drainage investigations would include salinity, pH, temperature, etc.

Instruments have also been developed which use a diaphragm arrangement to measure either positive or negative pressures. These instruments are sometimes referred to as transiometers. As the water table fluctuates, they alternately measure depth of water above the measuring point or negative pressure in unsaturated soils.

3–22. Plugged Observation Holes.—After a series of measurements, it may be noted that the water level no longer fluctuates in certain holes, that the fluctuation departs from its former pattern, or that the position of the water table and the magnitude of fluctuation has changed in nearby holes. Such holes may have become plugged by an accumulation of silt. Possible plugging can be detected by pouring water into the hole and measuring the rate at which it is accepted into the formation. A very slow rate, considering the soil in the formation, indicates a plugged hole. Usually these holes can be returned to usefulness by flushing the hole from the inside or by bailing. A stirrup pump can be used for flushing by attaching a small diameter plastic hose to it, inserting the hose in the hole, and pumping water into the hole. The water will then flow upward out of the hole between the casing and the plastic hose. The flushing action will loosen the material that forms the plug and wash it out or permit bailing it. Under some conditions, a hand auger sized to fit inside the casing has been used to clean material from a plugged well. Augering used in combination with bailing works well for some soils.

When a monitoring well has outlived its usefulness, environmental considerations and legal requirements call for proper disposal or abandonment. The well should be cut off 0.5 meter (2 feet) below ground surface and backfilled with concrete to preclude the possibility of providing an avenue for contamination of the ground water. State and local codes should be checked to be sure that all statutory requirements are met.

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⟨⟨Chapter IV

DRAINAGE INVESTIGATIONS

4-1. Scope of the Investigations.—The many types and diversity of drainage problems require a clear understanding of the purpose of a particular investigation at its outset. The scope of the investigation and the level of the report should be directed toward specific objectives. The objectives should be established with economy and timeliness in full perspective. The minimum amount of data needed for solution of the problems must be determined. Existing data must be evaluated and the best means for obtaining necessary additional data examined.

After becoming acquainted with the area and the available data, the scope of the investigation can be established. The scope will represent a balance between the available data and the amount and types of additional data required as dictated by the accuracy and completeness expected of the final report or plan, including the time and manpower available for the investigation.

The scope of the investigation and the resultant plan and report will be less detailed for a reconnaissance investigation than for an investigation immediately prior to construction. The work performed during a reconnaissance investigation should fit into a pattern that can be expanded into the more complete study required for construction.

Each drainage project or segment of construction must be justified as economically necessary. The drainage engineer's principal job is to devise an effective drainage system at minimum cost. The Bureau of Reclamation method of economic analysis appears in Reclamation Instructions, Series 110, Project Planning.

Some drainage problems are simple and their solution readily apparent; for others, a limited investigation will suffice. Most drainage problems, however, involve a thorough study of the complex relationships among soils, water, crops, salts, and irrigation practices.

4-2. Factors in an Investigation.—The main factors in any drainage investigation are topography, soils, salts, ground water, soluble trace elements, and the sources and quantities of excess water. Any investigation must answer the following questions:

- Is excess water or salt present now or anticipated in the future?
- Is an adequate outlet available for excess water and salt?

- What is the source of the excess water and salt?
- What is the depth of the drainable soil zone?
- What type of drainage system is best?
- How much water and salt must be removed?
- Can the soil be economically drained?
- Are soluble trace elements present in potentially toxic quantities?

4-3. Review of Existing Data.—The first step in the drainage investigation is to collect, review, and analyze existing data. Data on geology, soils, topography, well logs, water levels and their fluctuations, precipitation, salinity, ground-water quantity, and surface flow are a few of the pertinent items. Analysis of these data will ascertain their adequacy and establish the amount and kind of additional data required.

4-4. Field Reconnaissance.—The field reconnaissance is one of the most important steps in any investigation. Firsthand information and impressions are valuable in evaluating current conditions and programming additional investigations. If possible, in making a field reconnaissance, someone familiar with the area should accompany the investigator.

The initial field study should acquaint the investigator with data on the following items:

(*a*) Location and capacity of natural waterways.

(*b*) Location and condition of outlets.

(*c*) High watermarks or other information which may be used in evaluating floodflows.

(*d*) Location and characteristics of canals, laterals, wells, springs, ponds, reservoirs, or other possible ground-water sources.

(*e*) Local irrigation practices, such as method of water application and efficiency of irrigation.

(*f*) An estimate of the present water table level and information with regard to its fluctuation and direction of movement.

(*g*) Present cropping practices, crop conditions, and a notation of any trend toward possible future changes.

(*h*) Type, location, spacing, depth, and effectiveness of any drains in the specified study area and adjacent areas. The analysis of drains in adjacent areas is one of the most important items in the investigation. Existing drains in similar areas can often constitute the soundest foundation from which to determine drainage requirements in the specified study area.

(*i*) Topographic features which might obviously affect the location of drains.

(*j*) Geologic setting and features which will affect the design of drains.

(*k*) Indications of salinity or alkalinity, such as surface florescence, barren soil surface, certain plant populations, or abnormal cultural practices.

(*l*) Discussions with local people, particularly those residing in the cultivated or irrigated areas. They may provide important information on types of crops currently grown and trends, crop yields, irrigation practices, and the extent and effects of local floods.

(*m*) Status and scope of any existing drainage programs administered or undertaken by State, Federal, or private agencies.

The preliminary information collected from the above items for field reconnaissance is associated with the analyses of certain subsurface conditions that are introduced in this section but discussed in more detail in subsequent sections.

The analysis of subsurface conditions requires either a value for the depth to barrier or the knowledge that the barrier is at such a sufficient depth that it has a negligible effect on the drainage requirements. The logs of any existing wells may show the depth of barrier; otherwise, new holes must be drilled for barrier depth determination. Such holes should be located at strategic points on depth-to-barrier contour maps.

To graphically show the effect of subsurface characteristics on drain location, depth, and spacing, a series of ground-water profiles should be made showing the location, extent, and slope of the different strata. These features can then be analyzed in relation to the slope of the ground surface and to the existing or projected ground-water conditions. A sample set of profiles is shown on figure 4–1. Where pertinent soil strata (either fine-textured, slowly permeable material, or coarse-textured, highly permeable material) are continuous over a large area, a contour map of the surface of the stratum is often useful. Such a contour map is extremely helpful in planning a drainage system for an area underlain at depths of 1.8 to 3 meters (6 to 10 feet) by these materials. Contour maps and ground-water maps drawn on transparent paper can be used as overlays on a base map of the study area which shows ground surface elevations, canal and drain locations, and other pertinent data. When making these overlays, using a color system as suggested on figure 2–1 will simplify the interpretation. This method is often very helpful in locating new drains. These types of maps and profiles can be easily developed using a GIS (Geographic Information System).

4–5. Subsurface Investigations.—A good investigation of subsurface conditions represents a balance among: the available data; the amount and types of additional data required; and the time, money, and manpower available. Hydraulic conductivity measurements represent the greatest investment in time, money, and manpower, but the resulting data are the most important of all the data produced in the subsurface investigations. Therefore, hydraulic conductivity should be measured using the best techniques.

(a) Log of Drainage Holes.—Each hole or cutbank used in a particular drainage study should be completely logged so the description of soil characteristics has maximum usefulness in identifying and correlating similar soils. Figure 4–2 shows the type of log preferred for a drainage hole. Personnel logging holes should coordinate their efforts so that identical soil characteristics are recognized and uniformly described wherever possible.

(b) Projection of In-Place Hydraulic Conductivity Data to Similar Soil Horizons.—An in-place hydraulic conductivity test, when conducted in two or more

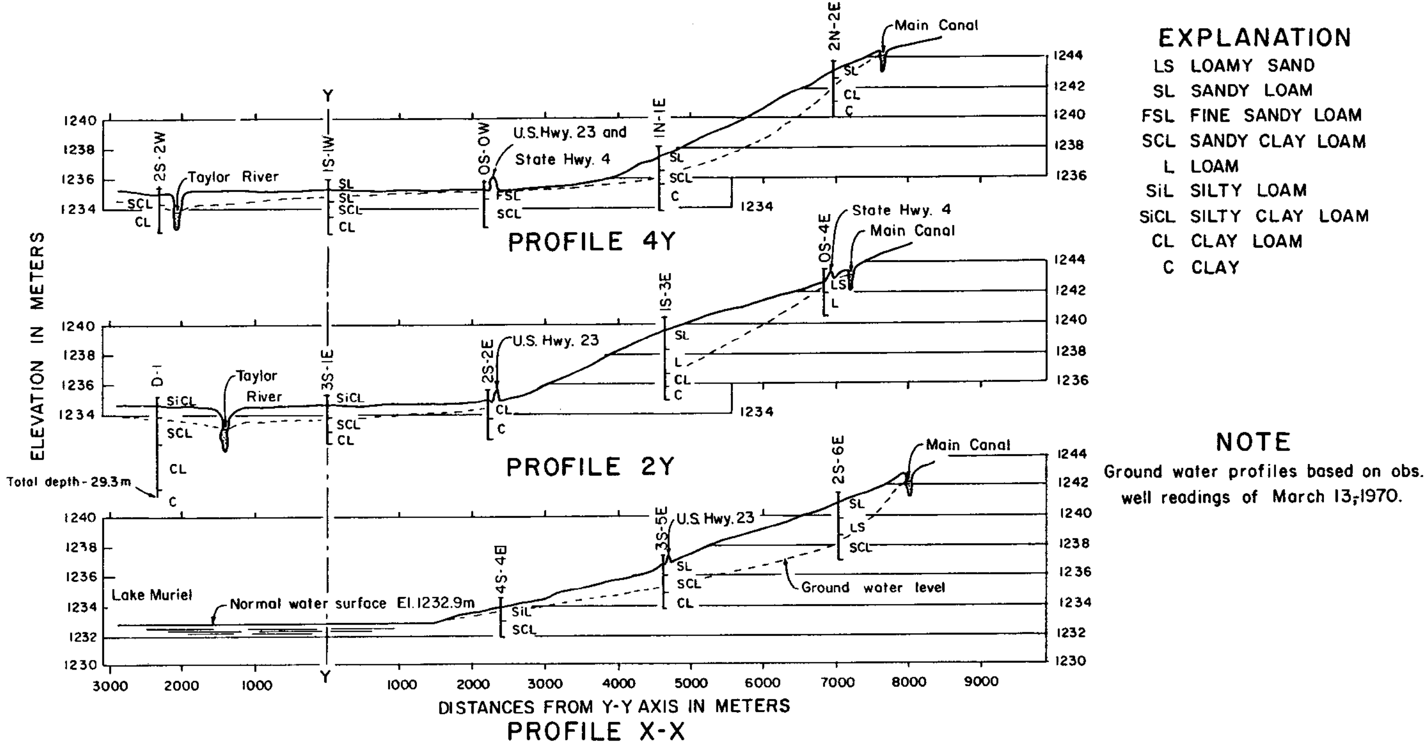


Figure 4–1.—Typical ground-water profiles. 103-D-1428.

HOLE NO.	LOCATION
Hydraulic Conductivity No. 1	T 120 N, R 64 W, 34 bbbb—Oahe Project, South Dakota

CREW	DATE
J. Smith, S. Williams	September 10, 1967

CROP	LAND CLASS
Wheat (harvested in July)	$\frac{3\text{ std}}{\text{C22BY } U_2f_2}$

DESCRIPTION OF MATERIAL								NOTES
TEXTURE	DEPTH	COLOR	STRUCTURE			PERM	S.Y.	
			TYPE	CLASS	GRADE			
SiL(SiCL) M.A. Sand 18% Silt 56% Clay 26%	4.61 to 6.61	7.5YR 5/4 Brown	Sub. Ang. Blocky	Fine to Medium	Moderate	Pump-in 0.5 in per hour		1. Medium cleavage lines between peds 2. Moist consistence-friable, slightly plastic 3. Few very fine and fine roots, concentrated along vertical ped faces 4. Many fine discontinuous vertical imped simple closed tubular pores 5. Very few clay films in tubular pores 6. pH 7.5 7. Slightly effervescent 8. Moisture less than field capacity
SiL(FsL) M.A. Sand 44% Silt 51% Clay 5%	6.61 to 8.81	10YR 6/2 C2P 10YR 5/6 and 5YR 5/4	Platy	Medium	Moderate	Pump-in 0.9 in per hour		1. Light brownish gray with common medium prominent mottles of yellowish brown and reddish brown 2. Fine cleavage lines between peds 3. Very few fine roots in the 6.6 to 7.0 ft zone 4. Few fine vesicular pores 5. Few clay films between plates 6. pH 8.0 7. Slightly effervescent 8. No visible moisture on auger or in pores

Figure 4–2.—Sample log of a drainage hole. 103-D-1637.

textures, gives a weighted value for the textures. This value can be used directly to design drains at the test site because the weighted hydraulic conductivity for the flow zone is used in design computations, rather than the values for individual strata. However, the weighted value is not readily transferable to other locations. If the test is conducted in only one texture for which the physical and chemical characteristics are known, the results can be averaged with other in-place data in similar soils of that texture to determine an average hydraulic conductivity. When the average hydraulic conductivities have been obtained for all the different texture-structure combinations in the project, the data can be used to estimate the weighted hydraulic conductivity at every site where a hole has been logged. By following this procedure, the weighted hydraulic conductivity values are available at a maximum number of sites with a minimum amount of field testing. This procedure is most valuable when estimates of drainage requirements are needed for large areas.

4–6. Identifying the Barrier Zone.—By definition, as used by the Bureau of Reclamation, a barrier zone is a layer which has a hydraulic conductivity one-fifth or less of the weighted hydraulic conductivity of the strata above it. Although this is a somewhat arbitrary standard, it has worked out satisfactorily in practice. Identifying and determining the depth to barrier zone in turn defines the thickness of material through which water may flow to a drain.

4–7. Geologic Influence.—Geologic processes often produce areas in which the soil mantle is underlain by material with markedly different characteristics than the overburden. The underlying material may have an irregular surface that shows significant relief. Material that is less permeable as compared to the overburden may affect ground-water movement. Deep, percolating water may perch on the material, or the lateral movement of ground water may be restricted.

If the surfaces of the underlying material have appreciable relief, ground water may be channeled in topographic lows, and the surrounding areas will be tributary to the channel. In some cases, the key to successfully draining the area is to tap the channel with drains and wells. On the other hand, the surface topography of the underlying material may act as dikes or dams to the lateral flow of water to natural or manmade outlets. Either case will require careful investigation in areas believed to have barrier material that has an unconformable contact surface with the overburden.

The normal observation hole system may not reveal the true subsurface condition. In areas known to be underlain by shale, or in areas where deep cuts have revealed undulating strata of impermeable material, more closely spaced holes will be necessary to locate and map the barrier surface.

Knowledge and understanding of the geologic processes which developed the soil mantle above the barrier zone are important in defining a drainage problem. Early recognition of the landforms in the area and how they developed will assist in developing the most efficient data-gathering plan. As an example, an elevated river terrace may require backhoe pits because of the size of the cobble and boulders. At the same time, ancient lakebed materials may be investigated with a hollow-stem drill rig; an alluvial fan may require a combination of both.

4-8. Water Source Studies.—*(a) General.*—The presence of excess water that creates a drainage problem can ordinarily be traced to:

(1) Precipitation.
(2) Irrigation applications.
(3) Seepage from surface water bodies.
(4) Hydrostatic pressure from an artesian aquifer.
(5) A combination of any of these sources.

Proper protective measures cannot be taken unless the source of the damaging water is known. If the source of the water is precipitation, the solution may involve additional surface drains; an overirrigation problem may require water use education as well as additional drains (recognizing that practically all arid soils require some irrigation in excess of consumptive use for salt control); canal lining can slow or stop seepage; pumped relief wells may alleviate hydrostatic pressure. Relief or interceptor drains will generally accompany all these possible solutions.

(b) Precipitation.—The precipitation record obtained in the study of rainfall-runoff relationships should be analyzed from the standpoint of its effect on both the surface runoff and the ground-water table. The precipitation distribution should be related to the fluctuations in water table elevations, and long-term precipitation records should be related to long-term hydrographs of water levels, where possible.

(c) Irrigation.—Drainage problems are most frequently traced to irrigation practices. In determining the possible contribution of excess irrigation water to the drainage problem, the aspects that should be investigated are:

(1) The effect of individual irrigations on the water table.
(2) The fluctuation of the water table throughout the irrigation season and during times of no irrigation.
(3) The changes in water table elevations over a period of years, both before and after the beginning of irrigation, if possible.

Irrigation practices should relate to soil types and crop needs and, ideally, only enough water should be applied to furnish crop needs and to maintain a salt balance.

(d) Seepage.—Seepage can be a major source of ground water moving into many drainage problem areas. Most seepage originates from irrigation development works such as canals, laterals, reservoirs, or the irrigation of higher lying lands. In some cases, seepage may result from rainfall or snowmelt on the high-lying areas. The comparison of ground-water fluctuations with water levels in canals and reservoirs, or with the application of irrigation water at higher levels, may indicate the source of the seepage water. The growth of tules, willows, or other water-loving plants downstream from possible sources of seepage indicates a high water table. Other methods of detecting seepage involve the use of radioisotopes, dyes, salts, observation holes, and piezometers.

(e) Hydrostatic Pressure.—In some areas, hydrostatic pressure in underlying aquifers may be damaging. Hydrostatic or artesian pressures are found where a slowly permeable layer overlies a saturated permeable layer that is under pressure. Hydrostatic pressure may force water upward through the slowly permeable layer or through fractures in this layer. Damaging amounts of artesian water may be present in areas where old artesian wells are leaking below the ground surface or are allowed to run freely without proper facilities to dispose of the surface flow.

4–9. Ground-Water Studies.—*(a) General.*—Studies of the water table produce information necessary to solve a drainage problem. Areas where a high water table has developed or is anticipated must be mapped. Information concerning depths, trends, and movements is essential to understand the problem. The water table investigation provides data on the position, extent, and fluctuations of the water table, the quantity and direction of movement of the ground water, and an indication of water sources and areas of discharge. Analyses of periodic measurements from observation holes and piezometers are the focus of the investigation.

The frequency of depth to water measurements in observation holes and piezometers depends on the particular problem under investigation. The frequency may vary from daily to quarterly readings, but in general, the readings should be made monthly. The objective of the measurements is to establish a record of the water table fluctuations over a period of time that will reflect all factors affecting the water table. At least one full annual cycle of readings is needed before locating and designing a drainage system.

Data on water table observations are meaningless without an analysis of their significance. The mere gathering of data is a needless expense unless the data are plotted in a form for study and interpretation of the results. Interpretation begins with the data gatherer, who must remain alert to abrupt changes in conditions and must attempt to account for them. A few notes made in the fieldbook can avoid confusion later.

In many cases, using automatic recorders at selected locations provides records for use in conjunction with other measurements. The use of recorders often permits longer time intervals between visits to the wellsite.

Drawings found useful in analyzing ground-water problems are ground-water table contour maps, depth-to-ground water maps, depth-to-barrier maps, water table profiles, piezometric profiles, and hydrographs.

(b) Ground-Water Table Contour Maps.—To prepare this type of map, all points at which ground-water elevations were taken should be marked on a map of the area. A contour map of the water table can then be prepared similar to the one shown on figure 4–3. The measurements of water table elevations should be made for all wells in the project area in the shortest possible time to ensure good correlation. The inclusive dates during which the elevations were read must be noted on the map.

Water table maps show the direction of water movement by the shape and position of the contour lines, indicate the areas of recharge and discharge, and

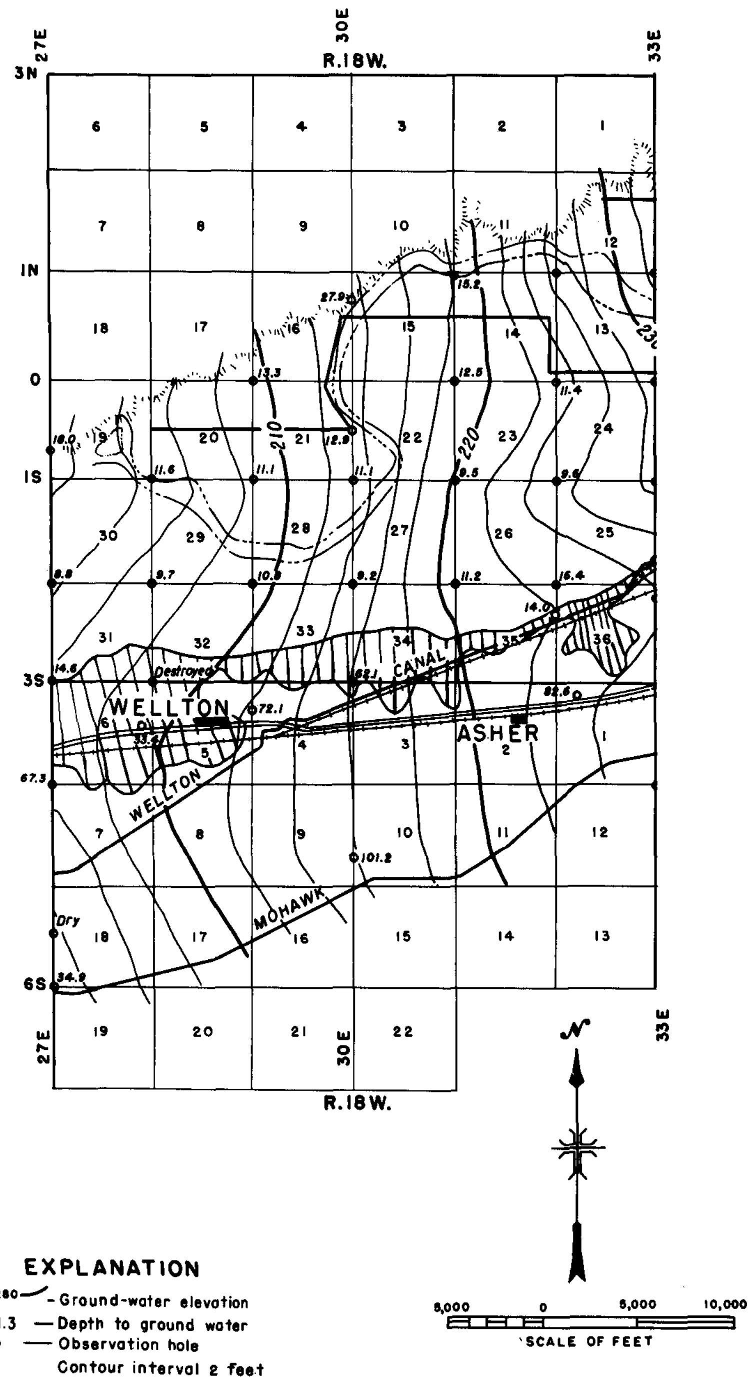


Figure 4–3.—Typical ground-water table contours. From drawing 103-D-703.

may give some indication of the relative hydraulic conductivity by the distance between contour lines. The maps should also include information on construction and depth of the well. This information is useful in assuring that the water table map shows contours on hydraulically interconnected ground-water bodies.

(c) Depth-to-Ground Water Maps.—One method of preparing these maps involves overlaying the water table contour map on a topographic map. This procedure can be done by marking each intersection of contours and noting the difference in their elevations at the intersection point. Using these values, a contour map which shows the depth to water below the ground surface at any point can be prepared. Another method of preparing a depth-to-ground water map is to mark the measured depths to water from the ground surface on a base map at each measuring point and prepare a contour map from these values. A typical depth-to-ground-water map is shown on figure 4–4.

(d) Depth-to-Barrier Maps.—A depth-to-barrier map can be prepared in a manner similar to a depth-to-ground water map if sufficient information is available on the location of the barrier. This type of map is useful in establishing drain locations, estimating quantity of ground-water movement, and providing other information needed for drainage calculations.

(e) Water Table Profiles.—A water table profile can be made for a series of observation holes. The base profile is prepared by plotting the ground surface elevation; the location and depth of the observation holes; and any springs, canals, or ponds that are in the profile. The profile is generally made downslope in the direction of water movement but can be made in any direction. The elevation of the water surface at each observation hole or other known point can be plotted on a print of this profile. The use of different colored pencils for readings taken at different times of the year facilitates a visual comparison of fluctuations in the water table along the profile.

A water table profile is even more useful if it also contains information on subsurface material. The logs obtained from installation of the observation holes can be plotted at each hole, and any other pertinent information can be plotted at the proper location. If soil textures are available, tentative correlations between holes may be possible. The elevation of the barrier in each hole should also be plotted on the profile, as this information will be helpful for locating drains and in calculating other drainage requirements.

(f) Piezometric Profiles.—Readings from several clusters of piezometers can be plotted on a profile drawn through the clusters. The elevation of the piezometric water table for each piezometer can be plotted at the elevation of the bottom of that piezometer. Lines drawn through points of equal piezometric water table elevation show lines of equipotential. Lines drawn from higher elevations through lower elevations and perpendicular to the equipotential lines form a flow network and show the direction of movement of water and, possibly, the source of the water. This procedure is particularly useful in locating an artesian water source.

(g) Hydrographs.—Drawings may be made showing the elevation of the water table with respect to time for any single observation hole, well, or piezome-

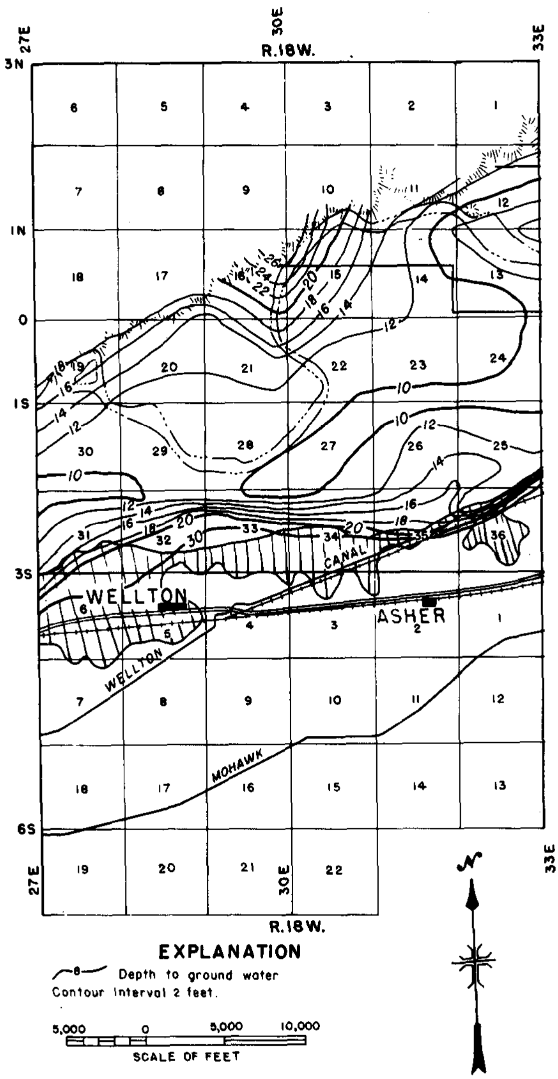


Figure 4-4.—Typical depth-to-ground water map. From drawing 103-D-661.

ter. Such a drawing clearly shows fluctuations in the water table as well as trends in water table movement. Figure 4–5 shows a typical hydrograph. When analysis of the hydrograph does not provide an explanation of certain problems, it may be helpful to superimpose additional data on the hydrograph for use in the analysis. Figure 4–6 shows the plotted data for a special problem where river stage, precipitation, periods of canal operation, and water deliveries were all included on the same hydrograph.

A useful tool in analyzing hydrograph data is to compare departures from normal weather data with hydrograph fluctuations. The plot often explains upward or downward trends in water levels.

Available geographic information system software designed for use on a work station makes development and modification of the maps, profiles, and hydrographs described in this section much easier than hand drafting methods.

4–10. Ground-Water Accretions to Drains.—In its natural state, ground water follows the hydrologic cycle wherein a portion of the precipitation falling on the land surface percolates downward to join the ground-water body. The ground-water body moves slowly from a higher to a lower elevation. Over a period of time, the underground basin fills with water until it spills into a natural outlet such as a spring or a stream. As a result of the cycle, a rise occurs in the water table during periods of high precipitation and deep percolation, causing an increase in flow at the natural outlet. A period of low precipitation causes a lowering of the water table and a decrease in flow. A stability is reached wherein the ground-water table and the natural discharge fluctuate within an established pattern.

When irrigation water is added to the land surface, thus increasing percolation, the pattern is upset. The water table rises and the discharge at the natural outlet increases. If water is added annually at a faster rate than it can travel to the outlet to be discharged, the water table will rise in search of outlets. When the water table approaches the land surface, agricultural production may be adversely affected, and additional manmade outlets in the form of drains must be installed. The drains keep the water table from encroaching into the root zone. A depth-to-water table of 0.9 to 1.5 meters (3 to 5 feet) is generally satisfactory, depending on local conditions including type of crops grown.

The data obtained by observing an operating drainage system can be used to verify the design capacity and drainage requirements for a new system, provided the soils, cropping pattern, climate, water management, and other conditions are similar. Before any data from an operating project are used, the effectiveness of existing drains should be investigated. Only when these drains are functioning as expected should the data be used to verify the design of new systems.

4–11. Outlet Conditions.—*(a) Physical Constraints.*—One of the most important considerations in all drainage planning is to determine the adequacy of the outlet for the system of drains. An inadequate outlet must be made adequate by channel construction or pumping of the discharge. Either of these measures may affect the overall feasibility of a drainage project.

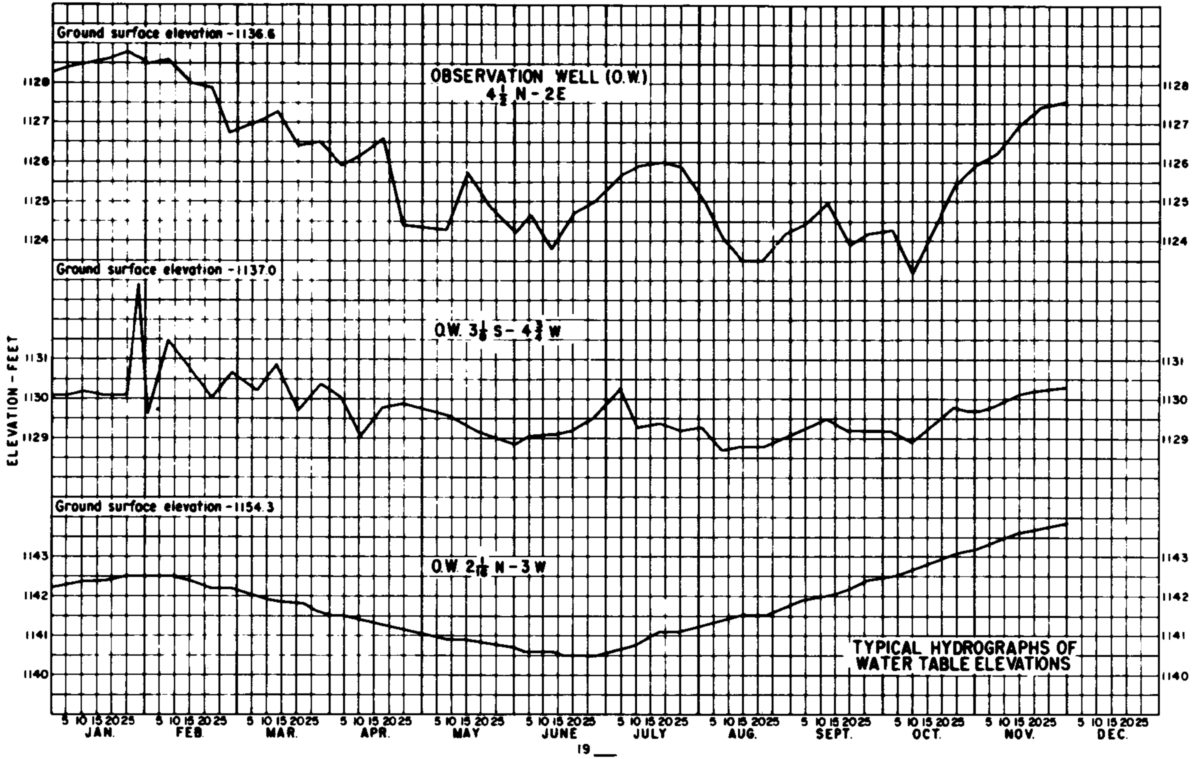


Figure 4–5.—Typical hydrographs of water table elevations. 103-D-780

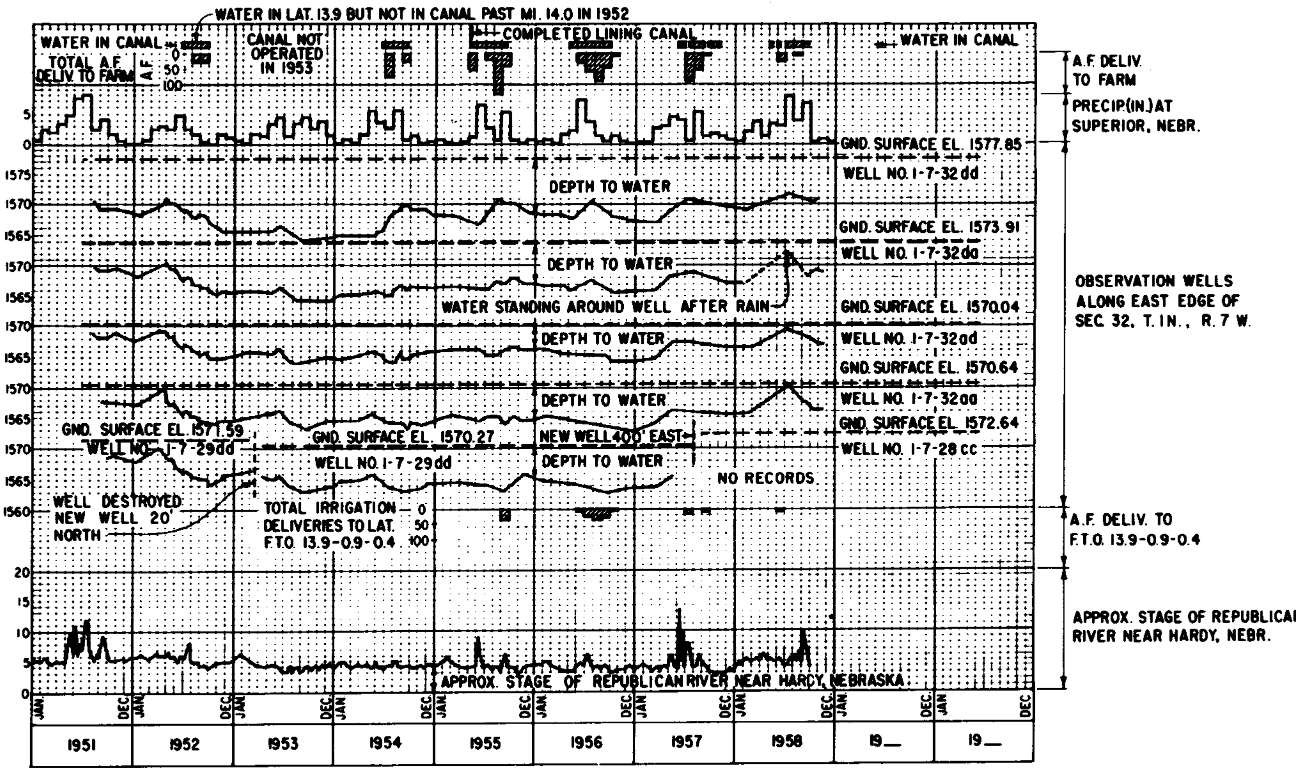


Figure 4–6.—Additional plotted data on an actual project hydrograph. 103-D-1638.

The investigations necessary to determine the adequacy of an outlet depend upon the characteristics of the stream or area which will serve as the outlet or disposal area. Where drainage systems will discharge into rivers, creeks, lakes, or other water bodies which are affected by high water, the elevation, frequency, and duration of the high water must be determined as accurately as possible, and the effect on the drainage system must be analyzed. These high-water elevations will limit the elevation of the hydraulic gradient at the lower end of the system. The water surface in gravity-drainage outlet works should coincide with the normal water surface of ponds, lakes, and reservoirs, unless studies show that high water will be of sufficient frequency and duration to be detrimental to drainage. Under usual circumstances, this means that the drained lands must lie about 3 meters (10 feet) or more above the outlet elevation if the lands are to be economically drained, although pumping can sometimes be justified.

High-water conditions can be obtained from gauge records, observation of watermarks on the banks of streams or lakes, and discussions with local residents. The adequacy of natural outlets can be determined by computing the estimated runoff from the entire area which they serve and checking their capacity.

There may be exceptional cases in which the effluent from surface drains may be disposed of by using sumps which allow the water to percolate into the ground and join the ground-water body. This method is possible only where the ground-water body itself discharges into a stream, other drainage features, or into an area where the water will not be a problem. The infiltration rate in these sumps must be high enough to support disposal of the necessary quantities to make the method economical. In some cases, inverted wells can be used to dispose of surface waste, provided adequate measures are taken to prevent aquifer contamination.

(b) Quality Requirements.—Quality of surface and ground water is an item of national concern. As most drainage systems discharge to surface waters, the drainage engineer needs to be aware of the effect drain effluent will have on those waters. State and national water-quality criteria are being refined to include trace elements and other potentially toxic constituents. Depending on the applicable water-quality standards, special discharge requirements may have to be met.

4-12. Drain Location.—There are no fixed rules or methods to direct the drainage engineer in locating every drain. Each location presents an individual problem which can be solved by analyzing the conditions involved. Wherever possible, outlet, suboutlet, and collector drains should be located in natural drainageways. Relief and interceptor drains should be located where they will produce the best drainage results. The location and spacing of drains require careful study and intuitive judgment on the part of the drainage engineer. As tentative drain locations are decided upon, they should be located on a map of the area. The centerlines of the drains should then be staked out on the site. Frequently, unmapped buildings, etc., at the construction site will make changes necessary in location or alignment of drains. In these instances, drain locations on the site should be changed as required and the tentative map locations revised to show the new alignments. Drain centerlines on the map should be scaled and stations marked for future reference. After the centerline has been staked on the ground, holes should be drilled along the centerline at various intervals down to the proposed drain depth to confirm that the drain is properly located in permeable material. Holes offset from the centerline should also be drilled for this purpose. Data collected from centerline drilling should be logged to provide information on construction conditions in addition to drainage parameters. This information should be provided to potential bidders as a part of the contract specifications. Those holes can also be used to confirm the gravel envelope design for the soils at actual drain depth. Stationing should start at the mouth of the outlet drain and proceed upstream. In some instances, first-order surveys may be required to establish centerlines, but quite often, in an open location, the line may be staked out visually with the use of range poles. In considering ditch locations, allowance should be made for sufficient right-of-way, usually 30 meters (100 feet).

4–13. Drain Numbering.—After drainlines have been laid out and staked, they should be given identifying numbers. No single numbering method fits all drain layout situations. One method adaptable to many situations is to locate station 0+00 of the suboutlet or collector drain with respect to land subdivisions and the junction of tributaries with respect to the suboutlet or collector. If station 0+00 of a collector drain is located in sec. 3, T. 7 N., R. 10 W., the number of the collector drain would become 3-7N-10W. Letters for the cardinal directions can be omitted if there is no possibility of confusion. If more than one collector drain discharges in sec. 3, the first could be 3A, the second 3B, etc. For example, if the first branch is located 975 meters (3,200 feet) upstream of the collector drain from station 0+00, the number of the tributary drain could be 3-7N-10W, 0.975 (3.2). If a tributary drain from both sides intersects the collector drain at this point, the one on the right (looking upstream) could be numbered 0.975R (3.2R) and the one on the left, 0.975L (3.2L). Junctions upstream from the tributary drain could be numbered the same way by adding to the previous number the distance to the upper junction from the lower junction in units and decimals of 1000 meters (feet). This system can be continued as necessary until the highest drain is numbered. It should be noted that using R and L does not conform to the hydraulic practice of assigning right and left when looking downstream, but does conform to drain surveying practice of starting the stationing at the outlet and proceeding upstream. If this method is not adaptable for a particular situation, another numbering method should be devised. Drain numbering is a valuable aid in locating the drains both on maps and in the field.

4–14. Existing Structures.—The location, elevations, and capacities of all existing bridges and culverts through which a proposed drain will pass should be determined. Bridge footings should be investigated and the elevations of road or railroad fills determined. The location of all utility lines and buildings which could have an effect upon the construction work should be noted and appropriate descriptions of structures and conditions obtained. Other possible structures that the designer should be aware of include buried water supply and powerlines to center-pivot sprinkler systems, and farm laterals both surface and buried, including parts of permanent sprinkler systems. Also, the trend to rural small acreage subdivisions requires care to ensure proper clearance of septic tanks and leach fields.

4-15. Economic Considerations of Drainage Problems.—Determining economic benefits has been primarily the responsibility of economists. The drainage engineer's responsibility has been to design drainage systems that do the best job for the least cost.

Drainage systems are most often justified by comparing the direct cost of the drains with the direct benefits of maintaining or increasing crop production. Net direct benefits of farm operation are compared with the total cost of the irrigation and drainage system. The comparison is usually made using the present worth of capitalized benefits and estimated costs. Benefits are capitalized over the life of the drain system; a 100-year life expectancy is used on most Bureau of Reclamation systems.

The economic analysis on a drainage system is usually left to economists; however, the engineer is often asked for a quick estimate of the economic feasibility of a project. To do this estimate, the engineer must have an estimate of net direct benefits by land class and the current interest rates for capitalization. In an area subject to salinization, the entire net benefit less the costs for the distribution system and operation and maintenance (O&M) can be used to justify drainage works. An example for a preliminary estimate follows:

Assume:

Interest rate = 5.5 percent
Average cost for irrigation works = $1,125 per hectare ($450 per acre)
Total drainage cost = $875 per hectare ($350 per acre)
O&M annual cost = $23.75 per hectare ($9.50 per acre)

Distribution of acreages by economic land class:

Class	*Hectares*	*Acres*
1	96	240
2	40	100
3	120	300
Total	256 hectares	640 acres

Net direct benefits by land class:

	Annual benefit		*Total annual benefit*
Class	*per hectare*	*per acre*	*[hectares (acres) × annual benefit]*
1	$181.25	$72.50	$17,400
2	156.50	62.60	6,260
3	107.75	43.10	12,930
			Total $36,590

Average annual benefit = $36,590/256 = $142.93 per hectare ($36,590/640 = $57.17 per acre)

Find: An estimate of the economic feasibility over the 100-year life expectancy of the system.

Present worth (PW of capitalized average annual benefit):

$$PW = \text{interest factor} \times \text{annual benefit}$$

$$PW = \frac{(1+i)^n - 1}{i(1+i)^n} \times \$142.93 = \$2{,}586.46 \text{ per hectare } (\$1{,}034.55 \text{ per acre})$$

where:

n = number of interest periods in years, and

i = interest rate at which compounding takes place over the period, n, expressed as a decimal fraction.

Present worth of capitalized annual O&M costs:

$$PW = \frac{(1+i)^n - 1}{i(1+i)^n} \times \$23.75 = \$429.78 \text{ per hectare} (\$171.91 \text{ per acre})$$

Cost summary:

Drainage	=	\$875 per hectare (\$350 per acre)
Irrigation	=	\$1,125 per hectare (\$450 per acre)
O&M	=	\$430 per hectare (\$172 per acre)
Total	=	\$2,430 per hectare (\$972 per acre)

$$\text{Benefit-cost (B/C) ratio} = \frac{\$2{,}586}{\$2{,}430}\left(\frac{\$1{,}034}{\$972}\right) = 1.06$$

Drainage projects having B/C ratios greater than 1 are generally considered feasible. However, this example is obviously borderline and may prove infeasible under a more detailed analysis, particularly if unquantified impacts on the environment are considered.

The above example assumes that no crop production can be expected shortly after the drainage problem develops. This assumption is reasonable in areas where saline conditions follow high ground water, and also assumes that irrigation is the best use of the land. In areas not affected, or only moderately affected by salts, the net benefit (if based on maximum production) must be adjusted downward to allow for reduced production because of poor drainage. In some cases, the benefits can be increased if drainage will increase yields over that used to determine net direct benefits. The exact amount of adjustment is difficult to determine. Theoretically, the total amount that could be spent on drainage would be the difference between maximum production without salts and production with a given level of salinity.

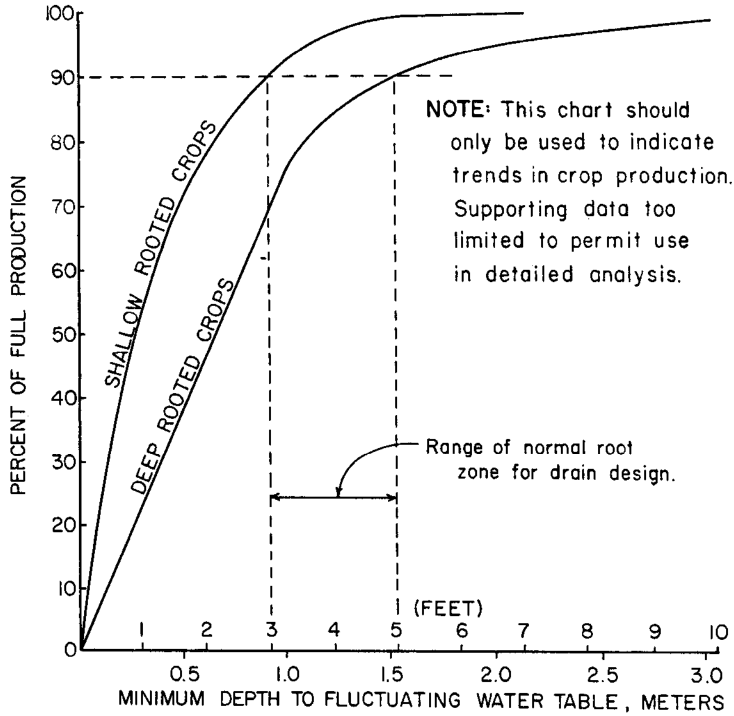


Figure 4–7.—Crop production response to a fluctuating water table. Drawing 103-D-1639.

Information regarding crop response to shallow, fluctuating water tables is limited. Figure 4–7 shows composite curves of available information on crop response to water table depths. The chart must be used judiciously, if at all, and is included in this manual only to indicate the general relationship between crops and water table levels. Most researchers report yield reductions when water tables fluctuate to levels less than 0.9 meter (3 feet) below ground surface.

If adequate data exist in the project area to develop charts similar to the one on figure 4–7, the average direct benefit presented in the previous example could be adjusted as follows:

Assume:

Annual benefits based on maximum production = \$142.93 per hectare (\$57.17 per acre)

Present minimum depth-to-water table = 0.67 meter (2.2 feet)

Crops are deep rooted.

From figure 4–7:

Percent of full production is 50 percent.

Adjusted annual benefit without drainage = \$142.93 × 0.50 = \$71.47 (\$28.59 per acre)

Annual benefit available for drainage = \$142.93 - \$71.47 = \$71.46 per hectare (\$28.58) per acre

Assuming the objective is to upgrade an operating project, the economic analysis could then be:

$$PW \text{ of annual benefit} = \frac{(1+i)^n - 1}{i(1+i)^n} \times \$71.46 = \$1{,}293 \text{ per hectare } (\$517 \text{ per acre})$$

Cost summary:

Drainage	=	\$875 per hectare (\$350 per acre)
O&M	=	\$430 per hectare (\$172 per acre)
Total	=	\$1,305 per hectare (\$522 per acre)
B/C ratio	=	1293/1305 (517/522) = 0.99

This approach would be valid, assuming present crop returns were sufficient to defray existing obligations and salts would not preclude production in the near future.

The approaches shown in the previous examples are highly simplistic and should be used only for preliminary estimates. Complete economic and repayment analyses for large projects should be made by qualified economists. This information, along with environmental considerations and other related factors, should be used in deciding the feasibility of drainage projects.

This manual does not address the problem of analyzing alternative costs for several approaches to a problem. For different methods of comparing costs of alternative plans and other information on making economic comparisons, see the Bureau of Reclamation publication *A Guide to Using Interest Factors In Economic Analysis of Water Projects* (Glenn and Barbour, 1970) and textbooks on engineering economics.

4-16. Drainage for Sprinkler Irrigation.—Sprinkler irrigation does not necessarily eliminate all possible drainage or salt problems. The leaching requirement must be considered in the design of all irrigation systems. If natural drainage is not adequate to remove the deep percolation without damage to plant roots, subsurface drains will be required.

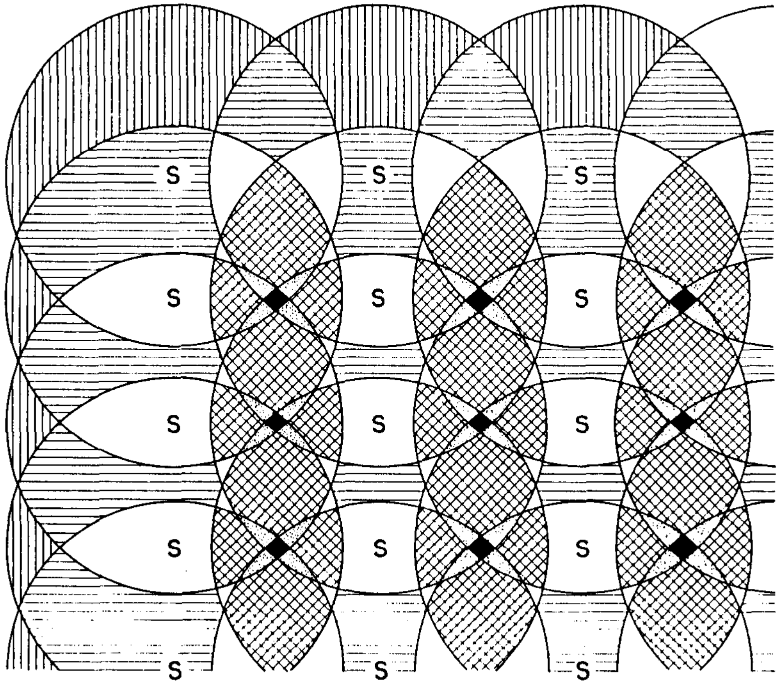
When the estimated deep percolation is based on the leaching requirement needed for salt balance, subsurface drainage requirements for sprinkler irrigation should be about the same as for good gravity irrigation. In areas of permeable surface soils having high infiltration rates, however, the minimum deep percolation under gravity irrigation will usually be more than required for salt balance. Consequently, the drainage requirements for gravity irrigation should be greater than for sprinkler irrigation. Properly designed sprinkler systems can offer a high degree of control for the total water application. Sprinkler application is not exactly uniform, however, and some areas receive more water than others. If the farmer does not apply sufficient water to bring the soil to field capacity over the entire field, crops in the drier areas may suffer from lack of moisture and will probably develop salt problems. If the farmer irrigates in a manner that ensures all areas sufficient water, some areas will receive more water than required which results in some deep percolation. Figures 4–8 and 4–9 show typical distribution patterns of two different sprinkler systems.

In the planning stage of a sprinkler-irrigated project, the drainage engineer must assume good sprinkler system design and careful operation. All subsurface investigations should be made, and the estimated drainage requirements should be determined to satisfy leaching requirements and normal deep percolation losses. Investigations should include ground-water movement from other areas, canal and lateral leakage, and studies of the water table fluctuations before and after irrigation. Measured deep percolation, if greater than that required for salt balance, should be used in designing the drainage system if the amount of deep percolation differs from planning stage estimates.

4–17. Tests for Estimating Deep Percolation From Sprinkler Systems.— The tests should be located in an area where the sprinkler lateral pressures are typical of the system. Several tests may be needed where large variations in pressure occur in the line because of topography or other factors.

Catch cans should be placed symmetrically in a grid covering an area sprinkled by two or three nozzles. These cans should be at least 10 centimeters (4 inches) in diameter and set at the center of 3- by 3-meter (10- by 10-foot) grids with the sprinklers placed at the grid corners. The cans should be set carefully with their tops parallel to the ground. Vegetation or other obstructions should not be permitted to interfere with entry of water into the cans. If necessary, the cans may be fastened to spikes to hold them upright. Water collected in the cans must be measured for two settings of the sprinkler line. The catch volume for each set must be added together to obtain the total catch volume in a grid square. Generally, all water caught in the cans can be assumed to infiltrate the soil. However, any significant runoff from the test field should be subtracted from the volume.

Measurements to be made are: (1) depth of water in the cans, (2) time for the water to accumulate, and (3) total time of irrigation per setting of the sprinkler line. If the water depth in the can is 50 millimeters (2 inches) or more, depths can be determined to plus or minus 2 millimeters (0.1 inch). For less than 50 millimeters (2 inches), the depths of catch should be determined from volumetric measurements to ensure accuracy.



TYPICAL SPRINKLER PATTERN

For 9m(30') spacing, 12m(40') radius, and 15m(50') move with less than an 8 km (5mi.) per hour wind at ground level.

S - Indicates location of sprinklers

NUMBER OF SPRINKLERS OVERLAPPING

0 2 3 4 5 6

SET TIME	14.3 HOURS
MAXIMUM TOTAL INFILTRATION	116mm (4.57")
DESIRED APPLICATION	76mm (3.0")
AVERAGE APPLICATION	95mm (3.75")
APPLICATION LESS THAN 3 INCHES	13 % OF PATTERN
APPLICATION EFFICIENCY	72 %
COEFFICIENT OF UNIFORMITY	80 %
AVERAGE DEEP PERCOLATION	21 %

Figure 4–8.—Typical sprinkler irrigation pattern. Drawing 103-D-1640.

Deep percolation is calculated by multiplying the catch rate (adjusted for losses if necessary) at each grid point by the average total time per set. The deep percolation is the difference between this product and the amount of moisture depleted since the last irrigation. Studies have indicated that deep percolation can vary over a wide range, from 9 to 30 percent of the amount of water infiltrating the soil surface. For a seasonal average, an overall farm efficiency of 65 percent can be expected with most sprinkler systems. A breakdown of farm losses under sprinkler irrigation could be as follows:

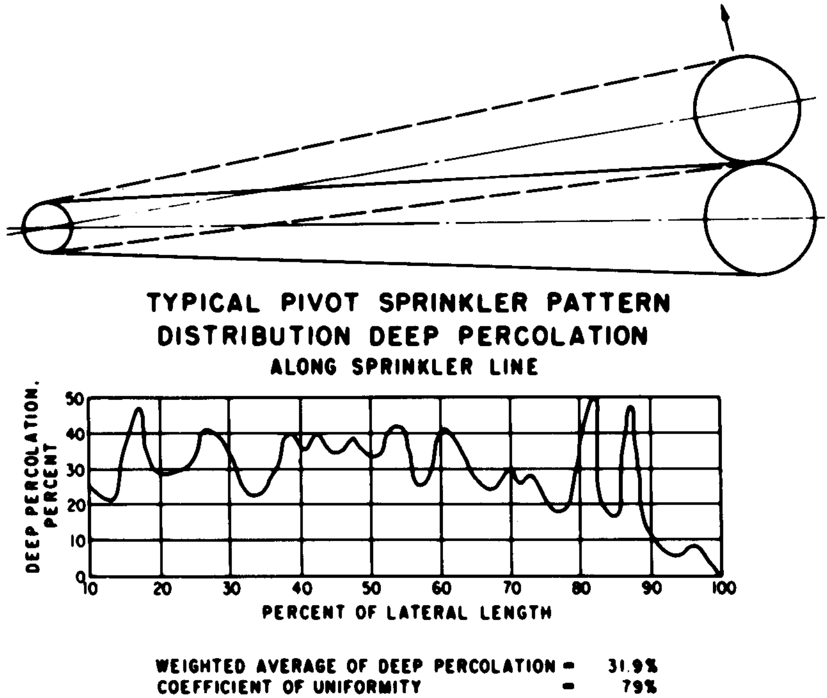


Figure 4–9.—Typical pivot sprinkler irrigation pattern. Drawing 103-D-1641.

	Percent
Evaporation and nonbeneficial consumptive use	10 to 15
Surface runoff	3 to 5
Deep percolation	15 to 22

The percentage losses shown above are based on the total amount of water delivered to the farm. This breakdown assumes the system is reasonably well designed for soil, topographic, and climatic conditions encountered in the field under study. The breakdown also assumes the farmer irrigates for a sufficient length of time to bring all of his land to field capacity upon each irrigation.

For very sandy soils in hot climates, deep percolation may be considerably higher than 22 percent of the total delivery because of the practice of using sprinklers to cool the crops. In very fine soils, surface runoff may exceed 5 percent, which can reduce deep percolation to quantities consistent with values obtained from gravity irrigation of fine-textured soils. Figure 2–6 summarizes the relationships between the deep percolation and infiltration rates and can be used for both sprinkler and gravity methods.

Limited information has been published regarding tests on pivot sprinklers; however, the information that has been gathered indicates that general values for deep percolation lie in the same range as for straight-line sprinkler systems. In evaluating a pivot system, the catch cans should be spaced the same distance as each sprinkler is spaced and beyond the last nozzle by a distance of one-half the radius of the circle covered by the last nozzle. The catch volume and time should be recorded for one complete pass of the sprinkler line.

4–18. Numerical Models.—Previous editions of this manual contained a section on building and using electric analog models for solving ground-water problems. Although electric models are still viable and useful tools, they are used infrequently these days. With the advent of low-cost digital computers, numerical models are more commonly employed to solve ground-water problems.

In the field of drainage and seepage, most numerical models use either the finite-difference method or the finite-element method to solve the governing partial differential flow equations. Numerical models are powerful tools for solving difficult problems. They can be used to solve complex problems involving nonhomogeneous anisotropic materials, highly variable problem geometry, spatial and temporal hydraulic stresses, and complex initial and boundary conditions for both saturated and unsaturated flow. Solute transport is increasingly more important, and models are available that provide this capability.

A number of robust, well-proven, and accepted general-purpose, finite-element and finite-difference codes are available at reasonable cost. Code selection should not be taken lightly; in choosing a code, cost should not be the sole criterion for selection. Some codes inherently deal with certain classes of problems better than others. Additionally, ease of use, documentation, and the availability of preprocessor and postprocessor utility programs can make the modeling task less burdensome.

The relative merits of the numerical method the code employs and the broad topic of constructing, calibrating, and verifying a numerical ground-water model are beyond the scope of this manual. The literature is replete with articles on these subjects.

Models can serve as an important framework into which all the available information can be integrated. Coarse, preliminary models and existing information can be used at the outset of a study to explore the sensitivity of parameters and to identify data deficiencies. When modeling is initiated early in a project, modeling and data collection can be coupled in an iterative process. Using the model as the framework for understanding, further data collection can be directed to specific areas of need, which results in a more thorough knowledge of the system and a more cost-effective use of available funds. Numeric models are not a panacea for a lack of information about the physical system. The model results are only as good as the data used and the assumptions made.

4–19. Bibliography.—

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⟨⟨Chapter V

DESIGN AND CONSTRUCTION

A. Spacing of Drains

5-1. Introduction.—Nearly all irrigated areas eventually require installation of some spaced drains. Proper spacing of these drains is very important but difficult in areas where field experience is inadequate or nonexistent. Spacing of drains that will be efficient, effective, and economical depends upon the full consideration of such factors as: depth of drain, depth to a slowly permeable barrier, hydraulic conductivity and specific yield of the soil, required depth of soil aeration for plant growth, effects of irrigation practices on deep percolation, length of irrigation season, number of irrigations, amount of deep percolation, climatic conditions, and irrigation water quality.

Every effort should be made to obtain information from operating systems in the vicinity of the study or in other areas where similar soil, topographic, climatic, and other related conditions permit comparisons. Such information may verify drainage requirements as determined from mathematical analyses. If wide variations exist in the spacing requirements between the field observations and the mathematical solution, field data should be checked to determine whether irrigation practices, moisture requirements, and water table conditions are satisfactory for optimum plant growth.

Most methods for estimating drain spacing are empirical and were developed to meet specific characteristics of a particular area. Some methods are based on assumptions of steady-state flow conditions where the hydraulic head does not vary with time. Other methods assume transient flow conditions where the hydraulic head changes with time. The very nature of precipitation and irrigation practices dictates that storage and discharge of ground water follow a transient or nonsteady-state flow regimen.

5-2. Transient Flow Method of Drain Spacing.—In the 1950's, the Bureau of Reclamation developed a method for estimating drain spacing based on transient flow conditions that relates the behavior of the water table to time and drain spacing. The validity of this method is demonstrated by the close correlation between actual spacing and drawdown values, and the corresponding predicted values. Reclamation's method of determining drain spacing accounts for time, water quantity, geology, and soil characteristics pertinent to the irrigation of specific areas. Although this method was developed for use in a relatively flat area, laboratory research and field experience show the method is applicable for areas having slopes up to 10 percent. Figures 5–1 and 5–2 compare measured values of drain spacing and water table heights with predicted values using Reclamation's methods.

5–3. Background of the Method.—In general, water tables rise during the irrigation season in response to deep percolating water from irrigation applications. In arid areas, water levels reach their highest elevation after the last irrigation of the season. In areas of year-round cropping, maximum levels occur at the end of the peak period of irrigation. The water table recedes during the slack or nonirrigation period and starts rising again with the beginning of irrigation the following year. Nearly all shallow water tables exhibit this cyclic phenomenon on an annual basis. Shallow water table rises also occur after each recharge to the ground water from precipitation or irrigation. Lowering of the water table occurs between recharges.

If annual discharge from an area does not equal or exceed annual recharge, the general cyclic water table fluctuation trend will progress upward from year to year. Specifically, the maximum and minimum water levels both reach progressively higher levels each year. When the annual discharge and recharge are about equal, the range of the cyclic annual water table fluctuation becomes reasonably constant. This condition is defined as "dynamic equilibrium."

Figure 5–3 shows two ground-water hydrographs that indicate how the above conditions developed under irrigation in two specific areas. The hydrograph for (A) on this figure shows the upward cyclic trend and the stabilization of the cyclic fluctuation. Dynamic equilibrium occurred when the maximum water table elevation reached a point sufficiently below ground level to preclude the need for artificial drainage. The hydrograph for (B) shows a similar upward trend of the water table in another area. At this location, the maximum 1956 water table elevation and the continued upward trend indicated the imminence of a damaging water table condition in 1957. Therefore, a drain was constructed early in 1957, and its effect in producing dynamic equilibrium at a safe water table level is evident in the graph.

Reclamation's method of determining drain spacing takes into account the transient regimen of the ground-water recharge and discharge. The method gives spacings which produce dynamic equilibrium below a specified water table depth. The method also provides for consideration of specific soils, irrigation practices, crops, and climatic characteristics of the area under consideration.

5–4. Data Required.—Figure 5–4 shows graphically the relationship between the dimensionless parameters $\frac{y}{y_o}$ versus $\frac{KDt}{SL^2}$ and $\frac{Z}{H}$ versus $\frac{KHt}{SL^2}$ based on the transient flow theory. This figure shows relationships midpoint between drains for cases where drains are located above or on a barrier.

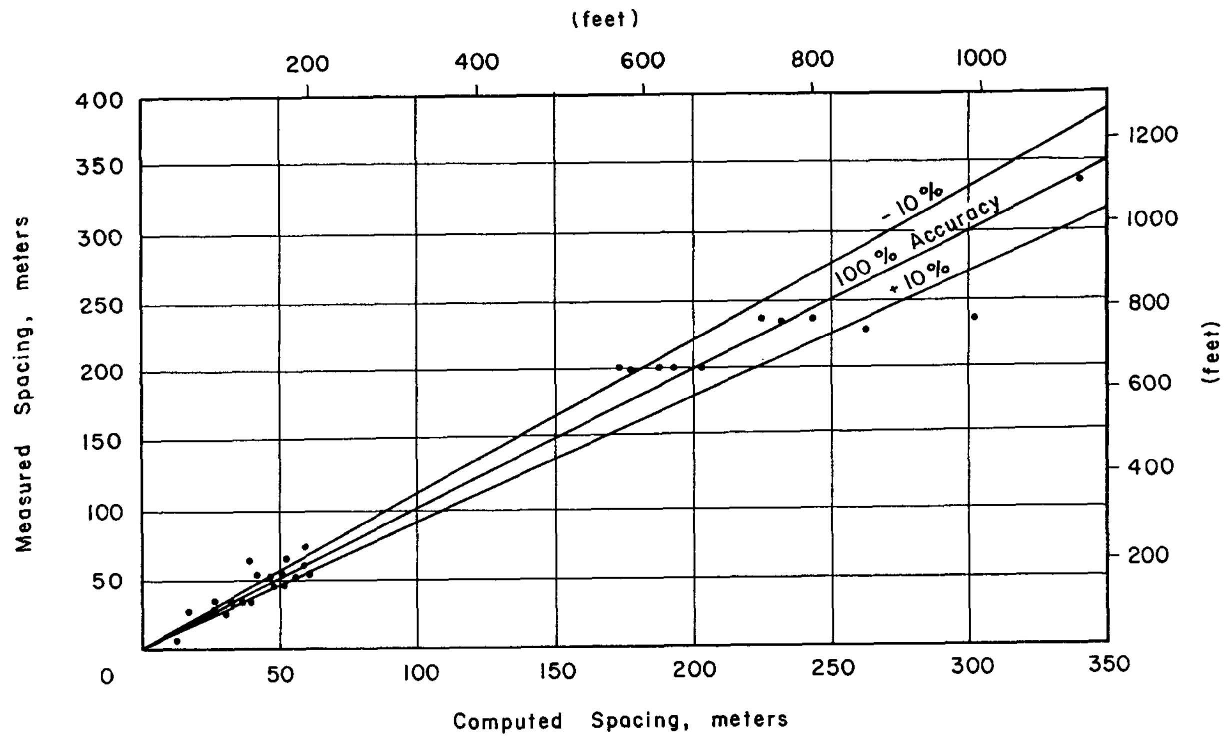


Figure 5–1.—Comparison between computed and measured drain spacings. Drawing 103–D–1649.

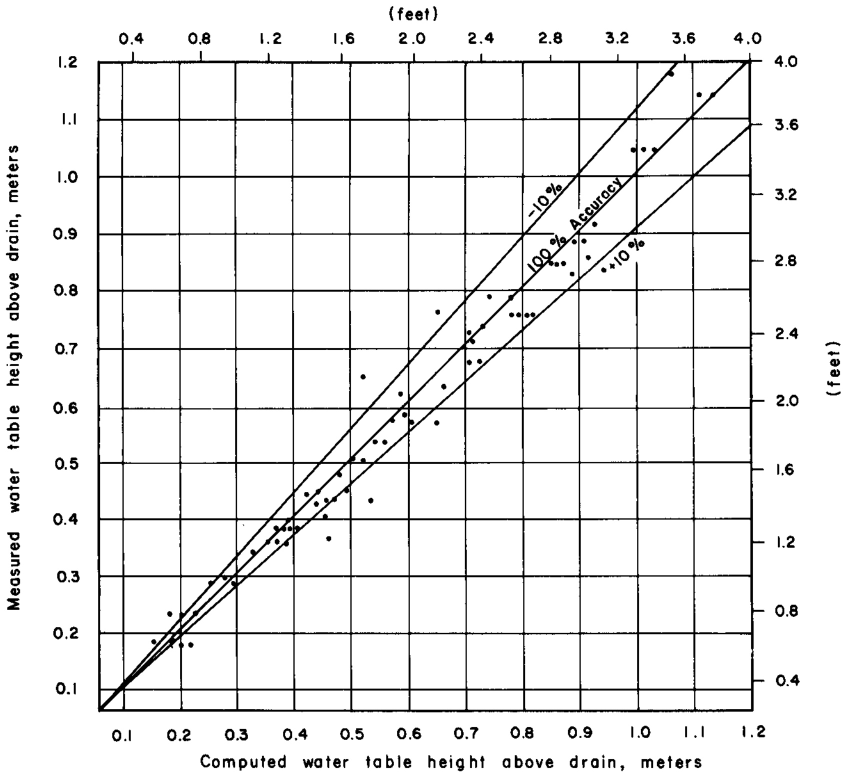


Figure 5-2.—Comparison between computed and measured water table heights above drains. Drawing 103-D-1650.

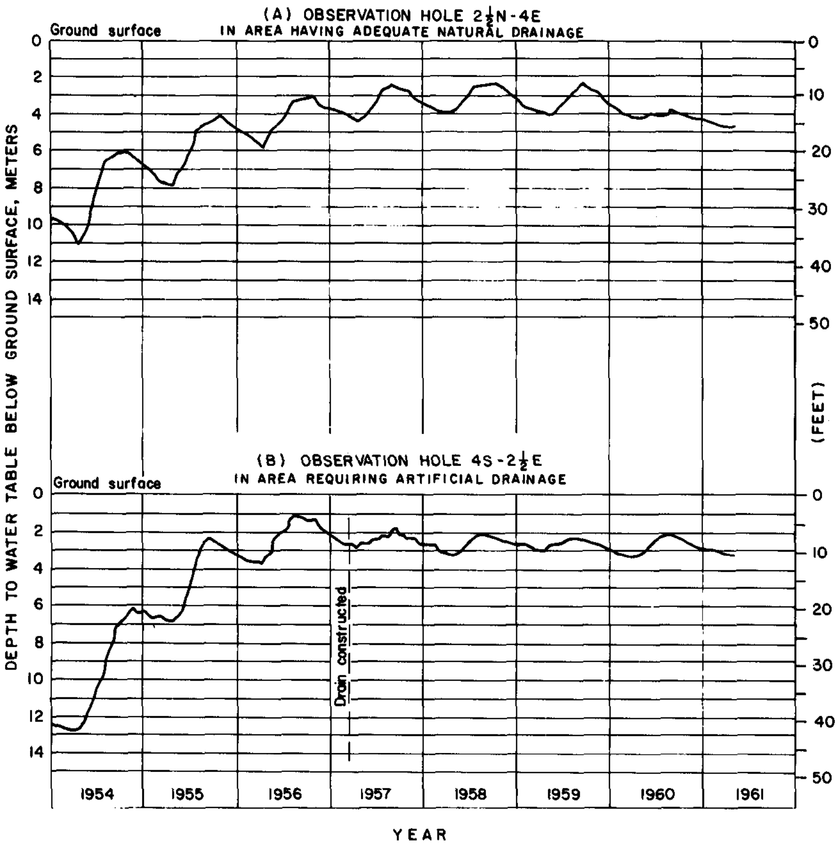


Figure 5–3.—Ground-water hydrographs. Drawing 103–D–777.

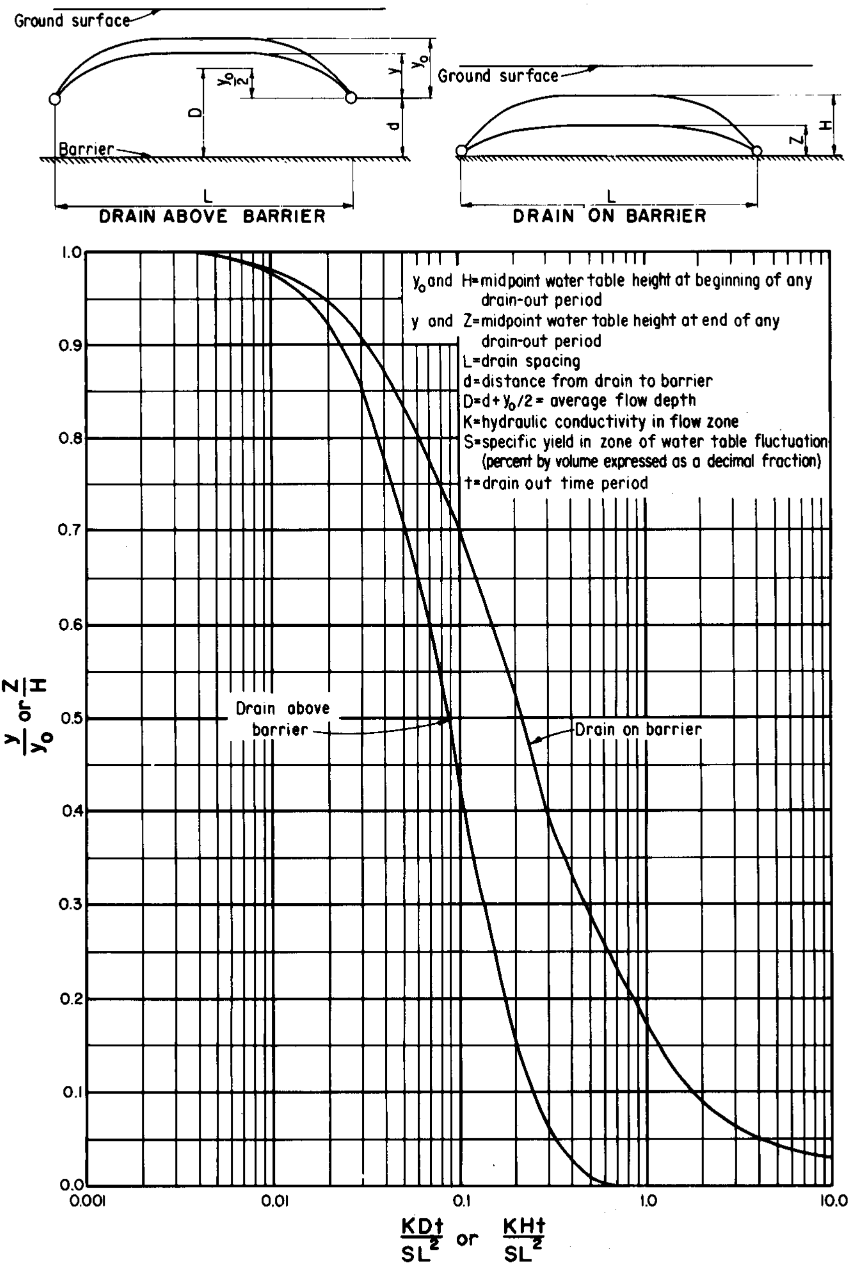


Figure 5–4.—Curves showing relationship of parameters needed for drain spacing calculations using the transient-flow theory. Drawing 103–D–1651.

Definitions of the various terms in the parameters are as follows:

(a) y_o, and H.—The water table height above the drain, midway between the drains and at the beginning of each individual drain-out period, is represented by y_o and H for drains above and on the barrier, respectively. As used in the drain spacing calculations, y_o and H represent the water table height immediately after a water table buildup caused by deep percolation from precipitation or irrigation. Parameter terms y_o and H also represent the height of the water table at the beginning of each new drain-out period during the lowering process which occurs in the nonirrigation season. The maximum values of y_o and H are based on the requirements for an aerated root zone which, in turn, are based on the crops and climatic conditions of each specific area.

(b) y and Z.—The water table height above the drain, midway between the drains and at the end of each individual drain-out period, is represented by y and Z for drains above and on the barrier, respectively. These terms represent the level to which the midpoint water table elevation falls during a drain-out period.

(c) Hydraulic Conductivity, K.—As used in this method, K represents the hydraulic conductivity in the flow zone between drains. Specifically, K is the weighted average hydraulic conductivity of all soils between the maximum allowable water table height and barrier, the barrier being a slowly permeable zone. The mathematical solution of the transient flow theory assumes homogeneous, isotropic soils in this zone. Such assumptions rarely exist; however, the use of a weighted K value has given a good correlation between measured and computed values for drain spacing and water table fluctuations. The K value is obtained by averaging the results from in-place hydraulic conductivity tests at different locations in the area to be drained.

(d) Specific Yield, S.—The specific yield of a soil is the amount of ground water that will drain out of a saturated soil under the force of gravity. S is approximately the amount of water held by a soil material, on a percent-by-volume basis, between saturation and field capacity. Specific yield, therefore, relates the amount of fluctuation of the water table to the amount of ground water added to or drained from the system. On the basis of considerable data, a general relationship has been developed between hydraulic conductivity and specific yield. This relationship is shown on figure 2–4 in chapter II, and values from this figure can be used to estimate specific yield values used in the drain spacing calculations in most cases.

Because the fluctuation of the water table in a drained area takes place in the soil profile zone between the drains and the maximum allowable water table height, it is reasonable to assume that the average specific yield in this zone will adequately reflect water table fluctuations. The use of figure 2–4 to estimate the specific yield requires that the weighted average hydraulic conductivity in this zone be determined.

The specific yield value, when used in the parameters of figure 5–4, accounts for the amount of drainout associated with lowering the water table. To determine the buildup of the water table from each increment of recharge, the depth of each recharge should be divided by the specific yield.

(e) Time, t.—This variable represents the drain-out time between irrigations or at specified intervals during the nonirrigation season. In an irrigated area, the time periods between irrigations have generally been established. Methods for estimating unestablished time periods are discussed in section 2–6. The drain spacing calculations should separate the longer nonirrigation season into two or three approximately equal time periods for accuracy in results.

(f) Flow Depth, D.—The flow depth is the average flow depth transmitting water to the drain. As shown on figure 5–4, D is equal to the distance from the barrier to the drain, plus one-half the distance from the drain to the midpoint water table at the beginning of any drainout period, $D = d + \frac{y_o}{2}$.

The theoretical derivation for the case where drains are located above a barrier was based on the assumption that the distance from the drain to the barrier, d, is large compared with the midpoint water table height, y_o. This poses a question regarding cases where the drains are above the barrier, but d is not large compared with y_o. In verifying the applicability of figure 5–4, studies have indicated when $\frac{d}{y_o} \leq 0.10$, the spacing computations should be made as if the drains were located on the barrier, and when $\frac{d}{y_o} \geq 0.80$, the computations should be made as if the drains were located above the barrier. A family of curves could be drawn between the two curves shown on figure 5–4, or a computer program could be used to account for the $\frac{d}{y_o}$ values between 0.10 and 0.80. The need for either of these refinements in the practical application of this method is not necessary.

(g) Drain Spacing, L.—The drain spacing is the distance between parallel drains. However, this distance is not calculated directly using this method. Values of L must be assumed until a solution by trial and error results in annual water table buildup and decline that will offset each other within acceptable limits. This resulting condition is defined as a state of dynamic equilibrium.

5–5. Convergence.—When ground water flows toward a drain, the flow converges near the drain. This convergency causes a head loss in the ground-water system and must be accounted for in the drain spacing computations. Figure 5–4 does not account for this convergency loss when the drain is above the barrier, and the drain spacing derived through the use of this curve is too large.

A method of accounting for convergence loss, developed by the Dutch engineer Hooghoudt, considers the loss in head required to overcome convergence in the primary spacing calculation. His method accounts for this head loss by using an equivalent depth, d', to replace the measured depth, d in the calculation of $D = d + \frac{y_o}{2}$. Hooghoudt's correction for convergence can be determined from the following equations:

$$d' = \frac{d}{1 + d/L(2.55 \ln d/r{-}c)} \quad \text{for } 0 < \frac{d}{L} \leq 0.31$$

$$d' = \frac{L}{2.55\,(\ln L/r{-}1.15)} \quad \text{for } \frac{d}{L} > 0.31$$

where:

d = distance from drain to barrier
d' = Hooghoudt's equivalent distance from drain to barrier
L = drain spacing
r = outside radius of pipe plus gravel envelope
c = $3.55 - 1.6\frac{d}{L} + 2\left(\frac{d}{L}\right)^2$
ln = $\log_e$ = Natural log

Curves have also been developed for determining d' and are shown on figures 5–5a, 5–5b, 5–6a, and 5–6b. These curves were developed for an effective drain radius, r, of 0.18 meter (0.6 foot) and should cover most pipe drain conditions. The effective drain radius is defined as the outside radius of the pipe plus the thickness of the gravel envelope. The use of the Hooghoudt method is also a trial and error process of assuming drain spacings. The d' value for the assumed spacing is obtained from figures 5–5a, 5–5b, 5–6a, or 5–6b and is used to obtain the corrected average flow depth, $D' = d' + \frac{y_o}{2}$. This method of correcting for convergence has been found to be most appropriate for use with Reclamation's method of determining drain spacing and discharge rates.

If the spacing that results from use of the equivalent depth d' is reduced by more than 5 percent from the spacing that results from use of the initial depth d, another iteration should be done using the initial depth d and the reduced spacing that resulted from the first d'.

If the drain spacing has been corrected for convergence and the drain discharge is to be computed from the formulas of section 5–11, the corrected average flow thickness, D', should be used.

Correction for convergence should also be made when using the steady-state drain spacing formulas of section 5–10.

The curve of figure 5–4 for the drain on the barrier is based on a solution with the convergence accounted for in the initial mathematical model. Therefore, no correction for convergence is required when using this curve.

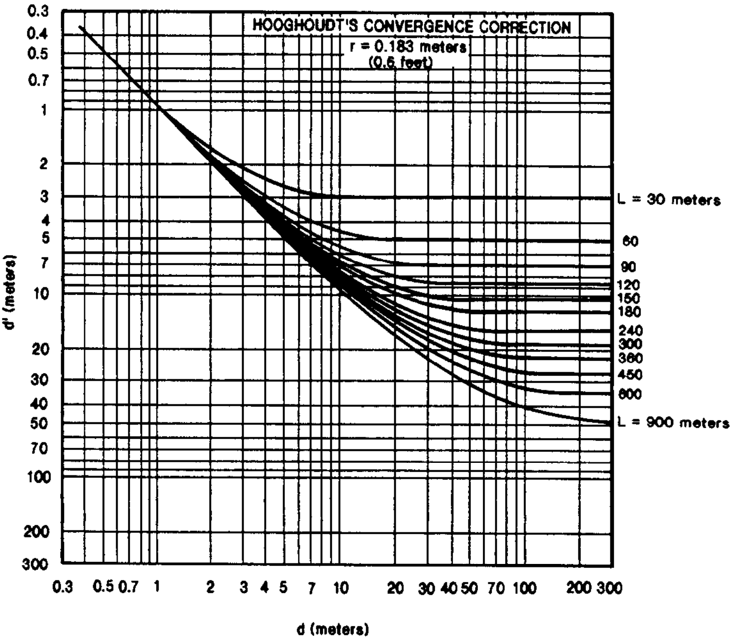


Figure 5–5a.—Curves for determining Hooghoudt's convergence correction (metric units). Drawing 103–D–1653.

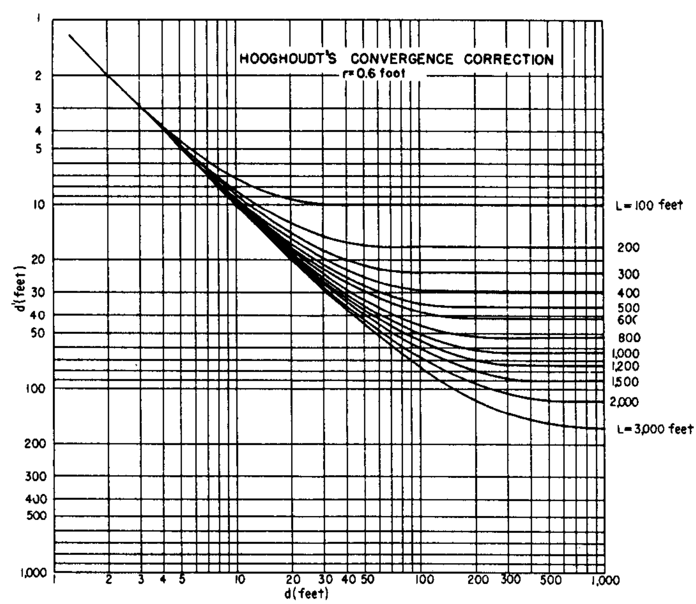


Figure 5–5b.—Curves for determining Hooghoudt's convergence correction (U.S. customary units). Drawing 103–D–1653.

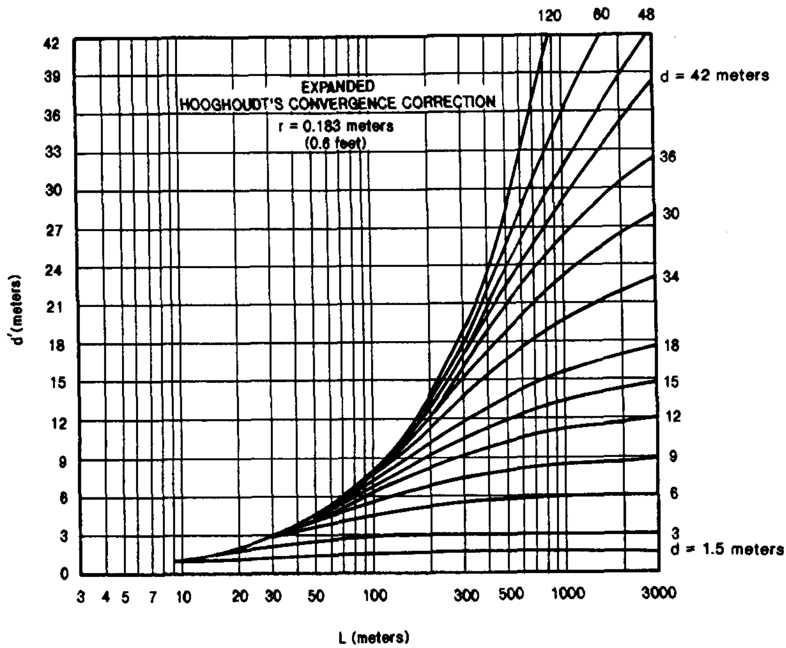


Figure 5–6a.—Expanded curves for determining Hooghoudt's convergence correction (metric units). Drawing 103–D–1654.

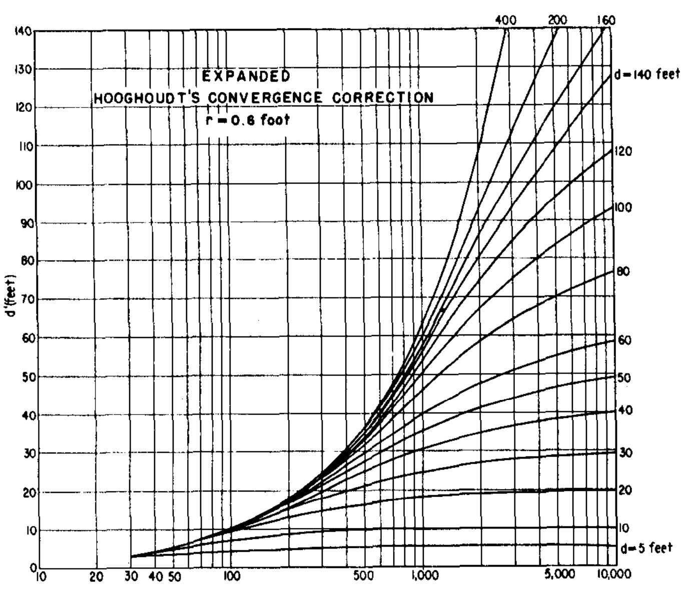


Figure 5–6b.—Expanded curves for determining Hooghoudt's convergence correction (U.S. customary units). Drawing 103–D–1654.

5–6. Deep Percolation and Buildup.—Deep percolation from any source causes a buildup in the water table. The methods of estimating drain spacing developed by the Bureau of Reclamation require that deep percolation and buildup in the water table from each source of recharge (rainfall, snowmelt, or irrigation application) be known or estimated and accounted for in the drain spacing calculations.

When a drainage problem exists on an operating project and drains are being planned, the buildup in the water table caused by irrigation applications can best be determined by field measurements. The water table depth should be measured at several locations in the area to be drained on the day before and on the day after several irrigation applications. The average buildup shown by these two measurements should be used in the spacing computations. These measurements obviate the need for theoretical estimates on the amount of deep percolation, and relate the buildup to the actual irrigation operations of the area to be drained.

In the planning stage of new projects or on operating projects where the measured buildup is not available, the amount of expected deep percolation must be estimated from each irrigation application. The buildup is computed by dividing the amount of deep percolation by the specific yield of the material in the zone where the water table is expected to fluctuate. Table 5–1 shows deep percolation as a percentage of the irrigation net input of water into the soil to be considered. These percentages are given on the basis of various soil textures and on infiltration rates of the upper root zone soils.

The following examples show how to use table 5–1 to obtain deep percolation and, in turn, the water table buildup:

Example 1:

Assume the irrigation application is known to be 150 millimeters (about 6 inches) per irrigation, soils in the root zone have a loam texture with an infiltration rate of 25 millimeters (1 inch) per hour, and about 10 percent of the 150-millimeter (6-inch) application runs off.

The net input of water into the soil per irrigation would then be 90 percent of the 150-millimeter (6-inch) application, or 135 millimeters (5.4 inches). From table 5–1, the deep percolation would be 20 percent for an infiltration rate of 25 millimeters (1 inch) per hour. Therefore, the deep percolation is $135 \times 0.20 = 27$ millimeters (1.08 inches). If the hydraulic conductivity in the zone between the root zone and the drain depth is 25 millimeters (1 inch) per hour, then the specific yield corresponding to this hydraulic conductivity is 10 percent, as given by figure 2–4. The buildup of the water table per irrigation is the deep percolation divided by the specific yield, or $\frac{27}{0.10} = 270$ millimeters (10.8 inches).

Table 5–1.—*Approximate deep percolation from surface irrigation (percent of net input).*

By texture

Texture	Percent	Texture	Percent
LS	30	CL	10
SL	26	SiCL	6
L	22	SC	6
SiL	18	C	6
SCL	14		

By infiltration rate

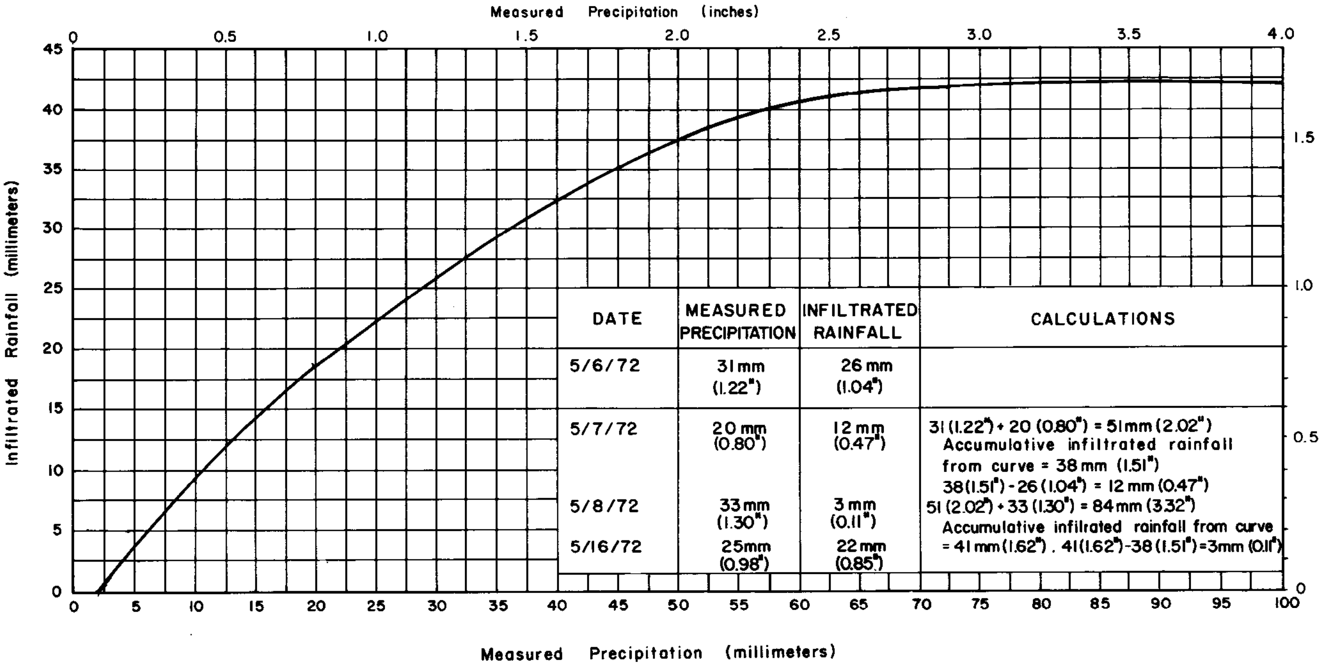
Inf. rate		Deep percolation,	Inf. rate		Deep percolation,
mm/h	(in/h)	percent	mm/h	(in/h)	percent
1.27	(0.05)	3	25.4	(1.00)	20
2.54	(.10)	5	31.8	(1.25)	22
5.08	(.20)	8	38.1	(1.50)	24
7.62	(.30)	10	50.8	(2.00)	28
10.2	(.40)	12	63.5	(2.50)	31
12.7	(.50)	14	76.2	(3.00)	33
15.2	(.60)	16	102.0	(4.00)	37
20.3	(.80)	18			

Example 2:

Assume the total readily available moisture in the root zone (allowable consumptive use between irrigations) has been determined as 107 millimeters (4.2 inches) and that the infiltration rate of the soil in the area is 25 millimeters (1 inch) per hour with a corresponding deep percolation of 20 percent.

The net input of water into the soil per irrigation will be $\frac{107}{0.80}$ = 134 millimeters (5.25 inches), where 0.80 = 1.00 – 0.20. The deep percolation will be 134 – 107 = 27 millimeters (1.05 inches). The buildup in the water table per irrigation would be this deep percolation amount divided by the specific yield in the zone between the drain and the maximum allowable water table.

Rainfall in arid areas is usually, but not necessarily, so small that the effects of deep percolation from this source during the irrigation season can be neglected. In semihumid areas, deep percolation from rain may be appreciable and must be accounted for in estimating subsurface drainage requirements. When it is apparent that precipitation is a significant source of soil moisture and deep percolation, the curve of figure 5–7 can be used to estimate the infiltrated precipitation. This infiltrated precipitation can then be used in a manner similar to that described in section 2–6 to determine the resultant irrigation schedule and the amount and timing of deep percolation from rainfall and irrigation. In areas that frequently have 3 or 4 days of rainfall separated by only 1 or 2 rainless days, the transient flow methods yield more accurate values for discharge if the accumulated deep



DATE	MEASURED PRECIPITATION	INFILTRATED RAINFALL	CALCULATIONS
5/6/72	31 mm (1.22")	26 mm (1.04")	
5/7/72	20 mm (0.80")	12 mm (0.47")	31 (1.22") + 20 (0.80") = 51 mm (2.02") Accumulative infiltrated rainfall from curve = 38 mm (1.51") 38 (1.51") - 26 (1.04") = 12 mm (0.47")
5/8/72	33 mm (1.30")	3 mm (0.11")	51 (2.02") + 33 (1.30") = 84 mm (3.32") Accumulative infilrated rainfall from curve = 41 mm (1.62"). 41 (1.62") - 38 (1.51") = 3 mm (0.11")
5/16/72	25 mm (0.98")	22 mm (0.85")	

Figure 5-7.—Curve for estimating infiltrated rainfall. Drawing 103-D-1655.

percolation from infiltrated precipitation is assumed to occur on the last day of rain.

Deep percolation from spring snowmelt occurs in some areas and should be accounted for where possible. In some areas, the buildup in the water table from this snowmelt can be measured in observation wells and used directly in the spacing computations. In other areas, the estimate may have to be based entirely on judgment and general knowledge of the area.

5–7. Using the Data.—The method of using the data described in section 5–3 to obtain dynamic equilibrium is briefly described in this section. A more detailed description is given in examples shown in subsequent sections. A computer program has also been developed by Reclamation personnel to perform drain spacing computations and analyze return flows for salinity studies.

The drain spacing computations have also been adapted for use on a personal computer. This program is called the Agricultural Drainage Planning Program (ADPP). The program manual and disks are available through the Superintendent of Documents, U.S. Government Printing Office.

Begin the calculations by assuming a drain spacing, L, and the assumption that the water table reaches its maximum allowable height, y_o, immediately after the last irrigation application of each season. At least two successive positions of the water table are calculated during the nonirrigation season (even in areas of year-round cropping, a slack period occurs sometime during the year). Then, the buildup and drainout from each irrigation is calculated for the irrigation season. If the assumed spacing results in dynamic equilibrium conditions, the water table height at the end of the series of calculations for the irrigation season will equal the maximum allowable water table height, y_o. If y_o after the last irrigation is not equal to the maximum allowable y_o the procedure is repeated with a different L. Normally, only two drain spacing assumptions are necessary to verify the dynamic equilibrium-producing spacing. A straight-lined relation between two assumed spacings and their resulting values of y_o after a complete annual cycle will permit determination of the proper spacing if the original assumptions are reasonably close.

Where the annual hydrograph peaks at some time other than the end of the irrigation season, the normal high point should be used as a starting point for calculations. This high point often occurs in the spring where sprinkler irrigation is used in semiarid or subhumid climates.

5–8. Drain Above the Barrier Layer.—The following example is given to illustrate the method of determining the drain spacing for a drain above the barrier. The following conditions are assumed:

(a) The distance from the barrier to the drain, d, is 6.7 meters (22 feet), and the depth of the drain is 2.4 meters (8 feet).

(b) The root zone requirement is 1.2 meters (4 feet), which gives a maximum allowable water table height, y_o, above the drain of $2.4 - 1.2 = 1.2$ meters ($8 - 4 = 4$ feet).

(c) The weighted average hydraulic conductivity in the zone between the barrier and the maximum allowable water table height is 127 millimeters (5 inches) per hour, or 3.05 meters (10 feet) per day.

(d) The hydraulic conductivity is uniform with depth. Therefore, the hydraulic conductivity in the zone between the maximum allowable water table height and the drain is also 127 millimeters (5 inches) per hour. From figure 2–4, the corresponding value of specific yield is 18 percent.

(e) The deep percolation from each irrigation (also assumed to be the same from a spring snowmelt) is 25.4 millimeters (1 inch), or 0.0254 meter (0.083 foot). The water table buildup from each increment of recharge is the deep percolation divided by the specific yield, or $\frac{.0254}{0.18} = 0.14$ meter (0.46 foot).

(f) The approximate dates of the snowmelt and the irrigation applications are as follows:

Irrigation or snowmelt (SM)	*Date*	*Time between irrigations, days*
SM	April 22	
First	June 6	45
Second	July 1	25
Third	July 21	20
Fourth	August 4	14
Fifth	August 18	14
Sixth	September 1	14
		132

Therefore, the nonirrigation period is 233 days (365 − 132). As previously mentioned, this period should be divided into two or three approximately equal periods; for this example, use two periods: one of 116 days and one of 117 days.

A drain spacing, L, of 442 meters (1,450 feet) resulted from two prior trial calculations. Assuming that the water table reaches the maximum allowable height immediately after the application of the last irrigation of each season, the computations begin at this point in time.

The first step in applying the method is to compute the $\frac{KDt}{SL^2}$ value for the first time period. Using this value, the value of $\frac{y}{y_o}$ is then found from figure 5–4. Knowing the initial y_o, we can then calculate y, the height to which the midpoint water table falls during this time period. This process is repeated for each successive time period, which results in a water table height for each successive recharge and drainout. The process is shown in tables 5–2a and 5–2b.

Table 5–2a.—*Computation of water table fluctuation in meters with drain above the barrier layer.*

①	②	③	④	⑤	⑥	⑦	⑧
Irrigation No.	Time period, t, days	Buildup per irrigation, meters	y_o, meters	D, meters	$\frac{KDt}{SL^2}$	$\frac{y}{y_o}$	y, meters
6							
	117		1.22	7.31	0.0742	0.575	0.701
	116		0.701	7.05	.0710	.590	0.414
SM		0.140					
	45		0.554	6.98	.0272	.870	0.482
1		.140					
	25		0.622	7.01	.0152	.958	0.596
2		.140					
	20		0.736	7.07	.0123	.978	0.720
3		.140					
	14		0.860	7.13	.0087	.985	0.847
4		.140					
	14		0.987	7.19	.0087	.985	0.972
5		.140					
	14		1.112	7.26	.0088	.985	1.095
6		.140					
			1.235				

Table 5–2b.—*Computation of water table fluctuation in feet with drain above the barrier layer.*

①	②	③	④	⑤	⑥	⑦	⑧
Irrigation No.	Time period, t, days	Buildup per irrigation, feet	y_o, feet	D, feet	$\frac{KDt}{SL^2}$	$\frac{y}{y_o}$	y, feet
6							
	117		4.00	24.00	0.0742	0.575	2.30
	116		2.30	23.15	.0710	.590	1.35
SM		0.46					
	45		1.82	22.91	.0272	.870	1.58
1		.46					
	25		2.04	23.02	.0152	.958	1.95
2		.46					
	20		2.41	23.20	.0123	.978	2.36
3		.46					
	14		2.81	23.41	.0087	.985	2.77
4		.46					
	14		3.22	23.61	.0087	.985	3.17
5		.46					
	14		3.63	23.82	.0088	.985	3.58
6		.46					
			4.04				

Explanation of each column:

Column ①.—Number of each successive increment of recharge, such as snowmelt (SM), rain, or irrigation.

Column ②.—Length of drainout period (time between successive increments of recharge or between incremental drainout periods).

Column ③.—Instantaneous buildup from each recharge increment (deep percolation divided by specific yield).

Column ④.—Water table height above drains at midpoint between drains immediately after each buildup or at beginning of incremental time periods during the nonirrigation season drainout (col. ⑧ of preceding period plus col. ③ of current period).

Column ⑤.—Average depth of flow, $D = d + \frac{y_o}{2}$ (d should be limited to $\frac{L}{4}$).

Column ⑥.—A calculated value representing the flow conditions during any particular drainout period: $\frac{K}{SL^2}$ × col. ⑤ × col. ②.

Column ⑦.—Value taken from the curve on figure 5–4.

Column. ⑧—Midpoint water table height above drain at end of each drainout period, col. ④ × col. ⑦.

Table 5–2 shows a final y_o = 1.235 meters (4.04 feet), which is approximately equal to the maximum allowable y_o of 1.22 meters (4.00 feet). Therefore, the spacing of 442 meters (1,450 feet) results in dynamic equilibrium. As stated in section 5–4, this spacing solution does not account for head loss due to convergence. Using Hooghoudt's method of correcting for convergence as given in section 5–4 and using figure 5–5, we find that for d = 6.7 meters (22 feet) and a drain spacing of 442 meters (1,450 feet), the equivalent depth, d', is 6.1 meters (20 feet). The D' to be used in the drain spacing computations is: $D' = d' + \frac{y_o}{2} = 6.1 + \frac{y_o}{2}$. The trial and error approach is again used to find the corrected spacing of 427 meters (1,400 feet). Table 5–3 shows the results of using D' with a spacing of 427 meters (1,400 feet).

The calculations in table 5–3 result in essentially the same water table heights, y_o, that were obtained in the previous calculations in table 5–2 and verify the 427-meter (1,400-foot) spacing as corrected for convergence. Figure 5–8 illustrates the water table fluctuation produced as a result of the conditions of this example.

Table 5–3a.—*Computation of water table fluctuation in meters with drain above the barrier layer using D′ as corrected by Hooghoudt.*

Irrigation No.	t, days	Buildup per irrigation, meters	y_o, meters	D', meters	$\frac{KD't}{SL^2}$	$\frac{y}{y_o}$	y, meters
6							
	117		1.22	6.71	0.0730	0.565	0.69
	116		0.689	6.44	.0695	.600	0.41
SM		0.140					
	45		0.554	6.73	.0267	.870	0.48
1		.140					
	25		0.622	6.41	.0149	.955	0.59
2		.140					
	20		0.736	6.46	.0120	.970	0.71
3		.140					
	14		0.856	6.52	.0085	.986	0.84
4		.140					
	14		0.987	6.59	.0086	.986	0.97
5		.140					
	14		1.112	6.65	.0087	.985	1.09
6		.140					
			1.235				

Table 5–3b.—*Computation of water table fluctuation in feet with drain above the barrier layer using D′ as corrected by Hooghoudt.*

Irrigation No.	t, days	Buildup per irrigation, feet	y_o, feet	D', feet	$\frac{KD't}{SL^2}$	$\frac{y}{y_o}$	y, feet
6							
	117		4.00	22.00	0.0730	0.565	2.26
	116		2.26	21.13	.0695	.600	1.36
SM		0.46					
	45		1.82	20.91	.0267	.870	1.58
1		.46					
	25		2.04	21.02	.0149	.955	1.95
2		.46					
	20		2.41	21.21	.0120	.970	2.34
3		.46					
	14		2.80	21.40	.0085	.986	2.76
4		.46					
	14		3.22	21.61	.0086	.986	3.17
5		.46					
	14		3.63	21.82	.0087	.985	3.58
6		.46					
			4.04				

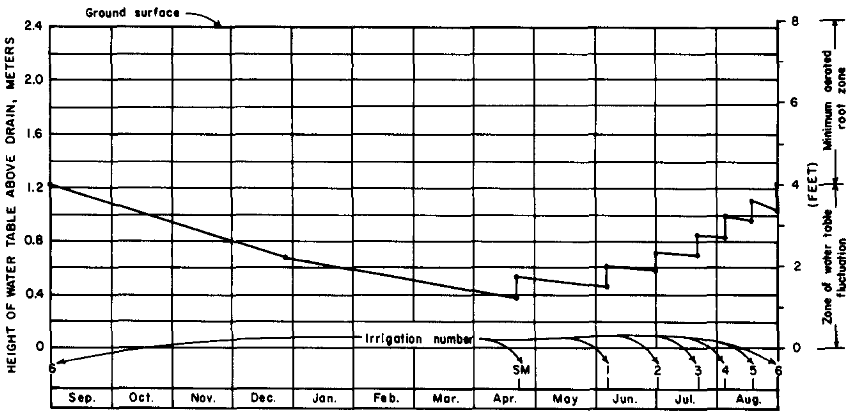


Figure 5–8.—Water table fluctuation chart for example problem. Drawing 103–D–776.

5–9. Drain on the Barrier Layer.—The following example is given to illustrate the method for determining the drain spacing for a drain on the barrier. All assumptions are the same as those in the example of section 5–8 except that d in this example is zero. The assumption of a drain spacing and subsequent computations of water table heights are also similar to those for a drain above the barrier.

A drain spacing of 125 meters (410 feet) is assumed, and subsequent computations are shown in tables 5–4a and 5–4b.

Table 5–4a.—*Computation of water table fluctuation in meters with drain on the barrier layer.*

Irrigation No.	Time period, t, days	Buildup per irrigation, meters	H meters	$\frac{KHt}{SL^2}$	$\frac{Z}{H}$	Z, meters
6						
	117		1.22	0.1546	0.590	0.719
	116		0.719	.0905	.720	0.518
SM		0.140				
	45		0.658	.0321	.900	0.591
1		.140				
	25		0.732	.0199	.945	0.691
2		.140				
	20		0.832	.0180	.950	0.789
3		.140				
	14		0.930	.0141	.975	0.911
4		.140				
	14		1.051	.0159	.970	1.015
5		.140				
	14		1.158	.0176	.955	1.103
6		.140				
			1.243			

Table 5–4b.—*Computation of water table fluctuation in feet with drain on the barrier layer.*

Irrigation No.	Time period, t, days	Buildup per irrigation, feet	H feet	$\frac{KHt}{SL^2}$	$\frac{Z}{H}$	Z, feet
6						
	117		4.00	0.1546	0.590	2.36
	116		2.36	.0905	.720	1.70
SM		0.46				
	45		2.16	.0321	.900	1.94
1		.46				
	25		2.40	.0199	.945	2.27
2		.46				
	20		2.73	.0180	.950	2.59
3		.46				
	14		3.05	.0141	.975	2.99
4		.46				
	14		3.45	.0159	.970	3.33
5		.46				
	14		3.80	.0176	.955	3.62
6		.46				
			4.08			

Table 5–4 shows a final H = 1.243 meters (4.08 feet), which is essentially equal to the maximum allowable H of 1.22 meters (4.00 feet). Therefore, the spacing of 125 meters (410 feet) results in dynamic equilibrium, and because no correction for convergence is required for this case, the final drain spacing is 125 meters (410 feet).

5–10. Other Uses for Transient Flow Curves.—The transient flow method is valid for either irrigated areas (dry climate) or humid areas. However, this manual emphasizes drainage for irrigation in dry climates.

At times, the drainage engineer is interested in the time necessary to lower a water table to some specified level, or may be asked for a drain spacing that will lower the water table to a specific depth in a specified time. The basic data regarding hydraulic conductivity, depth to barrier, specific yield, time, and drain spacing are as relevant in these problems as in the previously illustrated problems. The main difference is the simplicity in solving these problems as shown in the following examples:

Example 1: Drain above the barrier.

Assume: K = 0.305 meter (1.0 foot) per day, d = 6.1 meters (20 feet), depth to drain = 2.7 meters (9 feet), water table at ground surface at t = 0, specific yield = 7 percent, and existing drains are 91 meters (300 feet) apart.

Determine: Time required for the water table to drop 1.5 meters (5 feet) below the ground surface.

Because the water table is initially at the ground surface,

$$y_o = 2.7 \text{ meters (9 feet);}$$

$$D = d + \frac{y_o}{2} = 7.45 \text{ meters (24.5 feet);}$$

$$d' = 4.4 \text{ meters (14.5 feet) from figure 5–5; and,}$$

$$D' = d' + \frac{y_o}{2} = 5.75 \text{ meters (19 feet).}$$

$$y = 2.7 - 1.5 = 1.2 \text{ meters (4 feet)}$$

$$\frac{y}{y_o} = \frac{1.2}{2.7} = 0.444$$

From figure 5–4, $\frac{KD't}{SL^2} = 0.096$ when $\frac{y}{y_o} = 0.444$

Solving the parameter $\frac{KD't}{SL^2} = 0.096$ for t (metric and U.S. customary units):

$$\text{Metric, } t = \frac{0.096\,SL^2}{KD'} = \frac{0.096(0.07)(91)^2}{(0.305)(5.75)} = 31.7 \text{ days}$$

$$\text{U.S. customary, } t = \frac{0.096\,SL^2}{KD'} = \frac{0.096(0.07)(300)^2}{(1)(19)} = 31.8 \text{ days}$$

From the above calculations, the water table will drop 1.5 meters (5 feet) below the ground surface in about 32 days.

Example 2: Using example 1, determine the drain spacing required to drop the water table 1.5 meters (5 feet) below the ground surface in 20 days.

Using a similar approach, $\frac{KD't}{SL^2} = 0.096$, when $\frac{y}{y_o} = 0.444$.

$$\text{Then, } L = \left(\frac{KDt}{0.096S}\right)^{1/2} = \left[\frac{(0.3)(7.45)(20)}{(0.096(0.07)}\right]^{1/2} = 81.6 \text{ meters (267 feet)}$$

(uncorrected for convergence)

From figure 5–5, $d' = 4.00$ meters (13.1 feet) and $D' = d' + \frac{y_o}{2} = 5.35$ meters (17.5 feet).

$$L = \left[\frac{(0.3)(5.35)(20)}{(0.096)(0.07)}\right]^{1/2} = 69.1 \text{ meters (227 feet) (second trial)}$$

From figure 5–5, $d' = 3.9$ meters (12.8 feet) and $D' = 5.25$ meters (17.2 feet).

$$L = \left[\frac{(0.3)(5.25)(20)}{(0.096)(0.07)}\right]^{1/2} = 68.5 \text{ meters (224 feet) (corrected drain spacing).}$$

A drain spacing of 68.5 meters (224 feet) is required to lower the water table 1.5 meters (5 feet) below the ground surface in 20 days.

5–11. Drain Spacing Using Steady-State Formulas.—The theory of steady-state drainage considers a uniform, steady rate of recharge to the drainage system which, under specified conditions of depth of drain, depth to barrier, hydraulic conductivity, and drain spacing, will cause the water table between the drains to rise to and remain at some height so long as that rate of recharge continues.

For each set of physical conditions (depth of drain, depth to barrier, height of water table between drains, and hydraulic conductivity), there is a different drain spacing for each assumed value of steady recharge. Therefore, the validity of the drain spacing obtained by use of the steady-state formulas depends on the assumed steady recharge. The steady-state assumptions seldom represent the conditions produced as a result of the intermittent recharges from irrigation applications and the transient flow conditions. The method of determining the steady recharge rate is based on the experience of Reclamation engineers in comparing transient and steady-state solutions.

The steady-state drain spacing formula generally used in the irrigated areas of the United States is the Donnan formula.

$$\text{Donnan formula, } L^2 = \frac{4K\,(b^2 - a^2)}{Q_d} \qquad (1)$$

where:

L = drain spacing, meters (feet);
K = hydraulic conductivity, meters (feet) per day;
a = distance between drain depth and barrier, meters (feet);
b = distance between maximum allowable water table height between drains and the barrier, meters (feet); and
Q_d = recharge rate, cubic meters per square meter (cubic feet per square foot) per day.

Note: This formula is valid for any consistent set of units.

As previously mentioned, the validity of this formula depends upon the value of Q_d used. Through experience, engineers have found that Q_d should be derived by dividing the unit depth of deep percolation from an irrigation application by the number of days between irrigations during the peak portion of the irrigation season. This value of steady recharge should be used for the case where drains are above a barrier. Where drains are on a barrier, it has been found that this recharge rate should, generally, be divided by two.

The following examples show the use of the Donnan formula:

Example 1: Assume the conditions of the previous example in section 5–8, where the drains were located above the barrier and the transient flow method was used.

From section 5–8:

Deep percolation = 25 millimeters (1 inch) = .025 meter (0.083 foot);
Number of days between irrigations during peak of season = 14 days;
$d = 6.7$ meters (22 feet), maximum $y_o = 1.22$ meters (4 feet);

$D = d + \frac{y_o}{2} = 6.7 + \frac{1.22}{2} = 7.32$ meters (24 feet); and

$K = 3.05$ meters (10 feet) per day.

In steady-state nomenclature:

$a = d = 6.7$ meters (22 feet) and $a^2 = 44.9\ m^2$ (484 ft^2),
$b = d +$ max. $y_o = 6.7 + 1.22 = 7.92$ meters (26 feet) and $b^2 = 62.7\ m^2$ (676 ft^2), and

$Q_d = \frac{0.025}{14} = 0.0018$ meter (0.0059 feet) per day.

Using Donnan's formula:

$$L^2 = \frac{(4)(3.05)(62.7 - 44.9)}{0.0018} = 120{,}645\ m^2\ (1{,}300{,}000\ ft^2)$$

and $L = 347$ meters (1,140 feet) as compared to 442 meters (1,450 feet) by the transient flow method in section 5–8. Donnan's formula usually gives results that agree with the transient flow method within plus or minus 20 percent.

Example 2: Assume the conditions of the previous example in section 5–9, where the drains were located on the barrier and the transient flow method was used.

From section 5–9:

Deep percolation = .025 meter (0.083 foot),
Number of days between irrigations during peak of season = 14 days, $d = 0$, maximum $H = 1.22$ meters (4 feet), and $K = 3.05$ meters (10 feet) per day.

In steady-state nomenclature:

$a = d = 0$ and $a^2 = 0$;

$b = d + \text{max. } H = 0 + 1.22 = 1.22$ meters (4 feet) and $b^2 = 1.49$ m^2 (16 ft^2); and

$$Q_d = \frac{0.025}{14} = 0.0018 \text{ meter (0.0059 foot) per day.}$$

As mentioned previously, this value for Q_d should be divided by two for drains on the barrier. Then, $Q_d = \frac{0.0018}{2} = 0.0009$ meter (0.00295 foot) per day.

Using Donnan's formula:

$L^2 = \frac{(4)(3.05)(1.49)}{0.0009} = 20{,}200$ m^2 (217,000 ft ^{2}) and $L = 142$ meters (466 feet) as compared to 125 meters (410 feet) by the transient flow method in section 5–9.

The previous examples show that the steady-state method does not necessarily result in the same drain spacings as the transient flow methods. Because Q_d is an empirical value, this result is expected. The steady-state method does, however, give spacings which are reasonably close for use where quick estimates are needed or as good first approximations for the transient flow method. Very narrow spacings calculated by the steady-state method have been found invalid because of problems with the basic assumption of steady-state conditions. The drain spacings obtained using the steady-state method should be corrected for convergence, using the methods previously described in section 5–5.

5–12. Determining Discharge From Spaced Drains.—The discharge of spaced drains can be computed using the following formulas:

$$q_p = \frac{2\pi K y_o D}{86{,}400L} \text{ (for drains above a barrier)} \tag{1}$$

$$q_p = \frac{4KH^2}{86{,}400L} \text{ (for drains on a barrier)} \tag{2}$$

where:

q_p = discharge from two sides per unit length of drain, cubic meters per second per meter (cubic feet per second per foot);

y_o or H = maximum height of water table above drain invert, meters (feet);

K = weighted average hydraulic conductivity of soil profile between maximum water table and barrier or drain, meters (feet) per day;

D = average flow depth ($D = d + \frac{y_o}{2}$), meters (feet);

d = distance from drain to barrier, meters (feet); and
L = drain spacing, meters (feet).

The terms in the above formulas relate to the terms shown on figure 5–4.

Subsurface water flowing into an area from upslope sources can be evaluated quantitatively by use of the basic equation:

$$q_u = KiA \tag{3}$$

where:

q_u = unit flow, cubic meters (feet) per second;
K = weighted average hydraulic conductivity of the saturated strata above the barrier, meters (feet) per second;
i = slope (obtained from a ground-water table contour map along a line normal to the contours, because flow is in this direction); and
A = area through which flow occurs, square meters (feet).

Generally, the maximum water table height would be used to obtain the saturated depth from which K is obtained. This same depth would be used to obtain the area, A, for a unit width. The plane along which the area must be obtained is parallel to the contours or normal to the direction of flow.

An application of equation (3) is given in section 5–58.

The value of q_u in equation (3) is the total amount of moving water within the saturated profile above the barrier; however, an interceptor drain cannot be expected to pick up more than a portion of this water when the bottom of the drain is above the barrier. For practical purposes, the drain can be expected to intercept only that portion of the saturated profile above the water surface in the drain. Equation (3) then becomes:

$$q_u = KiA\frac{y}{y+d} \tag{4}$$

where:

q_u = volume rate of flow per unit length of drain from underflow sources;
K = hydraulic conductivity in meters (feet) per second;
i = slope of water table;
A = saturated area in square meters (feet) of flow in a unit length of width;
y = height in meters (feet) of maximum water surface immediately above proposed drain; and
d = distance in meters (feet) from drain invert to barrier.

The flow determined in this manner may originate from one or several upslope sources, depending on the circumstances. Some of these sources could be underflow from upslope irrigated farmland; seepage from canals at high elevations; or seepage from streams, lakes, or other water bodies. An evaluation of contributions from individual sources may be necessary, or a single computation for q_u may suffice. In making a single computation for q_u, the situation must be carefully considered to obtain either an average value or limiting high and low values. Water table contours will change throughout the year. It is important that records be available for at least a year so that an estimate of the values of i and A can be made.

Sometimes, the ground-water contribution from a surface water body such as a stream, pond, or lake must be evaluated. This evaluation may be done by analyzing surface and subsurface inflow, precipitation, transpiration and evaporation, imported and evaporated water, surface outflow, and the change in the surface storage.

Contributions to ground water by seepage from canals can be obtained by a ponding test. In this test, seepage loss can be measured by changes in volume, corrected as necessary for transpiration and evaporation losses. Other methods for estimating seepage losses are described in the following paragraphs.

In the planning phase of an irrigation project, consideration should be given to the effects seepage from unlined canals and laterals has on the drainage requirement. If lining is needed but not provided, additional drains may be required to protect nearby crops. A method of estimating the seepage losses from unlined canals and laterals is given in section 5–15.

To evaluate the benefits from reducing canal seepage to the ground water, the amount of this seepage must be known. The effect of canal lining on the drainage requirement can be determined and a cost comparison made between canal lining and drain construction. The drainage requirement may be reduced by lining the canals and in some instances may be eliminated. Lining of a canal does not permit the assumption that seepage is eliminated because even the best lining usually permits some seepage. The effect of canal lining on the drainage requirement will depend upon the capability of the formation to convey water in relation to the seepage rates.

Drains should be designed for the total accretions:

$$q = q_p + q_u \tag{5}$$

where:

q = cubic units of flow per unit of time per unit length of drain;
q_p = flow in above units due to deep percolation; and
q_u = flow in above units due to underflow from outside the area or due to seepage from surface water bodies.

5–13. Design Discharge for Collector Drains.—The discharge q in equation 5, determined for each unit length of pipe, can be used in the formula $Q = qL$, where Q is the discharge in cubic units per second at the end of a pipe L units long. This formula for Q is applicable for a length of pipe, L, which serves an area that can be irrigated within about 2 days. If q is the maximum rate of discharge per unit length of pipe, the formula gives the discharge only for the period that the water table is highest. At any other time, the rate of discharge will be less than maximum. For example, consider a collector drain receiving water from a group of drains serving an area that takes about 10 days to irrigate. Each of the branch drains will deliver water to the collector at a different rate, Q, depending on the value of q. The parcel which has been irrigated most recently will have the highest water table and the highest discharge, while the parcel irrigated first will have the lowest discharge. The other drains will discharge at rates somewhere between the highest and the lowest. The summation of the Q values from each branch drain, at a point on the collector drain, will be less than the maximum q multiplied by the total length of collector and all branch drains above that point.

The water table height and the resultant value of Q will fluctuate mainly because of the intermittent application of irrigation water, because the q value for canal seepage, underflow, etc., is nearly constant.

Little data exist on which to base a rationalization of the reduction in flow received by collector drains. In general, few drains will collect drainwater from more than about 2,000 hectares (5,000 acres) before they discharge into a deep, open suboutlet. The following equations will provide a reasonable design capacity for most collector drains:

$$\text{Drains above barrier:} \quad q = C\frac{2\pi K y_o D}{86{,}400L}\left(\frac{A}{L}\right) \tag{6}$$

$$\text{Drains on the barrier:} \quad q = C\frac{4KH^2}{86{,}400L}\left(\frac{A}{L}\right) \tag{7}$$

where:

q = discharge [cubic meters (feet) per second per unit area]; y_o, K, D, H, and L are as described in section 5–12;

A = area drained in square meters (feet); and

C = area discharge factor.

The factor C is the relationship between possible discharge and probable discharge, and is determined from table 5–5.

Table 5–5.—*Area discharge factors.*

Hectares drained	Acres drained	Factor, C
0–16	0–40	1.0
16–32	40–80	1.0–0.92
32–49	80–120	0.92–0.87
49–65	120–160	0.87–0.82
65–81	160–200	0.82–0.79
81–97	200–240	0.79–0.76
97–113	240–280	0.76–0.74
113–130	280–320	0.74–0.72
130–194	320–480	0.72–0.65
194–259	480–640	0.65–0.60
259–324	640–800	0.60–0.56
324–389	800–960	0.56–0.54
389–453	960–1,120	0.54–0.52
453–518	1,120–1,280	0.52–0.50
518–2,023	1,280–5,000	0.50

B. Interceptor Drains

5–14. Introduction.—The principal function of interceptor drains is to control ground-water levels on sloping lands. As a general rule, this control should be accomplished by pipe drains except where the drain must receive surface runoff. Open drains are more expensive to maintain than closed drains, and they also use producible land for their construction.

Interceptor drains are usually required at abrupt breaks in slope to control the water table on the lower slope. An interceptor drain should be placed on or as close to the barrier as practical, which usually means the drain is located at the toe of a break in slope. However, the drain can be located above the break if the drain is placed on the barrier.

Interceptor drains are required when the slope of the barrier converges with the ground surface slope. Under this condition, sufficient borings must be made to determine at what point the barrier is about 2.4 meters (8 feet) below the land surface. An interceptor drain at this location will intercept all water moving downhill. Specific conditions will determine the need for additional drains either upslope or downslope from the initial interceptor.

When there is an appreciable decrease in the hydraulic conductivity on the slope, the water table rises to compensate for the reduced conductivity by increasing the flow area. This may cause the water table to approach the land surface. As was the case where the barrier and ground surfaces converged, sufficient borings must be made to determine where the hydraulic conductivity changes. The interceptor drain is then located where it will be about 2.4 meters (8 feet) deep just upslope of the decrease in hydraulic conductivity. If the change is abrupt, the interceptor drain should be located in the more permeable material just before the change.

5–15. Location of First Drain Below an Unlined Canal or Lateral.—Data required to determine the location of the first drain below an unlined canal or lateral are:

(a) Channel sections and grades.

(b) Hydraulic conductivity of the material adjacent to the channel.

(c) Weighted hydraulic conductivity between permissible root zone depth and barrier.

(d) Depth to barrier.

(e) Slope of barrier and ground surface in the vicinity of the channel.

(f) Distance from the centerline of channel to the irrigated land, see figure 5–9.

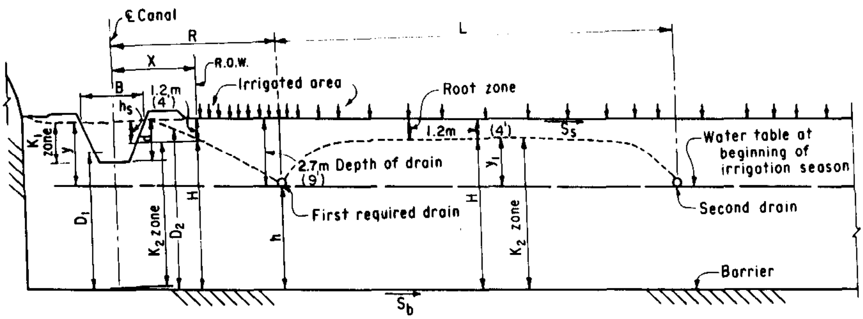


Figure 5–9.—Measurements needed for estimating location of first drain below an unlined canal or lateral. Drawing 103–D–1656.

The following steps show a method of determining the distance from the canal centerline to first drain:

Step 1. Estimate the channel seepage under free drainage conditions. The following formulas may be used for estimating in the absence of a better method.

$$q_1 = \frac{K_1\,(B+2d)}{3.5} \tag{8}$$

where:

q_1 = seepage in cubic meters (feet) per linear meter (foot) of channel per day, when water table is below channel bottom (free drainage condition);
K_1 = hydraulic conductivity adjacent to the channel section, meters (feet) per day;
d = depth of water in channel at normal operating level, meters (feet);
B = width of water in channel at normal operating level, meters (feet); and
3.5 = factor used to adjust hydraulic conductivity test values to seepage losses from ponding tests.

Example: For a canal section with a base width of 3 meters (10 feet) and 2:1 side slopes, find q_1 if K_1 = 0.46 meter (1.5 feet) per day and d = 0.76 meter (2.5 feet).

$$q_1 = \frac{0.46\,[6.1+(2\times 0.76)]}{3.5} = 1.0\ \text{m}^3/\text{m/d}\ (10.71\ \text{ft}^3/\text{ft/d})$$

For existing canals and laterals, q_1 can be measured, but care must be taken to ensure that free drainage exists below the canal or lateral. When a water table has developed under the canal or lateral, the depth to the water table must be measured at the same time as the seepage. Unless a thick, permeable aquifer underlies the canal, a ground-water mound will rise under the channel and eventually reach the same level as the water surface in the channel. The time required for this to occur can be estimated from the formula:

$$t = \frac{\pi K_2 y^2 D_1 S}{q_1^2} \tag{9}$$

where:

t = time in days for water table mound to rise from water table depth at beginning of irrigation season to water surface in canal;
K_2 = weighted hydraulic conductivity between root zone depth and barrier, meters (feet) per day;
y = distance from water table depth at beginning of irrigation season to normal water surface in the channel, meters (feet);
D_1 = distance in meters (feet) between water table depth (at beginning of irrigation season) and the barrier plus one-half y;
q_1 = seepage under free draining conditions, m³/m/d (ft³/ft/d); and
S = specific yield determined from hydraulic conductivity in the K_1 zone, percent by volume.

For example, if the distance between water table depth (at beginning of irrigation season) and the barrier is 6.1 meters (20 feet), $K_2 = 0.46$ m/d (1.5 ft/d), $v = 2.74$ meters (9 feet), $S = 12$ percent, and $q_1 = 1.0$ m³/m/d (10.71 ft³/ft/d) as previously calculated. Find the time, t, as defined above.

$$D_1 = 6.1 + \frac{2.7}{2} = 7.45 \text{ meters (24.5 feet), and}$$

$$t = \frac{(3.1416)(0.46)(2.74)^2(7.45)(0.12)}{(1.0)^2} = 10 \text{ days}$$

The use of q_1 in formula (9) does not account for the fact that the seepage rate begins to decrease when the water table mound reaches the bottom of the channel and will continue to decrease until the mound rises to the water surface elevation in the channel. At this point, the seepage rate becomes essentially constant and is called the terminal seepage rate, q_2. The seepage rate, q_2 can be determined by the formula:

$$q_2 = q_1 \left(\frac{B - 2d}{B + 2d} \right) \tag{10}$$

$$q_2 = 1.0 \left(\frac{6.10 - 1.52}{6.10 + 1.52} \right) = 0.601 \text{ m}^3\text{/m/d}$$

Often, an aerated root zone must be maintained at the edge of an irrigated area adjacent to an unlined channel. This situation may require a drain. The seepage from the channel and the additional capacity needed in the first drain because of the seepage can be determined by the formula:

$$q_3 = \frac{K_2 D_2 h_s}{X}$$

where:

q_3 = seepage in cubic meters (feet) per linear meter (foot) of channel per day when the selected root zone depth at the edge of the irrigated area is maintained by a drain;

K_2 = weighted hydraulic conductivity between root zone depth and barrier, meters (feet) per day;

D_2 = one-half the sum of the distances between: (1) barrier and water surface in channel, and (2) barrier and selected root zone depth at the edge of the irrigated area;

h_s = difference in elevation between selected root zone depth at the edge of the irrigated field and water surface in channel; and

X = distance from centerline of channel to the edge of the irrigated area.

Example: If h_S = 1.22 meters (4 feet) and X = 18.3 meters (60 feet), then

$$D_2 = \frac{(6.1 + 2.74) + (6.1 + 2.74 - 1.22)}{2} = 8.23 \text{ meters (27 feet), and}$$

$$q_3 = \frac{0.46 \times 8.23 \times 1.22}{18.3} = 0.252 \text{ m}^3\text{/m/d } (2.70 \text{ ft}^3\text{/ft/d})$$

Step 2: If the canal is on a sidehill where the ground-water movement is in one direction and where q_3 is less than q_2, use q_3 as the seepage factor in estimating the distance from the canal centerline to first drain. If movement is in two directions or from a canal on a ridge with irrigation on both sides, when q_3 is less than $\frac{q_2}{2}$, use q_3.

The example in this section has the canal on a sidehill with all ground-water flow in one direction and q_3 less than q_2; therefore, use the q_3 seepage of 0.252 cubic meters per linear meter (2.70 cubic feet per linear foot) of channel per day.

Step 3: Estimate the distance from the canal centerline to first required drain by the formula:

$$R = \frac{K_2(H^2 - h^2)}{2q_3} + X \tag{12}$$

where:

R = distance in meters (feet) from channel centerline to first required drain;
h = distance in meters (feet) between drain and barrier; and
H = distance in meters (feet) between barrier and maintained root zone depth at edge of irrigated area.
K_2, q_3, and X are as previously defined.

Example: If $h = 6.1$ meters (20 feet) and $H = 6.1 + (2.74 - 1.22) = 7.62$ meters (25 feet), then

$$R = \frac{0.46\,[(7.62)^2 - (6.1)^2]}{2 \times 0.252} + 18.3 = 37.4 \text{ meters (123 feet)}$$

Some irrigation recharge between the drain and the edge of the irrigated area above the drain has not been considered in the calculations. This recharge area is accounted for by using the 37.4 meters (123 feet) as the first estimate of the distance from channel centerline to first required drain. Irrigation recharge between the drain and the channel can be estimated and added to the canal seepage as follows:

(a) Deep percolation from irrigation during the peak period, 14 days between irrigations = 9.40 millimeters (0.37 inch).

(b) Average daily rate of recharge during irrigation season would then be $i = \frac{9.40}{14} = 0.67$ millimeter (0.00067 meter or 0.0022 foot) per day.

(c) Irrigation recharge to be drained between the drain and edge of irrigated area = $i(R - X)$ = (0.00067 (37.4 – 18.3) = 0.0128 cubic meter per linear meter (0.14 cubic foot per linear foot) of drain per day.

(d) Irrigation recharge plus canal seepage q_3, = 0.0128 + 0.252 = 0.265 $m^3/m/d$ (2.84 $ft^3/ft/d$).

The second estimate of the distance from channel centerline to the first drain using irrigation recharge plus canal seepage would be:

$$R = \frac{0.46\,[(7.62)^2 - (6.1)^2]}{2 \times 0.265} + 18.3 = 36.4 \text{ meters (120 feet)}$$

Irrigation recharge will now be $i(R - X) = (0.00067)(36.6 - 18.3) = 0.0123$ $m^3/m/d$ (0.13 $ft^3/ft/d$) and, if added to the canal seepage, q_3 would not change the second estimate of R.

Any additional parallel drains required to keep the water table below the acceptable level can be computed by the drain spacing methods described in part A of this chapter. These methods were developed for level lands but give an acceptable spacing for slopes up to about 10 percent.

5–16. Location of First Drain on Irrigated Sloping Land.—When an irrigated area lies on a slope, deep percolation from irrigation may cause shallow water tables and the need for spaced drains. When seepage from canals or laterals is negligible, a strip along the upper edge of the irrigated area may not require spaced drains because of the downhill movement of the water. However, some distance down the slope the water table will become too shallow for crop production and farming operations. This section describes a method, based on steady-state conditions, to determine the location of the first drain downslope.

When infiltration is steady, the water table will approach steady-state configurations as shown in the profiles on figures 5–10 and 5–11. The water table can be determined from these figures for combinations of surface and barrier slope. A sample solution follows:

(a) Assume seepage loss from lined canal is negligible.

(b) K = average hydraulic conductivity of soil profile under irrigated land = 5.08 centimeters (2 inches) per hour or 1.22 meters (4 feet) per day.

(c) t = irrigation season = 135 days.

(d) L = length of irrigated slope = 457 meters (1,500 feet).

(e) DP = deep percolation from irrigation and rainfall for 135 days = 0.091 meter (0.30 foot).

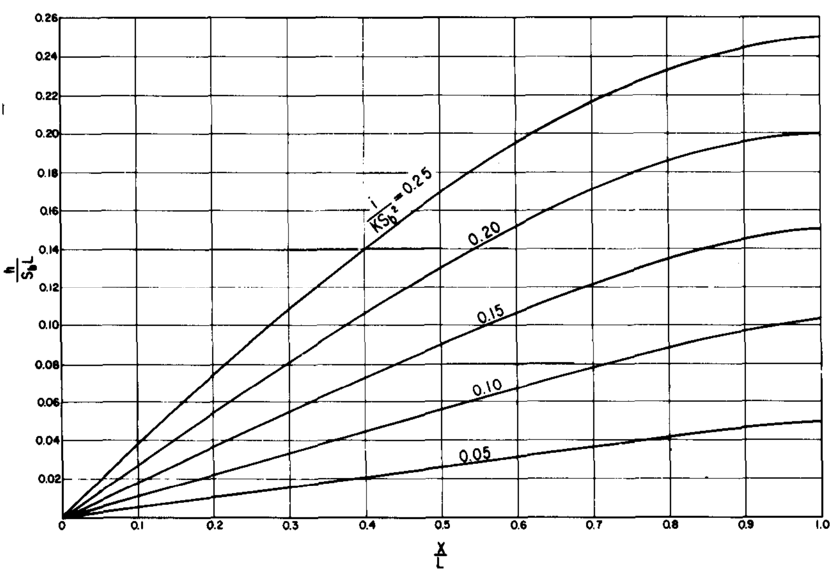


Figure 5–10.—Water table profiles on sloping barriers for $0.05 \leq \frac{i}{KS_b^2} \leq 0.25$. Drawing 103-D-1657.

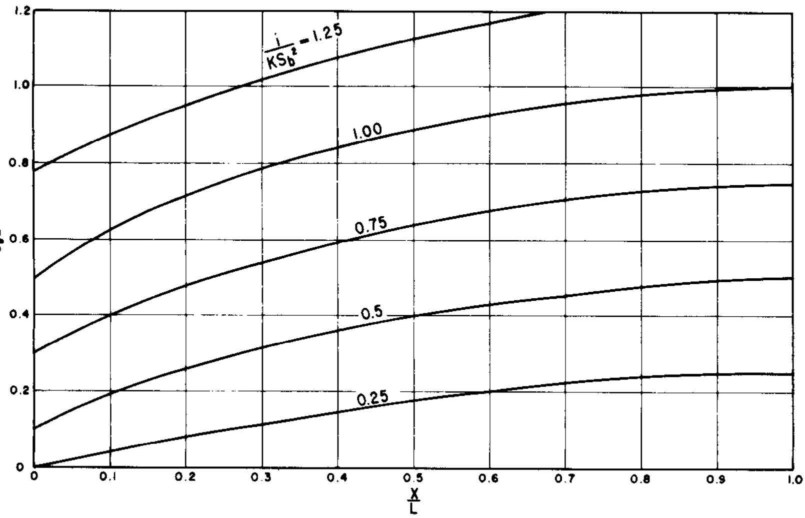


Figure 5–11.—Water table profiles on sloping barriers for $0.25 \leq \frac{i}{KS_b^2} \leq 1.25$. Drawing 103-D-1658.

(*f*) Average daily rate of recharge during irrigation season,

$$i = \frac{DP}{t} = \frac{0.091}{135} = 0.00068 \text{ m}^3/\text{m}^2/\text{d } (0.00222 \text{ ft}^3/\text{ft}^2/\text{d}).$$

(*g*) $S_S = 0.03$ m/m (ft/ft), slope of land surface.

(*h*) $S_b = 0.027$ m/m (ft/ft), slope of barrier layer.

(*i*) Db_1 = depth to barrier at upper end of irrigated area = 7.32 meters (24 feet).

(*j*) Db_2 = depth to barrier at lower end of irrigated area = 5.94 meters (19.5 feet).

(*k*) $\frac{i}{KS_b^2} = \frac{0.00068}{1.22(0.027)^2} = 0.76$

(*l*) h = height above barrier.

(*m*) Interpolate between the curves on figure 5–11 to plot the $\frac{i}{KS_b^2} = 0.76$ curve shown on figure 5–12.

(*n*) Plot the ground surface using the barrier as the abscissa (fig. 5–12).

$$\text{When } \frac{X}{L} = 0, \frac{h}{S_b}L = \frac{7.32}{(0.027)(457)} = 0.593, \text{ and}$$

$$\frac{X}{L} = 1, \frac{h}{S_b L} = \frac{5.94}{(0.027)(457)} = 0.481$$

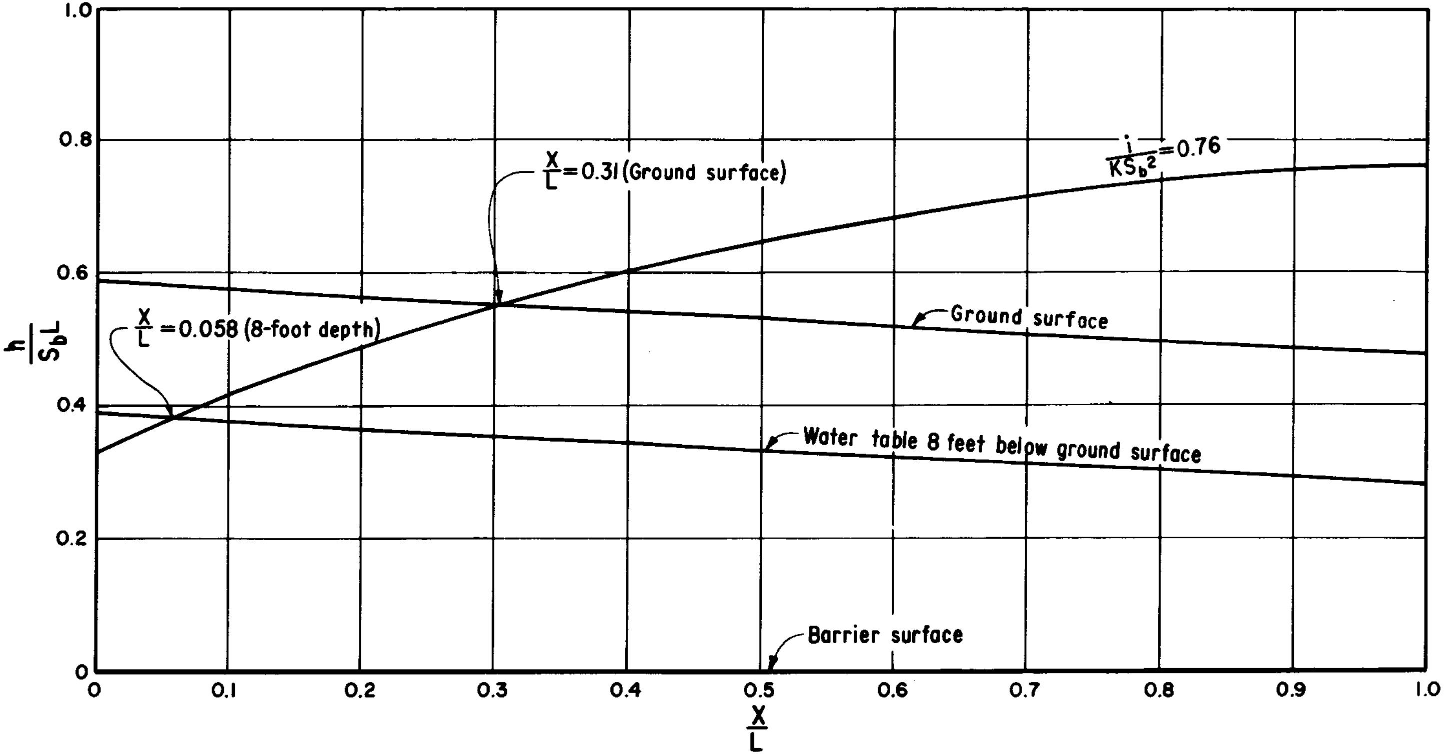


Figure 5–12.—Water table profile on a sloping barrier for $\frac{i}{KS_b^2} = 0.76$. Drawing 103–D–1659.

Plot these two points and draw a line between them to represent the ground surface. Where this line intersects the curve, read $\frac{X}{L} = 0.31$. The distance from the edge of the irrigated area to the point where the ground water appears at the ground surface is:

$$X = (457)(0.31) = 142 \text{ meters (465 feet)}$$

(o) Find the point where the water table will be 2.44 meters (8 feet) below the ground surface as follows:

When $\frac{X}{L} = 0, h = 7.32 - 2.44 = 4.88$ meters (16 feet)

therefore, $\frac{h}{S_bL} = \frac{4.88}{(0.027)(457)} = 0.395$

When $\frac{X}{L} = 1, h = 5.94 - 2.44 = 3.50$ meters (11.5 feet)

and $\frac{h}{S_bL} = \frac{3.50}{(0.027)(457)} = 0.284$

Plot these points on figure 5–12 as shown and draw a line between the points. Where the line intersects the curve, read $\frac{X}{L} = 0.058$ on the abscissa. The distance from the edge of the irrigated area to the point where the water table is 2.44 meters (8 feet) below the ground surface is:

$$X = (457)(0.058) = 26.5 \text{ meters (87 feet)}$$

(p) The shape of the water table without drains can be determined as follows: Make a table using the coordinates of the curve on figure 5–12 using $L = 457$, and $S_bL = 0.027 \times 457 = 12.34$ meters (40.5 feet).

Coordinates		$\frac{h}{S_bL}(12.34)$	$\frac{h}{S_bL}(40.5)$	$\frac{X}{L}(457)$	$\frac{X}{L}(1{,}500)$
		h		X	
$\frac{X}{L}$	$\frac{h}{S_bL}$	meters	feet	meters	feet
0.00	0.335	4.13	13.6	0	0
.05	.380	4.69	15.4	22.9	75
.06	.390	4.81	15.8	27.4	90
.10	.425	5.25	17.2	45.7	150
.15	.460	5.68	18.6	68.6	225
.20	.496	6.12	20.1	91.4	300
.25	.528	6.52	21.4	114.3	375
.30	.555	6.85	22.4	137.1	450
.31	.560	6.91	22.7	141.7	465

Plot h and X as shown on figure 5–13, where h is the vertical height of the water table above the barrier and X is the distance from the edge of the irrigated field.

(q) The drain spacing for the remainder of the area can be determined using methods described in part A of this chapter. The spacing calculations do not take into account sloping barriers; however, the results are reasonably reliable for slopes up to 10 percent.

The first 26.5 meters (87 feet) from the edge of the irrigated area will be drained by the downhill movement of water. This distance must be accounted for in the solution for drain spacing. The basic drain spacing, L, is about 305 meters (1,000 feet). Then, $L + 26.5 = 331.5$ meters (1,087 feet). To find depths between drains, slopes S_S, and S_b must be used.

For example:

$331.5\ S_S = 331.5 \times 0.030 = 9.95$ meters (32.61 feet)

$331.5\ S_b = 331.5 \times 0.027 = 8.95$ meters (29.36 feet)

The depth to the barrier at 331.5 meters (1,087 feet) will then be:

$7.32 - (9.95 - 8.95) = 6.32$ meters (20.74 feet)

The average depth to the barrier is:

$$\frac{7.32 + 6.32}{2} = 6.82 \text{ meters (22.37 feet)}$$

Using Donnan's steady-state equation, the distance between drain depth and barrier, a, for a drain depth of 2.44 meters (8.0 feet) is:

$$a = 6.82 - 2.44 = 4.38 \text{ meters (14.37 feet)}$$

Correcting a for convergence using Hooghoudt's methods:

$a' = 4.0$ meters (13 feet), and
$b = 4.0 + 1.22 = 5.22$ meters (17 feet).

Therefore:

$$L^2 = \frac{(4)(1.22)(5.22^2 - 4.0^2)}{0.00068} = 80{,}724, \text{ and}$$
$$L = 284 \text{ meters (932 feet).}$$

Transient flow methods should be used to check results of the steady-state analysis.

(r) The first drain is located $284 + 27 = 311$ meters (1,020 feet) from the upper edge of the irrigated field. The spacing is based on drawdown from two drains, but at 30 meters (97 feet) from the upper edge of the irrigated field, natural flow down the slope keeps the water table at 2.4 meters (8 feet). Therefore, no water

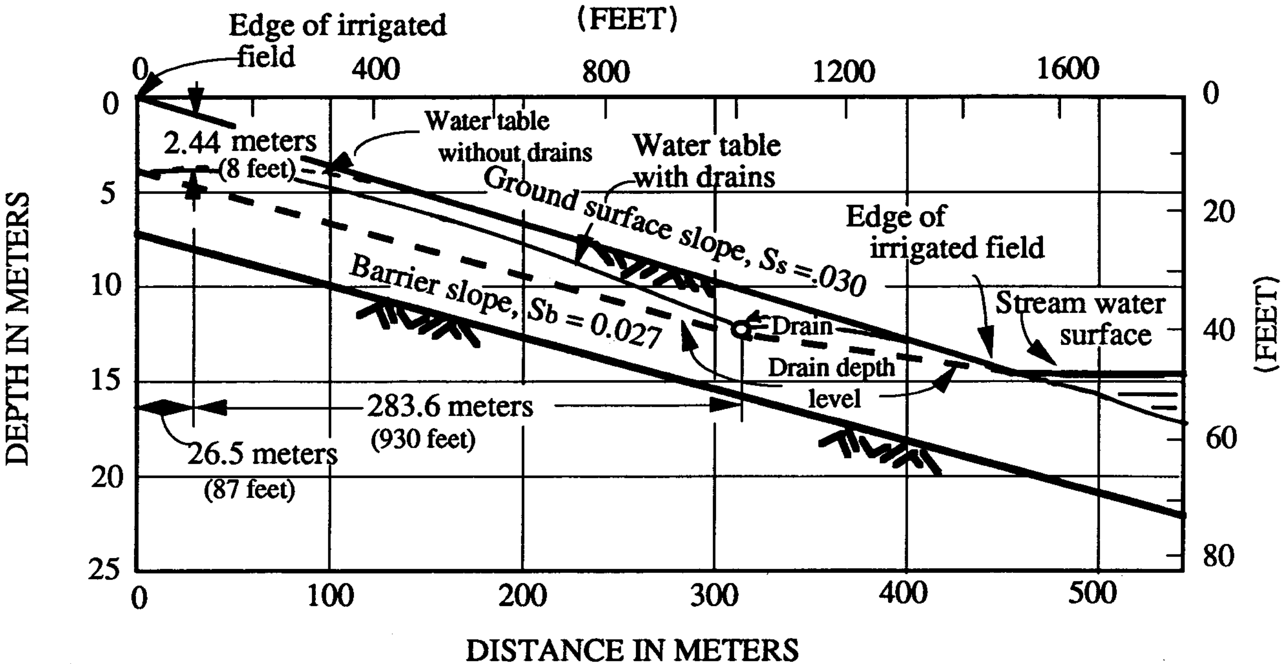


Figure 5–13.—Water table profile under steady-state conditions with and without drains. Drawing 103–D–1660.

would enter a drain at this point, and the effect is the same as having a drain at this point. The downslope drain will maintain a minimum 1.2-meter (4-foot) water table depth along the slope above the drain. The size of the first drain should be designed to handle all deep percolating water between the upper edge of the field plus normal flow from the downslope side—or about 1.5 times as much as a normal spaced drain.

For fields where one drain is not quite adequate but two drains would overdrain the area, the planners and designers must decide on what is best for the farmer and project—to install one or two drains. Generally, the decision is based on economics, but project or district policies may influence the decision. An economic study of the area would probably show that the use of only one drain, which would place the lower end of the field in nonirrigable status, would be more economical.

To determine the distance downslope from the last drain where the water table would be 1.2 meters (4 feet) from the land surface, the following procedure can be used:

(1) Measure the distance from the last drain to a natural drain. In the example, this is 488 – 311 = 177 meters (580 feet). Draw a line between the centerline of the drain and the water surface in the natural channel.

(2) At $\frac{L}{2}$ or 89 meters (292 feet) downslope from the drain, determine the depth from the barrier to the line connecting the drain to the water surface in the natural channel.

(a) Ground surface is (311 + 89) 0.03 = 12.0 meters (39.37 feet) below the top of the field.

(b) Elevation of the barrier is 0.027 (400) + 7.3 = 18.10 meters (59.38 feet) below the top of the field.

(c) Elevation of the last drain is 0.03 (311) + 2.4 = 11.73 meters (38.51 feet) below the top of the field.

(d) Elevation of the water surface in the natural drain is 14.6 meters (48 feet) below the top of the field (fig. 5–13).

(e) Elevation of the drain depth between the last drain and the natural drain is $\frac{L}{2} = \frac{14.8 + 11.73}{2} = 13.17$ meters (43.25 feet).

(f) Distance from drain depth to barrier at $\frac{L}{2}$ is:

$$a = 18.1 - 13.2 = 4.9 \text{ meters (16.08 feet).}$$

(3) Compute the height of the water table midway between drains:

L = 178 meters (583 feet)
a = 4.9 meters (16.09 feet)
a' = 4.3 meters (14.0 feet)
K = 1.22 meters (4 feet) per day
Q_d = 0.00068 $m^3/m^2/d$ (0.00222 $ft^3/ft^2/d$)

Using Donnan's equation:

$$L^2 = \frac{4K(b^2-a'^2)}{Q_d}$$

$$\text{or } b^2-a'^2 = \frac{L^2 Q_d}{4K}$$

$$b^2-a'^2 = \frac{(178)^2(0.00068)}{(4)(1.22)} = 4.42$$

$$a'^2 = 4.3^2 = 18.5$$

$$\text{then, } b^2 = 4.42 + 18.5 = 22.92$$

$$\text{and, } b = 4.79 \text{ meters (15.7 feet)}$$

$$\text{therefore, } y_o = b - a = 4.79 - 4.3 = 0.49 \text{ meter}$$

(1.61 feet), the height of the water table above the drain.

At a point 89 meters (292 feet) downslope from the last drain, the water table will be 13.2 – 0.49 – 12 = 0.71 meters (2.33 feet) below the ground surface, which is not adequate. By plotting a fourth degree parabola of the drawdown curve between drains when y_o = 0.49 meter (1.6 feet), the point where the water table will be 1.2 meters (4 feet) below the ground surface can be estimated as follows:

$\frac{X}{L}$	X*		y		Distance from ground surface to y	
	meters	feet	meters	feet	meters	feet
0	0	(0)			2.44	(8.0)
0.05	8.8	(29)	$0.3439y_o$ = 0.169	(0.553)	2.16	(7.1)
0.1	17.7	(58)	$0.5904y_o$ = 0.289	(0.949)	1.92	(6.3)
0.2	35.7	(117)	$0.8704y_o$ = 0.426	(1.399)	1.52	(5.0)
0.3	53.3	(175)	$0.9744y_o$ = 0.477	(1.566)	1.22	(4.0)
0.4	71.3	(234)	$0.9984y_o$ = 0.489	(1.605)	0.98	(3.2)
0.5	89.0	(292)	y_o = 0.490	(1.608)	0.73	(2.4)

* The pipe drain represents X = 0.

To obtain the distance from ground surface to y in the previous tabulation, the following calculations were necessary:

Elevation of ground surface (fig. 5–13) from the last drain downslope to the natural drain is:

$y_s = 9.30 + 0.03X$ meters $= 30.51 + 0.03X$ feet

where X is measured in meters and feet, respectively.

Elevation of drain level between drains is:

$$y_d = 11.73 + \frac{(14.6 - 11.73)X}{178} = \text{drain level elevation in meters.}$$

$$y_d = 38.51 + \frac{(48 - 38.51)X}{583} = \text{drain level elevation in feet.}$$

Depth to drain: $D_d = y_d - y_s = 2.44 - 0.01372X$ (in meters) or $= 8 - 0.01372X$ (in feet)

Depth to water: $D_w = D_d - y, D_w = y_d - y_s - y$, or $D_w =$ $2.44 - 0.01372X - y$ (in meters) below ground surface or $8 - 0.01372X - y$ (in feet) below ground surface.

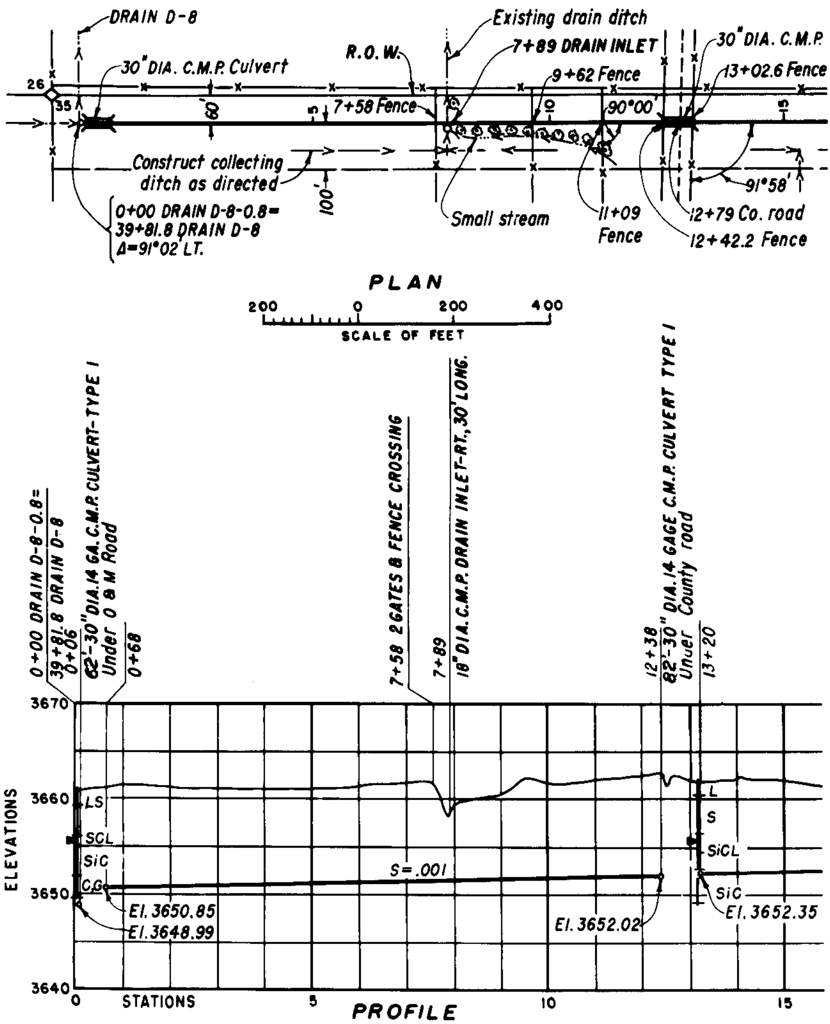
From the previous tabulation, the water table will be 1.2 meters (4 feet) below the ground surface at about 53 meters (175 feet) downslope from the drain. The area that would be inadequately drained for deep-rooted crops, if only one drain is installed, would be at about 125 meters (411 feet) on the lower edge of the field.

C. Open Drains

5–17. Introduction.—Open drains are ditches with an exposed water surface and are widely used for surface and subsurface drainage. Shallow surface drains are normally used for the removal of irrigation surface waste and storm water. This type of drain provides very little subsurface drainage and is considered simply a wastewater ditch or storm channel. Deep subsurface drainage ditches are used to provide subsurface drainage and as collectors for surface and subsurface drainage systems.

Many hydraulics textbooks thoroughly present the theory and details of open channel design; therefore, only those criteria that pertain to design of drains are presented here. Figure 5–14 shows a typical plan and profile of an open drain.

5–18. Open Channel Flow.—The area, A, of a drain section for any flow, Q, is found from the equation $A = \frac{Q}{v}$. The velocity, v, based on Manning's formula, can be found in the Bureau of Reclamation's Hydraulic and Excavation Tables (Bureau of Reclamation, 1957). These tables give velocities in feet per second for various coefficients of roughness, n, for trapezoidal channels. An $n = 0.030$ should be used for open ditches. When these tables are not available, the Manning formula can be used to determine the velocity.



HYDRAULIC PROPERTIES

$n = .030$

REACH	b	s	NORMAL Q	STORM Q	d	A	r	V
0+68 to 13+20	4.0	.001	5.5		0.9	4.82	0.66	1.14
				33	2.3	17.15	1.40	1.93

Normal Q in the hydraulic properties table is the estimated subsurface accretions plus the return flow from irrigation.
Storm Q in the hydraulic properties table is the normal Q plus the estimated surface run-off from a storm of approx. 5 year frequency.

Figure 5–14.—Typical plan and profile of an open drain. From drawing 103–D–663.

$$\text{Manning formula, } v = \frac{1.486}{n} r^{2/3} s^{1/2} \tag{13}$$

where:

v = velocity in feet per second,
r = hydraulic radius in feet,
s = slope of the drain in feet per foot, and
n = coefficient of roughness.

For velocities in meters per second, the Manning formula is

$$v = \frac{r^{2/3} s^{1/2}}{n} \text{ (metric form)} \tag{13a}$$

where:

v = velocity in meters per second,
r = hydraulic radius in meters,
s = slope of the drain in meters per meter, and
n = coefficient of roughness.

As an approximation, the velocities in feet per second as given in the Bureau of Reclamation's Hydraulic and Excavation Tables (1957) multiplied by 0.3 will give velocities in meters per second.

Values of A and r for small, V-shaped drainage ditches are shown in table 5–6.

5–19. Drain Velocities.—Maximum permissible velocities for open drains according to soil texture are as follows:

Soil texture	*Velocity, meters (feet) per second*	
Clay	1.2	(4.0)
Sandy loam	0.8	(2.5)
Fine sands	0.5	(1.5)

In some soils, a tractive force analysis may be necessary to determine the stability of the drainage channel. The objective is to construct a relatively stable channel which will neither erode nor be subject to deposition of objectionable amounts of sediment. The maximum permissible gradient under given topographic and soil conditions should always be used, provided the velocity is kept below that which would cause significant erosion from a 5-year storm. Where surface slopes are steep, structures must be provided to control velocities.

Table 5–6a.—*Cross-sectional area and hydraulic radius for small V-shaped ditches (metric units).* Drawing 103–D–682.

$1\frac{1}{2}$:1 SIDE SLOPES

DEPTH (meters)	0.0 / 0.5		0.1 / 0.6		0.2 / 0.7		0.3 / 0.8		0.4 / 0.9	
	A	r	A	r	A	r	A	r	A	r
0	0	0	0.015	0.042	0.060	0.083	0.135	0.125	0.240	0.166
	0.375	0.208	0.540	0.250	0.735	0.291	0.960	0.333	1.215	0.374
1	1.500	0.416	0.185	0.458	2.160	0.499	2.535	0.541	2.940	0.582
	3.375	0.624	3.840	0.666	4.335	0.707	4.860	0.749	5.415	0.790

2:1 SIDE SLOPES

DEPTH (meters)	0.0 / 0.5		0.1 / 0.6		0.2 / 0.7		0.3 / 0.8		0.4 / 0.9	
	A	r	A	r	A	r	A	r	A	r
0	0	0	0.020	0.045	0.080	0.089	0.180	0.134	0.320	0.179
	0.500	0.224	0.720	0.268	0.980	0.313	1.280	0.358	1.620	0.402
1	2.000	0.447	2.420	0.492	2.880	0.537	3.380	0.581	3.920	0.626
	4.500	0.671	5.120	0.716	5.780	0.760	6.480	0.805	7.220	0.850

$2\frac{1}{2}$:1 SIDE SLOPES

DEPTH (meters)	0.0 / 0.5		0.1 / 0.6		0.2 / 0.7		0.3 / 0.8		0.4 / 0.9	
	A	r	A	r	A	r	A	r	A	r
0	0	0	0.025	0.046	0.100	0.093	0.225	0.139	0.400	0.186
	0.625	0.232	0.900	0.279	1.225	0.325	1.600	0.371	2.025	0.418
1	2.500	0.464	3.025	0.511	3.600	0.557	4.225	0.604	4.900	0.650
	5.625	0.696	6.400	0.743	7.225	0.789	8.100	0.836	9.025	0.882

3:1 SIDE SLOPES

DEPTH (meters)	0.0 / 0.5		0.1 / 0.6		0.2 / 0.7		0.3 / 0.8		0.4 / 0.9	
	A	r	A	r	A	r	A	r	A	r
0	0	0	0.030	0.047	0.120	0.095	0.270	0.142	0.480	0.190
	0.750	0.237	1.080	0.285	1.470	0.332	1.920	0.379	2.430	0.427
1	3.000	0.474	3.630	0.522	4.320	0.569	5.070	0.617	5.880	0.664
	6.750	0.712	7.680	0.759	8.670	0.806	9.720	0.854	10.83	0.901

Table 5–6b.—*Cross-sectional area and hydraulic radius for small V-shaped ditches (U.S. customary units).* Drawing 103–D–682.

1½ : 1 SIDE SLOPES

DEPTH	0.0		0.2		0.4		0.6		0.8	
(feet)	*A*	*r*	*A*	*r*	*A*	*r*	*A*	*r*	*A*	*r*
0							0.54	0.75	0.96	0.33
1	1.50	0.42	2.16	0.50	2.94	0.58	3.84	0.67	4.86	0.75
2	6.00	0.83	7.26	0.92	8.64	1.00	10.14	1.08	11.76	1.16
3	13.50	1.25	15.36	1.33	17.34	1.41	19.44	1.50	21.66	1.58
4	24.00	1.66	26.46	1.75	29.04	1.83	31.74	1.91	34.56	2.00

2 : 1 SIDE SLOPES

DEPTH	0.0		0.2		0.4		0.6		0.8	
(feet)	*A*	*r*	*A*	*r*	*A*	*r*	*A*	*r*	*A*	*r*
0							0.72	0.27	1.28	0.36
1	2.00	0.45	2.88	0.54	3.92	0.63	5.12	0.72	6.48	0.80
2	8.00	0.89	9.68	0.98	11.52	1.07	13.52	1.16	15.68	1.25
3	18.00	1.34	20.48	1.43	23.12	1.52	25.92	1.61	28.88	1.70
4	32.00	1.79	35.28	1.88	38.72	1.97	42.32	2.06	46.08	2.14

2½ : 1 SIDE SLOPES

DEPTH	0.0		0.2		0.4		0.6		0.8	
(feet)	*A*	*r*	*A*	*r*	*A*	*r*	*A*	*r*	*A*	*r*
0							0.90	0.28	1.60	0.37
1	2.50	0.46	3.60	0.56	4.90	0.65	6.40	0.74	8.10	0.84
2	10.00	0.93	12.10	1.02	14.40	1.11	16.90	1.21	19.60	1.30
3	22.50	1.39	25.60	1.49	28.90	1.58	32.40	1.67	36.10	1.76
4	40.00	1.86	44.10	1.95	48.40	2.04	52.90	2.13	57.60	2.23

3 : 1 SIDE SLOPES

DEPTH	0.0		0.2		0.4		0.6		0.8	
(feet)	*A*	*r*	*A*	*r*	*A*	*r*	*A*	*r*	*A*	*r*
0							1.08	0.28	1.92	0.38
1	3.00	0.47	4.32	0.57	5.88	0.66	7.68	0.76	9.72	0.85
2	12.00	0.95	14.52	1.04	17.28	1.14	20.28	1.23	23.52	1.33
3	27.00	1.42	30.72	1.52	34.68	1.61	38.88	1.71	43.32	1.80
4	48.00	1.89	52.92	1.99	58.08	2.09	63.48	2.18	69.12	2.27

The ideal minimum gradient in a drain would have sufficient velocity at low flows to prevent deposition and growth of aquatic plants. This velocity would be in the range of 0.2 to 0.3 meter (0.75 to 1.0 foot) per second for prevention of silt and fine sand deposits, 0.5 to 0.6 meter (1.5 to 2.0 feet) per second for the prevention of weeds and grasses, and 0.8 meter (2.5 feet) per second or more to inhibit growth of aquatic plants. In areas where ideal velocities cannot be obtained, drains should be designed with a minimum velocity of about 0.3 meter (1.0 foot) per second for the normal flow. In some collector drains, pumping plants might be required where the gradient must be built into the drain. Pumping plants in drains have the disadvantages of constant maintenance, expense of operation, and icing during the wintertime. They should be used only when the velocities at normal flow are well below the minimum 0.3 meter (1.0 foot) per second. Gradients for natural outlet drains usually are not altered except where the channel straightening gradients allows increase.

Minimum grades require maximum maintenance; therefore, when gradients are used that result in velocities of 0.3 meter (1.0 foot) per second or less for normal flows, provisions should be made for shorter periods of time between drain cleaning.

5–20. Depth of Drain.—The depth of an open drain for carrying surface water is controlled by the quantity of water it carries. The depth of a deep, open subsurface drain is controlled by physical and hydraulic properties of the soils, permissible water table levels, construction equipment, and quantity of water it must carry. The most difficult design case is that of a drain which receives runoff water from tributary drains, while picking up ground water throughout its reach. The drain must be deep enough so that the normal water surface will be below the water table to allow the drain to pick up ground water. Also, the drain must be large enough to accept tributary drain discharge. The normal water surface elevation in the collector drain must not be higher than that in the tributary drain. Designing the capacity for carrying floodflows is usually no problem in a completely open drain system. When the first two requirements are satisfied, capacity is adequate to handle most floodflows. A floodflow may raise the water level temporarily in the drain to a point higher than the ground-water elevation. This water level inhibits the drain from picking up ground water, but crops would not be burned if the condition did not last for more than 48 hours. Where flash floods occur frequently and the soils are highly erosive, separate deep drainage and floodwater systems may be more economical.

If the tributary drains are closed drains, the normal water surface elevation in the open collector drain should be below the invert elevation of the closed drain by a distance sufficient to allow for some floodwater flow down the open drain without affecting the closed drain. This practice will prevent water from backing up in the closed drain. The additional distance should be 450 millimeters (18 inches), if practical, but can be as low as 150 millimeters (6 inches) if banks are stable or if the open drain depth would otherwise be unreasonable. An occasional, temporary rise in water level over the closed drain caused by floods is not detrimental.

In general, subsurface drains should be from 2.4 to 3 meters (8 to 10 feet) deep to provide the best economic balance between drain cost and drain spacing. On occasion, local conditions may require deeper or shallower drains. The most important condition would be location of the permeable and impermeable strata.

5–21. Drain Section.—The most hydraulically efficient open channel has maximum capacity for a given slope and cross-sectional area. The most efficient cross section has the smallest wetted perimeter. Based on these facts, a semicircular section would be the most efficient. However, for channels excavated in earth, the semicircular shape is impractical for various reasons, including construction difficulty. Trapezoidal cross sections are most often used and have been found to be the most economical section for earth channels.

In trapezoidal-shaped drains, stability of the side slopes depends on soil characteristics. The side slopes should be less than the angle of repose of the saturated material, at least as far up the slope as the maximum water table elevation. Side slopes may vary from a 3:1 slope or greater in a sandy soil to almost vertical side slopes in a highly organic soil.

In general, a berm between the edge of the cut and the roadway or spoil bank should not be provided because of the maintenance problems created. Berms, however, may be required where soils are unstable and the load of the fill would be detrimental. The minimum bottom width of drains is influenced by the types of excavating and maintenance equipment available for use. If a dragline is to be used, the minimum width should be about 0.9 meter (3 feet). Figure 5–15 shows typical drain sections and the relationship between roadways, spoil banks, and berms for drains of different sizes.

5–22. Drain Banks.—Drain banks should be constructed by depositing the excavated material in approximately horizontal layers to a thickness equal to the depth of the material as it is deposited by the excavating equipment. Excavated material should be placed over the full width of the bank to the prescribed slopes and not widened with loose material from the top.

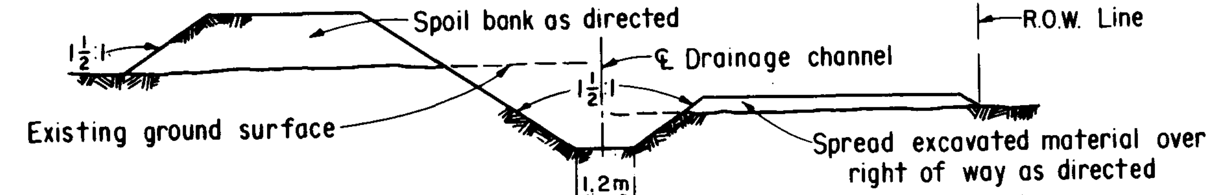
The crown of the banks should be graded to a reasonably uniform surface. The crown on at least one side should serve as a roadway. When excavated material is unstable and cannot be deposited within the prescribed slopes and widths, the material should be allowed to drain and dry before the banks are graded. Before the drain is accepted as completed, all banks should present a neat appearance.

5–23. Tributary Drain Intersections.—Open tributary drains should enter the collector drain with their water surfaces at the same elevation. If the tributary drain carries more than about 0.4 m^3/s (15 ft^3/s), the bottom grade must be curved downstream to make the flow lines of the drains more nearly parallel at the point of juncture. This curve is not required for tributaries with flows less than this, but it would improve the flow characteristics and reduce maintenance costs if applied.

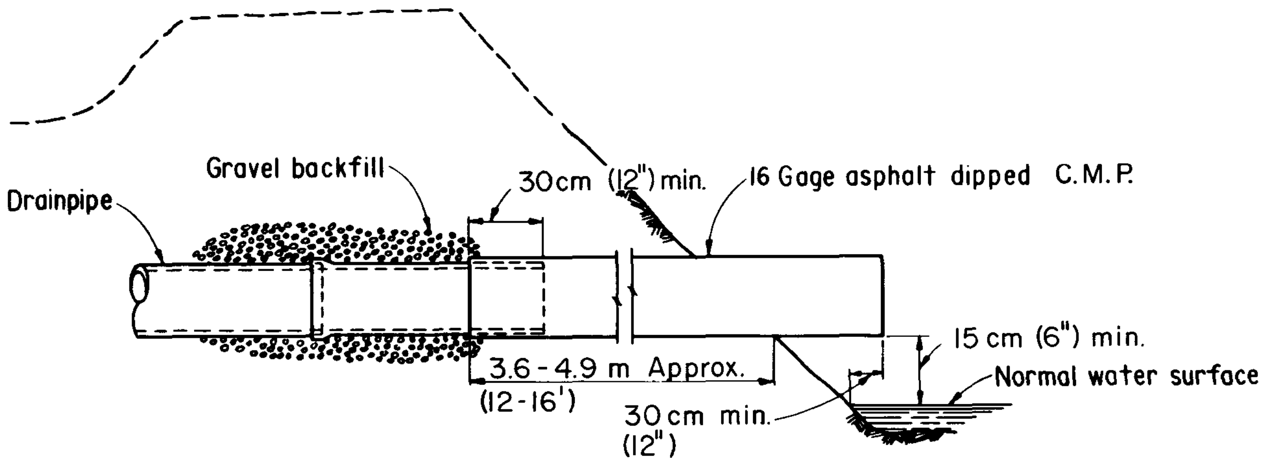
5–24. Surface Inlets.—Surface water should never be permitted to enter a deep drain by flowing down the side slopes. Spoil banks should be constructed to prevent this, and pipe inlets should be provided to control the inflow of surface water. Figure 5–16 shows some typical culverts and drain inlets and an acceptable method for installing a surface water pipe inlet to an open drain.

5–25. Transition Sections.—Changes in the channel depth or bottom width should not be made abruptly, but over a distance of 3 meters (10 feet) or more, depending upon the extent of the change. Where the depth changes, the slope of the transition should be gentle enough to prevent scouring. Transition sections should be located above the entrance of any side drains.

5–26. Design Capacities.—Surface drain channels should be designed for stormflow only with no allowance for irrigation waste because the magnitude of stormflow usually is so much greater than the magnitude of irrigation waste that



TYPICAL CROSS DRAINAGE CHANNEL SECTION



ELEVATION CLOSED DRAIN OUTLET

Figure 5-15.—Typical drain and collecting ditch sections (sheet 1 of 2). Drawing 103-D-1661.

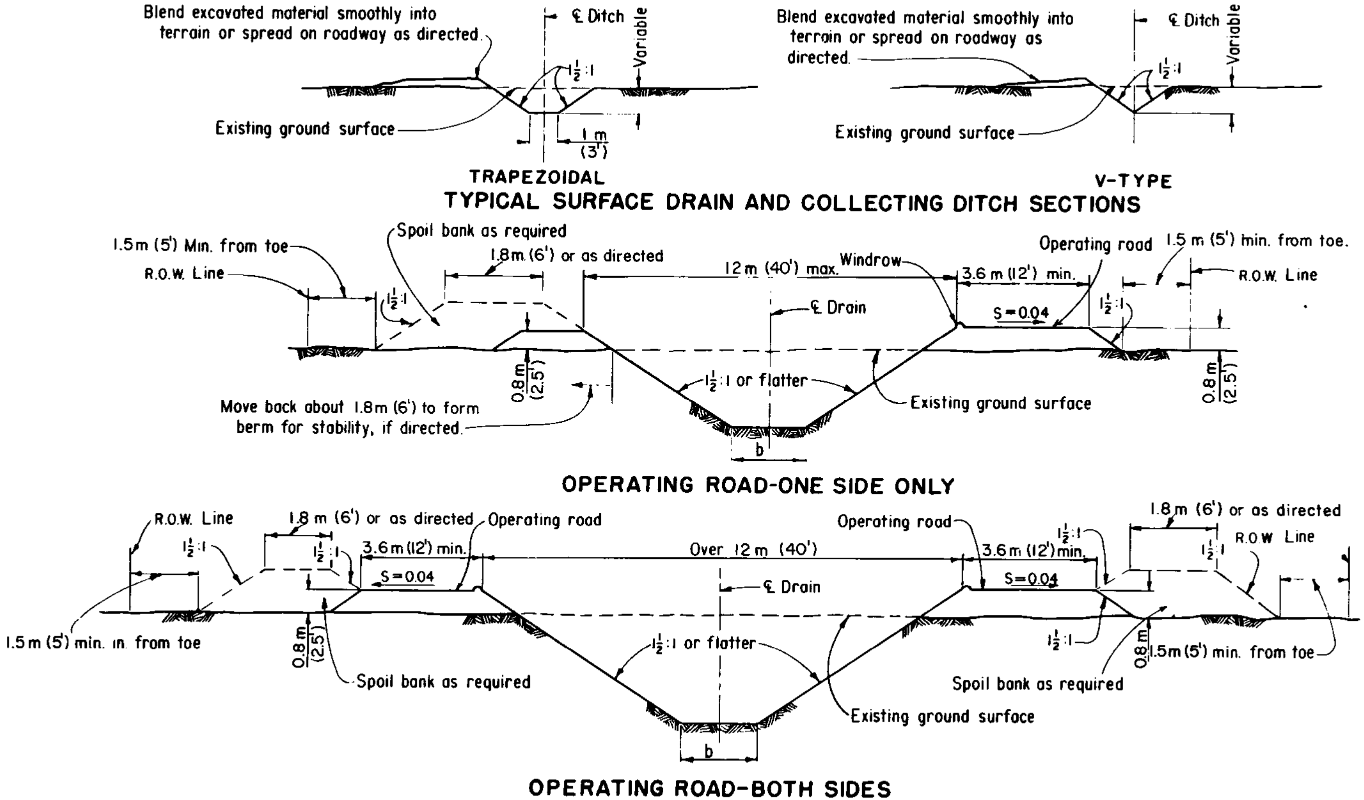


Figure 5–15.—Typical drain and collecting ditch sections (sheet 2 of 2). Drawing 103–D–1661.

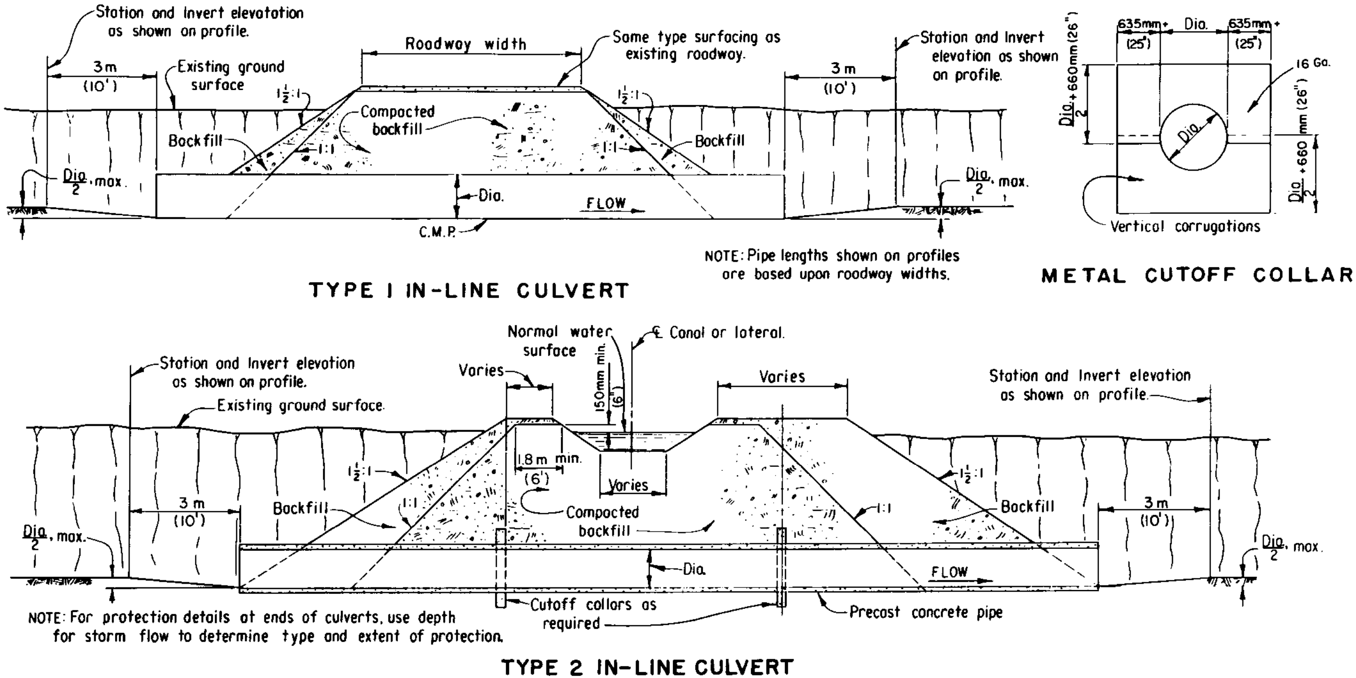
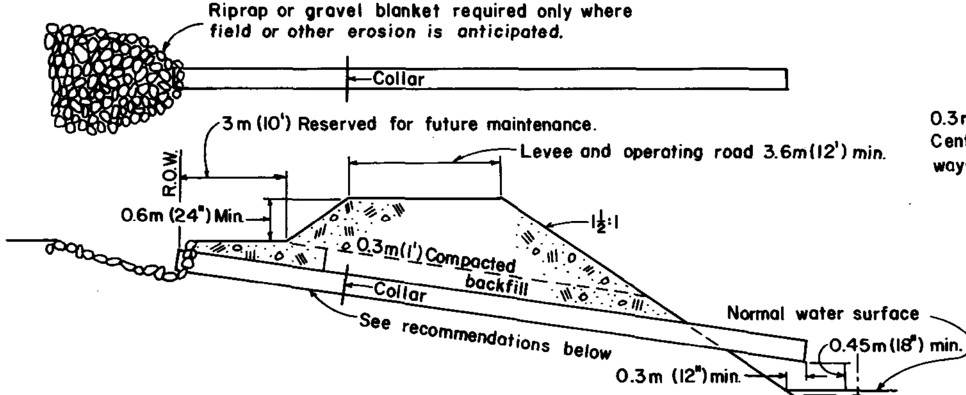
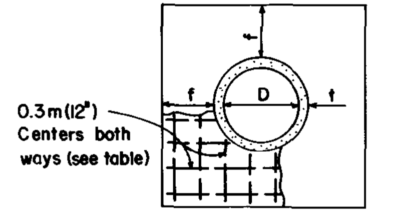


Figure 5-16.—Typical culverts and drain inlets (sheet 1 of 2). Drawing 103-D-1662.



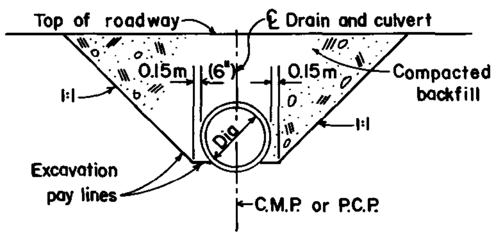
NOTE:
(1) For near salt free soil, heavy gage galvanized C.M.P. may be used.
(2) For saline or alkaline soils, galvanized pipe should not be used.
(3) Minimum pipe diameter is 0.45 m.

TYPICAL SECTION OF DRAIN INLET

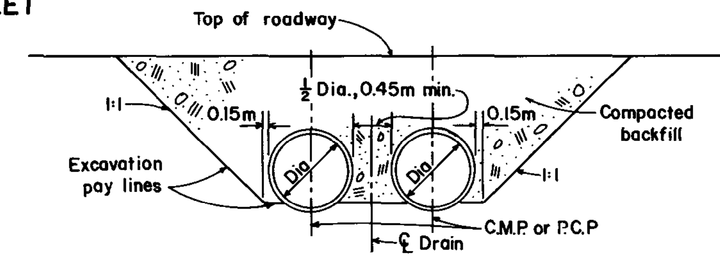


CONCRETE CUTOFF COLLAR

DIAMETER mm	(in.)	f meters	feet	Nominal t mm	(in.)	Collar Reinf.
460	(18)	0.460	1.50	65	2.50	#3
530	(21)	0.460	1.50	70	2.75	#3
610	(24)	0.460	1.50	75	3.00	#3
760	(30)	0.508	1.67	90	3.50	#3
910	(36)	0.610	2.00	100	4.00	#3
1070	(42)	0.686	2.25	115	4.50	#4
1220	(48)	0.762	2.50	125	5.00	#4
1370	(54)	0.762	2.50	135	5.50	#4
1520	(60)	0.762	2.50	150	6.00	#4



TYPICAL SECTION OF SINGLE BARREL CULVERT



TYPICAL SECTION OF DOUBLE BARREL CULVERT

NOTE: For protection details at ends of culverts, use depth for storm flow to determine type and extent of protection.

Figure 5–16.—Typical culverts and drain inlets (sheet 2 of 2). Drawing 103–D–1662.

the impact of irrigation waste would be negligible. In general, stormflows should be estimated for 5-year frequency storms unless available information justifies use of other flows. The minimum capacity of surface drains should be 0.08 to 0.14 cubic meters (3 to 5 cubic feet) per second. Ponding for stormflows in the field should be considered in surface drain capacity estimates. But ponding on arable land should not be permitted for periods exceeding 48 hours. Most crops submerged over 48 hours suffer reduced production, and many crops are destroyed completely.

Capacities of open interceptor and relief drains intended primarily for control of ground-water levels should be sufficient to carry the estimated ground-water accretion plus the estimated farm waste, with the water surface elevation of the drain at or below the required effective drainage depth. Storm water from fields, which may enter these drains through regular drain inlets, will not be considered in design unless stability is a problem, because neither the quantity nor the duration of flow would normally adversely affect the efficiency of the drain.

Capacities for open collector drains should be sufficient to carry normal flow of ground-water accretions, irrigation surface waste, estimated stormflow, and the quantities delivered to the collector drains by relief and interceptor drains.

Capacities for open outlet and suboutlet drains should be sufficient to carry the flows from the collector drains.

Wastewaters in canal wasteways are sometimes turned into open drains rather than being carried separately to a point of disposal. In this case, the capacity of the drain must be designed to include the expected amount of waste, which is usually the capacity of the canal.

5–27. Structures.—Open drain structures consist of inlets to the drain; drops and chutes; and road, railroad, and canal crossings. Actual structural design should be made in accordance with Reclamation policy and standards.

(a) Inlets.—Inlets should be made of corrugated metal pipe with a design coefficient of roughness, *n*, of 0.021. The pipe can be galvanized, asphalt dipped, or polymer coated, depending on the corrosivity of the soil. The corrosivity can be best determined by experience in the area with highway culverts, existing drainage structures, or similar means. The minimum pipe size should be 450 millimeters (18 inches) in diameter to minimize operation and maintenance costs. Velocity in the pipe should not exceed 3 meters (10 feet) per second, and the minimum pipe slope should be 0.01. The outlet end of the pipe should extend 300 millimeters (12 inches) beyond the edge of the normal water surface in the drain so that water from the pipe will not drain onto and erode the bank of the drain. This end of the pipe should also be at least 450 millimeters (18 inches) above normal water surface elevation in the drain, see figure 5–16. Multiple pipes may be used if required. Headwalls are not necessary, although riprap may be required on larger structures. Earth backfill should be compacted around the pipe for its full length and for 300 millimeters (1 foot) above the pipe. One collar is required for each pipe, as shown on figure 5–16.

(b) Drops and Chutes.—Conventional chute structures may be used where appropriate. Drop structures should be used as follows:

Differential drop in water surface		*Structure*
meters	feet	
0 to 0.6	0 to 2.0	No structure but some riprap
0.3 to 1.5	1.0 to 5.0	Rock cascade drop with sheet piling
1.5 and over	5.0 and over	Baffled apron or rectangular-inclined (R.I.) drops

(c) Crossings.—Crossing structures can be of either metal or concrete pipe depending on the importance of the crossing, which is measured by the capital loss that would result from its failure. In chemically active soils and waters that would be corrosive to the pipe, the pipe should be protected with an asphalt or similar coating. Crossing structures for major highways, railroads, and canals should be designed for flows from a 25-year storm; for less major crossings, flows from a 10-year storm can be used; and flows from a 5-year storm can be used for roads within a field or for farm ditches. Circular pipe culverts can be placed with a maximum of 50 percent of their diameter below gradeline; however, 25 percent or 0.3 meter (1 foot) maximum is the preferred limit. Pipe-arch, corrugated-metal culverts, if justified, can be placed with about 20 percent of the "rise" value below gradeline. The pipe should extend beyond the toe of the fill, and collars should be placed on the pipe as required. Maximum velocity for a full pipe should be about 1.5 meters (5 feet) per second. A siphon-type structure should not be used for drainage crossings because of the variation in flow. During low flows, any transported sediment will be deposited in the siphon, and without scheduled maintenance, the crossing will become plugged.

5–28. Natural Channels.—In many instances, a natural channel (Kouns and Pemberton, 1963) is used as an open drain for conveyance of irrigation surface wastewater and storm water. The addition of irrigation surface waste (or in some cases, subsurface drainage flow) will often change a normally dry stream to one with a continuous flow, at least for the irrigation season. This change corresponds to a change from an ephemeral stream to an intermittent or perennial stream. The continuous wetting of the natural channel banks may result in an unstable condition when a floodflow occurs.

The stability of the natural channel used as an open drain should be checked by a tractive force analysis based on particle-size analyses or plasticity indices of soil textures. Stability should be determined for 5-year frequency floodflow, plus irrigation waste flow. The tractive forces used to check stability, in addition to being affected by wetted banks, are also adjusted for the type of sediment transported by the channel. If instability is indicated, control structures will be required.

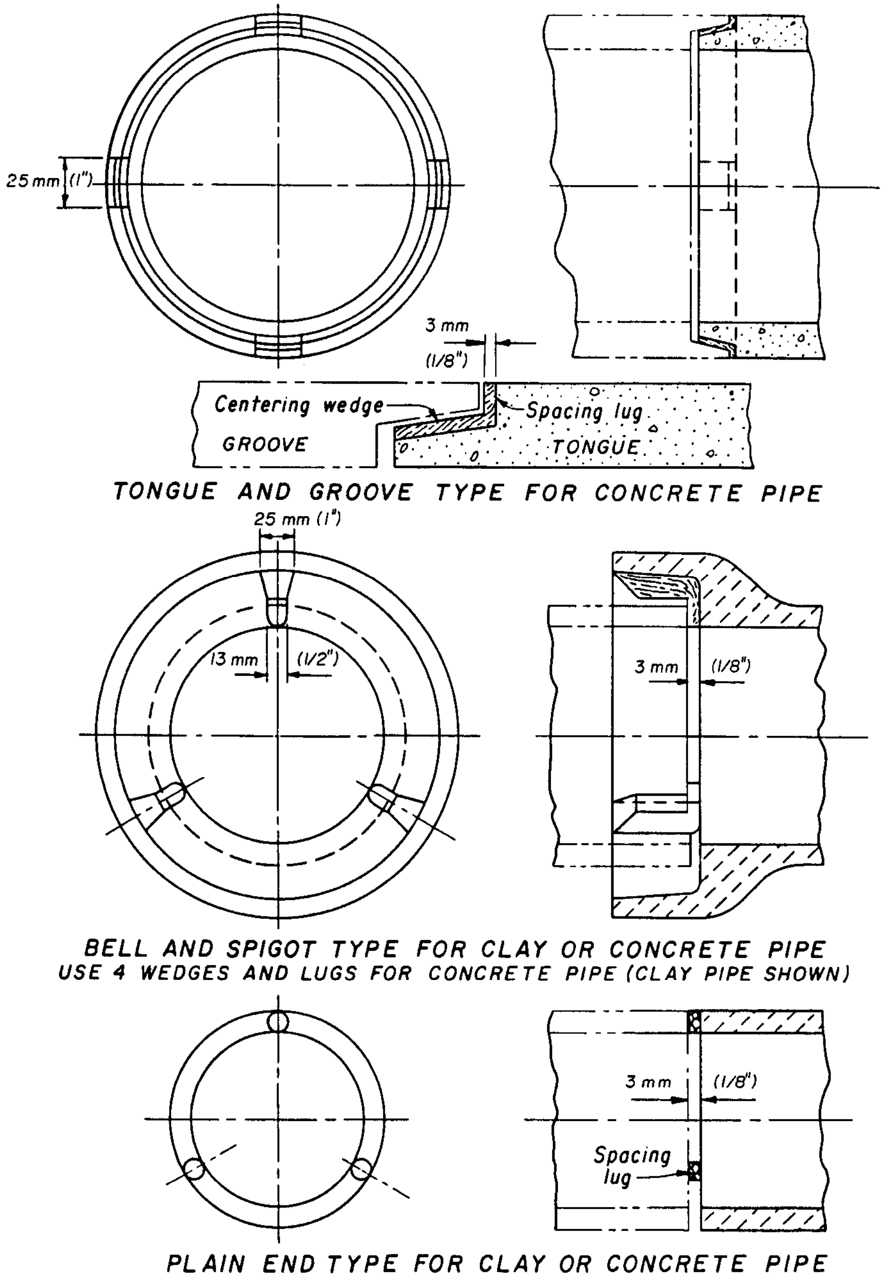


Figure 5–17.—Joint design for rigid pipe drains. Drawing 103–D–1663.

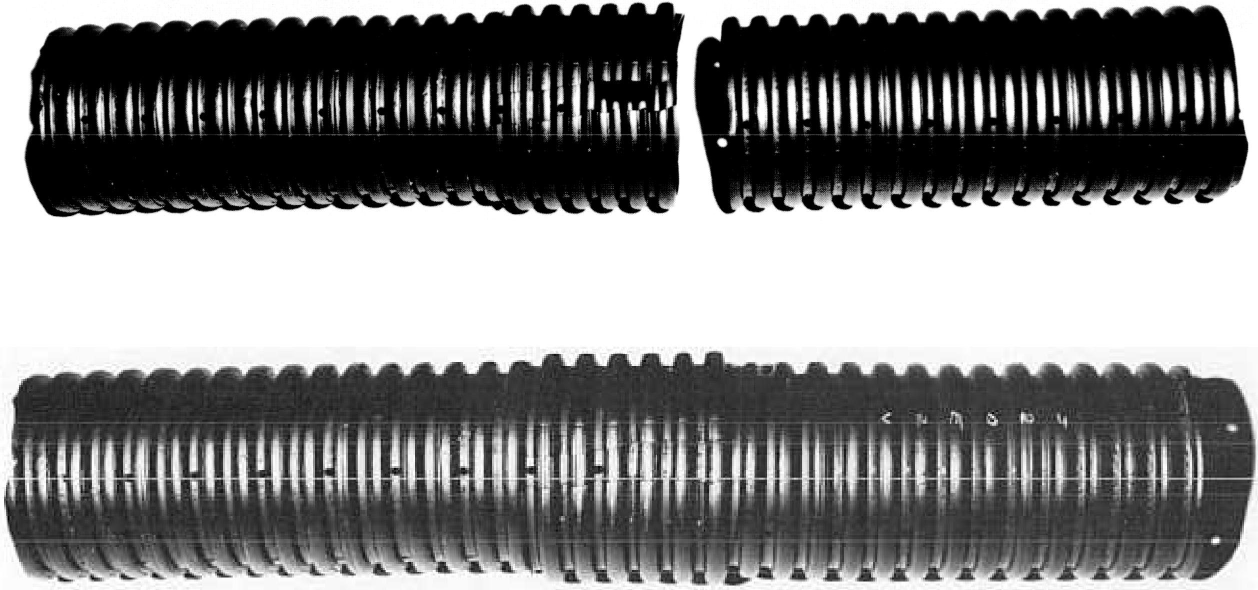


Figure 5–18.—Typical pipe and joint sections for corrugated plastic pipe drains. Two lengths of pipe are shown joined together by a joint section. Drawing P801–D–77015

5–29. Stage Construction.—Stage construction is sometimes used when an open drain must be excavated in saturated unstable material such as fine sands, fine sandy loams, silts, and silt loams. In stage construction, the portion of the drain section that will remain relatively stable is excavated and the banks allowed to drain and stabilize before the next stage of excavation is started. This process is continued until design grade is reached. Drains requiring this type of construction can be readily anticipated during the initial investigations of an area when casings must be installed to keep hand-augered holes, below the water table, open.

Estimating costs for stage construction presents problems for the engineer because of the difficulty in determining how many stages will be required and the time required between stages. If construction can be scheduled for the nonirrigation season, drawdown can be relatively rapid and the drain will stabilize quickly. If the nonirrigation season is short and the water table is constantly being recharged, the stage construction might extend over a 1- or 2-year period. For this situation, it would probably be more economical to make each stage a separate contract or schedule the work to be done by O&M (operation and maintenance) personnel when excavation conditions are suitable.

Stage construction costs for open drains vary with many factors but could go as high as 50 percent over what the drain would cost if completed in one stage.

D. Pipe Drains

5–30. Introduction.—Normally, pipe drains are used when they are lower in capital and annual costs than open drains. The computation of annual costs should include, in addition to the construction and maintenance costs, values for the right-of-way costs and for the loss of project income from land in open drains. Comparison of the environmental and esthetic values between open and pipe drains should also be made.

In general, pipe drains should only collect and remove ground water, but in special instances, they may have to carry storm water or excess irrigation surface waste. When waters other than ground water are collected, larger pipe must be used to carry the increased flow and to prevent clogging from surface debris. Pipes should be designed to flow only half full when surface water is collected.

5–31. Pipe for Drains.—Pipe drains consist of buried pipe with some type of openings in the pipe through which water can enter. The water is then carried in the pipe to a point of disposal. The pipe is usually manufactured from clay, concrete, plastic, or any of the suitable material that will not deteriorate rapidly with time.

Ordinarily, clay and concrete drainpipe is placed with 3-millimeter (1/8-inch) openings or cracks between the pipe lengths through which water enters the drain. Some rigid pipes are manufactured with holes or similar special provision for water entry, but they are usually too expensive for general use.

Pipe joints are sealed when pipe drains are laid under canals, railroads, highways, or near trees. Any one of the standard sealing methods used in laying sewer pipe is appropriate. Sealing prevents piping soil into the drain that may result in damage to the overlying structure, and keeps roots from entering and clogging the drain.

Concrete and clay drainpipe is manufactured with plain, tongue-and-groove, or bell-and-spigot ends. With the latter two end types, the adjoining sections interlock, making them easier to place and hold to grade and alignment than sections with plain ends. For all types of pipe ends, the openings between pipe sections must be maintained at about 3 millimeters (1/8 inch). To ensure that the joint spacing will be maintained, the bell and-spigot and tongue-and-groove pipe should be provided with wedges for centering, and lugs for spacing. A suggested arrangement for placing these wedges and lugs is shown on figure 5–17, but other methods can be used if approved by the Contracting Officer. It is suggested that 3-millimeter (1/8-inch) spacer lugs be used because smaller openings may not be sufficient and larger openings could allow entry of soil and envelope material.

Corrugated plastic pipe is manufactured in long rolls, or 6-meter (20-foot) joints, the length dependent on the diameter. Water enters the pipe through slots or holes cut in the valley portion of the corrugations. The openings are generally evenly spaced around the circumference of the pipe and must provide a minimum of 2,120 square millimeters of open inlet area per meter (1 square inch per foot) of pipe. A serious problem occurs when the pipe is stretched during the laying process, causing the slots or holes to widen, which allows the gravel envelope to enter the pipe. Stretching the pipe also has the disadvantage of reducing its strength. Figure 5–18 shows a typical section of corrugated plastic pipe. Nonperforated corrugated plastic pipe is used in those areas where sealed joints would be specified if concrete or clay pipe were used. Successive lengths of plastic pipe are connected by manufactured splicers or by splitting a length of the same diameter pipe and laying it around abutting ends of pipe, see figure 5–18. The split pipe is then wrapped with plastic tape or otherwise tied in place.

Corrugated plastic pipe is currently being manufactured in sizes from 75- to 900-millimeter (3- to 36-inch) nominal diameter. This size range is adequate for most agricultural drainage applications. The costs of construction at the drainage site will usually determine the type of material used for drainpipe.

5–32. Pipe Specifications.—Unreinforced concrete pipe specifications for closed drains may be either ASTM C 14, C 412, C 118, or C 444, latest revisions. In addition to the requirements of these specifications, the following requirements must be met:

(a) A minimum of 10 sacks of cement per cubic meter (7-1/2 sacks per cubic yard) of concrete must be used. A low-alkali cement is required for drainpipe except where it is positively known that the aggregates to be used are not sufficiently reactive to require the low-alkali limitation. When concrete aggregates are reactive, a low-alkali cement should be used to protect against disruptive expansion.

(b) All pipe should be steam cured for a minimum of 48 hours between 38 and 60 °C (100 and 140 °F) or should be kept moist cured for not less than 7 days. All surfaces of the pipe shall be kept moist continuously from the time of completion

of molding to the completion of the curing period. The ambient temperature within the curing enclosure shall not exceed 38 °C (100 °F) within 2 hours after completion of molding; thereafter, the temperature shall be brought to the specified curing temperature and maintained for the specified number of hours. The ambient temperature rise within the steam curing enclosure shall not exceed 17 °C (30 °F) per hour. Pipe shall be protected from temperatures below 5 °C (40 °F) before and during curing operations.

(*c*) A maximum absorption of not more than 6.5 percent, 5-hour boiling test, in accordance with paragraph 18, ASTM C 14, is required.

(*d*) Pipe shall be air-dried for not less than 30 days prior to placement in the ground unless otherwise directed by the Contracting Officer.

(*e*) Calcium chloride shall not be used in the cement for concrete pipe.

These additional requirements are considered necessary to produce pipe that will have a long, useful life. When concrete pipe is used for manholes or when reinforced concrete pipe is used under railroads or where it is known that concrete pipe drains will be exposed to sulfate concentrations amounting to more than 0.2 percent in soils or 1,000 parts per million dissolved in ground water, the concrete is to be made with type V cement. If the aggregates to be used are known to be reactive, low-alkali type V cement should be used. In areas where the sulfate environment is not severe, cement other than type V may be used.

Clay pipe specifications for closed drains may be either ASTM C 4, C 13, or C 200.

Plastic pipe for use in Reclamation drainage systems shall conform to Bureau of Reclamation Standard Specifications M-19 for Corrugated Polyethylene and Polyvinyl—Chloride Drainage Pipe, July 1992. Special consideration must be given to limiting the stretch of corrugated pipe to 5 percent during installation to prevent failure by collapse. Also, the slots or holes in the pipe should be carefully inspected to ensure they are free of tag ends or other material. Tag ends and poorly cut slots or holes offer collection points for silts, clays, mineral deposits, and bacteria that often seal off water inlet areas.

5–33. Collectors.—Deep, open drains or natural drainageways normally serve as the collector drain for pipe drain systems; however, pipe drains must sometimes be discharged into a sump and the drainage water disposed by pumping into shallow surface drains. A thorough study of collector and suboutlet conditions and requirements is an important consideration in planning a pipe drainage system which will function satisfactorily.

5–34. Depth of Pipe Drains.—The depth of pipe drains is always a major consideration, because the success or failure of the entire drainage system may depend upon this factor. The depth will usually depend upon the outlet elevation, the general topography of the ground surface, and the position of the aquifer or water-bearing strata in the soil profile—all in relation to the required ground-water elevation. Because the primary function of a pipe drain is to collect and remove ground water, the pipe should be placed, if possible, in a relatively coarse-textured stratum.

In cases of deep, uniform profiles, depths of drains can be determined by analyzing costs. To accurately apply this method, drainage engineers should have experience data to draw from regarding costs for excavation, gravel envelope, and furnishing and laying pipe. Another data factor needed, and probably the most important, is the travel speed of the drain-laying equipment used in the area.

If drains have been previously built in the area, analyses of the bid abstracts on those drains are a good starting point. Weighted average costs could be determined and tabulated to arrive at an estimated cost per foot of drain.

The tabulation could be simplified by combining related items and expressing the costs as a percent of the total as in the following example:

Summary of cost by item

Item	*Percent*
Earth work	42
Pipe	42
Gravel envelope	16
Total	100

Expressing the costs as a percentage of the total may be useful in projecting costs to nearby areas where drains have not been constructed; however, estimating costs based on construction estimates is more reliable.

Next, some idea of the rate of installing drains must be developed. Figure 5–19 shows the rates of installation by drain depth for three different trenchers as experienced on various Reclamation projects. The information from this figure, along with the drainage requirement per hectare (acre) drained, can be used to determine the cost per hectare (acre) related to the depth of drain. The following examples[1] illustrate typical procedures:

Example 1: High-speed trencher.
Assume:

(a) Average total cost of a 2.4-meter (8-foot) deep drain is $11.52 per linear meter ($3.51 per linear foot) and this cost is distributed as follows:

(1) Excavation—42 percent
(2) Pipe—42 percent
(3) Gravel envelope—16 percent

[1]The given costs may be different from current costs; however, the procedure in the examples is still valid.

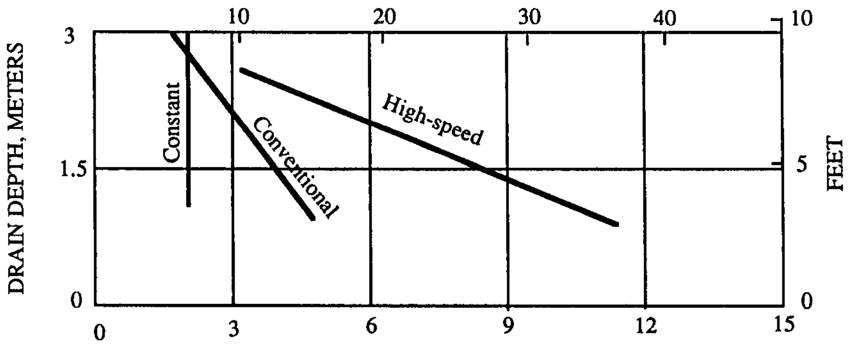


Figure 5–19.—Rate of installation of drains by drain depth for three different types of trenching machines. Drawing 103–D–1664.

(b) The drainage requirement varies with drain depth as shown below:

Drain depth,		*Drain spacing*		*Length of drain per*	*Length of drain per*
meters	*feet*	*meters*	*feet*	*hectare, meters*	*acre, feet*
1.4	4.5	108	355	92.6	123.0
1.5	5.0	152	498	65.8	87.5
1.7	5.5	184	605	54.3	72.0
1.8	6.0	211	693	47.4	62.9
2.0	6.5	234	768	42.7	56.8
2.1	7.0	255	835	39.2	52.2
2.4	8.0	288	945	34.7	46.1

(c) Cost per minute based on bid abstracts of operating the high-speed trencher can be calculated as follows:

Excavation cost = ($11.52/m)(0.42) = $4.84/m ($1.47/ft)
Cost per hectare (acre) for excavation = (4.84/m)(34.7 m/hectare) = $167/hectare ($68/hectare)
Rate of installation from figure 5–19 = 3 m/min (10 ft/min)
Cost of excavation per minute = ($4.84/m)(3 m/min) = $14.70/min

(d) Cost per meter (foot) of gravel envelope = ($11.52/m)(0.16) = $1.84/m ($0.56/ft)

Cost per meter (foot) of pipe = ($11.52/m)(0.42) = $4.84/m ($1.47/ft)

Using similar assumptions and methods for each drain depth, table 5–7 can be made.

Table 5–7.—*Cost relationships for drains installed with high-speed equipment.*1/

Drain depth, meters	Drain spacing, meters	Length per hectare, meters	Time per hectare, minutes	Cost in dollars per hectare				Cost, dollars per meter
				Excavation	Pipe	Envelope	Total	
1.4	108	92.6	9.22	136	450	170	756	8.16
1.5	152	65.8	7.22	106	321	121	548	8.33
1.7	184	54.3	6.60	96	264	99	459	8.46
1.8	211	47.4	6.47	96	230	86	412	8.70
2.0	234	42.7	7.02	104	208	79	391	9.16
2.1	255	39.2	7.59	111	190	72	373	9.51
2.4	288	34.7	11.39	168	168	64	400	11.53

Drain depth, feet	Drain spacing, feet	Length per acre, feet	Time per acre, minutes	Cost in dollars per acre				Cost, dollars per foot
				Excavation	Pipe	Envelope	Total	
4.5	355	123.0	3.73	55	182	69	306	2.49
5.0	498	87.5	2.92	43	130	49	222	2.54
5.5	605	72.0	2.67	39	107	40	186	2.58
6.0	693	62.9	2.62	39	93	35	167	2.66
6.5	768	56.8	2.84	42	84	32	158	2.78
7.0	835	52.2	3.07	45	77	29	151	2.89
8.0	945	46.1	4.61	68	68	26	162	3.51

1/ These costs and relationships may vary from correct values, but the procedures are similar.

From table 5–7, the drainage cost per hectare (acre) is at a minimum for drains placed about 2.1 meters (7 feet) below ground surface. The table also shows that the cost per meter (foot) increases with depth but gives no indication as to what optimum depth to place the drains. Figure 5–20 shows these cost relationships for a high-speed trencher.

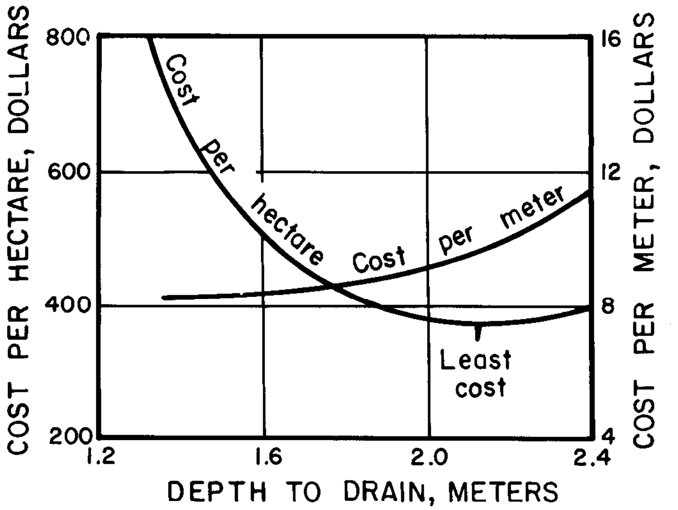


Figure 5–20a.—Cost relationships by drain depth for drains installed with a high-speed trencher (metric units). Drawing 103–D–1665.

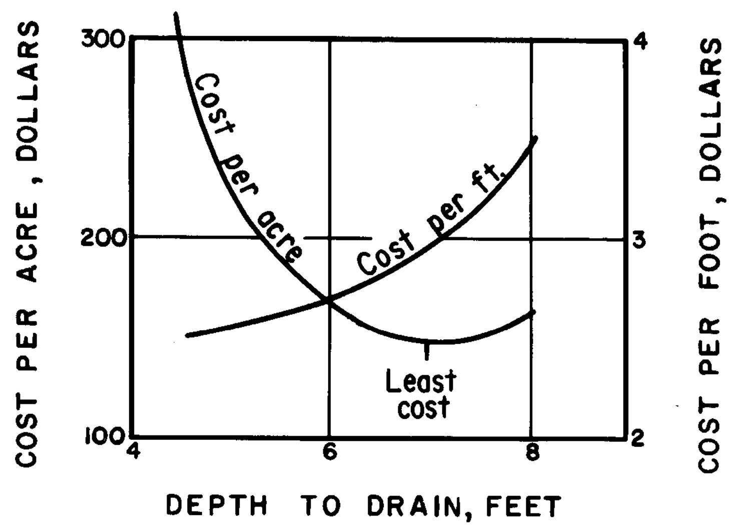


Figure 5–20b.—Cost relationships by drain depth for drains installed with a high-speed trencher (U.S. customary units). Drawing 103–D–1665.

Four other analyses were made using the same basic assumptions used in example 1 with the following alternatives:

Example 2—Conventional trencher with variable speeds.
Example 3—Constant speed trencher.
Example 4—Conventional trencher with half the unit pipe costs of example 1.
Example 5—Conventional trencher with half the unit excavation costs of example 1.

Figure 5-21 shows the relationships between cost per hectare (acre) and depth to drain for examples 1, 2, and 3. This figure indicates that drains installed with

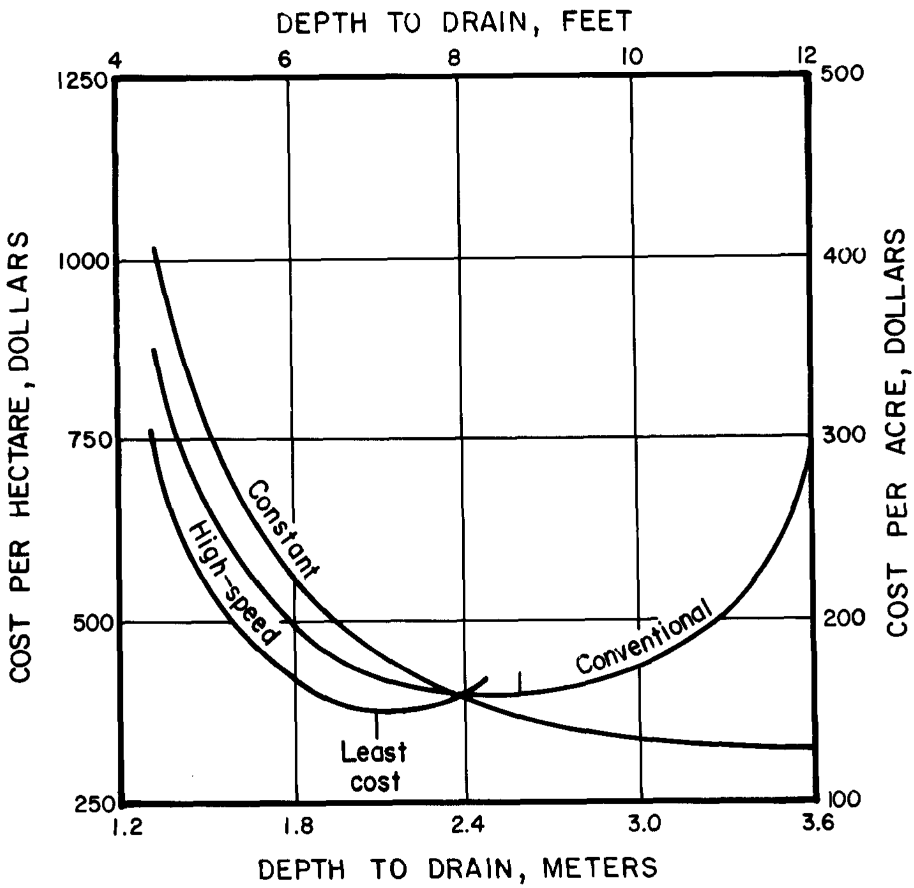


Figure 5-21.—Cost relationships by drain depth for three different trenchers. Drawing 103-D-1666.

high-speed trenchers at depths of about 2.1 meters (7 feet) will cost the least. If conventional trenchers are used, drains should be placed about 2.6 meters (8.5 feet) below ground level.

Figure 5–22 shows effects of reducing excavation and pipe costs by one-half, based on drains installed with a conventional trencher, examples 4 and 5. Reducing excavation costs by 50 percent does not affect selection of drain depth. However, reducing pipe costs by 50 percent changes optimum depth of drain to 2.4 meters (8.0 feet) instead of 2.6 meters (8.5 feet).

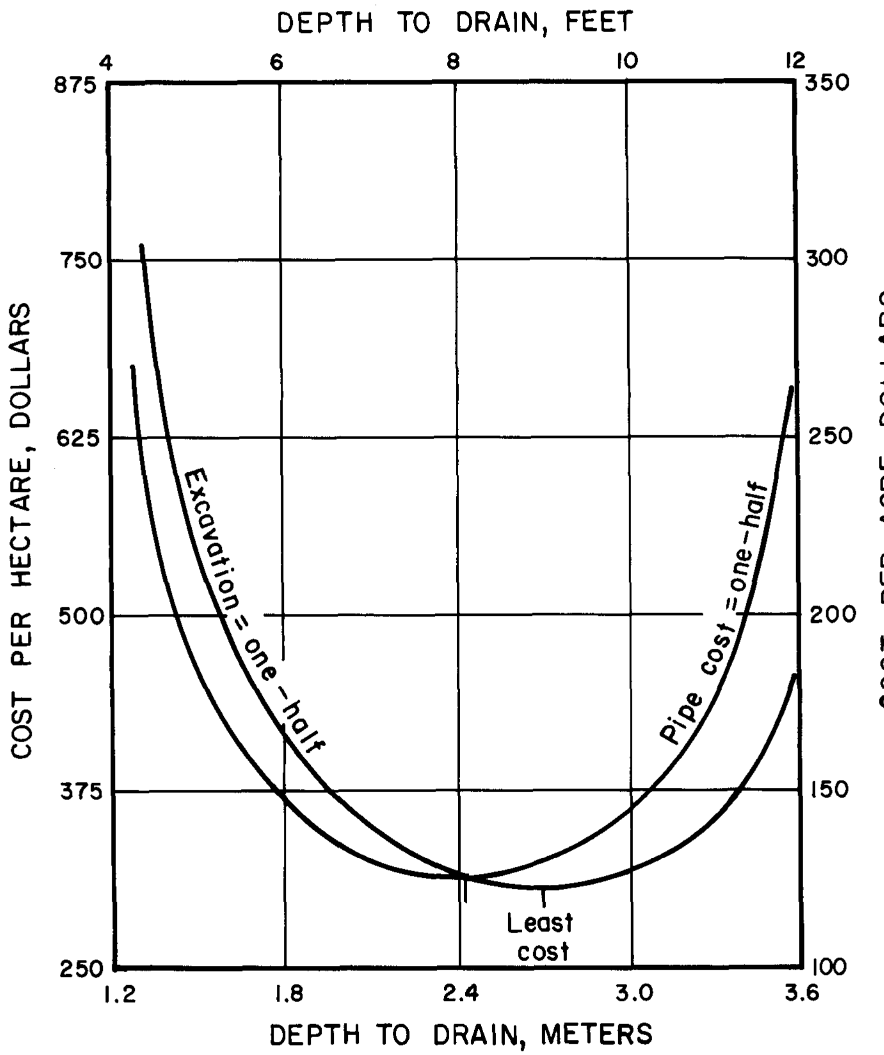


Figure 5–22.—Cost effects by drain depth as a result of reducing excavation and pipe costs by one-half for a conventional trencher. Drawing 103–D–1667.

Results from the preceding examples indicate that the rate of installing drains influences drain depths and costs more than any other single factor. Reducing the unit cost of excavation would have greater effect on reducing the total per-hectare (acre) cost than reducing the cost of pipe.

5–35. Grade and Alignment.—The proper installation and functioning of pipe drains require rigid control of grade and alignment. The minimum grade for a closed drain should be 1/1,000; however, steeper grades are more desirable. With steeper grades, the control required during construction can be less exacting and less chance of drain clogging exists. With the low flows that occur at various times in many pipe drains, any departure from established grade will result in solid material collecting in the lows which may eventually clog the drain. The maximum allowable departure from grade should not exceed 10 percent of the inside diameter of the drainpipe, and in no case should the departure exceed 0.03 meter (0.1 foot). Where departures occur, the rate of return to established grade should not exceed 2 percent of the pipe diameter per joint of concrete or clay pipe or per 0.9-meter (3-foot) length of plastic pipe. In determining the grade of a proposed drain, use a slope easy to work with in the field. For example, it is easier for the Contractor to establish and for the inspector to check grade if a slope of 0.002 is used instead of 0.00213.

The maximum allowable departure from alignment should not exceed 20 percent of the inside diameter of the drainpipe with a rate of return to the established line not to exceed 5 percent per joint of concrete or clay pipe, or a 0.9-meter (3-foot) length of plastic pipe.

5–36. Envelope Material.—Because all closed drains are pipe and may be located in all kinds of material, it is good practice to lay the pipe in a suitable envelope. Such an envelope is used to provide a permeable path for water to move into the pipe openings from the base material and to hold the base material in place. The graded envelope material also provides needed support for the flexible plastic pipe. This support in turn reduces the chances of excess deflection of the pipe and possible crushing during backfilling operations. The top of joints between plain-end pipe sections should be covered with asphalt building paper or plastic strips to prevent the finer particles of the envelope material from falling through the joint openings under the action of gravity. This covering is not recommended for bell-and-spigot or tongue-and-groove pipe, or perforated plastic pipe. An envelope less than 100 millimeters (4 inches) thick around the pipe probably would be sufficient, but because of the physical difficulty in placing envelope material uniformly to a small thickness, it is more economical to specify a 100-millimeter (4-inch) thickness.

Envelope gradation requirements for base materials of silt loams, sandy clay loams, and loams can usually be more flexible than for base materials that have textures of fine or very fine sands. Base material is that zone of soil material in which the drainpipe is physically located. The velocity at the interface between the finer textured base materials and the envelope material is so low that the fine-textured base material will not move into the envelope even under excessive leaching conditions. It has been observed that base materials having a predominance of particles which range in size from 0.05 to 0.4 millimeter tend to be easily moved. As a rule of thumb, this material will pass the No. 40 sieve and be retained by the No. 200 sieve. Velocities as low as 0.03 meter (0.1 foot) per second will move this size of material. For these soils, it is critical that placement of a properly designed and installed graded gravel envelope be a part of the drain construction process.

The gradation requirements should not be changed every time a different textured soil is encountered. From borings taken about every 180 meters (600 feet) along the centerline of a drain, the most permeable base material for significant lengths of the drain should be determined and the envelope designed for this material. Different gradation requirements can be specified if there are long sections of drain where the gradation and hydraulic conductivity of the base material indicate that a less expensive or easier to obtain envelope material can be used. However, a proper envelope material must be designed and used for these sections or the overall effectiveness of the drain might be impaired.

The envelope should be well graded, free of vegetable matter, clays, and other deleterious substances which could, in time, change the hydraulic conductivity of the envelope. For sieve analysis of the envelope material, 100 percent should pass the 38.1-millimeter (1-1/2-inch) clear, square screen openings, and not more than 5 percent should pass the 0.297-millimeter (No. 50 United States Standard Series) sieve. Because few pit-run sands and gravels meet these requirements, most envelope material must be machine sorted. Washing is required only when clean sand and gravel are not plentiful and the only source is from pits containing silt- or clay-coated material.

An envelope material is considered to be well graded when all particle sizes from the largest to the smallest are present. To determine whether a material is well graded, coefficients describing the slope and shape of the gradation curve have been defined as follows:

$$\text{Coefficient of uniformity, } C_u = \frac{D_{60}}{D_{10}}$$

and

$$\text{Coefficient of curvature, } C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})}$$

where:

D_{10}, D_{30}, and D_{60} = diameter of particles in millimeters (mm) passing the 10-, 30-, and 60-percent points on the envelope material gradation curve.

To be well graded, the coefficient of uniformity must be greater than 4 for gravels and greater than 6 for sands and, in addition, the coefficient of curvature must be between 1 and 3 for both gravels and sands.

In some locations, available sources of envelope material make the previous gradation limits uneconomical because the majority of the pit run material would pass the No. 30 sieve. For these locations, material passing the No. 200 sieve should be removed and a hydraulic conductivity test run on the remaining sample. Table 5–8 shows the gradation relationship between the base material and gravel envelope for most soils. These relationships are based on both field observations and laboratory work and have been found to work satisfactorily under the low-head conditions found near agricultural drains.

Table 5–8.—*Gradation relationship between base material and diameters of graded envelope material.*

Base material, 40 percent retained (diameter of particles, mm)	Gradation limitations for envelope (diameter of particles, mm)										
	Lower limits, percent retained						Upper limits, percent retained				
	0	40	70	90	95	100	0	40	70	90	100
0.02–0.05	9.52	2.0	0.81	0.33	0.3	0.074	38.1	10.0	8.7	2.5	0.59
0.05–0.10	9.52	3.0	1.07	0.38	0.3	0.074	38.1	12.0	10.4	3.0	0.59
0.10–0.25	9.52	4.0	1.30	0.40	0.3	0.074	38.1	15.0	13.1	3.8	0.59
0.25–1.00	9.52	5.0	1.45	0.42	0.3	0.074	38.1	20.0	17.3	5.0	0.59

Figures 5–23a, 5–23b, and 5–24 show excavation amounts for various widths and depths of trenches and the 100-millimeter (4-inch) gravel envelope amounts for various pipe sizes.

5–37. Determining Hydraulic Conductivity of Envelope Material.—In most cases, the hydraulic conductivity of the envelope material will be adequate when all the material is retained on the No. 30 screen. However, the presence and effect on hydraulic conductivity of any deleterious substances not readily visible can be determined by the following hydraulic conductivity test:

(a) *Equipment.*—Equipment required is as follows:

(1) 300-millimeter (12-inch) length of 200-millimeter (8-inch) irrigation pipe.
(2) Standard No. 30 screen.
(3) Four small metal screws.
(4) Silicone caulking.
(5) Constant head device such as an overflow pipe inserted 50 millimeters (2 inches) below the top of the irrigation pipe.

The irrigation pipe should fit easily into the standard screen. Fasten it in place with screws and seal with silicone caulk. Etch a line on the inside of the irrigation pipe 180 millimeters (7 inches) above the screen.

DRAIN TRENCH EXCAVATION CUBIC METERS

FOR VARIOUS DEPTHS AND WIDTHS

DEPTH (meters)	40 cm	50 cm	60 cm	70 cm	80 cm	90 cm	100 cm
0.05	0.020	0.025	0.030	0.035	0.040	0.045	0.050
0.10	0.040	0.050	0.060	0.070	0.080	0.090	0.100
0.15	0.060	0.075	0.090	0.105	0.120	0.135	0.150
0.20	0.080	0.100	0.120	0.140	0.160	0.780	0.200
0.25	0.100	0.125	0.150	0.175	0.200	0.225	0.250
0.30	0.120	0.150	0.180	0.210	0.240	0.270	0.300
0.40	0.160	0.200	0.240	0.280	0.320	0.360	0.400
0.50	0.200	0.250	0.300	0.350	0.400	0.450	0.500
0.60	0.240	0.300	0.360	0.420	0.480	0.540	0.600
0.70	0.280	0.350	0.420	0.490	0.560	0.630	0.700
0.80	0.320	0.400	0.480	0.560	0.640	0.720	0.800
0.90	0.360	0.450	0.600	0.630	0.720	0.810	0.900
1.00	0.400	0.500	0.540	0.700	0.800	0.900	1.000
1.50	0.600	0.750	0.900	1.050	1.200	1.350	1.500
2.00	0.800	1.000	1.200	1.400	1.600	1.800	2.000
2.50	1.000	1.250	1.500	1.750	2.000	2.250	2.500
3.00	1.200	1.500	1.800	2.100	2.400	2.700	3.000
3.50	1.400	1.750	2.100	2.450	2.800	3.150	3.500
4.00	1.600	2.000	2.400	2.800	3.200	3.600	4.000
4.50	1.800	2.250	2.700	3.150	3.600	4.050	4.500
5.00	2.000	2.500	3.000	3.500	4.000	4.500	5.000

GRAVEL ENVELOPE CUBIC METERS

CUBIC METERS PER LINEAR METER FOR VARIOUS PIPE SIZES								
10 cm	15 cm	20 cm	25 cm	30 cm	37.5 cm	45 cm	52.5 cm	60 cm
0.095	0.123	0.153	0.181	0.213	0.264	0.319	0.376	0.439

BASIS OF GRAVEL ENVELOPE COMPUTATIONS

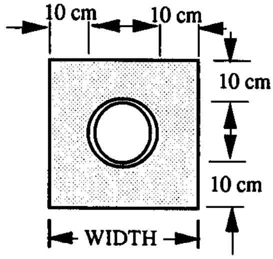


Figure 5–23a.—Excavation amounts for various trench widths and depths and 100-millimeter gravel envelope amounts for various pipe sizes (metric units). Drawing 103–D–684.

DRAIN TRENCH EXCAVATION YARDAGE
FOR VARIOUS DEPTHS AND WIDTHS

DEPTH (feet)	CUBIC YARDS PER LIN. FT. FOR WIDTH				
	18 in.	24 in.	27 in.	30 in.	36 in.
0.1	0.0056	0.0074	0.0083	0.0093	0.0111
0.2	.0111	.0148	.0166	.0185	.0222
0.3	.0167	.0222	.0250	.0278	.0333
0.4	.0222	.0296	.0333	.0370	.0444
0.5	.0278	.0370	.0416	.0463	.0555
0.6	.0333	.0444	.0500	.0556	.0666
0.7	.0388	.0518	.0583	.0648	.0777
0.8	.0444	.0592	.0666	.0741	.0888
0.9	.0499	.0666	.0750	.0833	.0999
1.0	.0555	.074	.083	.093	.111
2.0	.111	.148	.167	.185	.222
3.0	.167	.222	.250	.278	.333
4.0	.222	.296	.333	.370	.444
5.0	.278	.370	.417	.463	.556
6.0	.333	.444	.500	.556	.667
7.0	.388	.518	.583	.648	.778
8.0	.444	.592	.666	.741	.889
9.0	.497	.666	.750	.833	1.000
10.0	.555	.740	.833	.926	1.111
11.0	.610	.814	.916	1.019	1.222
12.0	.666	.888	1.000	1.111	1.333
13.0	.722	.962	1.083	1.204	1.444
14.0	.777	1.036	1.166	1.296	1.556
15.0	.833	1.110	1.249	1.389	1.667

GRAVEL ENVELOPE YARDAGE*

CUBIC YARDS PER LINEAR FOOT FOR VARIOUS PIPE SIZES								
4 in.	6 in.	8 in.	10 in.	12 in.	15 in.	18 in.	21 in.	24 in.
0.038	0.049	0.061	0.072	0.085	0.105	0.127	0.150	0.175

*Yardages are approximate but satisfactory for estimating purposes.

BASIS OF GRAVEL YARDAGE COMPUTATIONS

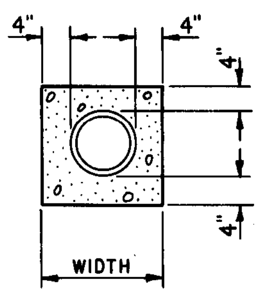


Figure 5–23b.—Excavation amounts for various trench widths and depths and 4-inch gravel envelope amounts for various pipe sizes (U.S. customary units). Drawing 103–D–684.

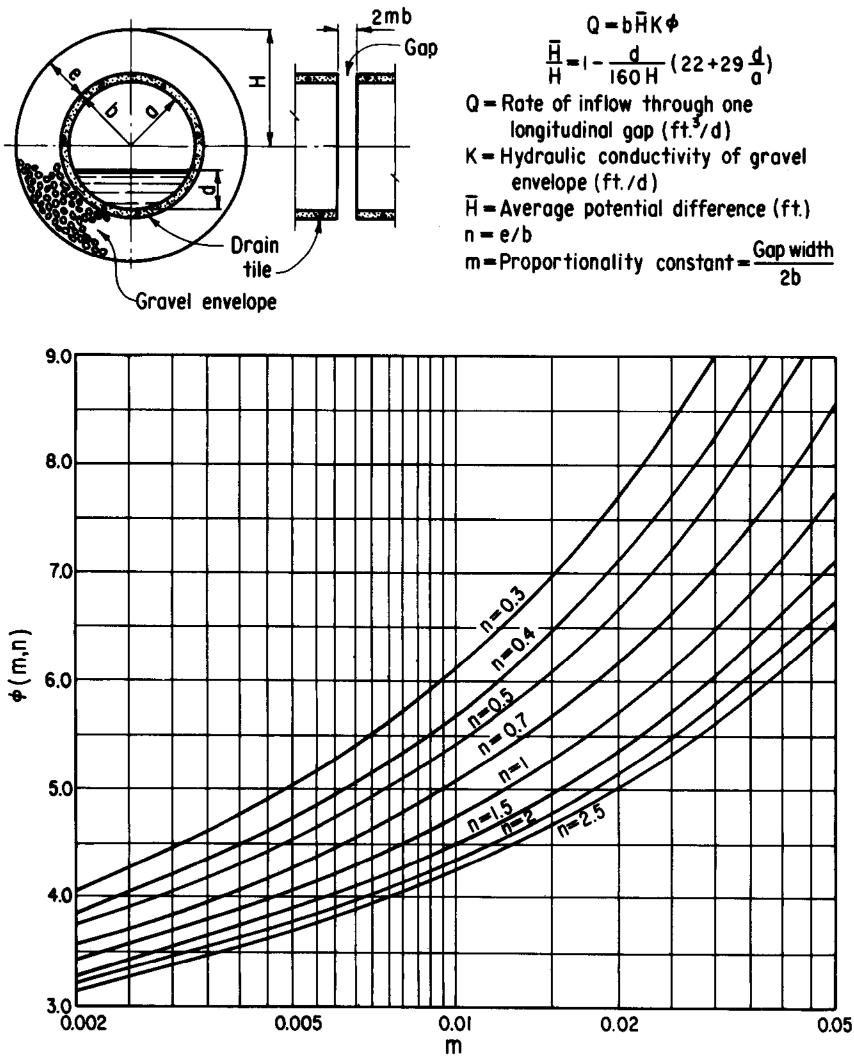


Figure 5–24.—Flow entering a spaced drain from a gravel envelope for concrete or clay pipe. Drawing 103–D–1668.

(b) Procedure.—

(1) Fill the irrigation pipe to the etched line with the envelope material. Drop it on a hard rubber pad 10 times from a height of about 25 millimeters (1 inch). Refill to the line with envelope.

(2) Slowly immerse the apparatus into a larger container of water until water rises above the envelope material and all air has been removed from the sample.

(3) Apply water to the top to maintain a constant head above the envelope material while the water outside the apparatus is removed.

(4) Maintain the constant head under free-flow conditions for a 5-minute interval.

(5) Catch, measure, and record the effluent for a 1-minute interval. Hold a constant head for another 25 minutes and again catch, measure, and record the effluent for 1 minute. Repeat this procedure after another 30 minutes of constant head. By the end of an hour, the presence of any material that might cause a reduction in hydraulic conductivity should be evident. In some of the less permeable envelope materials, a reduction in hydraulic conductivity may not become evident for 24 hours or more. Therefore, the test on any material that has a hydraulic conductivity of less than about 750 millimeters (30 inches) per hour at the end of 1 hour should be continued and measurements taken at the end of 12 and 24 hours. If a substantial reduction occurs in the hydraulic conductivity between the 12th and 24th hour, the test should be continued and a measurement taken at the end of 36 hours. If another substantial reduction in the hydraulic conductivity occurs between the 24th and 36th hour and the cause cannot be readily determined, the material should not be used for envelope material. To avoid difficulties from air bubbles, the water should be deaerated, especially if test is for extended periods.

(c) Calculations.—Use the Darcy flow equation in the form:

$$K = \frac{QL}{Aht} \tag{14}$$

where:

K = hydraulic conductivity in centimeters (inches) per hour;
Q = volume of water passing through the material in cubic centimeters (inches);
A = cross-sectional area in square centimeters (inches);
t = time in hours for which sample is collected (1/60th of an hour for most cases);
L = length of material column in centimeters (inches); and
h = height of water level above base of cylinder in centimeters (inches).

As a general guideline, a hydraulic conductivity rate of an envelope material which is 10 times the rate of the base material is adequate. It has also been observed that envelope materials which have hydraulic conductivity rates in excess of 150 meters (500 feet) per day [635 centimeters (250 inches) per hour] are difficult to place without segregation. If segregation occurs, voids develop in the envelope which allow fines from the base material to move into the drain.

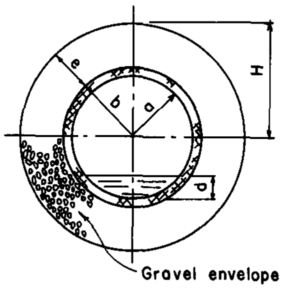
5–38. Gap Width, Length of Pipe Sections, and Hydraulic Conductivity of Envelope.—In designing a closed drain, it is assumed that: *(a)* the pipe will accept the drainage water when it arrives at the drainline, and *(b)* the pipe will carry away the water without a buildup of pressure within the pipe. Unless these assumptions are met, the drain will not function as intended, and the land may not be effectively drained. To meet the first assumption requires consideration of the relationship among the hydraulic conductivity of the gravel envelope, the length of pipe sections, and the gap width between pipe sections. To meet the second assumption requires that the pipe size and drain slope be sufficient to carry the water away after it enters the pipe. The design for the second assumption is explained in sections 5–46 and 5–47.

The theoretical relationship between rate of flow, hydraulic conductivity of the gravel envelope, and the head loss due to convergence of flow to the gap openings between lengths of pipe has been determined by W. T. Moody of the Bureau of Reclamation (Moody, 1960). His relationship is valid for all conditions of the closed drain, from empty to flowing full, but is not valid if the drain is under pressure. Moody concluded that increasing the hydraulic conductivity of the gravel envelope was a more effective method for increasing the rate of inflow than increasing the gap width. The curves and equations on figure 5–24 provide a means of analyzing the above relationships.

For corrugated plastic pipe having close, uniformly spaced slots or perforations throughout the length of the drain, figure 5–25 can be used to analyze the relationships developed by Moody. The curves on this figure were derived from electric analog studies performed by Reclamation personnel (Mantei, C. L., 1971, 1974).

The design curves in this section can be used in several ways. Generally, the rate of design inflow will be known before using these curves. If a certain length of pipe is more readily available than others, the minimum required hydraulic conductivity of the envelope can be determined. If the envelope material to be used is known and its hydraulic conductivity determined, the maximum permissible pipe length can be determined. Where the base material is highly permeable, it should be tested to determine if its hydraulic conductivity meets the requirements. If it does, there is no need to import envelope material because the excavated material will serve the purpose. Drains constructed of plastic drainpipe with a trencher require envelope material to be installed with the pipe to provide support for it during backfilling operations. For these conditions, it may be less costly to provide a graded envelope than to use excavated materials.

As an example, assume that a 100-millimeter (4-inch), corrugated plastic drain is to be installed and that it will run three-fourths full. The design inflow is 0.000013 cubic meter per second per meter (0.00014 cubic foot per second per foot) of drain. Assuming a 100-millimeter (4-inch) gravel envelope, the hydraulic conductivity needed for the drain can be determined and the suitability of the available envelope material can be checked in the laboratory.



$$Q = b\bar{H}K\phi$$

$$\frac{\bar{H}}{H} = 1 - \frac{d}{160\,H}\left(22 + 29\frac{d}{a}\right)$$

Q ≐ Rate of inflow per meter (ft.) of pipe, m^3/d ($ft.^3/d$)

K = Hydraulic conductivity of gravel envelope in m/d (ft./d)

H = Average potential difference, meter (ft.)

n = e/b

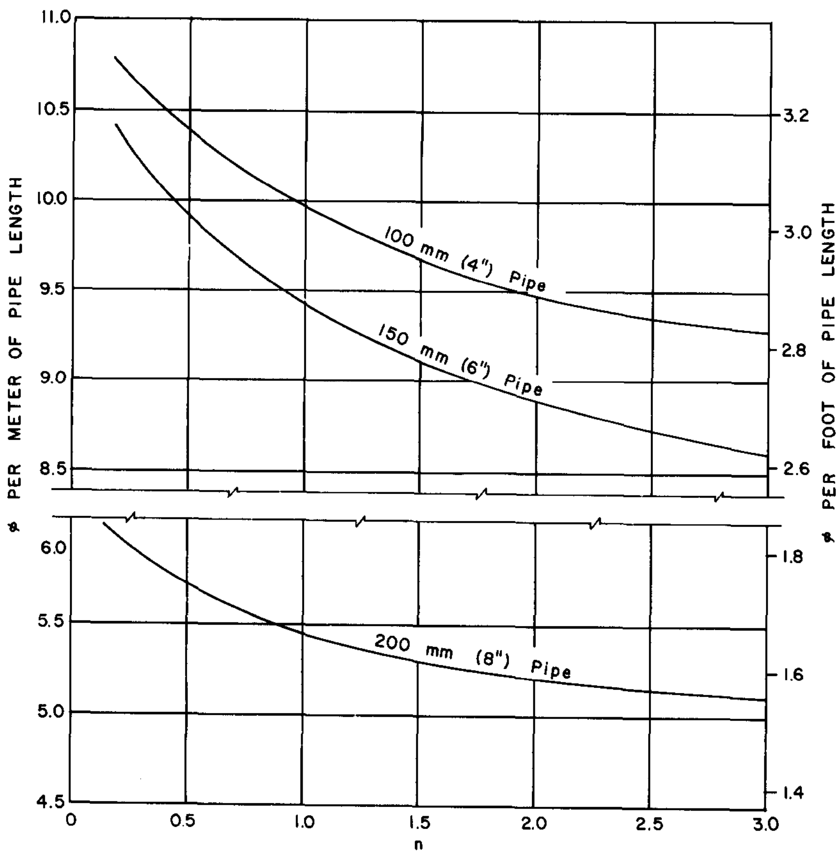


Figure 5-25.—Flow entering a spaced drain from a gravel envelope for plastic pipe. Drawing 103-D-1669.

Using the relationships shown on figure 5–25:

Q = 0.000013 $m^3/s/m$ (0.00014 $ft^3/s/ft$) = 1.12 $m^3/d/m$ (12.1 $ft^3/d/ft$)
b = 57.2 millimeters (2.25 inches) = 0.0572 meter (0.1875 foot)
e = 102 millimeters (4.00 inches)
a = 51 millimeters (2.00 inches)
d = 0.75 (102) = 76 millimeters (3.00 inches)
H = $b + e$ = 159 millimeters (6.25 inches)

$$\overline{H} = H\left[1 - \frac{d}{160H}(22 + 29\frac{d}{a})\right] = 127 \text{ millimeters (5.02 inches)} =$$

0.127 meter (0.418 foot),

$$n = \frac{e}{b} = 1.78$$

φ = (9.5) [from 100-millimeter (4-inch) pipe curve on figure 5–25], and

$$K = \frac{Q}{b\overline{H}\varphi} = \frac{1.12}{(0.0572)(0.127)(9.5)} = 16.2 \text{ meters (53.2 feet) per day} =$$

67.6 centimeters (26.6 inches) per hour.

The gravel envelope material requires a hydraulic conductivity of 67.6 centimeters (26.6 inches) per hour [16.2 meters (53.2 feet) per day] if a 100-millimeter (4-inch) envelope is used. The smallest diameter pipe used in a drainage system will always require the greatest hydraulic conductivity for the envelope material.

If the pit run material had a hydraulic conductivity of only 51 centimeters (20 inches) per hour [12.2 meters (40 feet per day)], the material should have to be processed to remove some of the fines to increase the hydraulic conductivity or else the thickness of the envelope would have to be increased. This increased thickness can be determined by substituting the measured hydraulic conductivity into the previous equation:

$$\overline{H} = \frac{Q}{Kb\varphi} = \frac{1.12}{(12.2)(0.0572)(9.5)} = 0.169 \text{ meter (0.556 foot)} = 169 \text{ milli-}$$

meters (6.7 inches).

Then, $\overline{H} = 169 = H\left[1 - \frac{d}{160H}(22 + 29\frac{d}{a})\right]$,

and H = 201 millimeters (7.9 inches).

Assuming the water level can be allowed to stand just at the top of the envelope with the pipe running full:

$H = b + e = 57.2 + e = 201$, and
$e = 143.8$. Use an envelope thickness, e, of 150 millimeters (6 inches).

The designer should compare the cost of using this extra envelope material against the cost of processing the pit run material before making his recommendations.

Many possible combinations of pipe diameter, pipe length, envelope thickness, and envelope hydraulic conductivity will satisfy the inflow requirements. All reasonable possibilities should be investigated to determine the most satisfactory and least expensive combination. However, compensating for low hydraulic conductivity by increasing the envelope thickness should be done cautiously. Never use envelope material having less hydraulic conductivity than the base material.

In the previous example, if the 100-millimeter (4-inch) diameter pipe were selected, it would be necessary to process the envelope material so that a 100-millimeter (4-inch) envelope could be used, and a 250-millimeter (10-inch) envelope would be required if the material was not processed. Also, a 150-millimeter (6-inch) diameter pipe could be used with a 100-millimeter (4-inch) gravel envelope of pit run material. Cost comparisons can be made on these different combinations as follows:

Furnishing and laying 100-millimeter (4-inch) pipe	\$1.57 per meter (\$0.48 per foot)
Furnishing and laying 150-millimeter (6-inch) pipe	\$2.13 per meter (\$0.65 per foot)
Furnishing and placing pit run material	\$5.56 per cubic meter (\$4.25 per cubic yard)
Furnishing and placing processed material	\$7.65 per cubic meter (\$5.85 per cubic yard)

The gravel envelope yardage would be:

100-millimeter (4-inch) pipe, 100-millimeter (4-inch) processed envelope	0.095 m^3/m (0.038 yd^3/ft)
100-millimeter (4-inch) pipe, 250-millimeter (10-inch) pit run envelope	0.376 m^3/m (0.15 yd^3/ft)
150-millimeter (6-inch) pipe, 100-millimeter (4-inch) pit run envelope	0.123 m^3/m (0.049 yd^3/ft)

Costs[1] for 100 meters (328 feet) and 100 feet (30 meters) of drainline are:

		100 meters	*100 feet*
100-millimeter (4-inch) pipe		$157.00	$48.00
100-millimeter (4-inch) processed envelope . . .		72.68	22.23
	Total	$229.68	$70.23
100-millimeter (4-inch) pipe		$157.00	$48.00
250-millimeter (10-inch) pit run envelope		209.06	63.75
	Total	$356.06	$111.75
150-millimeter (6-inch) pipe		$213.00	$65.00
100-millimeter (4-inch) pit run envelope		68.38	20.82
	Total	$281.38	$85.82

[1]Current costs may be different but the procedure of comparison is the same.

For this example, the most economical selection would be the 100-millimeter (4-inch) diameter pipe with a 100-millimeter (4-inch) processed gravel envelope.

5-39. Stability of a Pipe Drain Bed.—For a pipe drain to be as effective as predicted by the design data, it should be placed on a stable, undisturbed bed. This placement can be accomplished by installing the pipe in a dry trench where the base material remains undisturbed. However, pipe drains usually are not installed until after the ground-water table has risen higher than the bottom of the proposed drain, and many of the drainable agricultural soils become unstable when saturated.

There are a number of ways to stabilize a pipe drain bed, but only by dewatering the base material and installing the drains in stable soil conditions will the drain function at maximum effectiveness. When the base material in the vicinity of the pipe drain is disturbed, it usually becomes less permeable. Since most of the water entering the drain enters through the bottom portion of the pipe, any loss of hydraulic conductivity in this region increases head losses around the drain. This head loss causes a higher water table midway between spaced drains or upslope for interceptor drains. Unstable soils in the vicinity of the drain can be dewatered using well points. This method is expensive, but may be necessary for an effective concrete or clay drain.

Using a modern trenching machine, lightweight plastic pipe, and backfilling behind the trencher, there is seldom a need for dewatering the base material. However, when the base material is highly unstable, the shield may not prevent the base material from mixing with the envelope. This mixing results in an envelope with an indeterminate hydraulic conductivity and may cause the drain to malfunction.

When necessary, stabilization of drain beds can be accomplished with coarse gravel. In some instances, this method will require overexcavation; in others, the coarse gravel will work itself down into unstable material. Usually, the mixed material will have a lower hydraulic conductivity than the undisturbed base material, and the drain efficiency will be reduced. As a result, stabilization with coarse gravel could be less desirable than using well points when considering the life of the drainage system.

Stabilizing materials should conform to the following gradation:

Gradation of stabilizing material	*Percent*
Retained on 127-millimeter (5-inch) screen	0
Retained on 102-millimeter (4-inch) screen	0 to 20
Retained on 76.2-millimeter (3-inch) screen	0 to 30
Retained on 50.8-millimeter (2-inch) screen	20 to 50
Retained on 19.1-millimeter (3/4-inch) screen	20 to 50
Passing 4.76-millimeter (No. 4) screen	Less than 8

5-40. Laying Pipe Drains.—The finished bed for all pipe should be made smooth, including removal of material under the bell end of the bell-and-spigot-type joint, to ensure that the full length of pipe will be evenly and uniformly supported. When the bell-and-spigot-type joint is used, the bell end should always be upgrade. The pipe should be laid with the adjacent ends closely abutted against the spacing lugs. A drainpipe length should always be held in place by mechanical or other means until the next length of pipe is ready to be placed. Any pipe which is broken, cracked, or objectionable in any way should be discarded. Trenches that have been inadvertently overexcavated should be refilled with selected material and carefully compacted to original density or brought to grade with envelope material. During placement of the pipe, the water level in the trench should not exceed 50 percent of the pipe diameter above the invert of the pipe. Water may be removed from the trench by permitting it to flow through previously installed pipe. A screen cover should be placed over the exposed end of the pipe until the next length of pipe is placed. This screen should have a maximum mesh opening of 3.2 millimeters (1/8 inch).

Corrugated plastic pipe requires special precautions during laying operations. The plastic pipe must be well bedded and the bedding material should completely surround the pipe. The strength of the pipe depends upon the bedding material in addition to the design of the pipe corrugations. Care must be taken when laying the pipe to keep from stretching it more than 5 percent. Any greater stretch could cause deformation of the corrugations and permit collapse of the pipe during backfilling of the trench. Plastic pipe tends to float in water, so the trench should be backfilled as soon as possible after pipe installation. At sites where plastic pipe is being installed 0.6 meter (2 feet) or more below the water table, it may be necessary to add blinding material at the rear of the trenchers to prevent floating of the drainline.

When a portion of a pipe drain is not needed as a subsurface drain, such as under roads, laterals, and surface drains, or where roots could enter drain openings, the drains should be constructed with sealed joints. All joints should be sealed by hot-poured joint compounds, factory-fabricated joining connections, or rubber gaskets. Trenches must be kept free of water when joints are being sealed with the hot-poured compound. When plastic pipe is used, unperforated pipe with taped joints should be specified when sealed joints are required.

The upper end of pipe drains requires protection. Pipe drains can end in a manhole when the drain might be extended. If the drain will not be extended or if a manhole would be poorly located, a standard pipe plug packed with oakum should be used for terminating concrete or clay drains. Special end plugs are available for plastic pipe.

5–41. Inspecting and Testing Pipe Drains.—More pipe drains have proven to be ineffective because of poor inspection during construction than from poor location or design. The drain should be inspected for proper elevation below ground surface, grade, alignment, joint spacing, collapsing, broken or cracked pipe, and thickness of gravel envelope before backfilling. The inspector should ensure that the pipe drains and all manholes (including existing manholes used for outlets for new drains) are free of deposits of mud, sand, gravel, or other foreign matter, and are in good working condition. Unstable soils may preclude all but spot checks before backfilling.

Before being accepted as completed, each drain should be tested for obstructions. If a clean and unobstructed view of the complete bore of the pipe cannot be obtained between manholes by use of a high-powered light, a test plug having a diameter about 25 millimeters (1 inch) less than the drainpipe should be drawn through the drain. When a test plug is used, it should be rigid and tapered at both ends. The length of the plug, excluding tapered ends, should be twice the diameter of the pipe. The plug should be pulled by hand with a steady pull. A rope should be tied to both ends of the plug so that the plug can be backed out if necessary because of an obstruction. The rope also serves as a means for determining the location of the plug and obstruction if one is encountered. Pipe 380-millimeter (15-inch) diameter and larger should be inspected with a plug having a diameter which is 90 percent of the pipe diameter. For pipe 610 millimeters (24 inches) and larger, the use of a plug for inspection becomes difficult. This size pipe is seldom used for agricultural drainage systems. Visual inspection of large diameter pipe is recommended when practical. If not practical, then other means of ensuring no crushed, broken, separated joints or other obstructions exist will have to be used.

When concrete or clay pipe are used, an airfilled ball may be flushed through the drain in lieu of a rigid plug. Normally, the ball is used to locate obstructions, but due to the jetting action around the ball, small quantities of sand can be flushed out of the pipe. A waterhead of no more than 0.6 meter (2 feet) should be used when using this flushing method. The ball should float through the pipe and not be pulled. If pulled, the ball can pass through areas of pulled joints and partially filled pipe without being detected. The ball method does not work well on perforated plastic pipe.

5–42. Backfilling Pipe Drain Trench.—During backfilling, care should be taken to ensure that the drain is not disturbed either vertically or horizontally. The earth backfilling of the trenches should be done with material from the trench excavation. Backfill should be pushed diagonally into the trench and placed in concurrent horizontal lifts on both sides of the trench.

About 0.3 meter (1 foot) of fill should be carefully placed over the envelope before starting the general backfilling operations. This procedure ensures that backfill material does not drop directly onto the gravel envelope causing pipe displacement or failure. No more than about 300 meters (1,000 feet) of trench should be open at any one time. In unstable soils, this open trench length should be reduced to 8 meters (25 feet) or less. Rocks larger than 130 millimeters (5 inches) in diameter should not be permitted within 0.3 meter (1 foot) above the pipe, and frozen earth clods should not be permitted within 1.2 meters (4 feet) above the pipe. Special compaction of the backfill is not required except where pipe drains cross below irrigation or surface water drainage ditches or roads. At these locations, earth backfill should be compacted to a depth of 1 meter (3 feet) below the bottom of the ditch or roadbed being crossed. The compaction should be carried for such lengths along the trench that settlement or erosion under the road or ditch will not occur.

The top 0.6 meter (2 feet) of a trench in a field should be backfilled with topsoil that has been laid aside during the excavation of the trench. Excess backfill material, with all rocks, caliche, and other such material removed, should be deposited in a uniform windrow over the trench. Puddling the trench to restore the windrow to normal ground surface is permitted when carefully done. Under certain soil conditions, puddling can cause channeling of the water and movement of fine soils into the drain.

Upon completion of the drain, all canal, lateral, and farm ditch linings; fences; and concrete or asphalt roads should be restored to their original or improved condition.

5–43. Manholes.—Manholes are located in pipe drains to serve as junction boxes, silt and sand traps, observation wells, discharge measurement facilities, entrances to the drain for maintenance, and to permit easy location of the drain. There are no set criteria for the spacing of manholes. In general, a manhole should be used at junction points on a drain or at major changes in alignment on collector and suboutlet drains. Manholes are not required at every junction of closely spaced [less than 210 meters (700 feet)] relief or interceptor drains or collector drains. Manholes are usually not required at grade changes if the grade becomes steeper. Special effort should be made to locate manholes where they will not interfere with farming operations.

If a manhole cannot be justified for the purposes described above, a simple Y-section, T-section, or holes made in the collector pipe can be used to tie the relief or interceptor drains to the collector drain. Changes in pipe diameter should be made at a manhole, if convenient.

Manholes should extend a minimum of 300 millimeters (12 inches) and a maximum of 600 millimeters (24 inches) above the natural ground surface for easy recognition. They should be placed in fence rows or at other out-of-the-way locations if at all possible. Neither a manhole nor a cleanout is required at the upper end of a line, but this end of the line must be plugged. The location of the plugged end should be recorded both in fieldbooks and on as-built drawings.

When cleanout risers are used at the end of the line, they should be on a sufficient angle to permit entrance of cleaning equipment.

To compensate for the head losses within a manhole, the general practice has been to provide a drop at the invert elevation between the inlet and outlet pipes. This practice is satisfactory but not absolutely necessary and sometimes creates problems on level lands where the gradelines have to be greater than the gradients of the land surface. For this condition, the top of the inlet and outlet pipes can be placed at the same elevation. If design data show the inlet pipe to be at capacity at the manhole, the outlet pipe size will be increased and the necessary drop will be available in the larger pipe. If a size change is not required at the manhole, neither pipe will be at capacity and the slight head loss required will be available in the unused capacity of the pipes.

The base of the manhole should be about 450 millimeters (18 inches) below the bottom of the effluent pipe to form a trap that will catch any debris that may enter the line. Upon completion of a new drain, all traps should be cleaned out and the manhole covers set. Traps should also be cleaned periodically as a maintenance item.

Figure 5–26 shows a standard design for a manhole. Manholes may also be constructed of asphalt-dipped or polymer-coated corrugated metal pipe (CMP) where salinity of the soil and water is low and stability is a problem for heavy concrete pipe. Plastic manholes have also been successfully used.

5–44. Surface Inlets.—In general, surface water should not be allowed to enter a closed drain. In some instances, however, it may be necessary to dispose of small amounts of surface water in this manner. Even then, special precautions should be taken to remove weed and silt load from the surface water.

Topography may be such that an open drain can discharge directly into the closed drain, but more often the open drain will discharge into a manhole. In either case, every possible precaution should be taken to keep material from entering the closed drain which might clog it. The minimum precaution should be to install a self-cleaning trashrack in the open drain, which will prevent entry of large rocks, brush, and debris. A desilting pond should be provided if the water contains significant amounts of sediment.

5–45. Outlet Structures.—The outlet end of a closed drain, if not properly protected, will be undercut by the action of discharging water. This undercutting will cause the drain to shift out of proper grade and alignment and create costly maintenance problems. Complete blockage of the outlet may also occur. To prevent misalignment, 3.6 to 4.6 meters (12 to 15 feet) of heavy-gauge, asphalt-dipped, or polymer-coated CMP should be placed at the outlet end of closed drains. Corrugated, high-density polyethylene pipe is also used for drain system outlets. A screen should be placed on the pipe to keep rodents from entering. Some drain outlets require flap valves to keep high flows in the open drain from entering the pipe drain. All drain outlets should have a rodent screen installed over the end of the pipe. Figure 5–15 shows a typical closed drain outlet.

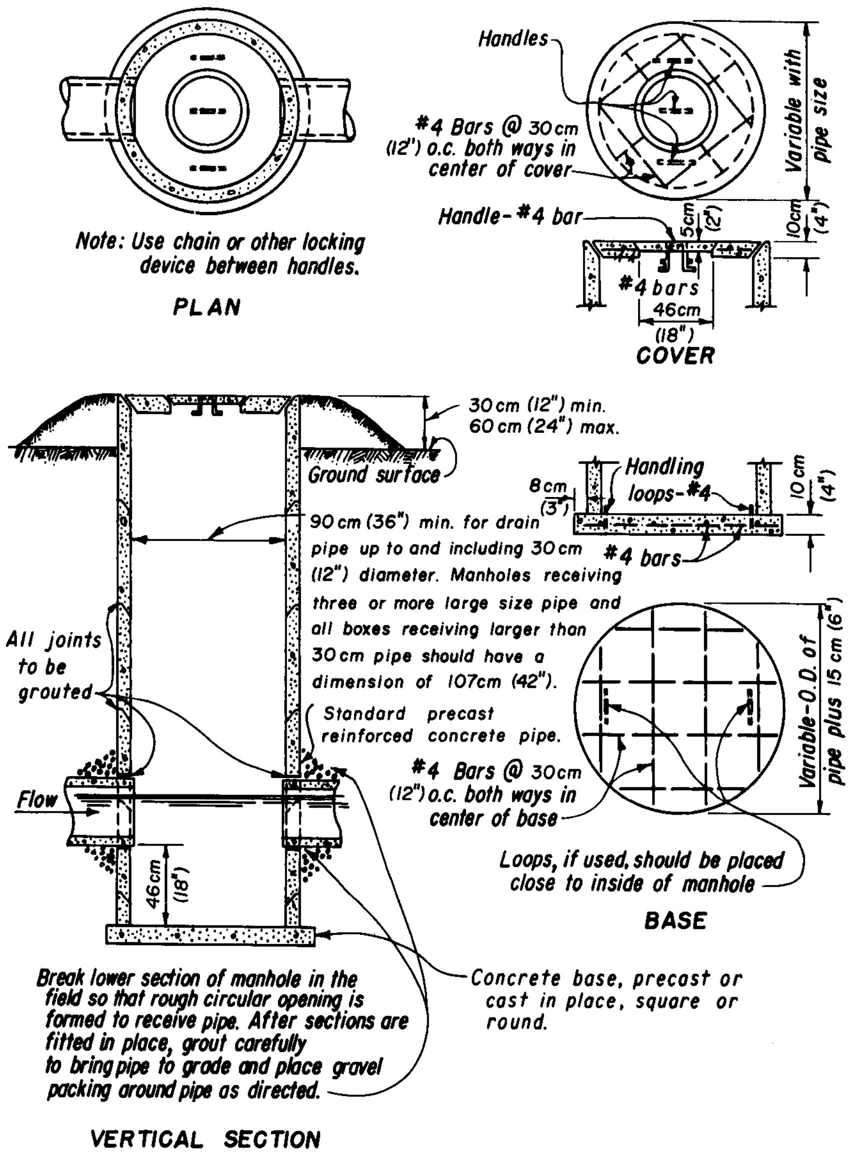


Figure 5–26.—Typical manhole design for a closed drain. Drawing 103–D–686.

5-46. Strength of Drainpipe.—*(a) General.*—Since closed drains in irrigated areas are usually placed at considerable depth below the ground surface, the ability of the pipe to carry the backfill load is an important consideration. Both concrete and clay pipe are made in several different strengths, so designs for the proper strength pipe are not only necessary to ensure the permanence of the drain, but also to permit use of the most economical pipe.

Figure 5-27 shows the load per linear meter (foot) on pipe from different backfilling materials for varying backfill depths and trench widths. The loads shown will vary slightly with the diameter of the pipe, so they are not exact, but they are within the limits of accuracy of other factors that affect the load and are satisfactory for use in design. The loads are based on Marston's formula as shown on figure 5-29. Note that trench widths are measured at the top of the pipe, and these values are used whether the trench sides are vertical or sloping. A nomograph for solving the Marston formula for rigid pipe is given on figure 5-28. A safety factor of 1.5 should be used to determine the strength of concrete or clay pipe required when strengths are determined by physical testing.

(b) Rigid pipe.—Table 5-9 shows the allowable crushing strength of various pipe laid in a gravel envelope. For pipe not laid in a gravel envelope, only 75 percent of these values should be used. The tabular values shown in table 5-9 assume that a class C bedding will result when using a gravel envelope. A class C bedding designates a shaped bed fitted to the lower part of the pipe. If a different class of bedding is provided, the tabular values can be adjusted accordingly. For more information on bedding classifications, see ASTM C 12.

The following procedure can be used to determine the strength of pipe required for a particular installation:

(1) Knowing the unit weight of soil, depth of trench, and width of trench at top of pipe, use figure 5-27 or 5-28 to determine the load per linear meter (foot) on the pipe.

(2) Knowing the diameter and type of pipe, use table 5-9 to determine the quality of pipe required to support the load.

Example: Assume the preliminary design indicates a 250-millimeter (10-inch) diameter concrete pipe is required and the depth of backfill over the pipe will be 2.6 meters (8.6 feet). For a 250-millimeter (10-inch) pipe with a 100-millimeter (4-inch) gravel envelope, a 610-millimeter (24-inch) wide ditch should be satisfactory; however, this ground is not expected to be stable, so a ditch width of 0.8 meter (2.5 feet) at top of the pipe is estimated. The backfill material will be saturated topsoil weighing 1,760 kilograms per cubic meter (110 pounds per cubic foot).

From figure 5-27, for a 2.6-meter (8.5-foot) cover, the load is:

(1990) (1.1) = 2,189 kilograms per linear meter (1,472 pounds per linear foot)

SATURATED TOPSOIL
WEIGHING 1600 KILOGRAMS PER CUBIC METER *

Depth of Backfill Over Top of Pipe (meters)	Trench Width at Top of Pipe 45 cm	52.5 cm	60 cm	62.5 cm	75 cm	82.5 cm	90 cm	105 cm	120 cm
1.5	7600	9440	11360	13280	15120	16960	18720	22720	26400
1.8	8480	10560	12720	14880	17200	19360	21760	26240	30880
2.1	9120	11520	13920	16480	19040	21680	24160	29600	34880
2.4	9680	12320	15040	17760	20720	23600	26400	32560	38720
2.7	10160	12960	15920	19040	21760	25280	28640	35280	42000
3.0	10480	13520	16720	20080	23520	26960	30560	37600	45280
3.4	10800	14000	17440	20880	24720	28400	32320	40000	48160
3.7	11040	14400	18000	21680	25760	29760	33920	42320	50960
4.0	11280	14720	18560	22400	26640	30880	35280	44320	53440
4.3	11440	14960	18880	22960	27360	21840	36560	46080	55840
4.6	11520	15200	19280	23520	28160	32800	37600	47680	57840
WET CLAY									
1.5	8480	10320	12240	14080	16240	18000	19760	23920	27680
1.8	9520	11760	14000	16240	18560	22240	23360	27600	32480
2.1	10480	12960	15600	18160	20800	23440	26080	31760	36560
2.4	11280	14080	16960	19420	22960	25760	28460	34480	41440
2.7	11920	15040	18240	21440	24800	28080	31520	38240	44800
3.0	12560	15920	19360	22880	26560	30240	33680	41360	48960
3.4	13040	16880	20400	24160	28080	32080	35300	44320	52240
3.7	13440	17280	21280	25440	29600	33760	38160	47200	56080
4.0	13840	17760	22000	26320	30880	35440	40240	49760	59200
4.3	14160	18320	22720	27280	32180	36880	41920	52000	62240
4.6	14480	18720	23360	28080	33200	38320	43520	54320	65280

* For backfill weighing 1500 kilograms per cubic meter, multiply load shownby 0.94, for backfill weighing 1700 kilograms per cubic meter, multiply load shownby 1.06 etc.

SATURATED TOPSOIL
WEIGHING 100 POUNDS PER CUBIC FOOT *

Depth of Backfill Over Top of Pipe (feet)	Trench Width at Top of Pipe 18 in	21 in	24 in	27i in	30 in	33 in	36 in	42 in	48 in
5	475	590	710	830	945	1060	1170	1420	1650
6	530	660	795	930	1075	1210	1360	1640	1930
7	570	720	870	1030	1190	1355	1510	1850	2180
8	605	770	940	1110	1295	1475	1650	2035	2420
9	635	810	995	1190	1380	1580	1790	2205	2625
10	655	845	1045	1255	1470	1685	1910	2350	2830
12	675	875	1090	1305	1545	1775	2020	2500	3010
12	690	900	1125	1355	1610	1860	2120	2645	3185
13	705	920	1160	1400	1665	1930	2205	2770	3340
14	715	935	1180	1435	1710	1990	2285	2880	3490
15	720	950	1205	1470	1760	2050	2350	2980	3615
WET CLAY									
5	530	645	765	880	1015	1125	1235	1495	1730
6	595	735	875	1015	1160	1290	1460	1725	2030
7	655	810	975	1135	1300	1465	1630	1985	2285
8	705	880	1060	1245	1435	1610	1790	2155	2590
9	745	940	1140	1340	1550	1755	1970	2390	2800
10	785	995	1210	1430	1660	1890	2105	2585	3060
11	815	1055	1275	1510	1755	2005	2260	2770	3265
12	840	1080	1330	1590	1850	2110	2385	2950	3505
13	865	1110	1375	1645	1930	2215	2515	3110	3700
14	885	1145	1420	1705	2010	2305	2620	3250	3890
15	905	1170	1460	1755	2075	2395	2720	3395	4080

* For backfill weighing 90 pounds per cubic foot, multiply load shownby 0.9, for backfill weighing 110 pounds per cubic foot, multiply load shownby 1.1 etc.

Based on the Marston formula for loads in treches: $W = CwB^2$

where:
W = Load on pipe in kilograms per linear meter (pounds per linear foot),
C = Coefficient of load on pipe,
w = Weight of fill in kilograms per cubic meter (pounds per cubic foot),
B = Width of ditch at top of pipe in meters (feet), and
H = Height of fill above top of pipe in feet.

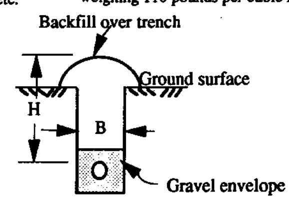


Figure 5–27.—Loads on concrete or clay pipe per linear meter (foot) for various backfill materials. Drawing 103D689.

EXAMPLE

A drain pipe line is to be placed in a trench 67.5 centimeters (27 inches) wide, B_d, at the top of the pipe with a cover, H, of 2.4 meters (8 feet) over the top of the pipe. The material is dry clay with a unit weight, w, of 1760 kilograms per cubic meter (110 pounds per cubic foot)
H/B_d = 2.4/0.675 = 3.56.
From nomograph, W_c = 1950 kilograms per linear meter (1300 pounds per linear foot).

BASED ON THE MARSTON FORMULA

$$W_c = C_d w B_d^2$$

where W_c = Vertical external load on a closed conduit due to fill material in kilograms per linear meter (pounds per cubic foot),
C_d = Load coefficient for type of backfill material,
w = Unit weight of material in kilograms per cubic meter (pounds per cubic foot),
B_d = Horizontal width of ditch at top of pipe in meters (feet) and,
H = Height of fill over top of pipe in meters (feet).

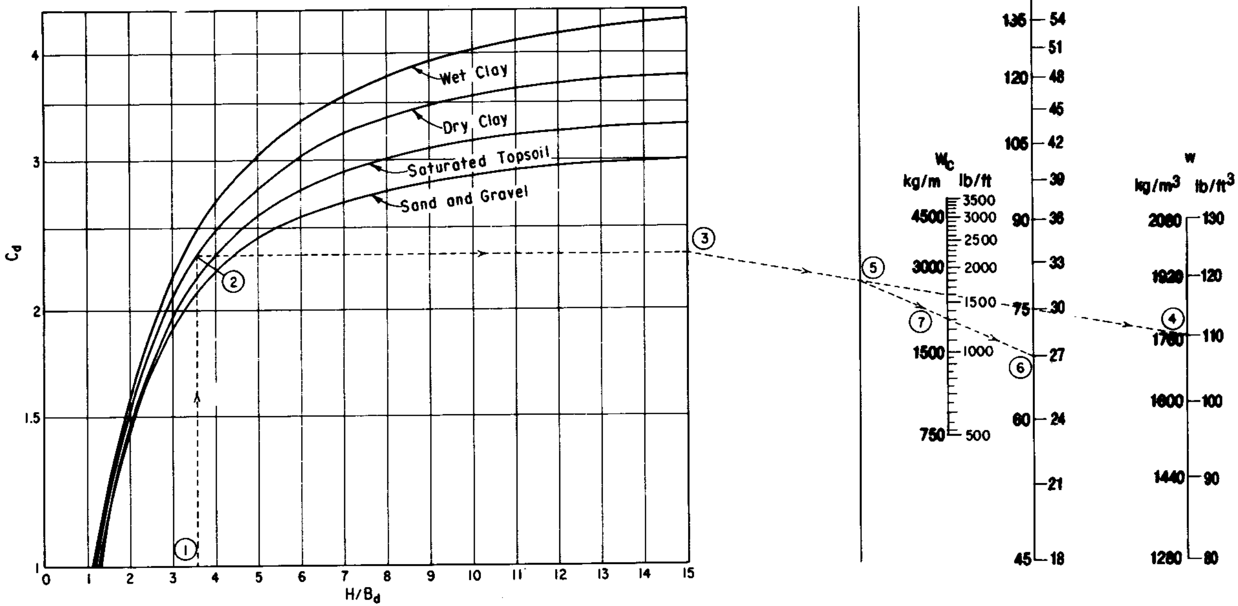


Figure 5-28.—Chart and nomograph for estimating backfill load on rigid pipe in trenches. Drawing 103-D-775.

Table 5–9a.—*Allowable crushing strength in kilograms per linear meter for rigid pipe drains in a gravel envelope (metric units).* Drawing 103–D–1670.

Pipe Diameter mm (in)	Clay Sewer Pipe[2]		Concrete Sewer Pipe[3]		Clay Drain Tile[4]			Concrete Drain Tile[5]		Concrete Drainage Pipe[6]	
	Standard Strength	Extra Strength	Class 1*	Class 2	Standard Strength	Extra Quality	Heavy Duty	Standard Quality	Extra** Quality	Standard	Heavy Duty
100 (4)	26.2	43.8	33.0	43.5	17.5	24.0	30.6	17.2	24.0	26.2	30.7
125 (5)	-----	-----	-----	-----	17.5	24.0	30.6	17.2	24.0	27.0	30.7
150 (6)	26.2	43.8	33.0	43.5	17.5	24.0	30.6	17.2	24.0	28.5	30.7
200 (8)	30.6	48.9	33.0	43.5	17.5	24.0	32.7	17.2	24.0	29.2	33.0
250 (10)	35.1	52.5	35.2	43.5	17.5	24.0	33.9	17.2	24.0	30.7	33.7
300 (12)	39.4	56.8	39.0	49.5	17.5	24.0	37.2	17.2	24.0	33.0	37.5
350 (14)	-----	-----	-----	-----	18.4	24.0	40.5	-----	24.0	35.2	40.5
375 (15)	43.8	63.4	43.5	57.0	19.0	25.0	43.3	-----	24.0	36.0	43.5
400 (16)	-----	-----	-----	-----	-----	26.2	45.9	-----	24.0	37.5	45.7
450 (18)	48.1	72.3	48.0	66.0	-----	28.5	51.1	-----	26.2	39.7	51.0
500 (20)	-----	-----	-----	-----	-----	-----	-----	-----	29.2	40.5	54.7
525 (21)	52.5	84.3	52.5	72.0	-----	31.6	58.6	-----	32.2	41.2	58.5
600 (24)	56.8	96.3	57.0	78.7	-----	34.9	65.7	-----	35.2	43.5	66.0
675 (27)	61.3	102.9	61.5	86.2	-----	39.3	72.9	-----	-----	-----	-----
750 (30)	72.3	109.5	66.0	94.5	-----	43.6	78.6	-----	43.5	-----	-----
825 (33)	78.3	120.4	69.0	96.0	-----	-----	-----	-----	-----	-----	-----
900 (36)	87.6	131.4	72.0	98.2	-----	-----	-----	-----	52.5	-----	-----

* Also Perforated Concrete Pipe[7]
** Also Special Quality

[1] The values listed in this table are 1.5 times the values given in the respective ASTM Specifications listed below which are minimum 3 edge bearing strengths.

Current ASTM Specification No.

[2] C700-91
[3] C14M-90
[4] C4-62 (Reapproved 1986)
[5] C412M-90
[6] C118M-90
[7] C444M-90

NOTE: When the crushing strength of the pipes listed will not meet an unusual load condition, reinforced concrete sewer or culvert pipe should be considered. See Federal Specifications No. SS-P-371, Type II, and ASTM C76-90.

Table 5–9b.—*Allowable crushing strength in pounds per linear foot for rigid pipe drains in a gravel envelope (U.S. customary units).* Drawing 103–D–1670.

Pipe diameter, inches	CLAY SEWER PIPE[1]		CONCRETE SEWER PIPE[2]		CLAY DRAIN TILE[3]			CONCRETE DRAIN TILE[4]		CONCRETE DRAINAGE PIPE[5]	
	Standard Strength	Extra Strength	Class 1*	Class 2	Standard Strength	Extra Quality	Heavy Duty	Standard Quality	Extra ** Quality	Standard	Heavy Duty
4	1,800	3,000	2,250	3,000	1,200	1,650	2,100	1,200	1,650	1,800	2,100
5	—	—	—	—	1,200	1,650	2,100	1,200	1,650	1,875	2,100
6	1,800	3,000	2,250	3,000	1,200	1,650	2,100	1,200	1,650	1,950	2,100
8	2,100	3,300	2,250	3,000	1,200	1,650	2,250	1,200	1,650	2,025	2,250
10	2,400	3,600	2,400	3,000	1,200	1,650	2,325	1,200	1,650	2,100	2,325
12	2,700	3,900	2,700	3,375	1,200	1,650	2,550	1,200	1,650	2,250	2,550
14	—	—	—	—	1,260	1,650	2,775	—	1,650	2,400	2,775
15	3,000	4,350	3,000	3,900	1,305	1,725	2,970	—	1,650	2,475	2,970
16	—	—	—	—	—	1,800	3,150	—	1,650	2,550	3,150
18	3,300	4,950	3,300	4,500	—	1,950	3,510	—	1,800	2,700	3,510
20	—	—	—	—	—	—	—	—	1,950	2,775	3,750
21	3,600	5,775	3,600	4,950	—	2,175	4,020	—	2,100	2,850	4,020
24	3,900	6,600	3,900	5,400	—	2,400	4,500	—	2,400	3,000	4,500
27	4,200	7,050	4,200	5,925	—	2,700	5,000	—	—	—	—
30	4,950	7,500	4,500	6,450	—	3,000	5,385	—	3,000	—	—
33	5,400	8,250	4,725	6,600	—	—	—	—	—	—	—
36	6,000	9,000	4,950	6,750	—	—	—	—	3,600	—	—

Current ASTM Specification No.

[1] C700-91
[2] C14-90
[3] C4-62(Reapproved 1986)
[4] C412-90
[5] C118-90
[6] C444-90

*Also Perforated Concrete Pipe[6]
**Also Special Quality

NOTES: When the crushing strength of the pipes listed will not meet an unusual load condition, reinforced concrete sewer or culvert pipe should be considered. See Federal Specifications No. SS-P-371, Type II and ASTM C76-74.

The three-edge bearing strength values have been multiplied by a load factor of 1.5 assuming Class C bedding.

Using table 5–9, the allowable crushing strength of all pipes listed, except standard clay and standard concrete draintile, will exceed the required strength.

(c) Plastic pipe.—For corrugated plastic pipe, the strength depends upon the bedding material. All plastic pipe drains should be surrounded by at least a 100-millimeter (4-inch) gravel envelope. The loading capacity should be determined by Marston's method for flexible pipe. Figure 5–29 shows load coefficients for various soils based on the ratio of the depth of fill to the trench width.

Flexible pipe deflects when loaded, which results in a transfer of the load to the bedding material. Safe loads for corrugated plastic pipe that meet Reclamation materials specifications are those loads that will cause 10 percent or less deflection as determined by:

$$\Delta = \frac{DCW_c r^3}{EI + 0.061E'r^3} \tag{16}$$

where:

Δ = Pipe deflection in millimeters (inches),
D = Deflection lag factor of 1.5,
C = Bedding constant of 0.10,
W_c = Vertical load on pipe as determined from figure 5–31,
r = Mean radius of pipe in millimeters (inches),
E = Modulus of elasticity of pipe in kilospascals (pounds per square inch),
E' = Modulus of soil reaction [4,826 kilopascals (700 pounds per square inch) for drains in gravel pack)], and
I = Moment of inertia of pipe corrugations in millimeters (inches) per linear millimeter (inch).

The product for EI is calculated using the equation:

$$EI = 0.149\frac{F'}{\Delta y} \tag{17}$$

where:

F' = Load per linear inch on a parallel plate test apparatus (sand-bearing strength is $1.5F'$)
Δy = Vertical deflection of pipe in millimeters (inches)

Figure 5–30 shows the backfill loadings on flexible and rigid pipe according to depth to top of pipe for a 450-millimeter (18-inch) wide trench. This figure shows loadings by pipe size and backfill material. The following tabulation shows the weight of backfill causing a 10-percent deflection on pipe meeting Reclamation specifications for corrugated polyethylene pipe, with a stiffness equal to 275 kilopascals (40 pounds per square inch) (sand bearing):

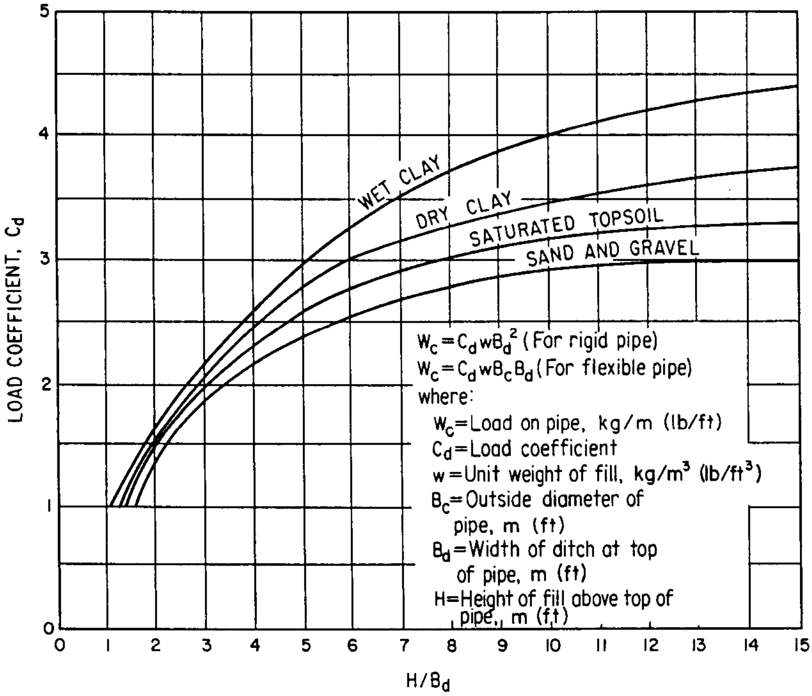


Figure 5-29.—Load coefficients for computing weight of backfill, based on Marston formula. Drawing 103-D-1671.

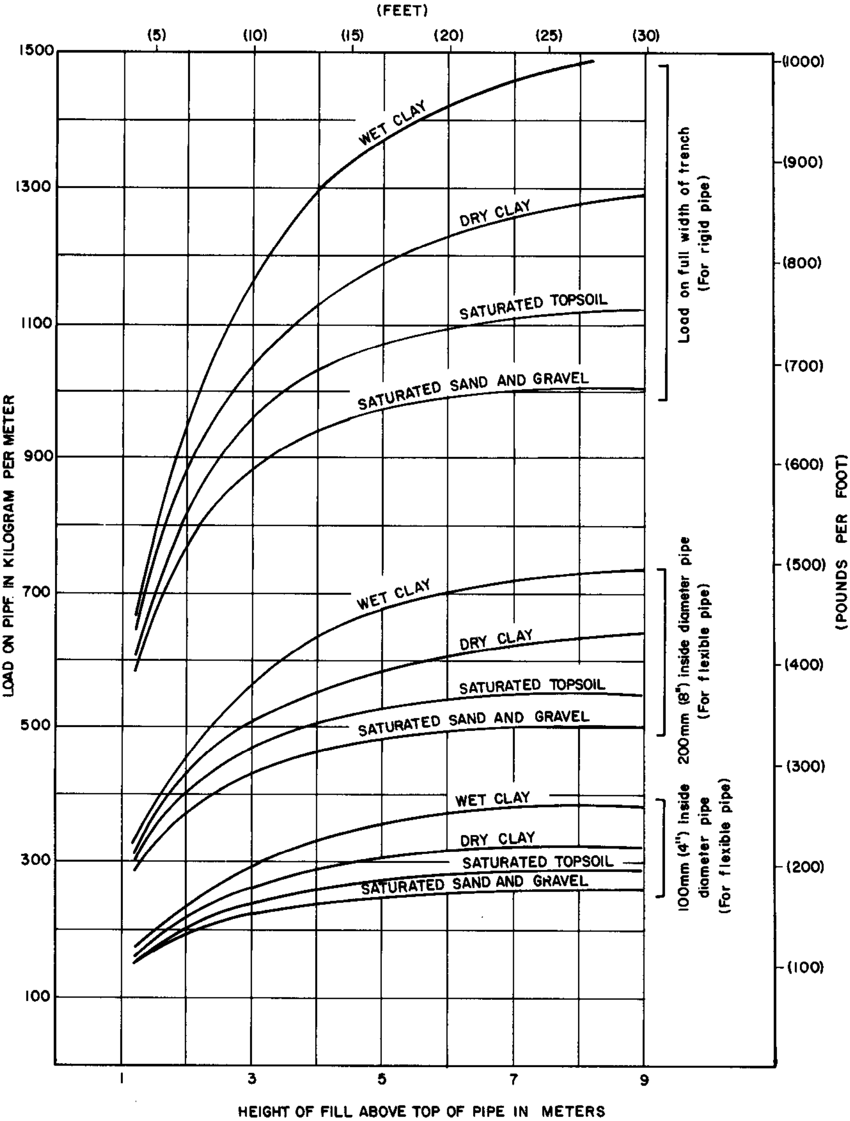


Figure 5–30.—Backfill loads on pipe in a 450-millimeter (18-inch) wide trench, based on Marston formula. Drawing 103–D–1672.

Inside diameter of pipe		*Deflection Δ*		*Vertical load*	
millimeters	*inches*	*millimeters*	*inches*	W_c, *lb/in*	W_c, *lb/ft*
100	4	10.2	0.4	125	1,500
125	5	12.7	0.5	156	1,872
150	6	15.2	0.6	188	2,256
200	8	20.3	0.8	250	3,000

The above loadings assume that the pipe is laid without any stretching of the corrugations. Fifteen percent stretch has been observed to cause collapse of the pipe when the stiffness was more than double that specified by Reclamation specifications. Reclamation specifications limit stretching of pipe upon installation to less than 5 percent.

5–47. Size of Pipe.—Using the formulas for ground-water accretion given in sections 5–12 and 5–13, the pipe drain is designed to run full. Pipe with less than a 100-millimeter (4-inch) inside diameter is not recommended. The 100-millimeter (4-inch) size should be used only in the upper reaches of a drain that will not have future requirements for extensions or branches.

Pipe sizes are determined from calculations involving the required discharge and the hydraulic gradient of the pipe drain. Using the required discharge and knowing the gradient of the line, the pipe size can be determined from the curves shown on figure 5–31. These curves are based on Manning's formula, equation (14) in section 5–18, using $n = 0.015$. This value has been found satisfactory for drains constructed with concrete, clay, and corrugated plastic pipe up to about 300-millimeter (12-inch) diameter. Manning's n values should be increased for larger diameter corrugated plastic pipe. An n value of 0.018 is recommended for 300- and 375-millimeter (12- and 15-inch) pipe and an n value of 0.020 for 450- and 600-millimeter (18- and 24-inch) pipe. Table 5–10 shows a sample pipe-sizing computation. Figure 5–32 shows a plan and profile of a typical closed drain.

Table 5–10.—*Sample pipe-sizing computation.*

Project Upper John Day – Drain System Member 26-13-34 D Date May 11, 1992 – Computed by GDS						
Col. 1 Sta. to sta.	Col. 2 Length	Col. 3 q	Col. 4 Q	Col. 5 Q_T	Col. 6 Slope	Col. 7 Pipe size
19+00–14.90	410	.000379	.155	.155	.001	6
14+90–10+100	490	.000095	.047	.202	.002	6
(Right subdrain entering the main at Sta. 10+00)						
Dr. 26-13-30-D-1.ORT						
7+50–0+00	750	.000189	.142	.142	.001	6
(Left subdrain entering the main at Sta. 10+00)						
Dr. 26-13-34-D-1.067 enters at Sta. 10+00						
5+25–0+00	525	.000189	.099	.099	.001	6
10+00–5+00	500	.000095	.048	.491	.003	8
5+00–0+00	500	.000095	.048	.539	.002	10

1 Stations that define the section of pipe to be sized from upstream down.
2 Length of pipe defined.
3 Accretion rate usually in ft^3/s per foot.
4 Accretions to the defined length of pipe col. 2 x col. 3.
5 Total accretions to downstream end of defined length of pipe, including all upstream contributions.
6 Slope of the defined length of pipe.
7 Pipe size in inches based on figure 5–31b.

5–48. Capacity of Pipe Drains.—The capacity of pipe drains usually has to be sufficient to carry ground-water accretion only. Collector and outlet pipe drains must, of course, also carry the flows delivered to them by other drains. In the rare case where open drains discharge into pipe drains, the pipe drains should be designed to run only half full, including the flow from the open drains. In studies involving capacities, areas, and velocities, the information shown on figure 5–33 is useful for designing pipe drains flowing partially full.

5–49. Design of a Drainage Sump and Pumping Plant.—Many areas requiring drainage do not have a gravity outlet; these areas can be drained using pumping plants at reasonable cost. Pumping plants are also used to provide an adequate grade in pipe systems. Drains can be excavated 2.7 to 3 meters (9 to 10 feet) deep at an economical cost, but the cost increases rapidly for greater depths. By excavating drains to about 3 meters (10 feet) and then pumping the water up 1.2 or 1.5 meters (4 or 5 feet), adequate grades can be obtained in large, flat areas. The main steps in the design of a drainage sump and pumping plant are: *(a)* determining maximum inflow into the sump; *(b)* determining amount of storage required; *(c)* determining pumping rate; *(d)* determining start, stop, and discharge levels; *(e)* determining type of storage required; and *(f)* selecting the pump and power unit.

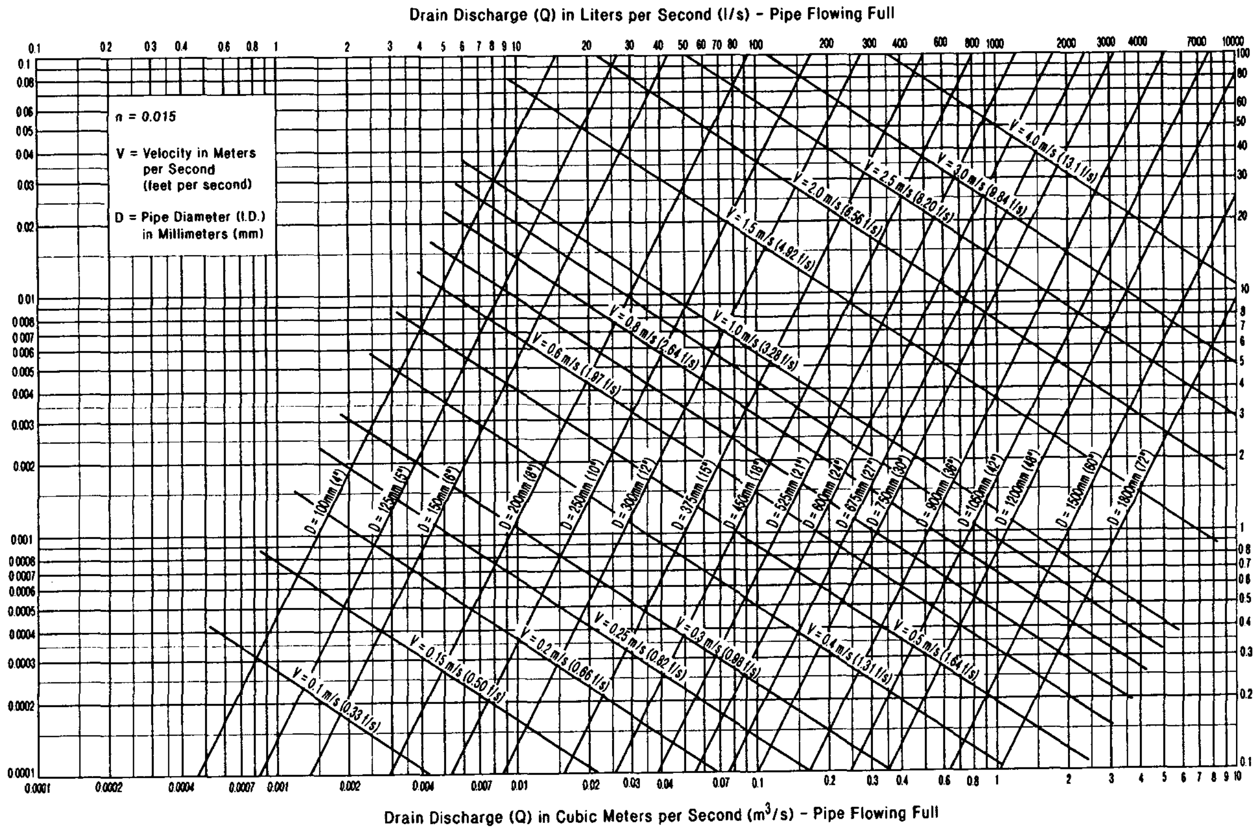


Figure 5–31a.—Flow in drains of various diameter based on slope (metric units). Drawing 103–D–666.

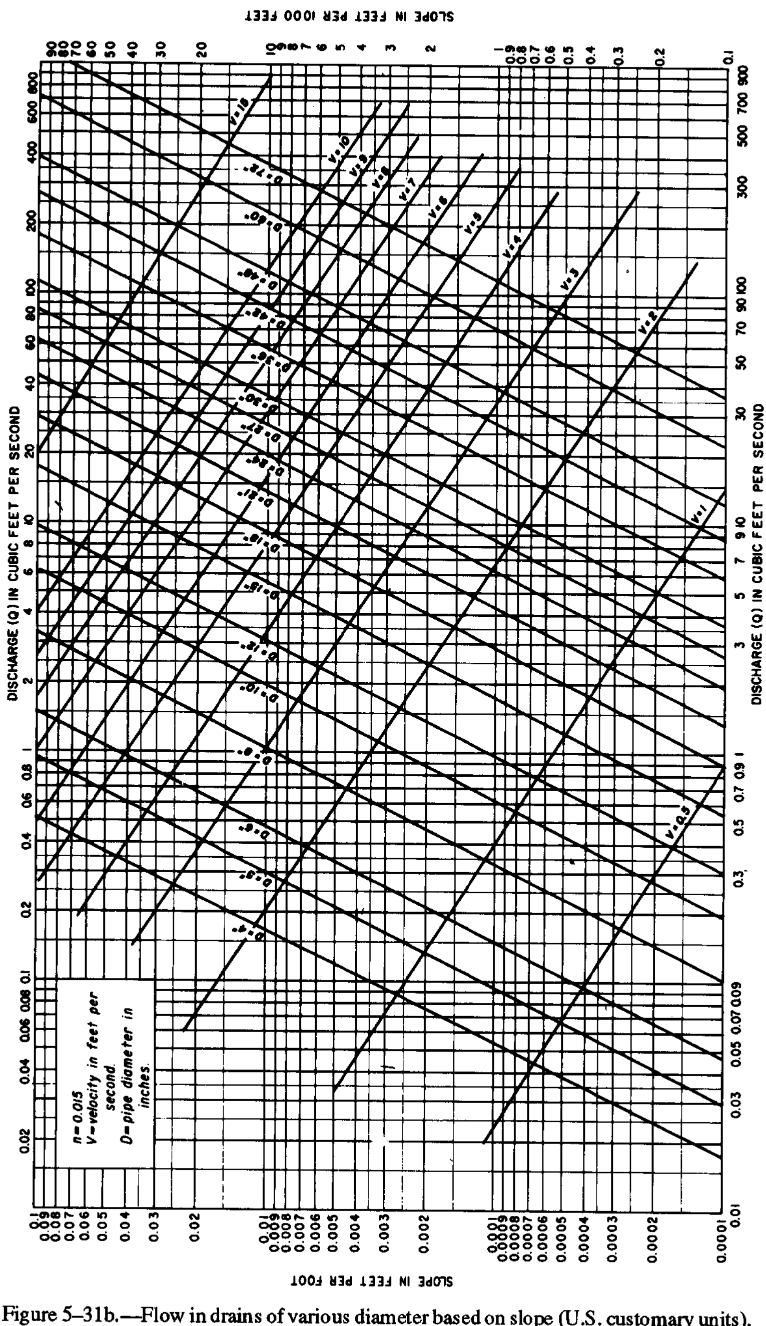
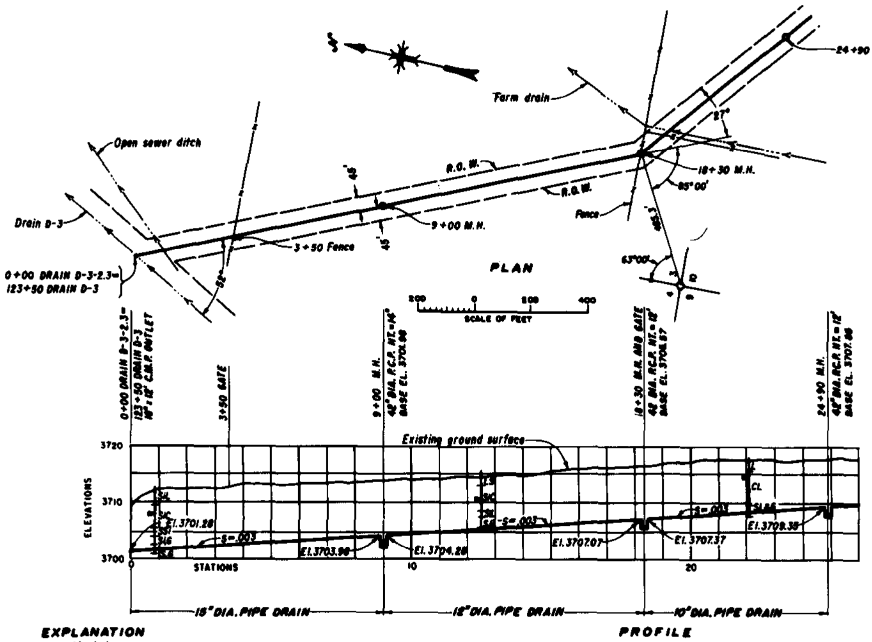


Figure 5–31b.—Flow in drains of various diameter based on slope (U.S. customary units). Drawing 103–D–666.



EXPLANATION

M.H. = Manhole
H.T. = Height of manhole

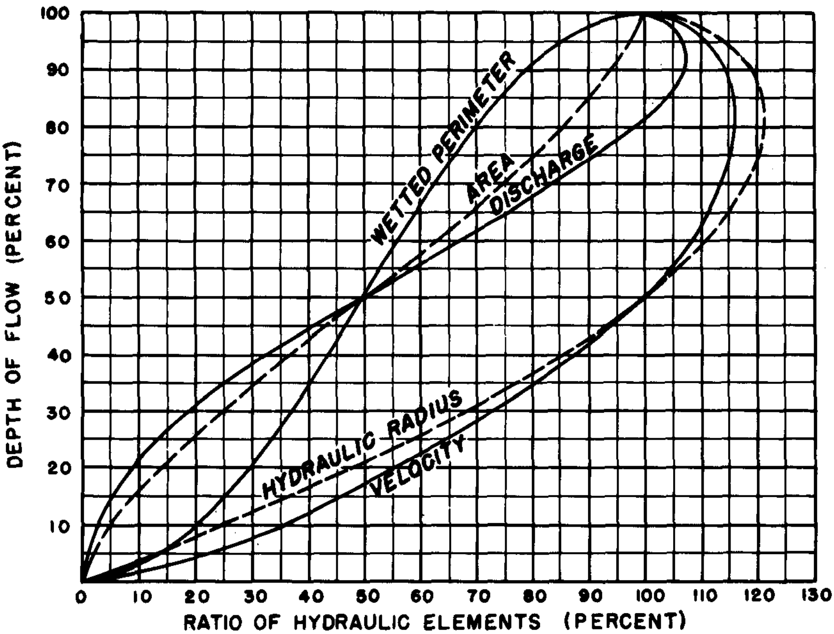
LOG SYMBOLS

Si — Silt
C — Clay
L — Loam
S — Sand
G — Gravel
▶ Water table elevation on May 21, 19___

HYDRAULIC PROPERTIES

REACH	DIA.	S	N	V	Q	
					PIPE FULL	EST. FLOW
0+00 to 9+00	15"	.003	.015	2.48	3.04	1.80
9+00 to 18+30	12"	.003	.015	2.15	1.69	1.30
18+30 to 24+90	10"	.003	.015	1.91	1.04	0.80

Figure 5–32.—Plan and profile of a typical closed drain. Drawing 103–D–667.



PIPE DIAMETER MILLIMETERS	(INCHES)	PIPE DIAMETER METERS	(FEET)	PIPE AREA METERS^2	(FEET^2)	HYDRAULIC RADIUS (FLOWING FULL OR HALF FULL) METERS	(FEET)
100	(4)	0.100	(0.333)	0.0078	(0.087)	0.0253	(0.083)
150	(6)	0.152	(0.500)	0.0176	(0.196)	0.0381	(0.125)
200	(8)	0.200	(0.667)	0.0314	(0.349)	0.0509	(0.167)
250	(10)	0.250	(0.833)	0.0491	(0.545)	0.0634	(0.208)
300	(12)	0.300	(1.000)	0.0707	(0.785)	0.0762	(0.250)
375	(15)	0.375	(1.250)	0.1104	(1.227)	0.0950	(0.312)
450	(18)	0.450	(1.500)	0.1590	(1.767)	0.1143	(0.375)
525	(21)	0.525	(1.750)	0.2164	(2.405)	0.1332	(0.437)
600	(24)	0.600	(2.000)	0.2827	(3.142)	0.1524	(0.500)
675	(27)	0.675	(2.250)	0.3578	(3.976)	0.1695	(0.556)
750	(30)	0.750	(2.500)	0.4418	(4.909)	0.1905	(0.625)
825	(33)	0.825	(2.750)	0.5346	(5.940)	0.2094	(0.687)
900	(36)	0.900	(3.000)	0.6362	(7.068	0.2286	(0.750)

Figure 5-33.—Hydraulic properties of drainpipe. Drawing 103-D-687.

The maximum inflow into the sump must be determined for the total drainage requirement of the area to be drained by the sump. For example, if the pumping plant must relift water from a drainage system with a total area of 259 hectares (640 acres), the following data must be known and computations made:

Known:

Drainage area = 259 hectares (640 acres)
Drain spacing, L = 183 meters (600 feet)
Hydraulic conductivity, K = 0.37 meter (1.2 feet) per day
Hooghoudt's equivalent depth, d' = 5.5 meters (18 feet)
Maximum distance between drain and root zone, y_o = 1.5 meters (5 feet)

$$\text{Average flow depth, } D' = d' + \frac{y_o}{2} = 6.25 \text{ meters (20.5 feet)}$$

Find: Maximum flow q into the sump in liters per second (gallons per minute).

Using equation (6) from section 5–13:

$$q = C\frac{2\pi K y_o D}{86{,}400L}\left(\frac{A}{L}\right)$$

$$q = 0.6\left[\frac{2\pi(0.37)(1.52)(6.25)}{(86{,}400)(183)}\right]\left[\frac{(254)(10{,}000)}{183}\right] = 0.01186 \text{ m}^3/\text{s } (0.415 \text{ ft}^3/\text{s})$$

$$q = 11.86 \text{ liters per second (188 gallons per minute)}$$

The cycling operation of the pump and motor to determine the amount of storage required is the next consideration in the sump design. The length of a complete cycle in minutes is equal to the standing time plus the running time. The pump and motor are most efficient if operated continuously, but 8- to 12-minute cycles are almost as efficient. For general design, a 12-minute cycle or five cycles per hour is considered satisfactory.

Using five cycles per hour means there will be five starts per hour with even on-and-off times of 6 minutes each for maximum inflow. During low flows, the off-time will be much longer than the running time, but as long as the running time does not drop below about 3 minutes, the plant efficiency is satisfactory and motor breakdowns are kept to a minimum.

For the motor to have equal on-and-off times, the storage must be equal to the amount that would run into the sump in one-half the cycling time, which would be 6 minutes when 12-minute cycles are used. Therefore, the sump must have a storage capacity, V, of:

$$V = 6 \times 60 \times 9 = 6 \times 60 \times 11.86 = 4{,}270 \text{ liters (1,128 gallons)} = 4.27 \text{ cubic meters (151 cubic feet)}$$

The pumping rate can now be determined from the equation:

$$P = \frac{S + It}{t}$$

where:

P = Pumping rate at maximum inflow in liters per second (gallons per minute)
t = Running time of pump and motor in minutes for maximum inflow based on the selected complete cycling time with equal on-and-off times
S = Sump storage volume in liters (gallons)
I = Inflow rate in liters per second (gallons per minute)

Then, $P = \dfrac{4{,}270 + 11.86 \times 60)}{6 \times 60} = 23.7$ L/s (376 gal/min)

The minimum and maximum water levels in the sump must be determined for individual outlet conditions. In general, the maximum water level for starting the pump should be at about the top of the pipe drain discharging into the sump. Never should it exceed one-half the pipe diameter over the top of the drain. The minimum elevation should be from 0.6 to 1.2 meters (2 to 4 feet) above the base of the sump. Pump lifts are the difference in elevation between water level in the sump and the discharge elevation, see figure 5–34.

The volume of required storage plus the criteria that the minimum water level should be 0.6 to 1.2 meters (2 to 4 feet) above the bottom of the sump determines the size of the sump. Generally, the sump will be cylindrical and placed vertically, but can also be placed horizontally. Assuming the pipe drain enters 3 meters (10 feet) below the ground surface and that the sump will be both cylindrical and vertical, the distance between the start and stop elevations, D, should be small to keep the depth of the sump reasonable. For example, assume $D = 0.6$ meter (2 feet). Knowing the volume of required storage, V, to be 4.27 cubic meters (151 cubic feet), the diameter of the sump, d, is computed from:

$$d^2 = \frac{V}{0.7854D}$$

$$d^2 = \frac{4.27}{(0.7854)(0.6)} = 97.5 \text{ m}^2 \text{ (9.06 ft}^2\text{), and}$$

$d = 3.01$ meters (9.9 feet) [use a 3-meter (10-foot) diameter sump]

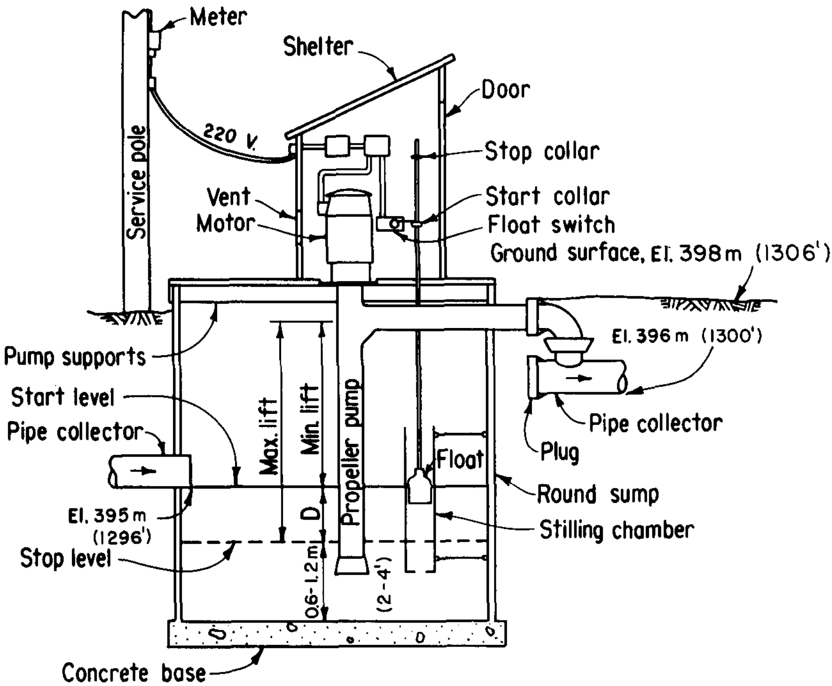


Figure 5–34.—Typical arrangement of an automatic drainage relift pumping plant. Drawing 103–D–1673.

Figure 5–34 shows the required design elevations and arrangement of equipment for an automatic drainage pumping plant.

For planning estimates, the pump and power units can be selected from reliable pump and motor manufacturers, using their literature and charts to determine the most efficient pump and motor. For construction specifications on small units, see the Bureau of Reclamation's *Ground Water Manual* (1977).

Multiple pumps can be used for large areas. When pumps of equal size are used, they can be operated to cycle only one pump at a time. The storage requirement is computed using the capacity of only one pump. If the pumps are not of equal capacity, the storage should be designed for the capacity of the largest pump.

E. Special Drain Types

5–50. Introduction.—Certain conditions require special types of drainage methods. These methods include relief wells, inverted wells, and pumped wells. Detailed instructions for investigating, planning, and installing wells are given in Reclamation's *Ground Water Manual* (1977).

5–51. Relief Wells.—In some areas, confining layers of a deep artesian aquifer may be sufficiently permeable to allow water to move upward and cause a high water table. Removal of this water by a normal drainage system usually requires very closely spaced drains and is generally uneconomical. In some cases, relief wells drilled into the artesian aquifer and outletting in the bottom of a deep drain will relieve the artesian pressure sufficiently to lower the ground-water table to safe levels. Ordinarily, a single well of this type does not relieve enough head on the system to be effective over a large area. The investigation for relief well systems must be thorough to ensure success. Artesian pressures must be located and identified, and pressure reductions must be estimated and verified before undertaking a relief well program.

5–52. Pumped Wells.—Under certain conditions, pumped wells in an unconfined water table offer an efficient solution for a drainage problem. In some cases, the pumped wells may provide all the drainage necessary, while in others the wells may furnish only supplemental drainage for critical areas. Pumped wells may be located to discharge water directly into an irrigation system for reuse, or they may discharge into a drainage channel. Drainage by pumping is feasible only in localities having extensive underlying aquifers of ample thickness. The wells must have large areas of influence with nominal drawdown to be effective and economical. Pumped wells in artesian areas may prove especially effective. Artesian pressures can be lowered over a widespread area by pumping.

Power costs are a critical factor in determining the feasibility of drainage by pumping, and the possibility of obtaining more favorable rates by using power only during low demand periods should be investigated.

5–53. Inverted or Recharge Wells and Infiltration Galleries.—In an inverted or recharge well, water flows into the earth instead of flowing from it. When used for drainage, the inverted well is the outlet for the drainage system.

The inverted wells must penetrate a permeable zone capable of accepting the quantities of drainage water either by storage or by carrying it away by natural flow. Extensively fractured basalts or cavernous limestones are typical examples of such permeable zones. Coarse sands and gravels may be suitable if they have good hydraulic properties.

Typical well construction is used for inverted wells, but sediment must be removed from the drainage water before it enters the inverted well. Sediments will clog the aquifer in the vicinity of the well and will gradually reduce the effectiveness of the well. Dissolved gas caused by turbulent flow or chemical reactions between the aquifer and the recharge water can also clog the aquifer and reduce well efficiency. Studies to determine methods of prolonging the life of recharge wells are being made with increasing frequency because the subject of artificial recharge in restoring water levels in overpumped basins or in stopping the encroachment of seawater is becoming more important. Care must be taken to ensure that existing aquifers are not polluted by the inverted well systems.

Infiltration galleries can be used for the same purposes as recharge wells. They are most often used to restore ground-water levels for pumping at a later time.

They are constructed similarly to agricultural drains using perforated plastic pipe installed in a graded gravel envelope. Depth and spacing of the system depend on the physical characteristics of the site.

As with recharge wells, sediment must be removed from the water to prevent clogging of the galleries. All local, State, and Federal water-quality criteria must be met to prevent pollution of the ground-water system.

F. Investigation and Layout for Drains

5–54. Introduction.—An analysis of a "sample farm" will be used to illustrate the methods and procedures used in drainage investigations. The sample farm developed waterlogged conditions after about 3 years of irrigation. Figure 5–35 shows the layout, surface topography, and irrigation facilities of the farm. Although this illustration uses a "sample farm," a more typical Reclamation drainage system would include several farm units or ownerships.

5–55. Investigation Procedure.—The first step in investigation is to lay out a grid system covering the waterlogged area. A 120- to 180-meter (400- to 600-foot) grid is generally sufficient to provide a detailed ground-water contour map and adequate hydraulic conductivity data. The grid should be designed to include any suspected source of seepage from canals and adjacent areas.

On the sample farm, ground surface elevations were determined at each 120-meter (400-foot) grid point, and elevations were taken at the bottom and at the indicated water surfaces of the wasteway, irrigation canals, and farm laterals. Holes were augered at each of the grid points to a depth of at least 3 meters (10 feet) and to a depth of 6 meters (20 feet) at the 240-meter (800-foot) grid points. The depth of the water table was measured at each grid point. Figure 5–36 shows the water table conditions at the time of the investigation. Each hole was logged for texture, structure, and any other pertinent information such as color changes, mottling, plasticity, stickiness, visible salt crystals, and unstable conditions.

Based upon water table location and soil profile data, three general types of conditions were recognized, each requiring a different combination of hydraulic conductivity test methods. Figure 5–37 shows the location of the test sites and the combination of hydraulic conductivity methods required at each site. Typical soil profiles of subareas A, B, and C are shown on figure 5–38.

The water table in subarea A was about 2.1 meters (7 feet) from the surface at the time of the investigation, but the farmer reported that it rose to within 0.3 to 0.6 meter (1 or 2 feet) of the surface during the period of heaviest irrigation. These high water table conditions indicated the need for horizontal hydraulic conductivities under saturated conditions in the 0.6- to 2.1-meter (2- to 7-foot), sandy-clay loam zone. Because this zone was dry, a shallow well pump-in test would be used. Below 2.1 meters (7 feet), in the sandy loam layer, the horizontal hydraulic conductivities under saturated conditions could be determined by the auger-hole test. For the pump-in tests, three additional 1.8-meter (6-foot) deep holes were augered at grid points D-1, C-3, and B-4. For the auger-hole test, the original 3-meter (10-foot) deep holes at these locations were used.

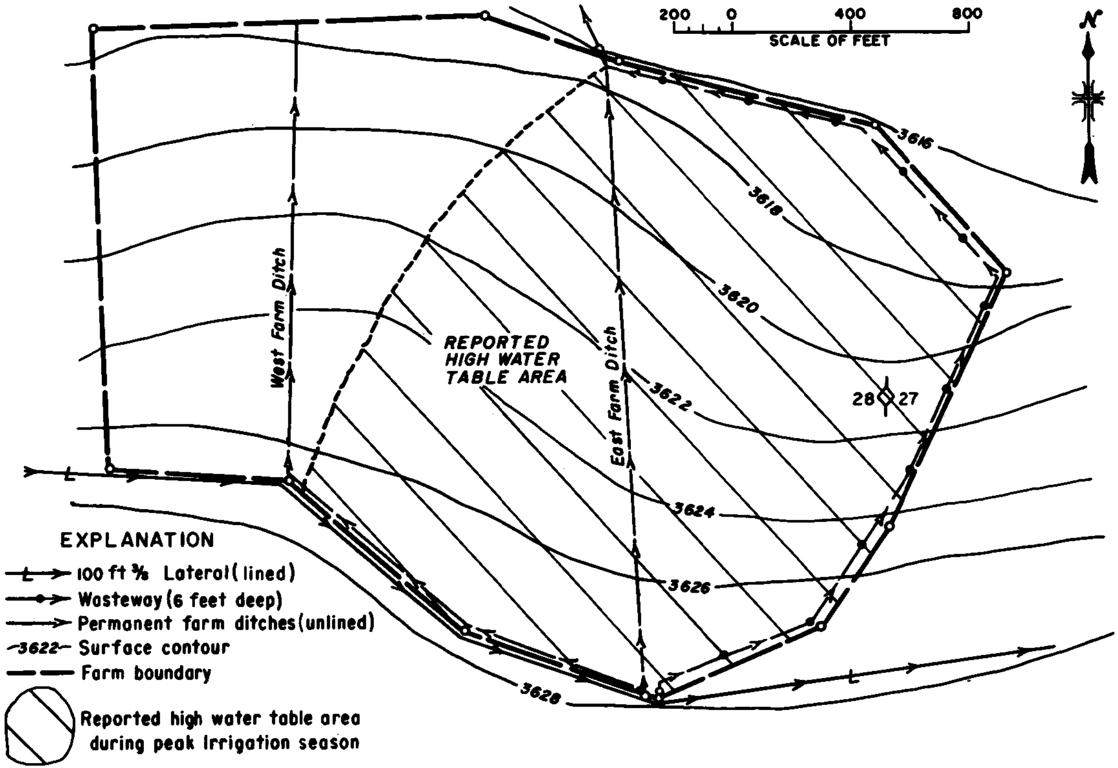


Figure 5–35.—Layout, surface topography, and irrigation facilities of the sample farm. Drawing 103–D–670.

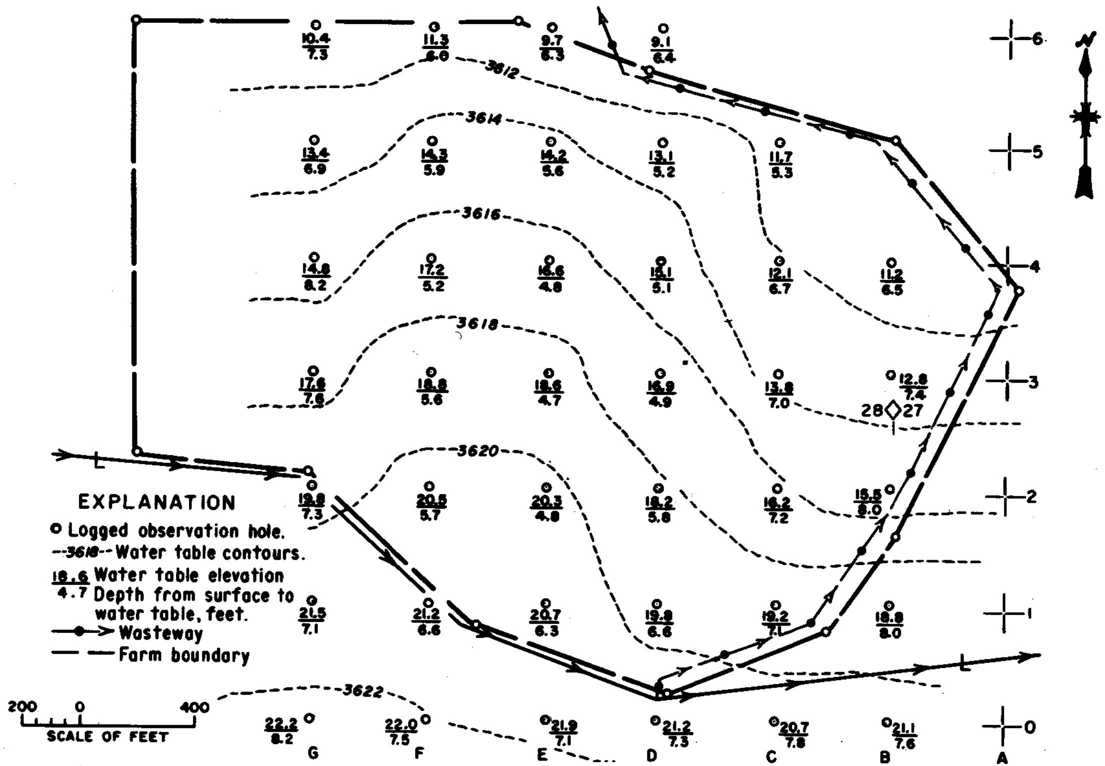


Figure 5–36.—Water table conditions of the sample farm. Drawing 103–D–671.

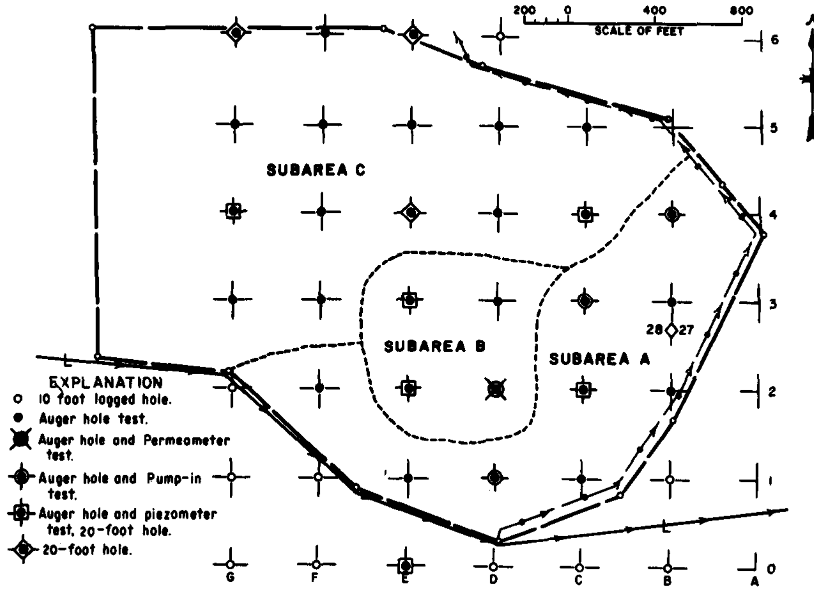


Figure 5-37.—Sample farm grid system and location of test sites. Drawing 103-D-672.

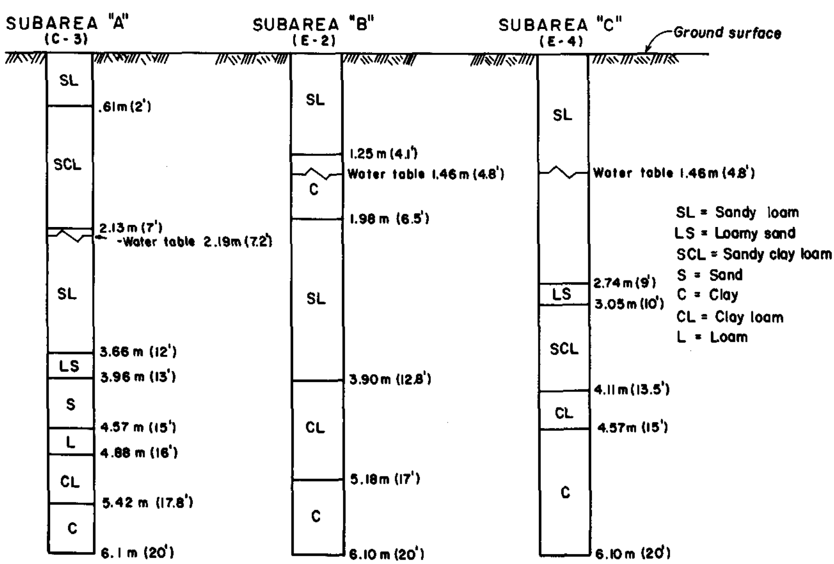


Figure 5-38.—Typical soil profiles of sample farm subareas. Drawing 103-D-673.

The water table in subarea B was at about 1.46 meters (4.8 feet); the clay layer from the 1.2- to 2.0-meter (4- to 6.5-foot) level could cause a perched water table during the irrigation season. To check this possibility, the vertical hydraulic conductivity of the clay layer was measured. This measurement required use of the ring permeameter test, and tests were run at grid points D-2 and E-3. During the tests, the water table at E-3 rose into the 150-millimeter (6-inch) test zone and the test had to be abandoned. Because the clay layer appeared homogeneous and isotropic at E-2, the piezometer test was substituted for the ring permeameter test. This test gave a value for horizontal hydraulic conductivity, and in view of the homogeneity of the clay, the vertical hydraulic conductivity could then be assumed to be about the same.

Because the 1.2- to 2.7-meter (4- to 9-foot) profile in subarea C was homogeneous and the water table was at 1.46 meters (4.8 feet), the auger-hole test was used for determining the hydraulic conductivity in this zone, and the piezometer test was used for determining the hydraulic conductivity of the clay loam and clay zones below 4.1 meters (13.5 feet).

Points on the 240-meter (800-foot) grid were used to determine the probable barrier layer. This determination required measuring the hydraulic conductivity of the various layers below the prospective drain depth. At these depths, the auger-hole test was not practical because of the depth of the layers, so the piezometer test was used and tests were run at C-2, C-4, E-0, E-2, and G-4. Figure 5–39 shows the location of all test sites and the hydraulic conductivity data.

5–56. Moisture Holding Capacity in the Root Zone.—The three subareas of the sample farm were examined for the most critical moisture-holding capacity within a 1.2-meter (4-foot) root zone. Subarea C was found to be the most critical. In this subarea, the available moisture was 29.5 millimeters (1.16 inches) in the first 0.3 meter (1.0 foot), 31.75 millimeters (1.25 inches) in the second 0.3 meter (1.0 foot), 36.83 millimeters (1.45 inches) in the third 0.3 meter (1.0 foot), and 36.83 (1.45 inches) in the fourth 0.3 meter (1.0 foot).

The total readily available moisture (TRAM) in the 1.2-meter (4-foot) root zone was calculated as outlined in section 2–6 of this manual. The critical quarter in this case is the first 0.3 meter (1.0 foot), and the TRAM in the sample profile is:

$$\text{TRAM} = (29.5 \times 0.70)/0.40 = 51.6 \text{ millimeters (2.03 inches)}$$

5–57. Annual Irrigation Schedule.—The irrigation schedule for the sample farm, as for any farm, varies from year to year because of crop rotation, size of farm, weather, and planting dates. However, for a specific climate, irrigation and cropping practices usually follow a pattern. Over the long term, the features determining irrigation schedules tend to be about the same each year. Therefore, an average irrigation schedule often is used in drain design. An irrigation schedule for the crop most generally grown and having the greatest drainage requirement is used in the drain design. On the sample farm, that crop is alfalfa.

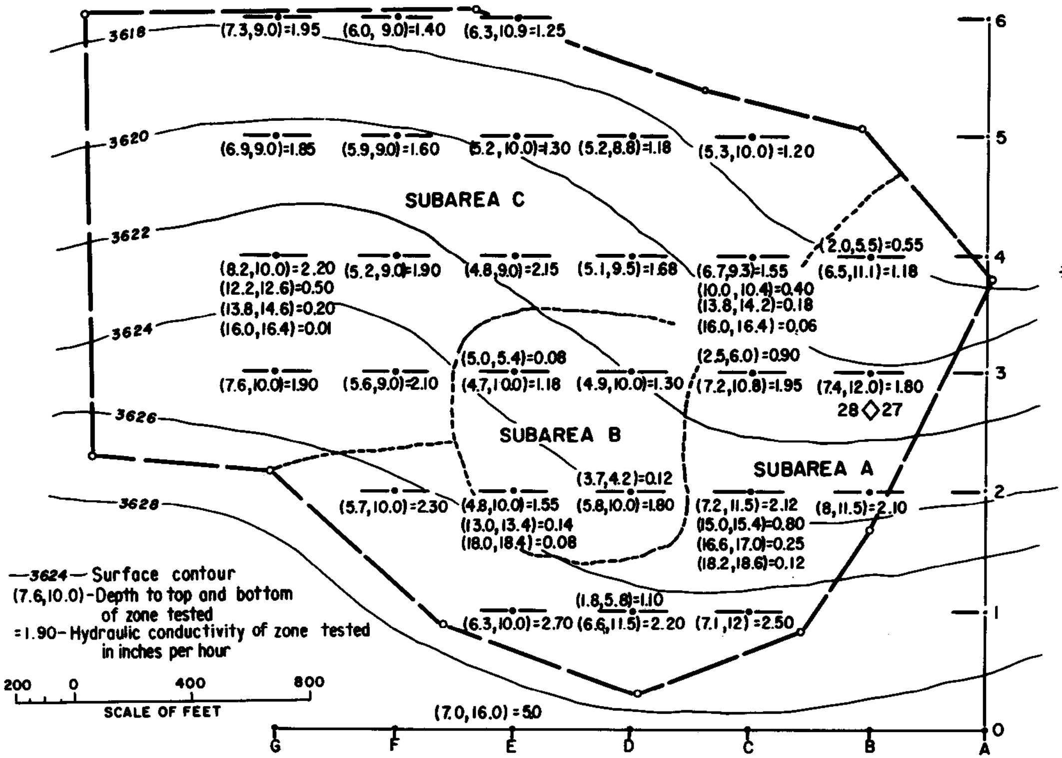


Figure 5–39.—In-place hydraulic conductivity data for sample farm. Drawing 103–D–674.

Using methods shown in section 2–6 of this manual, the consumptive use (CU) and irrigation schedule for various crops grown on the farm are shown in the following tabulations:

Calculations for average consumptive use and irrigation requirement for sample farm

Crops	Average percent grown per year	Growing season	Percent of moisture extracted per quarter of root zone: 1st	2nd	3rd	4th
Alfalfa	40	May 15 to Sept. 21				
Corn	20	May 15 to Sept. 15				
Beans	20	May 15 to Aug. 15	40	30	20	10
Small grains	20	May 15 to Aug. 15				

Consumptive use and irrigation requirement for alfalfa

Month	Growing days	CU		Daily CU	
		Millimeters	Inches	Millimeters	Inches
May	16	53.8	2.13	2.29	0.09
June	30	123.7	4.83	4.06	.16
July	31	138.7	5.46	4.32	.17
August	31	123.4	4.86	1.52	.06
September	15	45.7	1.80	1.52	.06

Consumptive use and irrigation requirement for beans and small grains

Month	Growing days	CU		Daily CU	
		Millimeters	Inches	Millimeters	Inches
May	16	54.1	2.13	2.29	0.09
June	30	122.7	4.83	4.06	.16
July	31	138.7	5.46	4.32	.17
August	15	59.9	2.36	1.02	.04

Typical irrigation schedules for the area of concern may have already been developed by commercial irrigation scheduling service companies. From an historical perspective, this type of irrigation schedule should be adequate for drain system design.

5–58. Irrigation Deliveries and Deep Percolation From Irrigation.— Records show that irrigation deliveries are made to the sample farm at the rate of 0.14 cubic meters (5 cubic feet) per second, or 504 cubic meters (4.96 acre-inches) per hour, and that 84 hours are needed to irrigate the 50.6-hectare (125-acre) farm. The depth of water delivered is:

$$\frac{84 \times 504}{50.6} = 85 \text{ millimeters (3.33 inches)}$$

The soil moisture was assumed to be at field capacity after snowmelt in the spring, May 15. The irrigation schedule for alfalfa is shown because this will be used in estimating the drain spacing.

Irrigation schedule for alfalfa

Date	Farm delivery	
	millimeters	inches
5/15	84.6	3.33
6/3	84.6	3.33
6/14	84.6	3.33
6/25	84.6	3.33
7/5	84.6	3.33
7/15	84.6	3.33
7/25	84.6	3.33
8/4	84.6	3.33
8/15	84.6	3.33
8/2	84.6	3.33
9/9	84.6	3.33
Total	930.6	36.63

Because the soil holds 51.6 millimeters (2.03 inches) of total readily available moisture at field capacity, the irrigation efficiency is:

$$\text{Farm efficiency} = \frac{51.6}{84.6} \times 100 = 61 \text{ percent}$$

Of this, about 10 percent, or 8.4 millimeters (0.33 inch), runs off as surface waste, leaving 76.2 millimeters (3.00 inches) to infiltrate the soil. This means about 24.6 millimeters (0.97 inch) will deep percolate to the ground-water table upon each irrigation. Deep percolation = 76.2 – 51.6 = 24.6 millimeters (3.00 – 2.03 = 0.97 inch) per irrigation. The total annual deep percolation for 11 irrigations, assuming that rainfall is negligible, will be about 271 millimeters (10.7 inches).

Observation well data from the site may also be useful in estimating deep percolation from an irrigation event. Changes in water table elevation before and after an irrigation event can be used to calculate deep percolation amounts. Neutron Probe data, which indicate deep percolation values, also may be available from irrigation scheduling service companies.

5-59. Other Water Sources Causing High Water Table Conditions.— Precipitation in the sample farm area is low and erratic, so it was not considered a contributing source to the ground water. The remaining sources of high ground water during the irrigation season are: *(a)* ground water moving into the area as subsurface flow from the adjacent farm to the south, and *(b)* seepage from unlined canals and laterals.

(a) Deep percolation from adjacent areas.—The ground-water contours on figure 5–37 indicate that subsurface water is moving into the sample farm from the south. An estimate of the volume of this water can be made using the Darcy principle:

$$Q = KiA \tag{19}$$

where:

Q = Flow in cubic meters (feet) per second per linear meter (foot)
K = Hydraulic conductivity in meters (feet) per second
i = Slope of the water surface in meters per meter (feet per foot)
A = Cross-sectional area in square meters (feet) of the water-bearing stratum for a 1 meter (foot) width

A hydraulic conductivity of 12.7 centimeters (5 inches) per hour [3.05 meters (10 feet per day)] was indicated by the auger-hole test at grid point E-0. A slope, i, of 0.004 meter per meter (foot per foot) and an area, A, of 2.44 square meters per linear meter (8 square feet per linear foot) of boundary were determined from information taken from the north-south profile on the E-line shown on figure 5–40. Then, $Q = 3.05 \times 0.004 \times 2.44 = 0.0298$ cubic meter per linear meter (0.32 cubic foot per linear foot) per day. As the south boundary of the sample area was about 792 meters (2,600 feet) wide, the total water moving into the farm could be $0.0298 \times 792 = 23.6$ cubic meters $(0.32 \times 2{,}600) = 23.6$ cubic meters (832 cubic feet) per day, but flows up to 31.7 cubic meters (1,120 cubic feet) per day can be expected according to records. This is equivalent to 0.00317 hectar-meter (0.31 acre-inch) per day. Assuming an average irrigation cycle of 12 days, and that this flow would occur under the entire farm area of 50.6 hectares (125 acres), the drainage requirement would be about $\frac{0.00317 \times 12}{50.6} = 0.75$ millimeter per hectare (0.03 inch per acre) per irrigation. This small amount of water can be easily removed through the spaced drain system. If the amount is on the same order of magnitude as deep percolation from irrigation, an analysis should be made to determine whether an interceptor drain should be constructed at the upper boundary of the sample farm.

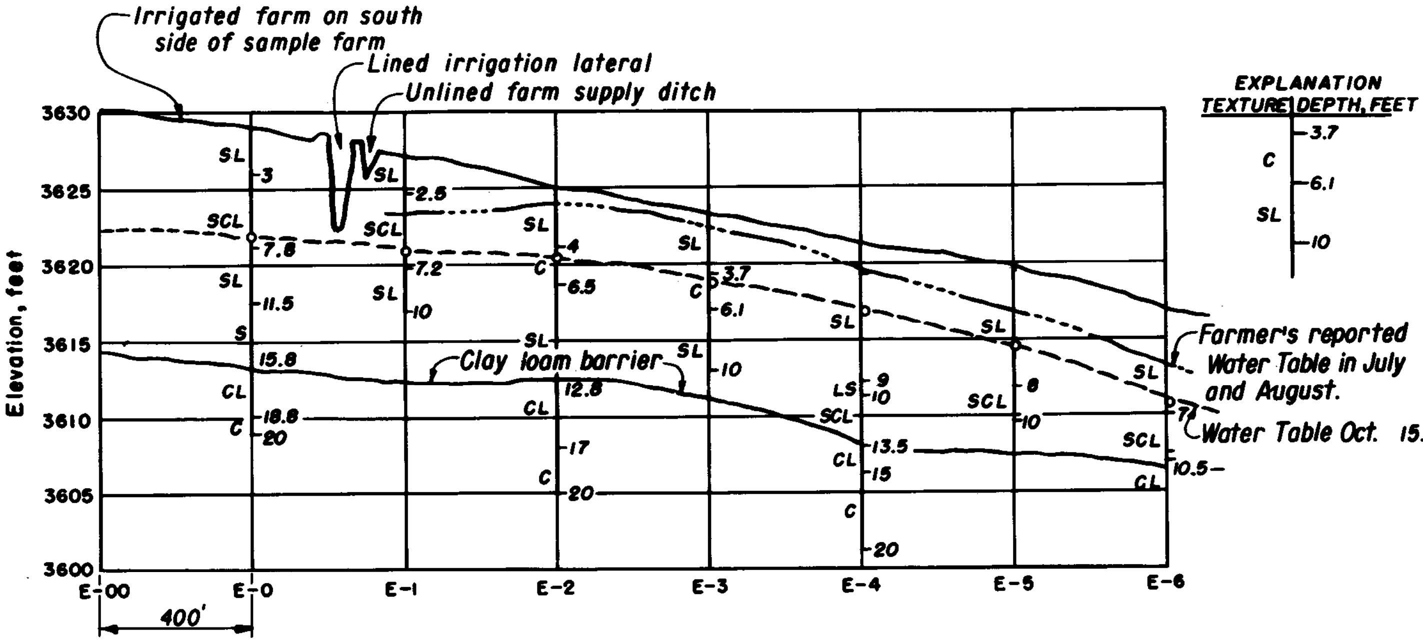


Figure 5–40.—North-south profile on E-line of sample farm. Drawing 103–D–675.

(b) Deep percolation from farm ditches.—The seepage from farm ditches can be estimated from equation (8) of section 5–15:

$$q_1 = \frac{K_1 (B + 2d)}{3.5}$$

With irrigation deliveries at the rate of 0.14 cubic meters (5 cubic feet) per second through V-shaped farm ditches constructed in sandy loam soils, the velocity should not exceed about 0.61 meter (2.0 feet) per second. Assuming that the side slopes are 1-1/2 to 1, the cross-sectional area required can be computed from the formula:

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

where:

A = Cross-sectional area in square meters (square feet)
Q = Irrigation delivery rate in cubic meters (cubic feet) per second
V = Velocity in meters (feet) per second

$$A = \frac{0.14}{0.61} = 0.23 \text{ square meters (2.5 square feet)}$$

From table 5–6, the depth of water, d, in the V-shaped farm ditch would be about 0.4 meter (1.3 feet), and the width of the water surface, B, would be 1.2 meters (3.9 feet). From the in-place tests, the hydraulic conductivity in the farm ditch section would be about 3.05 centimeters (1.2 inches) per hour or 0.73 meter (2.4 feet) per day.

Then:

$$q_1 = \frac{0.73\,(1.2 + 0.8)}{3.5} = 0.417 \text{ cubic meters per day per linear meter}$$

(4.45 cubic feet per day per linear foot) of channel

Seepage in cubic feet per second per mile:

$$\frac{0.417 \times 1{,}000}{86{,}400} = 0.00483 \text{ m}^3\text{/s per kilometer } (0.272 \text{ ft}^3\text{/s per mile})$$

The time required for irrigation of the sample farm is 88 hours, and during this time about 1.21 kilometers (0.75 mile) of farm ditch is carrying water. The seepage loss from the ditch during each irrigation over the 50.6 hectares (125 acres) is:

$$\text{Metric} - \frac{0.00483 \times 1.21 \times 88 \times 3{,}600 \times 1{,}000}{10{,}000 \times 50.6} = 3.66 \text{ millimeters (0.14 inch)}$$

$$\text{English} - \frac{0.27 \times 0.75 \times 88 \times 3{,}600 \times 12}{43{,}560 \times 125} = 0.14 \text{ inch (3.66 millimeters)}$$

The total deep percolation, including that from adjacent areas and the farm ditch, is 24.6 (0.97) + 0.76 (0.03) + 3.66 (0.14) = 29.0 hectare-millimeters per hectare (1.14 acre-inches per acre) for each irrigation.

5–60. Determination of Barrier Zone.—An accurate appraisal of barrier zones is important in the drain spacing solution, but barrier zone identification is not always easy or clear cut. The definition given in section 4–6 defines a barrier zone as a layer which has a hydraulic conductivity value one-fifth or less than that of the weighted average hydraulic conductivity of the layers above it. Table 5–11 shows the barrier layer computations for six subareas of the sample farm as shown on figure 5–41.

5–61. Depth of Drains.—Figure 5–41 shows areas with similar drainage conditions and the in-place hydraulic conductivity data for each area. Study of these data indicates that drains about 2.75 meters (9 feet) deep would be in the most permeable material. Also, the benefits for drain depths over 2.75 meters (9 feet) deep start decreasing when compared to construction costs. See section 5–33 for methods of analyzing economic drain depths.

5–62. Drain Spacing Determinations and Drain Locations.—Drain spacing is determined by the methods described in part A of this chapter. The following tabulation shows calculated drain spacings rounded to the nearest 3 meters (10 feet) for each of the subareas:

Drain spacings on sample farm

	Drain spacing	
Subarea	*meters*	*feet*
A–1	73	240
A–2	107	350
B	52	170
C–1	76	250
C–2	91	300
C–3	107	350

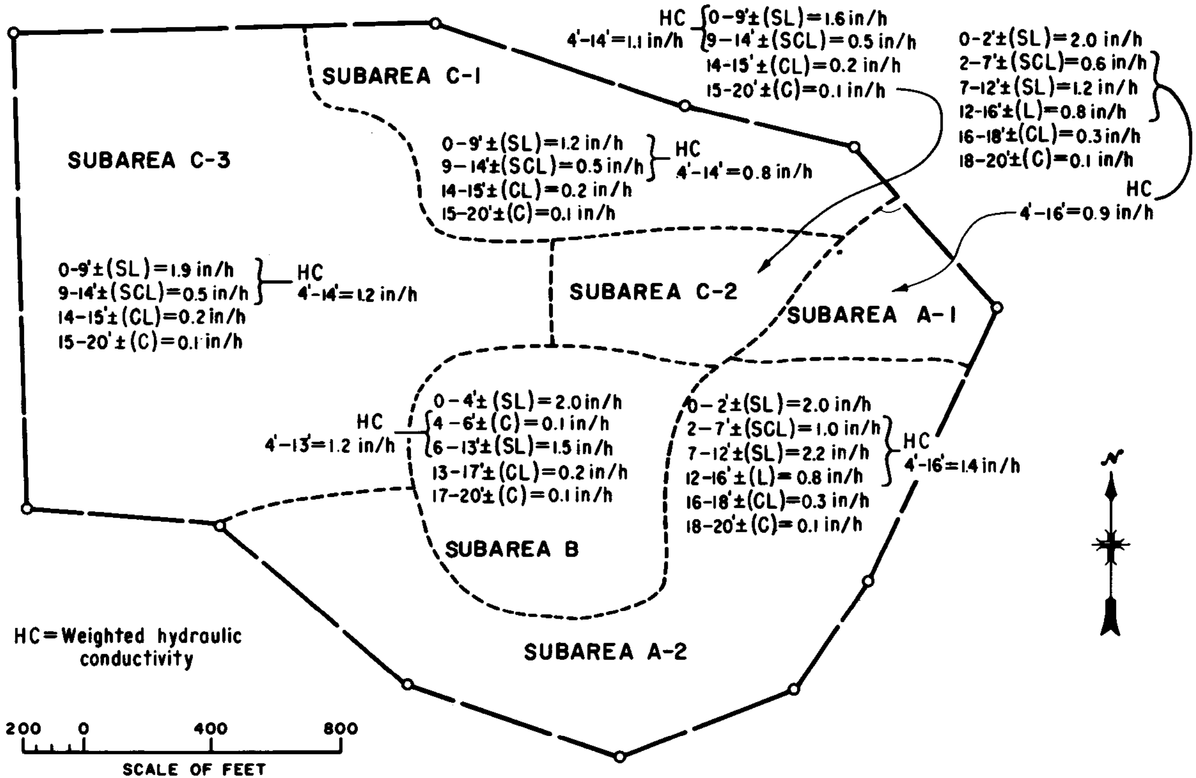


Figure 5–41.—Subareas of the sample farm having similar drainage conditions. The weighted, average, in-place hydraulic conductivity data are shown for each subarea. Drawing 103–D–676.

Table 5–11.—*Computations showing selection of barrier layer.*

Sub-area	Depth		Texture	K_2, hydraulic conductivity		K_2, weighted hydraulic conductivity		$K_1 \times 5$ compared with K_2 of layers above	Remarks
	meters	feet		cm/hr	in/h	cm/hr	in/h		
C–1	1.2–2.7	4–9	SL	3.05	1.2	3.05	1.2		
	2.7–4.3	9–14	SCL	1.27	0.5	2.16	0.85	1.27 (0.5) × 5 = 6.35 (2.5) > 3.05 (1.2)	
	4.3–4.6	14–15	CL	0.51	0.2	2.01	0.79	0.51 (0.2) × 5 = 2.55 (1.0) > 2.16 (0.85)	
	4.6–6.1	15–20	C	0.25	0.1	1.42	0.56	0.25 (0.1) × 5 = 1.25 (0.5) < 2.01 (0.79)	Barrier
C–2	1.2–2.7	4–9	SL	4.06	1.6	4.06	1.6		
	2.7–4.3	9–14	SCL	1.27	0.5	2.67	1.05	1.27 (0.5) × 5 = 6.35 (2.5) > 4.06 (1.6)	
	4.3–4.6	14–15	CL	0.51	0.2	2.49	0.98	0.51 (0.2) × 5 = 2.55 (1.0) < 2.67 (1.05)	Barrier
	4.6–6.1	15–20	C	0.25	0.1	1.78	0.70		
A–1	1.2–2.1	4–7	SCL	1.52	0.6	1.52	0.60		
	2.1–3.7	7–12	SL	3.05	1.2	2.46	0.97	3.05 (1.2) × 5 = 15.25 (6.0) > 1.52 (0.60)	
	3.7–4.9	12–16	L	2.03	0.8	2.34	0.92	2.03 (0.8) × 5 = 10.15 (4.0) > 2.46 (0.97)	
	4.9–5.5	16–18	CL	0.76	0.3	2.11	0.83	0.76 (0.3) × 5 = 3.80 (1.5) > 2.34 (0.92)	
	5.5–6.1	18–20	C	0.25	0.1	1.88	0.74	0.25 (0.1) × 5 = 1.25 (0.5) < 2.11 (0.83)	Barrier
A–2	1.2–2.1	4–7	SCL	2.54	1.0	2.54	1.0		
	2.1–3.7	7–12	SL	5.58	2.2	4.45	1.75	5.58 (2.2) × 5 = 27.9 (11.) > 2.54 (1.0)	
	3.7–4.9	12–16	L	2.03	0.8	3.63	1.43	2.03 (0.8) × 5 = 10.15 (4.8) > 4.45 (1.75)	
	4.9–5.5	16–18	CL	0.76	0.3	3.23	1.27	0.76 (0.3) × 5 = 3.80 (1.5) > 3.63 (1.43)	
	5.5–6.1	18–20	C	0.25	0.1	2.87	1.13	0.25 (0.1) × 5 = 1.25 (0.5) < 3.23 (1.27)	Barrier
B	1.2–1.8	4–6	C	0.25	0.1	0.25	0.10		Barrier
	1.8–4.0	6–13	SL	3.81	1.5	3.05	1.20	3.81 (1.5) × 5 = 19.05 (7.5) > 0.25 (0.1)	
	4.0–5.2	13–17	CL	0.51	0.2	2.26	0.89	0.51 (0.2) × 5 = 2.55 (1.0) < 3.05 (1.2)	Barrier
	5.2–6.1	17–20	C	0.25	0.1	1.88	0.74		
C–3	1.2–2.7	4–9	SL	4.83	1.9	4.83	1.90		
	2.7–4.3	9–14	SCL	1.27	0.5	3.05	1.20	1.27 (0.5) × 5 = 6.35 (2.5) > 4.83 (1.90)	
	4.3–4.6	14–15	CL	0.51	0.2	2.79	1.10	0.51 (0.2) × 5 = 2.55 (1.0) < 3.05 (1.2)	Barrier
	4.6–6.1	15–20	C	0.25	0.1	2.03	0.80		

For maximum effectiveness, the drains should be located in the more permeable layers. The in-place hydraulic conductivity data were used to determine the most desirable drain locations. For example, if spacing requirements could be satisfactorily met, a drain should not be located through the less permeable area represented by grid points D-3 and E-3. In this case, the drains could be located on either side of this less permeable area and still meet the drain spacing requirements. Figure 5–42 shows the location of the drains for the sample farm.

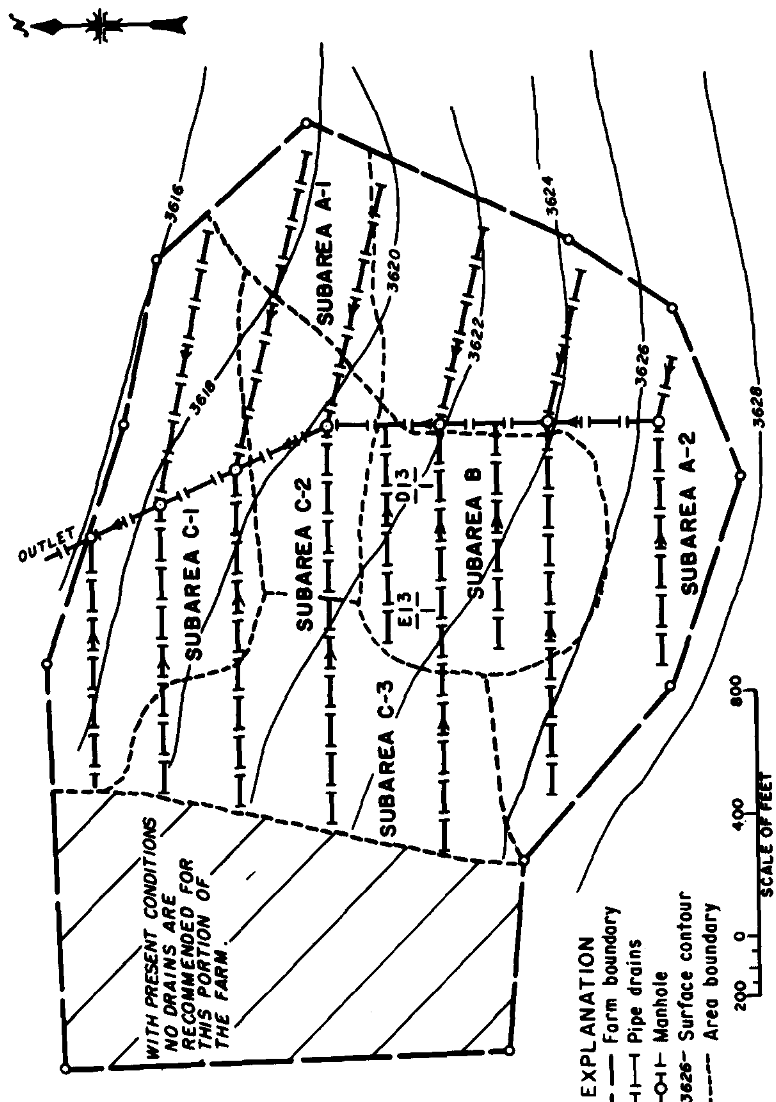


Figure 5–42.—Location of pipe drains on the sample farm. Drawing 103–D–679.

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⟨⟨Chapter VI

OPERATION AND MAINTENANCE OF DRAINAGE SYSTEMS

6–1. Introduction.—Efficient drainage systems must ultimately be provided on all irrigation projects when natural drainage conditions are inadequate to remove surplus water and salt. This surplus water may include waste from the irrigated farms, surface runoff from snow and rainfall, seepage and leakage from project canals and distribution systems, artesian water, and percolation from farm irrigation. Timely performance of preventive and regular maintenance on project drainage systems is absolutely necessary if the systems are to perform their intended functions. Project drainage systems should be thoroughly examined periodically to determine if they are functioning properly and if maintenance is required.

Occasionally, operation and maintenance forces on Bureau of Reclamation projects are required to design and construct open and pipe drains. These drains should be designed and constructed under the same criteria used when the work is done by Reclamation engineers.

6–2. Buried Pipe Drainage Systems.—Buried pipe drainage systems, properly installed, generally need little care to keep them operating satisfactorily; however, newly constructed systems require close vigilance during the early years of operation. Proper care of the system during this early period will increase the effectiveness of the drains and will often eliminate the need for future costly maintenance. Drainage system failures or partial failures are usually associated with unstable soil conditions which cause shifts in pipe alignment and grade; collapsed pipe; pulled joints; and plugged outlets, pipes, and manholes.

(a) Pipe drain outlets.—All pipe outlets should be inspected in the spring and after heavy rainstorms to ensure that the pipe still has a freefall into the open drain and that no erosion has occurred on the side slopes which could cause the outlet pipe to be displaced.

Flap gates, when required on the pipe outlet to keep floodwater in the open drain from backing up into the pipe, should be inspected at least once a month. Rodent screens that have been installed on pipe outlets should be checked periodically to be sure they are in place. Rodent screens may require periodic cleaning to remove moss and algae growth. Placing the screen in the outlet pipe so that it is out of direct sunlight may reduce the problem. Also, self-cleaning models are available through plastic pipe manufacturers. Where rodent screens have not been installed, the pipe outlet should be inspected periodically for rodent nests. All pipe outlets should be protected by fencing if farm animals are allowed in the area.

(b) Manholes or sand traps.—Manholes are used at any point on a pipe drain where they can be justified, and at junctions and major changes in alignment. It is very important that the manholes be kept clean; particularly during the initial operation of the system. Manholes should be inspected once a week when the drains are first laid, because failure to clean them has caused many drainage systems to become plugged. Pumps can be used to remove sand from manholes. Any erosion or settlement around the outside of the manhole should be repaired immediately. Manholes should not be used as surface waste disposal outlets, and no one should be permitted to remove the top 1.0-meter (3-foot) section, replace the cover, and thereby bury the structure without written consent of the control agency. Water levels should not be allowed in the manholes higher than the top of the inlet pipe.

When using mechanical cleaning rods in manholes, care should be taken so that the whipping motion of the cleaning cable does not damage the ends of the inlet and outlet pipes. Silt and sand trapped in the manhole should be cleaned following any drain-cleaning upstream.

Manhole covers should be fastened securely at all times, except during cleaning operations or inspection, to keep trash out and to prevent small children and animals from falling into the manhole.

(c) General maintenance of pipe drains.—A record should be established immediately after a drain is completed to track the amount of flow at each manhole and at the drain outlet. This tracking can be done by measuring the depth of water in the pipes that discharge into the manholes and by actually measuring discharge at the drain outlet. A sudden drop in discharge at any of the measuring points warrants additional investigations because there is a good possibility a segment of the drain has been completely or partially plugged. The area along the pipe drain should be inspected for sinkholes, wet spots, or tree growth, which are good indicators of potential trouble locations.

If a small sinkhole is discovered, it should be backfilled and inspected later for any additional settlement. If a large sinkhole is found, a fairly large hole should be dug down to the drain because large sinkholes often develop over broken pipe or over joints that have separated. Broken pipe should be replaced immediately. Joints that have pulled apart can be repaired satisfactorily by placing pipe butts (broken pieces of pipe) over the joint and backfilling around the joint with gravel.

Crushed pipe is a problem with plastic drains, and is usually the result of problems during construction. The most common problem is excessive stretch during construction; also, trencher breakdown or getting stuck allows the box to settle on the pipe. All collapsed drainpipe should be removed and replaced.

Wet spots that suddenly appear over pipe drains are good indicators that the drain has been completely or partially plugged. If the drain is only partially plugged with sediment, the plug can often be removed by placing a ball somewhat smaller than the pipe into the pipe upstream from the wet spot. This method has been used very successfully to flush sand and silt from pipe drains. Sewer rods can also be used both in concrete and clay pipe to probe and clear the drain. In recent years, high-pressure jets have been developed that have been particularly useful in cleaning plastic pipe drains. In some cases, a plug in the drain will have to be located and removed by uncovering and replacing a section of the drain.

Broken pipe, pulled joints, or plugged drains should be repaired as soon as possible so that the drainage system will function as intended. Plugs in older pipe drains are usually caused by tree or plant roots. Copper sulfate injected into the drain system will usually kill the roots, and by using a cleaning tool operated from the downstream side, the dead roots can be broken off and washed out to the nearest manhole for removal. When manholes are not available, a hole should be excavated to the drain downstream from the plug and one or more pipe joints removed so that the cleaning equipment can be inserted into the pipe. When using this method, a screen should always be placed over the pipe opening on the downstream side to prevent roots or other material from entering this portion of the drain.

Corrugated plastic drainpipe can easily be replaced using couplers and wire or tape. In case of an obstruction, instead of removing the pipe, it is often easier to cut an opening or window in the top of the pipe. After the obstruction is removed or other work performed, the hole is easily repaired. A cover piece is cut from a spare piece of pipe and then fastened in place with wire or tape. The window or joint areas are then covered with plastic sheet and the gravel envelope material replaced. Regardless of material, the disturbed area of pipe should be bedded in and covered with a minimum of 10 millimeters (4 inches) of gravel similar in gradation to the original envelope. (Sanders and Crooks, 1985).

Periodic checks should be made along the pipe drains to ensure that trees and shrubs have not started to grow over or near the drains. New growth should be killed by spraying with acceptable chemicals, if practicable. If trees and shrubs are growing near the drains that cannot be removed, the drain should be treated with copper sulfate to kill the roots. The first treatment should be made in April or early May, and if the roots are a serious problem, a second treatment should be made in August. The copper sulfate will not stop new root growth, so this treatment will have to be made annually. State water-quality standards must be followed closely when drains are treated with copper sulfate or other chemicals.

6–3. Open Drainage System.—Open drains require regular maintenance to keep them functioning as designed. The frequency and degree of this maintenance depend upon the climate, amount of rainfall, and the depth that the ground-water table must be kept below the ground surface. Shallow surface drains in stable material generally require only spot cleaning annually and a complete cleaning about every 5 years. In unstable soils, annual cleaning might be required along the bottom of the drains to maintain design depth, particularly if pipe drains discharge into the open drain. In the more stable soils and deep open drains, chemicals used periodically will prevent or kill weeds, willows, and tules. The weeds should be removed after they have been killed by chemicals so that the drain section is kept clean. All open drains will require some degree of maintenance after a large storm. A special problem is keeping open drains clear of tumbleweeds, which can cause serious erosion problems around structures.

All spoil banks should be planted to grass and should be releveled and replanted after bank cleaning. This replanting is done mainly to stabilize the excavated material to keep it from blowing or washing back into the drain and to provide a suitable roadway for maintenance. The side slopes of the open drain, particularly the sides above the water surface, should also be planted to grass and fertilized every 2 years. Maintenance roads require spot repair in the spring and after large storms.

Inlet openings, made through open drain banks for surface water, should be installed using pipe inlets or lined channels. Properly installed, these inlets usually require inspections only after large storms or when the open drain is being cleaned. Under no condition should an unlined cut be allowed through the drain bank. When pipes smaller than 450-millimeter (18-inch) diameter are used for these surface inlets, they should be inspected frequently during the spring to see if weeds have plugged the pipe. All grade control structures should be inspected periodically to check for undercutting or settlement and to determine that the trashracks and baffles are not plugged with weeds.

All livestock watering accesses to the drain should be covered with rock riprap or paved with concrete and fenced. All fences across the drain section should be inspected and cleaned of weeds and trash each spring and after large storms.

Wide-bottomed, shallow floodway channels should be grassed on the bottom and sides. The grass should be clipped to a height of about 10 centimeters at least once a year. The banks and sides should be fertilized as needed. Grazing on these grassed areas should be controlled, particularly in early spring.

Natural waterways used as drains should be left in their natural state as much as possible. Spot filling of eroded sections with rock or gravel should keep the channel stable, and smaller sections that erode under perennial flows should be rock lined. All inlets for surplus irrigation or rainfall runoff should consist of pipe inlets with riprap placed under the pipe.

6-4. Wastewater Disposal Ponds.—Wastewater disposal ponds are effective only in areas where the ponds can be bottomed in permeable sands and gravel with an adequate natural outlet or can be of such size as to store and evaporate drainwaters entering the pond. The ponds will operate as intended provided the silt which accumulates in the bottom is removed periodically. A record should be kept on the discharge of ponds. Staff gauges can be installed and readings taken at regular intervals to determine how fast the water seeps out of the pond. When the rate of discharge decreases considerably, it is time to clean the ponds. A good grass cover should be maintained on the dikes around ponds by periodic fertilization and watering if required.

Inlet structures, which have been constructed to bring surface wastewater from the fields into the ponds, should be kept in good repair. Settling basins or silt traps ahead of the inlet structure should be kept clean to minimize the need for cleaning the ponds.

6–5. Drainage Observation Wells.—Observation wells, properly installed, require minimum maintenance. However, any sudden change in the water-table depth or a constant water-table depth over a 3- or 4-month period usually indicates a plugged well. The work involved in cleaning the well can vary from pumping silt and sand from the well to pulling the pipe in the well and installing it in a new hole. The most common need for maintenance results from the pipe in the well being bent or pulled out by farm or highway equipment. To keep a reliable and complete record of the water table, these damaged wells should be reinstalled and protected by a 100- by 100-millimeter (4- by 4-inch) painted post. All automatic recorders installed on observation wells require constant maintenance to keep the clock and recorder operating properly.

6–6. Policy and Basic Requirements.—For additional information, see Reclamation Instructions Series 520 Drainage, Part 521, Policy and Basic Requirements.

6–7. Bibliography.—

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⟨⟨Chapter VII

SPECIAL DRAINAGE PROBLEMS

7–1. Return Flow Analysis Using the Transient Flow Concept.—A study of ground water hydrographs in an irrigated area generally shows that a water table rises during the irrigation season and reaches its highest elevation after the last irrigation of the season or, in an area of year-round cropping, at the end of the peak portion of the irrigation season. The water table then recedes during the slack or nonirrigation portion of the year and rises again during the irrigation season the following year.

If the annual discharge from an irrigated area does not equal recharge, the trend of the cyclic water table fluctuation will be progressively upward from year to year. When annual discharge and recharge become equal, the highest level and the range of water table fluctuation become reasonably constant from year to year. This condition is defined as "dynamic equilibrium." The method of drainage analysis developed by the Bureau of Reclamation takes into account the transient regimen of the ground-water recharge and discharge.

Figure 5–4, based on the Bureau's mathematical treatment of the transient flow concept, shows graphically the relation (at the midpoint between parallel drains) between the dimensionless parameters. The curves on figure 5–4 for these parameters represent the solution for the case where drains are above a barrier and on a barrier, respectively.

The discharge formulas for parallel drains are:

$$q = \frac{2\pi KyD}{L} \text{ (drains above barrier)}$$

$$q = \frac{4KH^2}{L} \text{ (drains on barrier)}$$

where:

q = drain discharge in cubic meters (feet) per linear meter (foot) of drain per day,

K = hydraulic conductivity in cubic meters (feet) per square meter (foot) per day [meters (feet) per day], and

y, D, L, and H are as defined in section 5–4.

These discharge formulas are combined with drain spacing computations in the development of area discharge curves for use in the design of drains and analysis of return flows. The discharge formulas, together with the spacing computations or an analysis of natural drainage in the area, can be used to compute the monthly distribution of discharge from a subsurface drainage system and to check whether dynamic equilibrium exists.

An alternate approach to determining outflow is accomplished by calculating the change in volume between successive drops in the water table and then dividing by the time period between readings:

$$\text{Volume} = 0.8\,(y_o - y) \times L \times S$$

where:

y_o = initial water table height,
y = final water table height,
L = drain spacing, and
S = specific yield.

The following is an example of drain spacing computations and the development of area discharge and monthly distribution discharge curves. The pertinent soil, crop, irrigation, drain design, and climatic characteristics are briefly described below:

(a) Drain depth is 2.4 meters (8 feet); maximum permissible height of water table midway between drains, y_o, is 1.2 meters (4 feet) above drain. This height provides a minimum root zone of 1.2 meters (4 feet).

(b) Hydraulic conductivity of the subsoil, in the zone where the water table will fluctuate, is 38 centimeters (15 inches) per hour [9.1 meters (30 feet) per day] with a corresponding specific yield of 23 percent.

(c) The depth from the drain to the impermeable barrier, d, is about 10 meters (33 feet). This depth corresponds to an equivalent depth, d', of 9.1 meters (30 feet) when spacing computations are corrected for convergency by Hooghoudt's method, discussed in section 5–5.

(d) The weighted average hydraulic conductivity in the zone between the maximum allowable water table and the impermeable barrier is 48 centimeters (19 inches) per hour, or 11.6 meters (38 feet) per day.

(e) Soil texture of the root zone is sandy loam. Deep percolation under normal irrigation practices on sandy loam soils amounts to about 28 percent of the irrigation application.

(f) The tabulation below shows the crops grown in the area, amount of water for each crop per irrigation, amount of deep percolation for each crop per irrigation, and the buildup in the water table caused by each irrigation.

(g) The irrigation schedule, shown on figure 7–1, shows the number and timing of irrigations for each crop as reported by the farmers in the area. Safflower-vegetable and barley-vegetable crops are double cropped on the same

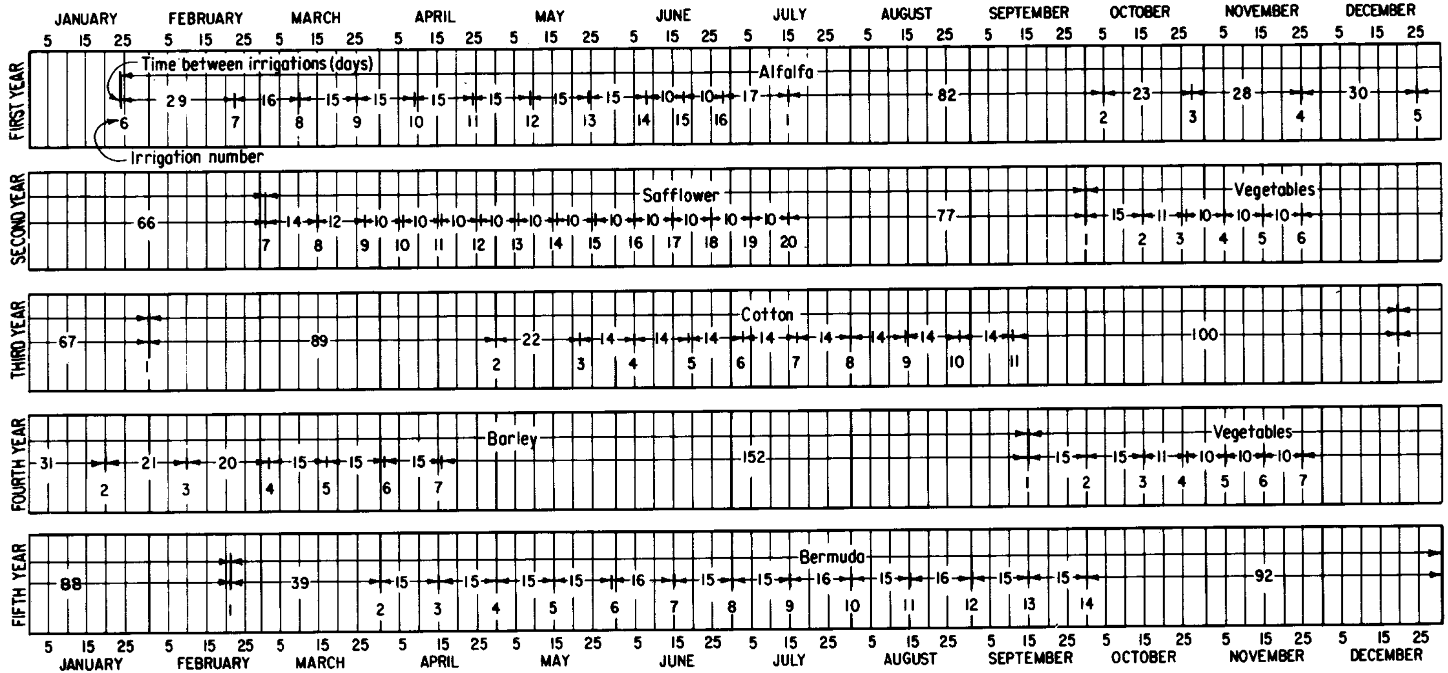


Figure 7–1.—Irrigation schedule for the example 5-year crop rotation program. 103-D-1674.

land. In the computations, assume that a 5-year crop rotation is practiced in the order shown from top to bottom on figure 7–1. The days between irrigations, used in the computations, are also shown on figure 7–1.

(h) Climatic conditions of the area are arid with only about 7.6 centimeters (3 inches) of annual precipitation. Deep percolation from precipitation can, therefore, be ignored. In areas where deep percolation from precipitation can be expected, the amount and timing of such deep percolation must be considered as recharge in the computations, as described in sections 5–5 and 5–57 of this manual.

(i) Assume the water table has reached dynamic equilibrium.

	Irrigation application,		Deep percolation,		Water table buildup,	
Crop	millimeters	inches	millimeters	inches	meters	feet
Alfalfa	140	5.5	39	1.54	0.17	0.56
Safflower	130	5.0	36	1.40	0.15	0.51
Vegetables	130	5.0	36	1.40	0.15	0.51
Cotton	130	5.0	36	1.40	0.15	0.51
Barley	115	4.5	32	1.26	0.14	0.46
Bermuda	140	5.5	39	1.54	0.17	0.56

The water table reaches the maximum allowable height, y_o, above the drain immediately after the last irrigation of the season or at the end of the peak portion of the irrigation season. Therefore, the average flow depth, D, can be computed for the first drain-out period. With this flow depth and the values of K, t, S, and a predetermined value of L, the value of the parameter $\frac{KDT}{SL^2}$ can be computed for the first time period. With this value, the corresponding parameter $\frac{y}{y_o}$ can be obtained from the curve for drains above barrier on figure 5–4. Knowing the initial water table height, v, at the beginning of the time period, the value of y, the height to which the midpoint water table falls during the time period, can be computed. This procedure is repeated for each successive time interval. If dynamic equilibrium exists, the water table must again reach, but not exceed, the initial height at the same time in the following year. See section 5–7.

Table 7–1 shows computations for the following 5-year crop rotation: (1) alfalfa, (2) safflower and vegetables, (3) cotton, (4) barley and vegetables, and (5) bermuda. In table 7–1, the columns contain the following information:

Column 1.–Crop under consideration.

Column 2.–Designation of each successive increment of ground-water recharge for each crop, see figure 7–1.

Column 3.–Length of drain-out period or time between recharge in days.

Column 4.–Buildup of water table in meters (feet) due to each recharge.

Column 5.–Water table height at midpoint between drains immediately after a recharge or at the beginning of each drain-out period (column 9 of preceding period plus column 4 of current period).

Column 6.—D is the average depth of flow, $d' + \frac{y_o}{2}$, where d' is the distance from drain to barrier corrected for convergency by Hooghoudt's method.

Column 7.—Computed value for flow conditions during drain-out period ($\frac{K}{SL^2}$ × column 3 × column 6).

Column 8.—Taken from curve of figure 5–4 for corresponding value of $\frac{KDt}{SL^2}$.

Column 9.—Midpoint water table height above drain at end of each drain-out period (column 5 × column 8).

Figure 7–2 shows the water table fluctuation for each crop in the rotation as produced by a 488-meter (1,600-foot) drain spacing. This figure illustrates the fact that a single drain spacing cannot be expected to be the optimum for all crops grown in rotation in the same field. In this example, the maximum permissible water table height occurs with two of the crops. Therefore, the 488-meter (1,600-foot) spacing is the maximum allowable for optimum production.

Table 7–2 shows how the discharge formula, $q = 2\pi \frac{KyD}{L}$ is used with calculated water table heights to compute discharge rates at the beginning and end of each drain-out period.

Figure 7–3 shows fluctuations in discharge rate produced from a crop of alfalfa under the following conditions: (1) entire area is irrigated at one time (maximum discharge rate), and (2) area is too large to be irrigated at one time, but portions are irrigated alternately so that the entire area is irrigated within the time period between irrigations (average discharge rate).

The design capacity of individual drainlines should be the maximum rate obtained from the curve of figure 7–3 for condition (1) above, because all or any portion of an individual line could be irrigated at one time. Collector and outlet drains which serve areas too large to be irrigated at one time should be designed for the maximum rate obtained from the curve of figure 7–3 for condition (2) above.

In this example, crops are in a 5-year rotation, and each farm unit has equal areas in each of the crops. As mentioned previously, no drain spacing can be optimum for all crops; similarly, no drainline capacity can be optimum for all crops, which means that both drain spacing and capacity should be provided for the crop with the greatest drainage requirement; in this example, safflower. The maximum discharge rate for safflower, as shown in table 7–2, is 2.01 cubic meters per day per meter (21.6 cubic feet per day per foot) of drain. The Bureau of Reclamation normally expresses this rate in cubic meters (feet) per second per kilometer (mile) of drain, as follows:

Table 7–1a.—*Drain spacing computations with convergence correction included for the example 5-year crop rotation program (metric units). L = 488 meters, K = 11.6 meters per day, S = 23 percent, and d′ = 9.1 meters. (Sheet 1 of 2.) 103-D-1679-1.*

1	2	3	4	5	6	7	8	9
Crop	Irrigation Number	Time, Days	Buildup, Meters	Yo, Meters	D, Meters	$\frac{KDt}{SL^2}$	$\frac{y}{yo}$	y, Meters
	16	17	0.171	1.219	9.754	0.0351	0.812	0.990
	1	82	0.171	1.161	9.726	0.1690	0.220	0.255
	2	23	0.171	0.426	9.357	0.0455	0.742	0.316
	3	28	0.171	0.488	9.388	0.0556	0.673	0.328
	4	30	0.171	0.499	9.394	0.0597	0.645	0.322
	5	30	0.171	0.493	9.391	0.0597	0.645	0.318
	6	29	0.171	0.490	9.388	0.0576	0.665	0.326
	7	16	0.171	0.498	9.388	0.0318	0.840	0.418
	8	15	0.171	0.586	9.437	0.0300	0.850	0.498
	9	15	0.171	0.668	9.479	0.0301	0.850	0.568
	10	15	0.171	0.741	9.513	0.0302	0.850	0.630
Alfalfa	11	15	0.171	0.801	9.543	0.0303	0.850	0.681
	12	15	0.171	0.850	9.571	0.0304	0.850	0.723
	13	15	0.171	0.894	9.592	0.0305	0.850	0.759
	14	10	0.171	0.930	9.610	0.0203	0.920	0.856
	15	10	0.171	1.027	9.656	0.0204	0.920	0.944
	16	17	0.171	1.113	9.702	0.0349	0.810	0.902
	1	82	0.171	1.075	9.680	0.1680	0.225	0.242
	2	23	0.171	0.412	9.351	0.0456	0.740	0.305
	3	28	0.171	0.475	9.382	0.0556	0.670	0.318
	4	30	0.171	0.489	9.388	0.0597	0.650	0.318
	5	66	0.171	0.489	9.388	0.1310	0.320	0.157
	7	14	0.155	0.311	9.299	0.0275	0.870	0.271
	8	12	0.155	0.426	9.357	0.0237	0.893	0.380
	9	10	0.155	0.536	9.412	0.0199	0.920	0.493
	10	10	0.155	0.648	9.467	0.0200	0.920	0.596
	11	10	0.155	0.752	9.519	0.0202	0.920	0.692
	12	10	0.155	0.848	9.568	0.0202	0.920	0.780
Safflower	13	10	0.155	0.934	9.610	0.0204	0.920	0.860
	14	10	0.155	1.016	9.653	0.0204	0.920	0.935
	15	10	0.155	1.092	9.690	0.0205	0.919	1.003
	16	10	0.155	1.158	9.723	0.0206	0.919	1.064
	17	10	0.155	1.219	9.754	0.0206	0.919	1.120
	18	10	0.155	1.275	9.781	0.0207	0.917	1.169
	19	10	0.155	1.326	9.808	0.0208	0.917	1.216
	20	77	0.155	1.375	9.830	0.1603	0.240	0.330
	1	15	0.155	0.484	9.385	0.0298	0.851	0.412
	2	11	0.155	0.568	9.427	0.0219	0.912	0.518
Vegetables	3	10	0.155	0.672	9.479	0.0200	0.920	0.618
	4	10	0.155	0.775	9.531	0.0202	0.920	0.713
	5	10	0.155	0.867	9.577	0.0203	0.920	0.798
	6	67	0.155	0.954	9.623	0.1360	0.313	0.299

Table 7–1a.—*Drain spacing computations with convergence correction included for the example 5-year crop rotation program (metric units). L = 488 meters, K = 11.6 meters per day, S = 23 percent, and d′ = 9.1 meters. (Sheet 2 of 2.) 103-D-1679-2.*

1	2	3	4	5	6	7	8	9
Crop	Irrigation Number	Time, Days	Buildup, Meters	Yo, Meters	D, Meters	$\frac{KDt}{SL^2}$	$\frac{y}{yo}$	y, Meters
	1	89	0.155	0.454	9.370	0.1770	0.205	0.093
	2	22	0.155	0.248	9.269	0.0431	0.755	0.187
	3	14	0.155	0.342	9.315	0.0276	0.870	0.298
	4	14	0.155	0.453	9.370	0.0278	0.870	0.394
	5	14	0.155	0.549	9.418	0.0279	0.870	0.478
Cotton	6	14	0.155	0.633	9.461	0.0280	0.870	0.550
	7	14	0.155	0.705	9.498	0.0281	0.866	0.611
	8	14	0.155	0.766	9.528	0.0282	0.865	0.662
	9	14	0.155	0.817	9.552	0.0283	0.865	0.707
	10	14	0.155	0.862	9.577	0.0284	0.866	0.747
	11	100	0.155	0.902	9.595	0.2032	0.145	0.131
	1	31	0.140	0.271	9.278	0.0609	0.640	0.173
	2	21	0.140	0.313	9.299	0.0414	0.770	0.241
	3	20	0.140	0.381	9.336	0.0395	0.783	0.298
Barley	4	15	0.140	0.438	9.363	0.0297	0.860	0.377
	5	15	0.140	0.517	9.403	0.0299	0.860	0.445
	6	15	0.140	0.585	9.437	0.0299	0.855	0.500
	7	76	0.140	0.640	9.464	0.1523	0.260	0.166
		76	0	0.166	9.226	0.1485	0.272	0.045
	1	15	0.155	0.200	9.245	0.0294	0.860	0.172
	2	15	0.155	0.327	9.309	0.0296	0.860	0.281
	3	11	0.155	0.436	9.363	0.0218	0.916	0.400
Vegetables	4	10	0.155	0.555	9.421	0.0199	0.924	0.513
	5	10	0.155	0.668	9.479	0.0200	0.923	0.616
	6	10	0.155	0.771	9.531	0.0202	0.921	0.710
	7	88	0.155	0.865	9.577	0.1785	0.188	0.163
	1	39	0.171	0.334	9.307	0.0769	0.555	0.185
	2	15	0.171	0.356	9.318	0.0296	0.860	0.306
	3	15	0.171	0.477	9.379	0.0298	0.860	0.410
	4	15	0.171	0.581	9.431	0.0300	0.850	0.494
	5	15	0.171	0.665	9.472	0.0301	0.850	0.565
	6	16	0.171	0.736	9.508	0.0322	0.840	0.618
Bermuda	7	15	0.171	0.789	9.535	0.0303	0.850	0.671
	8	15	0.171	0.842	9.561	0.0304	0.850	0.716
	9	16	0.171	0.887	9.583	0.0325	0.836	0.742
	10	15	0.171	0.913	9.596	0.0305	0.849	0.775
	11	16	0.171	0.946	9.613	0.0326	0.835	0.790
	12	15	0.171	0.961	9.621	0.0306	0.849	0.816
	13	15	0.171	0.987	9.633	0.0306	0.849	0.838
	14	92	0.171	1.009	9.645	0.1879	0.180	0.182

Table 7–1b.—*Drain spacing computations with convergence correction included for the example 5-year crop rotation program (U.S. customary units). L = 1,600 feet, K = 38 feet per day, S = 23 percent, and d′ = 30 feet. (Sheet 1 of 2.) 103-D-1679-1.*

1	2	3	4	5	6	7	8	9
Crop	Irrigation Number	Time, Days	Buildup, Feet	Yo, Feet	D, Feet	$\frac{KDt}{SL^2}$	$\frac{y}{y_o}$	y, Feet
	16	17	0.56	4.00	32.00	0.0351	0.812	3.25
	1	82	0.56	3.81	31.91	0.1690	0.220	0.84
	2	23	0.56	1.40	30.70	0.0455	0.742	1.04
	3	28	0.56	1.60	30.80	0.0556	0.673	1.08
	4	30	0.56	1.64	30.82	0.0597	0.645	1.06
	5	30	0.56	1.62	30.81	0.0597	0.645	1.04
	6	29	0.56	1.61	30.80	0.0576	0.665	1.06
	7	16	0.56	1.62	30.80	0.0318	0.840	1.36
	8	15	0.56	1.92	30.96	0.0300	0.850	1.64
	9	15	0.56	2.20	31.10	0.0301	0.850	1.87
	10	15	0.56	2.43	31.21	0.0302	0.850	2.06
Alfalfa	11	15	0.56	2.62	31.31	0.0303	0.850	2.23
	12	15	0.56	2.79	31.40	0.0304	0.850	2.37
	13	15	0.56	2.93	31.47	0.0305	0.850	2.49
	14	10	0.56	3.05	31.53	0.0203	0.920	2.80
	15	10	0.56	3.36	31.68	0.0204	0.920	3.10
	16	17	0.56	3.66	31.83	0.0349	0.810	2.96
	1	82	0.56	3.52	31.76	0.1680	0.225	0.79
	2	23	0.56	1.35	30.68	0.0456	0.740	1.00
	3	28	0.56	1.56	30.78	0.0556	0.670	1.05
	4	30	0.56	1.61	30.80	0.0597	0.650	1.04
	5	66	0.56	1.60	30.80	0.1310	0.320	0.51
	7	14	0.51	1.02	30.51	0.0275	0.870	0.89
	8	12	0.51	1.40	30.70	0.0237	0.893	1.25
	9	10	0.51	1.76	30.88	0.0199	0.920	1.62
	10	10	0.51	2.13	31.06	0.0200	0.920	1.96
	11	10	0.51	2.47	31.23	0.0202	0.920	2.27
	12	10	0.51	2.78	31.39	0.0202	0.920	2.56
Safflower	13	10	0.51	3.07	31.53	0.0204	0.920	2.82
	14	10	0.51	3.34	31.67	0.0204	0.920	3.07
	15	10	0.51	3.58	31.79	0.0205	0.919	3.29
	16	10	0.51	3.80	31.90	0.0206	0.919	3.49
	17	10	0.51	4.00	32.00	0.0206	0.919	3.68
	18	10	0.51	4.19	32.09	0.0207	0.917	3.84
	19	10	0.51	4.36	32.18	0.0208	0.917	3.99
	20	77	0.51	4.50	32.25	0.1603	0.240	1.08
	1	15	0.51	1.59	30.79	0.0298	0.851	1.35
	2	11	0.51	1.86	30.93	0.0219	0.912	1.70
Vegetables	3	10	0.51	2.21	31.10	0.0200	0.920	2.03
	4	10	0.51	2.54	31.27	0.0202	0.920	2.34
	5	10	0.51	2.85	31.42	0.0203	0.920	2.62
	6	67	0.51	3.13	31.57	0.1360	0.313	0.98

Table 7–1b.—*Drain spacing computations with convergence correction included for the example 5-year crop rotation program (U.S. customary units). L = 1,600 feet, K = 38 feet per day, S = 23 percent, and d′ = 30 feet. (Sheet 2 of 2.) 103-D-1679-1.*

1	2	3	4	5	6	7	8	9
Crop	Irrigation Number	Time, Days	Buildup, Feet	Yo, Feet	D, Feet	$\frac{KDt}{SL^2}$	$\frac{y}{y_o}$	y, Feet
Cotton	1	89	0.51	1.49	30.74	0.1770	0.205	0.31
	2	22	0.51	0.82	30.41	0.0431	0.755	0.62
	3	14	0.51	1.13	30.56	0.0276	0.870	0.98
	4	14	0.51	1.49	30.74	0.0278	0.870	1.30
	5	14	0.51	1.81	30.90	0.0279	0.870	1.57
	6	14	0.51	2.08	31.04	0.0280	0.870	1.81
	7	14	0.51	2.32	31.16	0.0281	0.866	2.01
	8	14	0.51	2.52	31.26	0.0282	0.865	2.18
	9	14	0.51	2.69	31.34	0.0283	0.865	2.33
	10	14	0.51	2.84	31.42	0.0284	0.866	2.46
	11	100	0.51	2.97	31.48	0.2032	0.145	0.43
Barley	1	31	0.46	0.89	30.44	0.0609	0.640	0.57
	2	21	0.46	1.02	30.51	0.0414	0.770	0.79
	3	20	0.46	1.25	30.63	0.0395	0.783	0.98
	4	15	0.46	1.44	30.72	0.0297	0.860	1.24
	5	15	0.46	1.70	30.85	0.0299	0.860	1.46
	6	15	0.46	1.92	30.96	0.0299	0.855	1.64
	7	76	0.46	2.10	31.05	0.1523	0.260	0.55
		76	0	0.55	30.27	0.1485	0.272	0.15
Vegetables	1	15	0.51	0.66	30.33	0.0294	0.860	0.57
	2	15	0.51	1.08	30.54	0.0296	0.860	0.93
	3	11	0.51	1.44	30.72	0.0218	0.916	1.32
	4	10	0.51	1.83	30.91	0.0199	0.924	1.69
	5	10	0.51	2.20	31.10	0.0200	0.923	2.03
	6	10	0.51	2.54	31.27	0.0202	0.921	2.34
	7	88	0.51	2.85	31.42	0.1785	0.188	0.54
Bermuda	1	39	0.56	1.10	30.55	0.0769	0.555	0.61
	2	15	0.56	1.17	30.58	0.0296	0.860	1.01
	3	15	0.56	1.57	30.78	0.0298	0.860	1.35
	4	15	0.56	1.91	30.96	0.0300	0.850	1.62
	5	15	0.56	2.18	31.09	0.0301	0.850	1.85
	6	16	0.56	2.41	31.21	0.0322	0.840	2.02
	7	15	0.56	2.58	31.29	0.0303	0.850	2.19
	8	15	0.56	2.75	31.38	0.0304	0.850	2.34
	9	16	0.56	2.90	31.45	0.0325	0.836	2.42
	10	15	0.56	2.98	31.49	0.0305	0.849	2.53
	11	16	0.56	3.09	31.55	0.0326	0.835	2.58
	12	15	0.56	3.14	31.57	0.0306	0.849	2.67
	13	15	0.56	3.23	31.61	0.0306	0.849	2.74
	14	92	0.56	3.30	31.65	0.1879	0.180	0.59

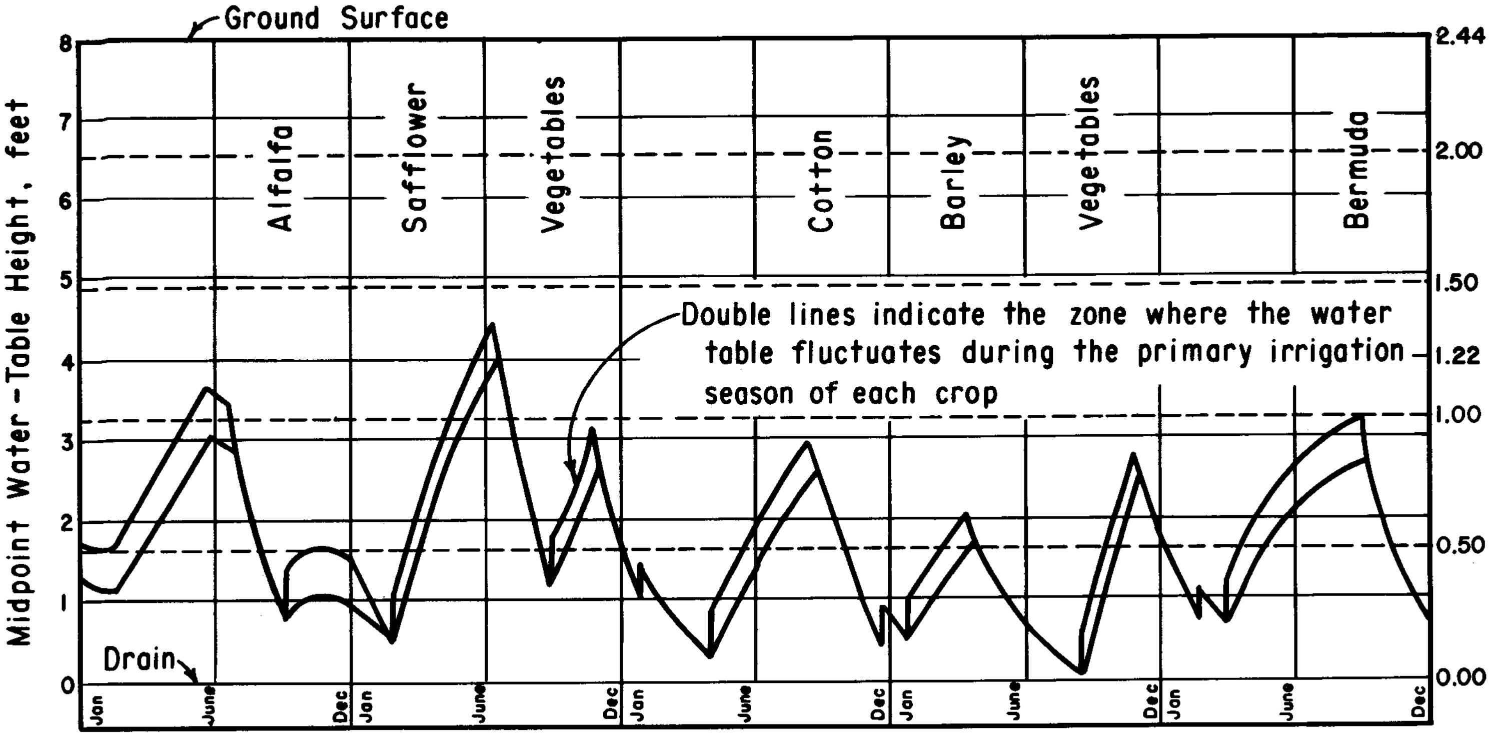
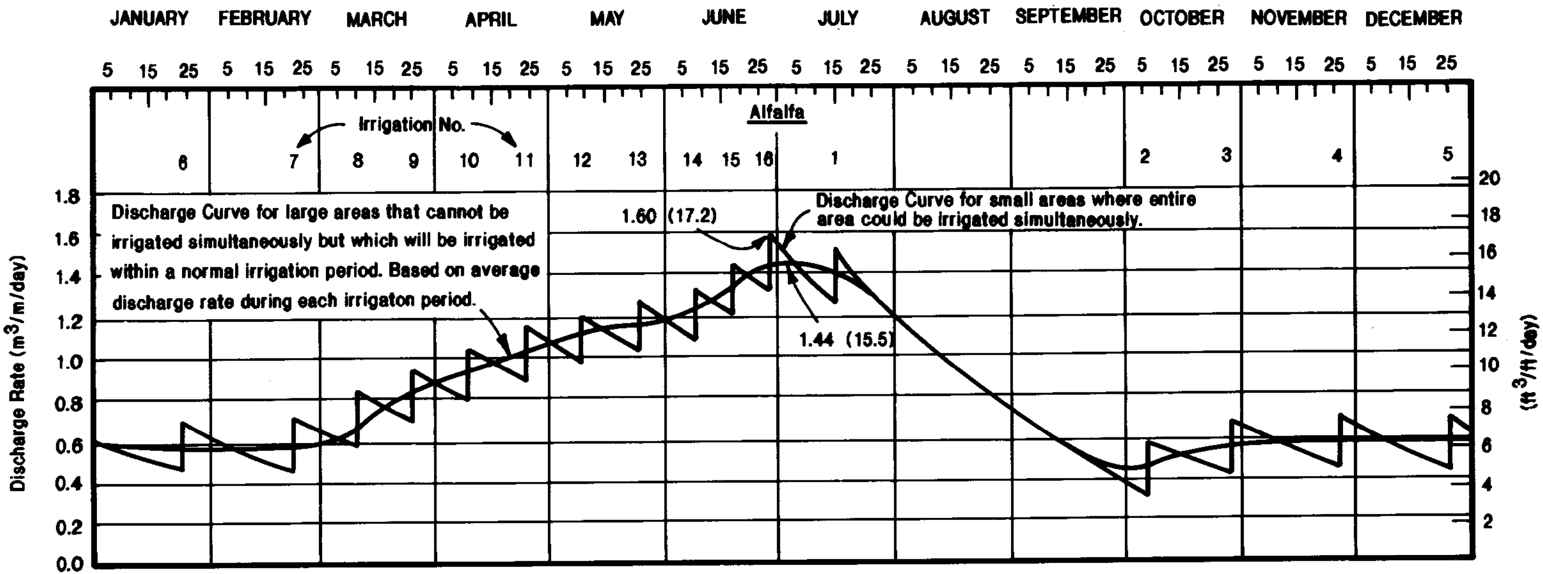


Figure 7–2.—Water table fluctuation for each crop in the example 5-year crop rotation program. 103-D-1675.



Discharge Rate for the design of individual spaced relief drains

$\frac{(1.60)(1{,}000)}{86{,}400} = 0.0185$ m^3/s per kilometer (1.052 ft^3/s per mile)

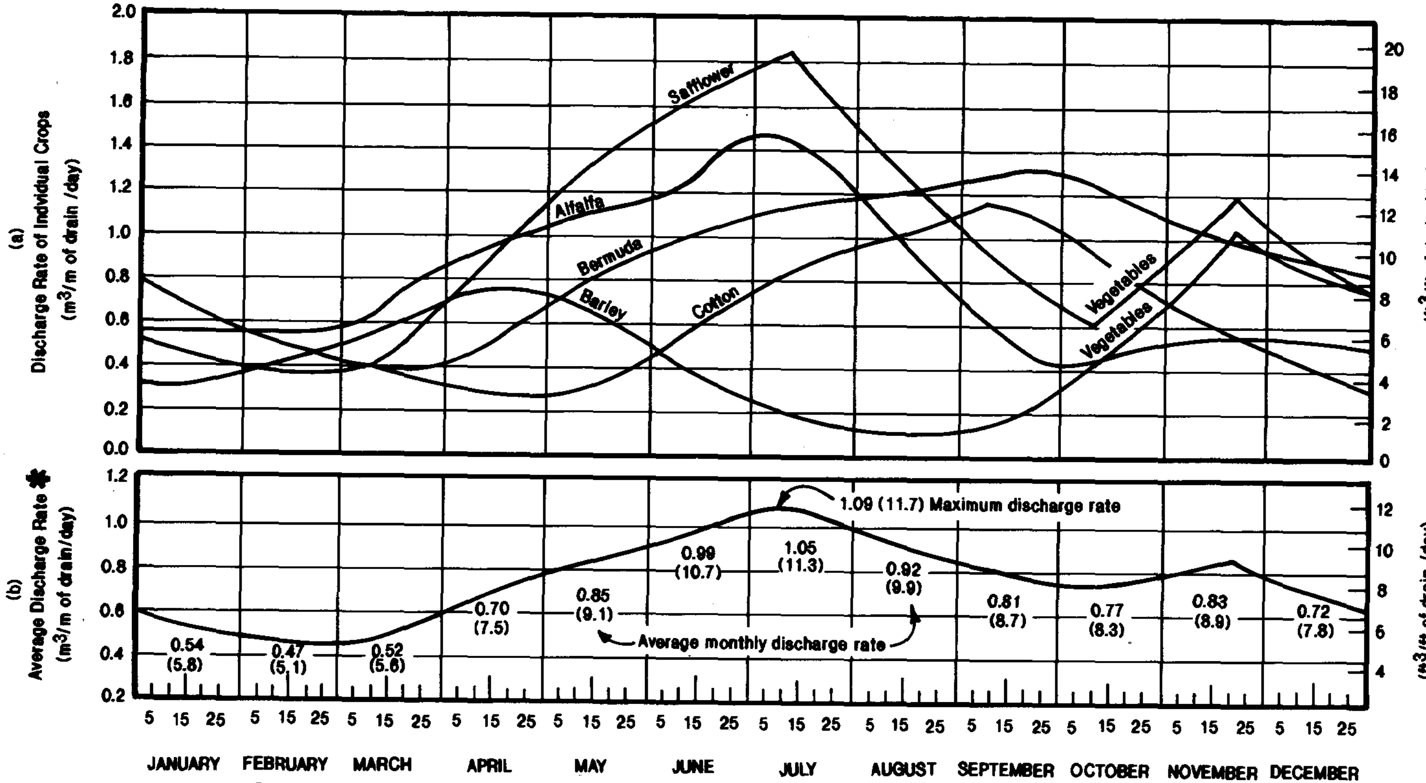
where 1,000 = meters per kilometer and 86,400 = seconds per day

Discharge Rate for the design of collector and outlet drains

$\frac{(1.44)(10{,}000)}{(86{,}400)(488)} = 0.00034$ m^3/s per hectare (0.0049 ft^3/s per acre)

where 10,000 = square meters per hectare,
86,400 = seconds per day and
488 = drain spacing in meters = 1,600 feet

Figure 7–3.—Fluctuations in discharge rate produced from a crop of alfalfa. 103-D-1676.



* Based on equal distribution of each crop. Other crop distribution would require proper weighting of values from the top chart.

Figure 7–4.—Discharge rates for each crop in the example 5-year crop rotation program.

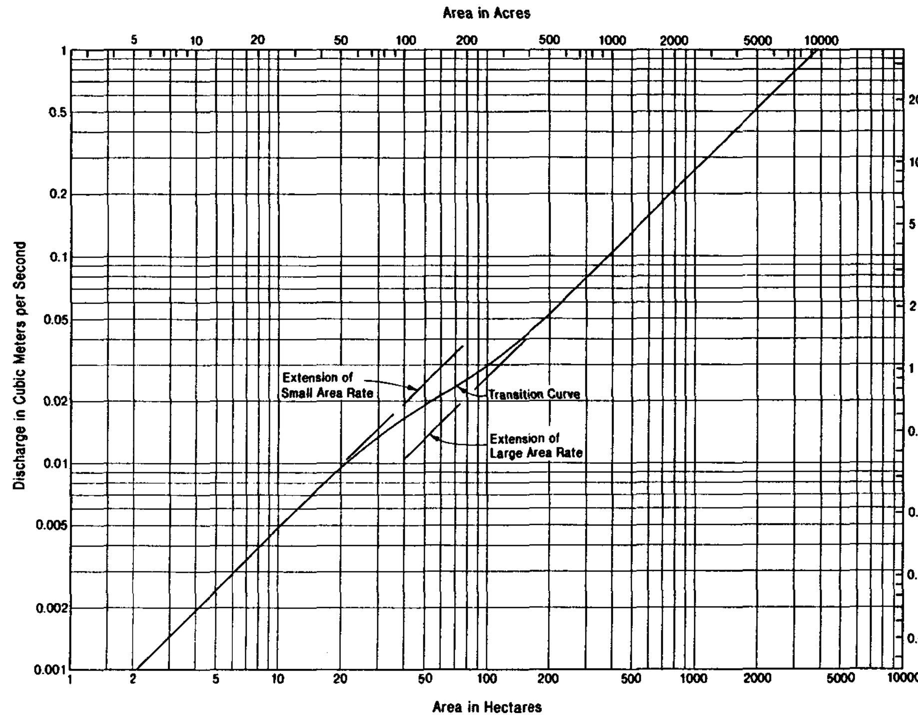


Figure 7–5.—Area discharge curve. 103-D-1678.

$$\frac{(2.01)(1{,}000)}{86{,}400} = 0.023 \text{ m}^3\text{/s per kilometer}(1.3 \text{ ft}^3\text{/per mile})$$

Discharge can also be expressed as cubic meters (feet) per second per hectare (acre), as follows:

$$\frac{(2.01)(10{,}000)}{(86{,}400)(488)} = 0.00048 \text{ m}^3\text{/s per hectare } (0.0068 \text{ ft}^3\text{/s per acre})$$

The maximum rate for safflower, 0.00048 m³/s per hectare (0.0068 ft³/s per acre) is used in deriving the area discharge curve for small areas up to about 16 hectares (40 acres).

The design of collector drains can be based on the maximum weighted average discharge rate produced by all crops used in the 5-year rotation, see figure 7–4. Figure 7–4 shows the average discharge rate by crop at various time intervals. A curve for any distribution of crops can be derived by weighting the discharge from each crop according to the acreage in that crop. In this example, figure 7–4 represents the average discharge rate from an area too large to be irrigated at one time and with equal acreages in the various crops of the 5-year rotation. The maximum discharge, 1.09 cubic meters (11.7 cubic feet per foot) of drain per day, from figure 7–4 can be used to develop the design capacity for collector and outlet drains, as follows:

$$\frac{(1.09)(10{,}000)}{(86{,}4000)(488)} = 0.00026 \text{ m}^3\text{/s per hectare } (0.00369 \text{ ft}^3\text{/s per acre})$$

This rate is normally considered to apply to areas larger than about 200 to 240 hectares (500 to 600 acres). An area discharge curve for designing the subsurface drainage system can be developed by plotting the rate for individual drainlines for areas up to 16 hectares (40 acres) and the rate for collector and outlet drains for areas above 200 to 240 hectares (500 to 600 acres). A smooth curve is drawn to connect the 16- (40-) and 200-hectare (500-acre) curves. The area discharge curve of figure 7–5 was derived in this manner.

Figure 7–4 can be used to derive the average monthly discharge rate and to confirm that the 488-meter (1,600-foot) spacing produces dynamic equilibrium. The discharge volume for each month of the year can be determined as follows:

For January:

$$\frac{(0.539)(10{,}000)(31)}{488} = 342.4 \text{ cubic meters hectare per month}$$

where:

0.539 is the average discharge rate in cubic meters per meter of drain per day (5.8 cubic feet per foot of drain per day), and

10,000 is the number of square meters in a hectare

Table 7–2a.–*Discharge computations for the example 5-year crop rotation program (metric units). (Sheet 1 of 4.) 103-D-1680-1.*

Crop	Irrigation Number	Discharge (q), m^3/m/day	Average Discharge m^3/m/day *
Alfalfa	16	$(0.149)^{**}(1.219)(9.754) = 1.772$ $(0.990)(9.639) = 1.422$	
	1	$(1.161)(9.725) = 1.682$ $(0.255)(9.272) = 0.352$	
	2	$(0.426)(9.357) = 0.594$ $(0.316)(9.302) = 0.438$	
	3	$(0.488)(9.388) = 0.683$ $(0.328)(9.308) = 0.455$	
	4	$(0.499)(9.394) = 0.698$ $(0.322)(9.305) = 0.446$	
	5	$(0.493)(9.391) = 0.690$ $(0.318)(9.303) = 0.441$	0.556
	6	$(0.490)(9.388) = 0.685$ $(0.326)(9.307) = 0.452$	0.569
	7	$(0.498)(9.388) = 0.697$ $(0.418)(9.353) = 0.583$	0.640
	8	$(0.586)(9.437) = 0.824$ $(0.498)(9.393) = 0.697$	0.761
	9	$(0.668)(9.479) = 0.943$ $(0.568)(9.428) = 0.798$	0.871
	10	$(0.741)(9.513) = 1.050$ $(0.630)(9.459) = 0.888$	0.969
	11	$(0.801)(9.545) = 1.139$ $(0.681)(9.485) = 0.962$	1.051
	12	$(0.850)(9.571) = 1.212$ $(0.723)(9.506) = 1.024$	1.118
	13	$(0.894)(9.592) = 1.278$ $(0.759)(9.524) = 1.077$	1.178
	14	$(0.930)(9.610) = 1.332$ $(0.856)(9.572) = 1.221$	1.277
	15	$(1.027)(9.656) = 1.478$ $(0.944)(9.616) = 1.353$	1.416
	16	$(1.113)(9.702) = 1.609$ $(0.902)(9.595) = 1.290$	1.450
	1	$(1.075)(9.680) = 1.551$ $(0.242)(9.265) = 0.334$	0.943
	2	$(0.412)(9.351) = 0.574$ $(0.305)(9.297) = 0.422$	0.498
	3	$(0.475)(9.382) = 0.664$ $(0.318)(9.303) = 0.441$	0.553
	4	$(0.489)(9.388) = 0.684$ $(0.318)(9.303) = 0.441$	0.563
	5	$(0.489)(9.388) = 0.684$ $(0.157)(9.223) = 0.216$	0.450
	7	$(0.311)(9.299) = 0.431$ $(0.271)(9.280) = 0.375$	0.403

* For the time period between irrigations

** $2\pi K/L = 2\pi(11.6)/488$

Table 7–2a.–*Discharge computations for the example 5-year crop rotation program (metric units). (Sheet 2 of 4.) 103-D-1680-2.*

Crop	Irrigation Number	Discharge (q), m^3/m/day	Average Discharge m^3/m/day *
Safflower	8	(0.149)** (0.426)(9.351) = 0.594	0.561
		(0.380)(9.334) = 0.528	
	9	(0.536)(9.412) = 0.752	0.721
		(0.493)(9.391) = 0.690	
	10	(0.648)(9.467) = 0.914	0.876
		(0.596)(9.442) = 0.838	
	11	(0.752)(9.519) = 1.067	1.023
		(0.692)(9.490) = 0.978	
	12	(0.848)(9.568) = 1.209	1.159
		(0.780)(9.534) = 1.108	
	13	(0.934)(9.610) = 1.337	1.282
		(0.860)(9.574) = 1.227	
	14	(1.016)(9.653) = 1.416	1.400
		(0.935)(9.612) = 1.339	
	15	(1.092)(9.690) = 1.577	1.509
		(1.003)(9.646) = 1.441	
	16	(1.158)(9.723) = 1.678	1.606
		(1.064)(9.676) = 1.534	
	17	(1.219)(9.754) = 1.772	1.696
		(1.120)(9.704) = 1.619	
	18	(1.275)(9.781) = 1.858	1.777
		(1.169)(9.729) = 1.695	
	19	(1.326)(9.808) = 1.938	1.853
		(1.216)(9.752) = 1.767	
	20	(1.375)(9.830) = 2.014	1.236
Vegetables		(0.330)(9.309) = 0.458	
	1	(0.484)(9.385) = 0.677	0.626
		(0.412)(9.350) = 0.574	
	2	(0.568)(9.427) = 0.798	0.762
		(0.518)(9.403) = 0.726	
	3	(0.672)(9.479) = 0.949	0.910
		(0.618)(9.453) = 0.870	
	4	(0.775)(9.531) = 1.101	1.055
		(0.713)(9.501) = 1.009	
	5	(0.867)(9.577) = 1.237	1.186
		(0.798)(9.539) = 1.134	
	6	(0.954)(9.623) = 1.368	0.891
Cotton		(0.299)(9.294) = 0.414	
	1	(0.454)(9.370) = 0.634	0.381
		(0.093)(9.191) = 0.127	
	2	(0.248)(9.269) = 0.343	0.300
		(0.187)(9.238) = 0.257	
	3	(0.342)(9.315) = 0.475	0.444
		(0.298)(9.293) = 0.413	
	4	(0.453)(9.370) = 0.632	0.590
		(0.394)(9.341) = 0.548	

* For the time period between irrigations
** $2\pi K/L = 2\pi(11.6)/488$

Table 7–2a.–*Discharge computations for the example 5-year crop rotation program (metric units). (Sheet 3 of 4.) 103-D-1680-3.*

Crop	Irrigation Number	Discharge (q), m^3/m/day	Average Discharge m^3/m/day *
Cotton	5	(0.149)** (0.549)(9.418) = 0.770 (0.478)(9.383) = 0.668	0.719
	6	(0.633)(9.461) = 0.892 (0.550)(9.419) = 0.772	0.832
	7	(0.750)(9.498) = 0.998 (0.611)(9.450) = 0.860	0.929
	8	(0.766)(9.528) = 1.087 (0.662)(9.475) = 0.935	1.011
	9	(0.817)(9.552) = 1.163 (0.707)(9.498) = 1.000	1.082
	10	(0.862)(9.577) = 1.230 (0.747)(9.518) = 1.059	1.145
	11	(0.902)(9.595) = 1.290 (0.131)(9.210) = 0.180	0.735
Barley	1	(0.271)(9.278) = 0.375 (0.173)(9.231) = 0.238	0.306
	2	(0.313)(9.299) = 0.434 (0.241)(9.265) = 0.333	0.383
	3	(0.381)(9.336) = 0.530 (0.298)(9.293) = 0.413	0.471
	4	(0.438)(9.363) = 0.611 (0.377)(9.333) = 0.524	0.568
	5	(0.517)(9.403) = 0.724 (0.445)(9.367) = 0.621	0.673
	6	(0.585)(9.437) = 0.823 (0.500)(9.394) = 0.700	0.761
	7	(0.640)(9.464) = 0.902 (0.166)(9.227) = 0.228	0.565
		(0.045)(9.167) = 0.061	0.145
Vegetables	1	(0.200)(9.245) = 0.276 (0.172)(9.230) = 0.237	0.256
	2	(0.327)(9.309) = 0.454 (0.281)(9.285) = 0.389	0.421
	3	(0.436)(9.363) = 0.608 (0.400)(9.344) = 0.557	0.583
	4	(0.555)(9.421) = 0.779 (0.513)(9.401) = 0.719	0.749
	5	(0.668)(9.479) = 0.943 (0.616)(9.452) = 0.868	0.906
	6	(0.771)(9.531) = 1.095 (0.710)(9.499) = 1.005	1.050
	7	(0.865)(9.577) = 1.234 (0.163)(9.226) = 0.224	0.729
	1	(0.334)(9.307) = 0.463 (0.185)(9.237) = 0.255	0.359

* For the time period between irrigations

** $2\pi K/L = 2\pi(11.6)/488$

Table 7–2a.–*Discharge computations for the example 5-year crop rotation program (metric units). (Sheet 4 of 4.) 103-D-1680-4.*

Crop	Irrigation Number	Discharge (q), m^3/m/day	Average Discharge m^3/m/day *
Bermuda	2	(0.149)** (0.356)(9.318) = 0.494 (0.306)(9.297) = 0.424	0.459
	3	(0.477)(9.379) = 0.667 (0.410)(9.349) = 0.571	0.619
	4	(0.581)(9.431) = 0.816 (0.494)(9.391) = 0.691	0.745
	5	(0.665)(9.472) = 0.939 (0.565)(9.427) = 0.794	0.866
	6	(0.736)(9.508) = 1.043 (0.618)(9.453) = 0.870	0.957
	7	(0.789)(9.535) = 1.121 (0.671)(9.480) = 0.948	1.034
	8	(0.842)(9.561) = 1.200 (0.716)(9.502) = 1.014	1.107
	9	(0.887)(9.583) = 1.267 (0.742)(9.515) = 1.052	1.159
	10	(0.913)(9.596) = 1.305 (0.775)(9.532) = 1.101	1.203
	11	(0.946)(9.613) = 1.355 (0.790)(9.539) = 1.123	1.239
	12	(0.961)(9.621) = 1.378 (0.816)(9.552) = 1.161	1.269
	13	(0.987)(9.633) = 1.147 (0.838)(9.563) = 1.194	1.305
	14	(1.009)(9.645) = 1.450 (0.182)(9.235) = 0.250	0.850

* For the time period between irrigations

** $2\pi K/L = 2\pi(11.6)/488$

Table 7–2b.–*Discharge computations for the example 5-year crop rotation program (U.S. customary units). (Sheet 1 of 4.) 103-D-1680-1.*

Crop	Irrigation Number	Discharge (q), $ft^3/ft/day$	Average Discharge $ft^3/ft/day$ *
Alfalfa	16	(0.149)**(4.00)(32.00) = 19.1 (3.25)(31.62) = 15.3	
	1	(3.81)(31.91) = 18.1 (0.84)(30.42) = 3.8	
	2	(1.40)(30.70) = 6.4 (1.04)(30.52) = 4.7	
	3	(1.60)(30.80) = 7.3 (1.08)(30.54) = 4.9	
	4	(1.64)(30.82) = 7.5 (1.06)(30.53) = 4.8	
	5	(1.62)(30.80) = 7.5 (1.04)(30.52) = 4.7	6.1
	6	(1.61)(30.80) = 7.4 (1.06)(30.53) = 4.8	6.1
	7	(1.62)(30.81) = 7.5 (1.36)(30.68) = 6.2	6.8
	8	(1.92)(30.96) = 8.9 (1.64)(30.81) = 7.5	8.2
	9	(2.20)(31.09) = 10.1 (1.87)(30.93) = 8.6	9.3
	10	(2.43)(31.21) = 11.2 (2.06)(31.06) = 9.5	10.3
	11	(2.62)(31.31) = 12.2 (2.23)(31.11) = 10.3	11.2
	12	(2.79)(31.40) = 13.1 (2.37)(31.18) = 11.0	12.1
	13	(2.93)(31.47) = 13.7 (2.49)(31.24) = 11.5	12.6
	14	(3.05)(31.53) = 14.3 (2.80)(31.39) = 13.0	13.6
	15	(3.36)(31.68) = 15.7 (3.10)(31.53) = 14.4	15.1
	16	(3.66)(31.83) = 17.2 (2.96)(31.48) = 13.9	15.5
	1	(3.52)(31.76) = 16.6 (0.79)(30.40) = 3.6	10.1
	2	(1.35)(30.67) = 6.2 (1.00)(30.50) = 4.5	5.8
	3	(1.56)(30.78) = 7.1 (1.05)(30.52) = 4.8	6.0
	4	(1.61)(30.81) = 7.4 (1.04)(30.52) = 4.8	6.0
	5	(1.60)(30.80) = 7.3 (0.51)(30.25) = 2.3	4.8
	7	(1.02)(30.51) = 4.6 (0.89)(30.44) = 4.0	4.3

* For the time period between irrigations

** $2\pi K/L = 2\pi(38)/1600$

Table 7–2b.–*Discharge computations for the example 5-year crop rotation program (U.S. customary units). (Sheet 2 of 4.) 103-D-1680-2.*

Crop	Irrigation Number	Discharge (q), $ft^3/ft/day$	Average Discharge $ft^3/ft/day$ *
	8	(0.149)** (1.40)(30.70) = 6.4	6.0
		(1.25)(30.62) = 5.7	
	9	(1.76)(30.88) = 8.1	7.8
		(1.62)(30.81) = 7.4	
	10	(2.13)(31.06) = 9.9	9.5
		(1.96)(30.98) = 9.0	
	11	(2.47)(31.23) = 11.5	11.0
		(2.27)(31.13) = 10.5	
	12	(2.78)(31.39) = 13.0	12.5
		(2.56)(31.28) = 11.9	
	13	(3.07)(31.53) = 14.4	13.8
		(2.82)(31.41) = 13.2	
	14	(3.34)(31.67) = 15.8	15.1
Safflower		(3.07)(31.53) = 14.4	
	15	(3.58)(31.79) = 17.0	16.2
		(3.29)(31.64) = 15.5	
	16	(3.80)(31.90) = 18.1	17.3
		(3.49)(31.74) = 16.5	
	17	(4.00)(32.00) = 19.1	18.3
		(3.68)(31.84) = 17.5	
	18	(4.19)(32.10) = 20.0	19.2
		(3.84)(31.92) = 18.3	
	19	(4.36)(32.18) = 20.9	20.0
		(3.99)(32.00) = 19.0	
	20	(4.50)(32.25) = 21.6	13.3
		(1.08)(30.54) = 4.9	
	1	(1.59)(30.79) = 7.3	6.7
		(1.35)(30.67) = 6.2	
	2	(1.86)(30.93) = 8.6	8.2
		(1.70)(30.85) = 7.8	
	3	(2.21)(31.10) = 10.2	9.8
Vegetables		(2.03)(31.02) = 9.4	
	4	(2.54)(31.27) = 11.8	11.4
		(2.34)(31.17) = 10.9	
	5	(2.85)(31.43) = 13.3	12.8
		(2.62)(31.31) = 12.2	
	6	(3.13)(31.57) = 14.7	9.6
		(0.98)(30.49) = 4.5	
	1	(1.49)(30.74) = 6.8	4.1
		(0.31)(30.15) = 1.4	
Cotton	2	(0.82)(30.41) = 3.7	3.3
		(0.62)(30.31) = 2.8	
	3	(1.13)(30.56) = 5.1	4.8
		(0.98)(30.49) = 4.5	
	4	(1.49)(30.74) = 6.8	6.3
		(1.30)(30.65) = 5.9	

* For the time period between irrigations

** $2\pi K/L = 2\pi(38)/1600$

Table 7–2b.–*Discharge computations for the example 5-year crop rotation program (U.S. customary units). (Sheet 3 of 4.) 103-D-1680-3.*

Crop	Irrigation Number	Discharge (q), ft^3/ft/day	Average Discharge ft^3/ft/day *
Cotton	5	(0.149)**(1.81)(30.90) = 8.3	7.8
		(1.57)(30.79) = 7.2	
	6	(2.08)(31.04) = 9.6	9.0
		(1.81)(30.90) = 8.3	
	7	(2.32)(31.16) = 10.8	10.0
		(2.01)(31.00) = 9.3	
	8	(2.52)(31.26) = 11.7	10.9
		(2.18)(31.09) = 10.1	
	9	(2.69)(31.34) = 12.6	11.7
		(2.33)(31.16) = 10.8	
	10	(2.84)(31.42) = 13.3	12.4
		(2.46)(31.23) = 11.4	
	11	(2.97)(31.48) = 13.9	7.9
		(0.43)(30.21) = 1.9	
Barley	1	(0.89)(30.45) = 4.0	3.3
		(0.57)(30.28) = 2.6	
	2	(1.02)(30.51) = 4.6	4.1
		(0.79)(30.40) = 3.6	
	3	(1.25)(30.63) = 5.7	5.1
		(0.98)(30.50) = 4.5	
	4	(1.44)(30.72) = 6.6	6.1
		(1.24)(30.62) = 5.7	
	5	(1.70)(30.85) = 7.8	7.2
		(1.46)(30.73) = 6.7	
	6	(1.92)(30.96) = 8.9	8.2
		(1.64)(30.82) = 7.5	
	7	(2.10)(31.05) = 9.7	6.1
		(0.55)(30.27) = 2.5	1.6
		(0.15)(30.07) = 0.7	
Vegetables	1	(0.66)(30.33) = 3.0	2.8
		(0.57)(30.28) = 2.6	
	2	(1.08)(30.54) = 4.9	4.6
		(0.93)(30.46) = 4.2	
	3	(1.44)(30.72) = 6.6	6.3
		(1.32)(30.66) = 6.0	
	4	(1.83)(30.91) = 8.4	8.1
		(1.69)(30.84) = 7.8	
	5	(2.20)(31.10) = 10.2	9.8
		(2.03)(31.01) = 9.4	
	6	(2.54)(31.27) = 11.8	11.4
		(2.34)(31.17) = 10.9	
	7	(2.85)(31.42) = 13.3	7.9
		(0.54)(30.33) = 2.4	
	1	(1.10)(30.61) = 5.0	3.9
		(0.61)(30.33) = 2.8	

* For the time period between irrigations

** $2\pi K/L = 2\pi(38)/1600$

Table 7–2b.–*Discharge computations for the example 5-year crop rotation program (U.S. customary units). (Sheet 4 of 4.) 103-D-1680-4.*

Crop	Irrigation Number	Discharge (q), $ft^3/ft/day$	Average Discharge $ft^3/ft/day$ *
Bermuda	2	$(0.149)^{**}(1.17)(30.61) = 5.3$ $(1.01)(30.52) = 4.6$	5.0
	3	$(1.57)(30.80) = 7.2$ $(1.35)(30.68) = 6.2$	6.7
	4	$(1.91)(30.96) = 8.8$ $(1.62)(30.82) = 7.4$	8.1
	5	$(2.18)(31.10) = 10.1$ $(1.85)(30.94) = 8.5$	9.3
	6	$(2.41)(31.22) = 11.2$ $(2.02)(31.02) = 9.3$	10.3
	7	$(2.58)(31.30) = 12.0$ $(2.19)(31.10) = 10.1$	11.1
	8	$(2.75)(31.39) = 12.9$ $(2.34)(31.18) = 10.9$	11.8
	9	$(2.90)(31.46) = 13.6$ $(2.42)(31.21) = 11.3$	12.4
	10	$(2.98)(31.50) = 14.0$ $(2.53)(31.27) = 11.8$	12.9
	11	$(3.09)(31.55) = 14.6$ $(2.58)(31.29) = 12.0$	13.3
	12	$(3.14)(31.57) = 14.8$ $(2.67)(31.33) = 12.5$	13.6
	13	$(3.23)(31.61) = 15.2$ $(2.74)(31.37) = 12.8$	14.0
	14	$(3.30)(31.65) = 15.6$ $(0.59)(30.30) = 2.8$	9.2

* For the time period between irrigations
** $2\pi K/L = 2\pi(38)/1600$

Assuming the area under consideration contains 1510 hectares (3,730 acres) of irrigated land, then the discharge during January is:

$$(342.4)(1510) = 516\ 860 \text{ cubic meters (419 acre-feet)}$$

Table 7–3 shows the discharge for each month in the year, and table 7–4 shows the recharge for each crop.

Table 7–3.—*Monthly distribution of discharge from 1510 hectares (3,730 acres).*

Month	Discharge hectare-meters	Discharge acre-feet	Month	Discharge hectare-meters	Discharge acre-feet
January	51.8	420	July	100.5	815
February	41.0	332	August	88.2	715
March	50.0	405	September	75.2	610
April	64.9	526	October	74.0	600
May	81.2	658	November	76.7	622
June	92.5	750	December	69.7	565
			Total	865.7	7,018

Table 7–4.—*Recharge by crop.*

Crop	Number of annual irrigations	Recharge: Per irrigation millimeters	Recharge: Per irrigation inches	Recharge: Annually meters	Recharge: Annually feet
Alfalfa	16	39.1	1.54	0.625	2.05
Safflower	14	35.6	1.40	0.497	1.63
Vegetables	6	35.6	1.40	0.213	0.70
Cotton	11	35.6	1.40	0.390	1.28
Barley	7	32.0	1.26	0.225	0.74
Vegetables	7	35.6	1.40	0.250	0.82
Bermuda	14	39.1	1.54	0.549	1.80
			Total	2.749	9.02

Average per hectare (acre) annual $= \frac{2.749}{5} = 0.550$ meter (1.80)

The annual recharge for the 1510 hectares (3,730 acres) is then 1510×0.55 $(3{,}730 \times 1.80) = 830.5$ hectare meters (6,733 acre-feet), which compares favorably with the computed annual discharge of 8657 cubic dekameters (7,018 acre-feet). The annual discharge is within about 4 percent of the annual recharge, which indicates that dynamic equilibrium essentially exists under the specified conditions.

The Bureau of Reclamation has developed computer programs using this concept to analyze water table buildup from present water table positions to levels where dynamic equilibrium is reached. This concept allows the drainage engineer to develop highly sophisticated models to estimate quantity and quality of return flows from irrigation projects.

7–2. Two-Layer Aquifers.—Drains should always be installed in the most permeable zone that is within an economical excavation depth, usually within about 3 meters (10 feet) of the ground surface. Often fine-textured soils overlie soils of much higher permeability. When the more permeable zone is too deep to reach with normal drain construction equipment, the drain must be installed in the less permeable material. However, this type of two-layer drainage can work efficiently. Sand tank models have shown that the water moves vertically down to the more permeable layer, horizontally through the permeable layer, then back up almost vertically to the drain, as shown on figure 7–6. On projects with two-layered conditions, Reclamation has used numerical models to generate drain spacings for representative conditions for the area. No general solution with proven reliability over a wide range of conditions has been developed.

7–3. Moody's Nonlinear Solutions.—Chapter 5 presents the Bureau of Reclamation's transient drain spacing method from a practical application standpoint. For design purposes, the transient solution has been reduced to two dimensionless curves, one for drain on barrier and one for drain above barrier.

Section 5–3f gives criteria for choosing the proper case (on barrier or above barrier) for design purposes and introduces the concept of "a family of curves" between the two limiting curves but suggests such refinement is of little practical application.

W. T. Moody (1966) solved the general nonlinear problem using a numerical solution based on finite difference methods for intermediate cases and for drains on barrier. His results are given as three families of curves representing: (a) dimensionless water table height versus dimensionless time, (b) dimensionless discharge versus dimensionless time, and (c) dimensionless volume of water removed versus dimensionless time. Within a curve family, Moody introduced the curve parameter, m, to represent the ratio of initial maximum water table height above drain level to the corresponding height above barrier. For drains on barrier, $m = 1$, and for drains far above the barrier, $m = 0$. Thus, in varying m from zero to one, the entire range of possibilities is represented.

Moody's work is a powerful extension to Reclamation's drain-spacing method as the work contributes to the overall understanding of hydraulics of spacing when the drains are near the barrier. The three families of curves are presented here in support of the practical applications discussed in chapter 5.

7–4. Agricultural Drainage Planning Program (ADPP).—ADPP is a menu-driven computer program that assists drainage system design and the analysis of existing drainage systems. ADPP has two components: "Drainage Design Under Uncertainty," a risk analysis program that uses Donnan's

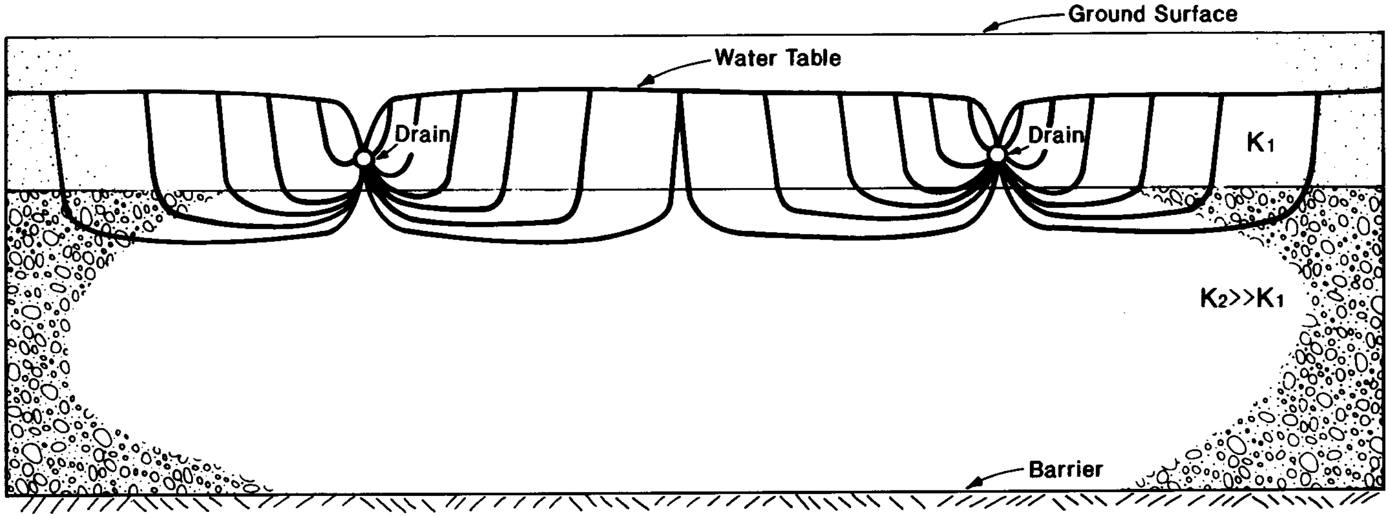


Figure 7–6.—Water movement in two-layer aquifers.

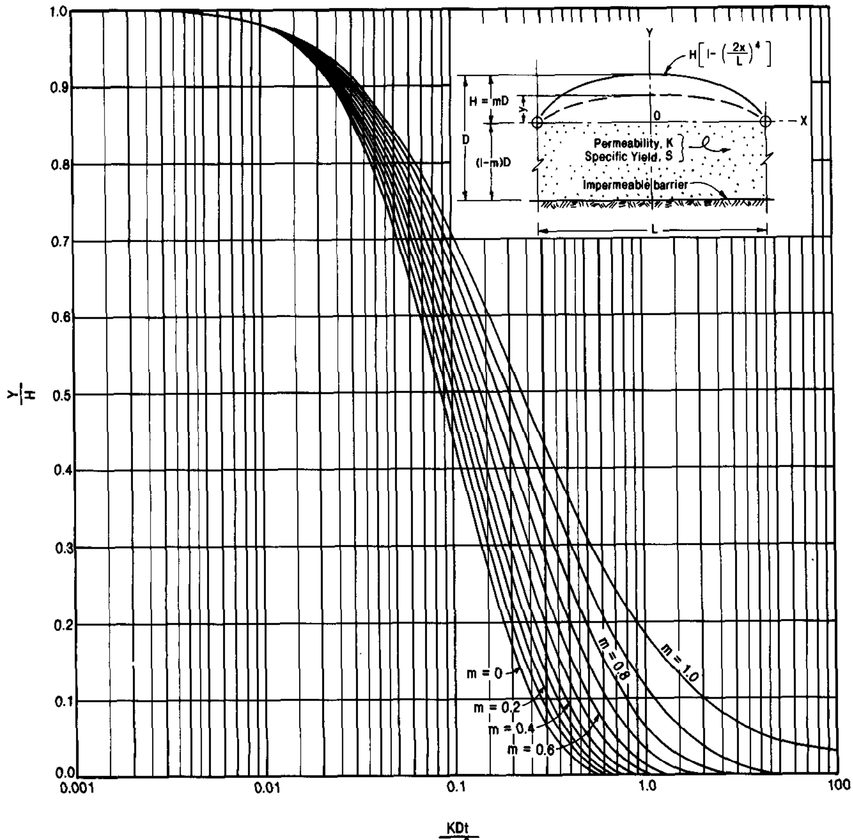


Figure 7–7.—Dimensionless curves of maximum water-table height, y, versus time, t, for parallel drains at various distances above an impermeable barrier.

Steady-State Equation; and "Transient-State Drain Spacing," a program that uses the Glover transient-state equation to compute drain spacings.

"Drainage Design Under Uncertainty" should be used to assess the reliability of a range of designs or a specific design. "Transient-State Drain Spacing" should be used for the drain system design.

The program is based on procedures described in this manual. ADPP is written in FORTRAN and is compiled to run on MS-DOS computer systems. It can run on an IBM XT, IBM AT, or larger compatible personal computer.

The software is contained on three floppy disks packaged with a user's manual. The complete package is available through the Superintendent of Documents, U.S. Government Printing Office.

The "Transient-State Drain Spacing" component uses the transient-state equation for drain spacing as developed by Lee Dumm, Ray Winger, Jr., and Robert Glover of the U.S. Bureau of Reclamation. Hooghoudt's Correction for Convergence is used to account for convergence loss.

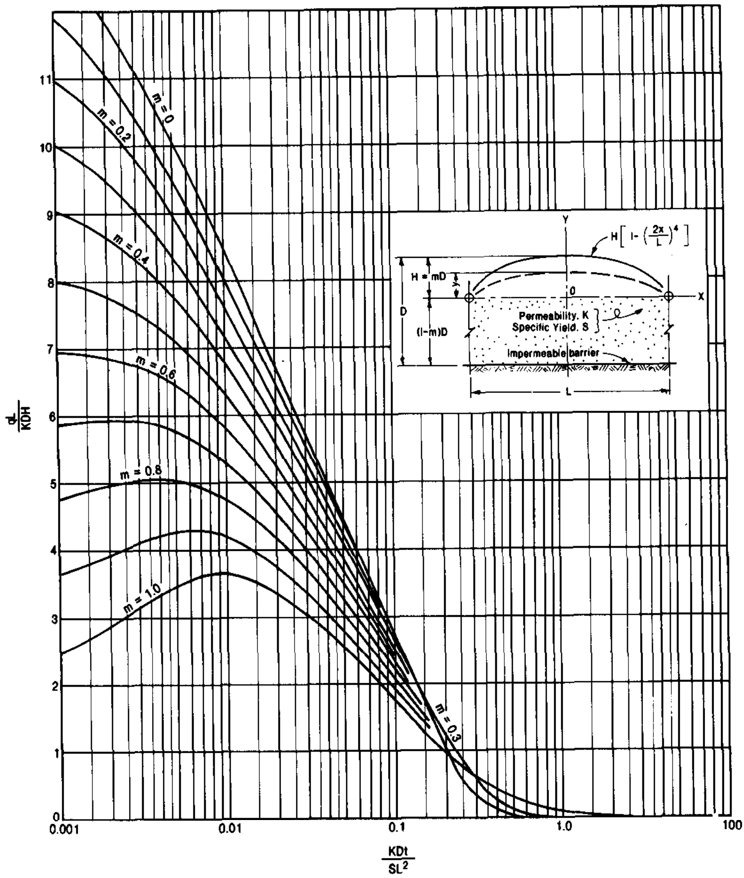


Figure 7–8.—Dimensionless curves of rate of discharge, q, versus time, t, for parallel drains at various distances above an impermeable barrier.

The program will calculate a drain spacing and provide a table showing computation of water table fluctuation. The table of water table fluctuation is similar to tables 5–3 and 5–4. Using the drain spacing (computed or entered by user), the table shows the buildup per irrigation, the height of the water table (Y_o), the flow conditions during a drain-out period (KDt/SL^2), and the midpoint water table height above drain at the end of each drain-out period (Y). The user can use this table to determine the drain spacing effectiveness.

This program may be used to obtain drain spacings based on the field data and the deep percolation. In those cases where there are physical constraints on the "ideal" drain design, this component will provide information on the water table for different drain spacings and/or depths, allowing the user to make a more informed decision on design. It has also been found useful to calculate drain

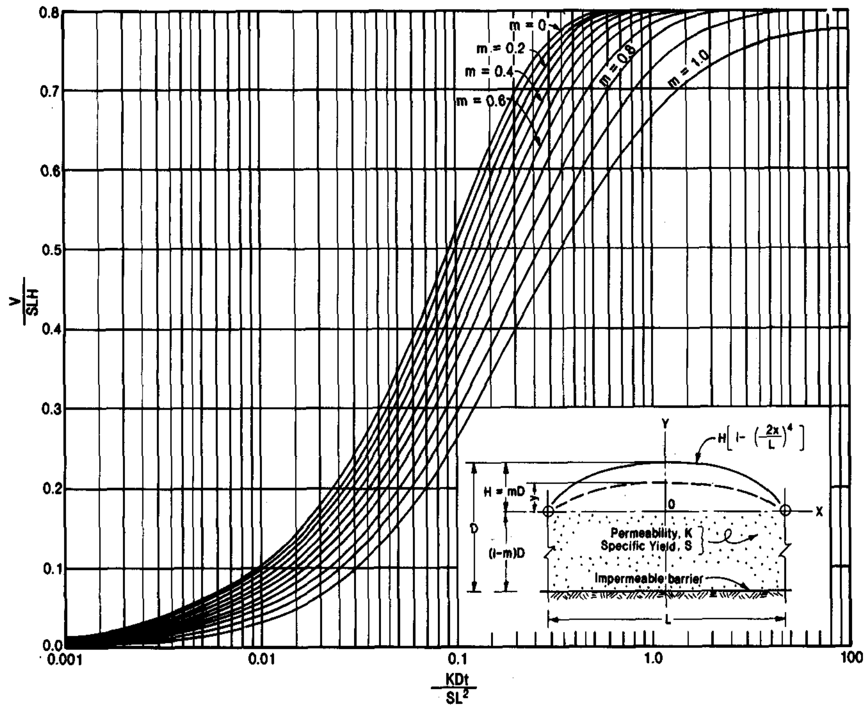


Figure 7–9.—Dimensionless curves of volume of water removed, V, versus time, t, for parallel drains at various distances above an impermeable barrier.

spacings with the transient-state analysis component, and then employ the uncertainty component to determine the reliability of the spacing.

Data required to use the program to compute drain spacing are:

(a) Permeability, in meters (feet) per day.

(b) The maximum allowable water table above the drain at the midpoint of the drain in meters (feet).

(c) The distance from the drain to barrier, in meters (feet).

(d) Specific yield, a decimal number.

(e) The radius of the drain, including pipe and gravel envelope, in meters (feet).

(f) Depth to the drain, in meters (feet).

(g) Schedule of deep percolation events by month and day.

(h) Deep percolation amount for each event in millimeters (inches).

The "Drainage Design Under Uncertainty" component is based on Donnan's steady-state equation. Normal design procedures use average site values for hydraulic conductivity (K), depth of flow zone (D), and recharge rate (Q_d). The use of these values results in a computed drain spacing which should control the depth to water table at a desired level. A problem with using average values of system performance is that they give no information about the expected variation of actual performance but the average value.

The risk approach to drain design addresses the uncertainties (normal variations) of K, D, and Q_d, and expresses them as the uncertainty of drain performance. Drain performance is measured as its effectiveness in controlling depth to water table. This analysis uses the FOSM (first-order, second moment) approach. The FOSM method assumes that the information contained in the mean value and the variance is sufficient to describe the uncertainty in the problem. For a more detailed description, the reader is referred to Garcia and Strzepek (1985) and Strzepek, Garcia, and Christopher (1987).

The drain design approach developed in this package allows the designer to look at the reliability of the drain to meet the specified depth to water table given the normal variation in the input parameters. The analytical package will also provide for the least cost design for each level of reliability. The cost model used to develop the least cost design is described in section 5–34 of this manual.

The package for drain design assumes that the designer has performed all the data collection and analysis. The package requires the designer to have the mean and variance on soil parameters, a design value of the depth to the water table, and all economic and physical data for the design process.

(a) Field Data.—Eight data items are requested:

(1) Type of pipe–plastic, concrete, or clay.
(2) Drain radius in meters (feet).
(3) Depth to barrier from drain, d, in meters (feet).
(4) Standard deviation of, d, in meters (feet).
(5) Hydraulic conductivity of soils, K, in meters/day (feet/day).
(6) Standard deviation of K in meters/day (feet/day).
(7) Recharge rate, Q, in meters/day (feet/day).
(8) Standard deviation of Q in meters/day (feet/day).

Reliable analysis of a drainage system requires that these data be site specific and be based on field measurements.

(b) Cost Data.—Information requested is:

(1) Interest rate to be used (percent).
(2) Life of the system in years.
(3) Cost of operation and maintenance per linear meter (foot).

The interest rate is to be entered as a percentage, not a decimal number (i.e., if 8 percent, use 8, not 0.08). These data are used in cost analysis by the program.

(c) Pipe Cost.—Pipe costs can be computed as an average cost of all sizes of pipe, or as a distribution of various pipe sizes.

(d) Data for Trenching Machines.—This screen requests the type of machine that will be used to install drains. Two types are used by the program: a constant-speed machine and a variable-speed machine.

(1) Constant speed.—If this option is chosen, the program requests the rate of installation in meters/minute (feet/minute) and the cost per minute of installation.

(2) Variable speed.—If this option is chosen, the program requests the maximum depth of installation in meters (feet), the minimum rate of installation, the cost per minute of installation, and the slope of depth versus installation rate (a decimal number). The normal range of values for most trenchers is 0.10 to 0.20.

(e) Analysis Evaluation.—The user is given the option of entering a drain design for a risk analysis or requesting an analysis of a range of drain designs.

If the user decides to enter a drain design, the program prompts for the spacing to be considered, the depth to be considered, and the critical depth to water. The critical depth to water is the allowable height of water above the drain at midpoint between drains. As used in the program, the critical depth to water may not be exceeded. This technique results in a very conservative drain spacing and a deeper drain depth.

If the user requests an analysis of a range of designs, the program prompts for minimum depth, maximum depth, increments in depth, minimum spacing, maximum spacing, and increments in spacing. The smaller the increments given, the longer the program will take to calculate.

(f) Uncertainty Analysis Option.—The user may request that the uncertainty analysis be calculated on a risk analysis of the reliability of the drains or on a loss function analysis.

The risk analysis option looks at the reliability of the drainage design in maintaining a water table that is kept within the critical depth to water. The user is given an option of finding a given reliability or of producing a table and graph of reliability versus cost. If the user requests a given reliability, the program prompts for the reliability. This reliability is a percentage, not a decimal number. If the user requests a table of reliability versus cost, the program prompts for the minimum reliability, the maximum reliability, and the increment to be used.

When using the risk analysis portion of ADPP, the user should bear in mind that the values are relative. Also, that the dollar value of crop loss for each increment of water table rise above the control level is subjective. For the traditional Reclamation drain system design, the reliability range is 55 to 65 percent. This range means that there is a 60-percent chance that the water table will never exceed the design control level.

This portion of the program is most useful for assisting designers and managers in determining the relative level of risk they are assuming in using a given quality of data for system design. The value versus cost of collecting better quality design data can be evaluated. Reclamation plans to use the Risk Analysis Program as a tool to aid drainage engineers in evaluating data collection needs which result in a successful drain system design.

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APPENDIX

SI METRIC

INTERNATIONAL SYSTEM (SI METRIC)/U.S. CUSTOMARY CONVERSION TABLES

Length

To convert from	*To*	*Multiply by*
angstrom units	nanometers (nm)	0.1
	micrometers (μm)	1.0×10^{-4}
	millimeters (mm)	1.0×10^{-7}
	meters (m)	1.0×10^{-10}
	mils	$3.937\ 01 \times 10^{-6}$
	inches (in)	$3.937\ 01 \times 10^{-9}$
micrometers	millimeters	1.0×10^{-3}
	meters	1.0×10^{-6}
	angstrom units (Å)	1.0×10^{4}
	mils	0.039 37
	inches	$3.937\ 01 \times 10^{-5}$
millimeters	micrometers	1.0×10^{3}
	centimeters (cm)	0.1
	meters	1.0×10^{-3}
	mils	39.370 08
	inches	0.039 37
	feet (ft)	$3.280\ 84 \times 10^{-3}$
centimeters	millimeters	10.0
	meters	0.01
	mils	0.3937×10^{3}
	inches	0.3937
	feet	0.032 81
inches	millimeters	25.40
	meters	0.0254
	mils	1.0×10^{3}
	feet	0.083 33

feet	millimeters	304.8
	meters	0.3048
	inches	12.0
	yards (yd)	0.333 33
yards	meters	0.9144
	inches	36.0
	feet	3.0
meters	millimeters	1.0×10^{3}
	kilometers (km)	1.0×10^{-3}
	inches	39.370 08
	feet	3.28
	yards	1.093 61
	miles (mi)	$6.213\ 71 \times 10^{-4}$
kilometers	meters	1.0×10^{3}
	feet	$3.280\ 84 \times 10^{3}$
	miles	0.621 37
miles	meters	$1.609\ 34 \times 10^{3}$
	kilometers	1.609 34
	feet	5280.0
	yards	1760.0
nautical miles (nmi)	kilometers	1.8520
	miles	1.1508

Area

To convert from	*To*	*Multiply by*
square millimeters	square centimeters (cm^2)	0.01
	square inches (in^2)	1.550×10^{-3}
square centimeters	square millimeters (mm^2)	100.0
	square meters (m^2)	1.0×10^{-4}
	square inches	0.1550
	square feet (ft^2)	$1.076\ 39 \times 10^{-3}$
square inches	square millimeters	645.16
	square centimeters	6.4516
	square meters	6.4516×10^{-4}
	square feet	69.444×10^{-4}

square feet	square meters	0.0929
	hectares (ha)	9.2903×10^{-6}
	square inches	144.0
	acres	$2.295\ 68 \times 10^{-5}$
square yards	square meters	0.836 13
	hectares	8.3613×10^{-5}
	square feet	9.0
	acres	$2.066\ 12 \times 10^{-4}$
square meters	hectares	1.0×10^{-4}
	square feet	10.763 91
	acres	2.471×10^{-4}
	square yards (yd^2)	1.195 99
acres	square meters	4046.8564
	hectares	0.404 69
	square feet	4.356×10^{4}
hectares	square meters	1.0×10^{4}
	acres	2.471
square kilometers	square meters	1.0×10^{6}
	hectares	100.0
	square feet	107.6391×10^{5}
	acres	247.105 38
	square miles (mi^2)	0.3861
square miles	square meters	$258.998\ 81 \times 10^{4}$
	hectares	258.998 81
	square kilometers (km^2)	2.589 99
	square feet	$2.787\ 84 \times 10^{7}$
	acres	640.0

Volume—Capacity

To convert from	*To*	*Multiply by*
cubic millimeters	cubic centimeters (cm^3)	1.0×10^{-3}
	liters (L)	1.0×10^{-6}
	cubic inches (in^3)	$61.023\ 74 \times 10^{-6}$
cubic centimeters	liters	1.0×10^{-3}
	milliliters (mL)	1.0
	cubic inches	$61.023\ 74 \times 10^{-3}$
	fluid ounces (fl oz)	33.814×10^{-3}

milliliters	liters	1.0×10^{-3}
	cubic centimeters	1.0
cubic inches	milliliters	16.387 06
	cubic feet (ft^3)	$57.870\ 37 \times 10^{-5}$
liters	cubic meters	1.0×10^{-3}
	cubic feet	0.035 31
	gallons	0.264 17
	fluid ounces	33.814
gallons	liters	3.785 41
	cubic meters	$3.785\ 41 \times 10^{-3}$
	fluid ounces	128.0
	cubic feet	0.133 68
cubic feet	liters	28.316 85
	cubic meters (m^3)	$28.316\ 85 \times 10^{-3}$
	cubic dekameters (dam^3)	$28.316\ 85 \times 10^{-6}$
	cubic inches	1728.0
	cubic yards (yd^3)	$37.037\ 04 \times 10^{-3}$
	gallons (gal)	7.480 52
	acre-feet (acre-ft)	$22.956\ 84 \times 10^{-6}$
cubic miles	cubic dekameters	$4.168\ 18 \times 10^{6}$
	cubic kilometers (km^3)	4.168 18
	acre-feet	3.3792×10^{6}
cubic yards	cubic meters	0.764 55
	cubic feet	27.0
cubic meters	liters	1.0×10^{3}
	cubic dekameters	1.0×10^{-3}
	gallons	264.1721
	cubic feet	35.314 67
	cubic yards	1.307 95
	acre-feet	8.107×10^{-4}
acre-feet	cubic meters	1233.482
	cubic dekameters	1.233 48
	cubic feet	43.560×10^{3}
	gallons	325.8514×10^{3}
cubic dekameters	cubic meters	1.0×10^{3}
	cubic feet	$35.314\ 67 \times 10^{3}$
	acre-feet	0.810 71
	gallons	$26.417\ 21 \times 10^{4}$

cubic kilometers	cubic dekameters	1.0×10^6
	acre-feet	$0.810\,71 \times 10^6$
	cubic miles (mi^3)	0.239 91

Temperature

degrees Celsius (°C)	t_c
kelvin (K)	t_k
degrees Fahrenheit (°F)	t_f
degrees Rankine (R)	t_r

$$t_c = (t_f - 32)/1.8$$
$$= t_k - 273.15$$

$$t_k = t_c + 273.15$$
$$= (t_f + 459.67)/1.8$$
$$= t_r/1.8$$

$$t_f = t_c/1.8 + 32$$

$$t_r = 1.8\, t_k$$
$$= 1.8\, t_c + 491.68$$

Acceleration

To convert from	*To*	*Multiply by*
feet per second squared	meters per second squared (m/s^2)	0.3048
	G's	0.031 08
meters per second squared	feet per second squared (ft/s^2)	3.280 84
	G's	0.101 97
G's (standard gravitational acceleration)	meters per second square	9.806 65
	feet per second square	32.174 05

Velocity

To convert from	*To*	*Multiply by*
feet per second	meters per second (m/s)	0.3048
	kilometers per hour (km/h)	1.097 28
	miles per hour (mi/h)	0.681 82

meters per second	kilometers per hour	3.60
	feet per second (ft/s)	3.280 84
	miles per hour	2.236 94
kilometers per hour	meters per second	0.277 78
	feet per second	0.911 34
	miles per hour	0.621 47
miles per hour	kilometers per hour	1.609 34
	meters per second	0.447 04
	feet per second	1.466 67
feet per year (ft/yr)	millimeters per second (mm/s)	$9.665\ 14 \times 10^{-6}$
feet per day	centimeters per second	3.505×10^{-4}

Force

To convert from	*To*	*Multiply by*
pounds	newtons (N)	4.4482
kilograms	newtons	9.806 65
	pounds (lb)	2.2046
newtons	pounds	0.224 81
dynes	newtons	1.0×10^{-5}

Mass

To convert from	*To*	*Multiply by*
grams	kilograms (kg)	1.0×10^{-3}
	ounces (avdp)	0.035 27
ounces (avdp)	grams (g)	28.349 52
	kilograms	0.028 35
	pounds (avdp)	0.0625
pounds (avdp)	kilograms	0.453 59
	ounces (avdp)	16.00
kilograms	kilograms (force)–second squared per meter ($kgf{\bullet}s^2/m$)	0.101.97
	pounds (avdp)	2.204 62
	slugs	0.068 52

slugs	kilograms	14.5939
short tons	kilograms	907.1847
	metric tons (t)	0.907.18
	pounds (avdp)	2000.0
metric tons (tonne or megagram)	kilograms	1.0×10^3
	pounds (avdp)	$2.204\ 62 \times 10^3$
	short tons	1.102 31
long tons	kilograms	1016.047
	metric tons	1.016 05
	pounds (avdp)	2240.0
	short tons	1.120

Volume per Unit Time
Flow

To convert from	*To*	*Multiply by*
cubic feet per second	liters per second (L/s)	28.316 85
	cubic meters per second (m^3/s)	0.028 32
	cubic dekameters per day (dam^3/d)	2.446 57
	gallons per minute (gal/min)	448.831 17
	acre-feet per day (acre-ft/d)	1.983 47
	cubic feet per minute (ft^3/min)	60.0
gallons per minute	cubic meters per second	0.631×10^{-4}
	liters per second	0.0631
	cubic dekameters per day	5.451×10^{-3}
	cubic feet per second (ft^3/s)	2.228×10^{-3}
	acre-feet per day	4.4192×10^{-3}
acre-feet per day	cubic meters per second	0.014 28
	cubic dekameters per day	1.233 48
	cubic feet per second	0.504 17
cubic dekameters per day	cubic meters per second	0.011 57
	cubic feet per second	0.408 74
	acre-feet per day	0.810 71

Viscosity

To convert from	*To*	*Multiply by*
centipoise	pascal-second (Pa•s)	1.0×10^{-3}
	poise	0.01
	pound per foot-hour (lb/ft•h)	2.419 09
	pound per foot-second (lb/ft•s)	$6.719\ 69 \times 10^{-4}$
	slug per foot-second (slug/ft•s)	$2.088\ 54 \times 10^{-5}$
pascal-second	centipoise	1000.0
	pound per foot-hour	$2.419\ 09 \times 10^{3}$
	pound per foot-second	0.671 97
	slug per foot-second	20.8854×10^{-3}
pound per foot-hour	pascal-second	$4.133\ 79 \times 10^{-4}$
	pound per foot-second	$2.777\ 78 \times 10^{-4}$
	centipoise	0.413 38
pounds per foot-second	pascal-second	1.488 16
	slug per foot-second	31.0809×10^{-3}
	centipoise	$1.488\ 16 \times 10^{3}$
centistokes	square meters per second (m^2/s)	1.0×10^{-6}
	square feet per second (ft^2/s)	$10.763\ 91 \times 10^{-6}$
	stokes	0.01
square feet per second	square meters per second	9.2903×10^{-2}
	centistokes	9.2903×10^{4}
stokes	square meters per second	1.0×10^{-4}
rhe	1 per pascal-second (1/Pa•s)	10.0

Force per Unit Area
Pressure—Stress

To convert from	*To*	*Multiply by*
pounds per square inch	kilopascals (kPa)	6.894 76
	[1]meters-head	0.703 09
	[2]mm of Hg	51.7151
	[1]feet of water	2.3067
	pounds per square foot (lb/ft^2)	144.0
	std. atmospheres	68.046×10^{-3}
pounds per square foot	kilopascals	0.047 88
	[1]meters-head	4.8826×10^{-3}
	[2]mm of Hg	0.359 13
	[1]feet of water	16.0189×10^{-3}
	pounds per square inch	6.9444×10^{-3}
	std. atmospheres	$0.472\,54 \times 10^{-3}$
short tons per square foot	kilopascals	95.760 52
	pounds per square inch (lb/in^2)	13.888 89
[1]meters-head	kilopascals	9.806 36
	[2]mm of Hg	73.554
	[1]feet of water	3.280 84
	pounds per square inch	1.422 29
	pounds per square foot	204.81
[1]feet of water	kilopascals	2.998 98
	[1]meters-head	0.3048
	[2]mm of Hg	22.4193
	[2]inches of Hg	0.882 65
	pounds per square inch	0.433 51
	pounds per square foot	62.4261
kilopascals	newtons per square meter (N/m^2)	1.0×10^{-3}
	[2]mm of Hg	7.500 64
	[1]meters-head	0.101 97
	[2]inches of Hg	0.2953
	pounds per square foot	20.8854
	pounds per square inch	0.145 04
	std. atmospheres	9.8692×10^{-3}

kilograms (f) per square meter	kilopascals	$9.806\,65 \times 10^{-3}$
	[2]mm of Hg	73.556×10^{-3}
	pounds per square inch	1.4223×10^{-3}
millibars (mbar)	kilopascals	0.10
bars	kilopascals	100.0
std. atmospheres	kilopascals	101.325
	[2]mm of Hg	760.0
	pounds per square inch	14.70
	[1]feet of water	33.90

Mass per Unit Volume
Density and Mass Capacity

To convert from	*To*	*Multiply by*
pounds per cubic foot	kilograms per cubic meter (kg/m^3)	16.018 46
	slugs per cubic foot ($slug/ft^3$)	0.031 08
	pounds per gallon (lb/gal)	0.133 68
pounds per gallon	kilograms per cubic meter (kg/m^3)	119.8264
	slugs per cubic foot	0.2325
pounds per cubic yard	kilograms per cubic meter	0.593 28
	pounds per cubic foot (lb/ft^3)	0.037 04
grams per cubic centimeter	kilograms per cubic meter	1.0×10^{3}
	pounds per cubic yard	1.6856×10^{3}
ounces per gallon (oz/gal)	grams per liter (g/L)	7.489 15
	kilograms per cubic meter	7.489 15
kilograms per cubic meter	grams per cubic centimeter (g/cm^3)	1.0×10^{-3}
	metric tons per cubic meter (t/m^3)	1.0×10^{-3}
	pounds per cubic foot (lb/ft^3)	62.4279×10^{-3}
	pounds per gallon	8.3454×10^{-3}
	pounds per cubic yard	1.685 56

[1] Column of H_2O (water) measured at 4 °C.
[2] Column of Hg (mercury) measured at 0 °C.

long tons per cubic yard	kilograms per cubic meter	1328.939
ounces per cubic inch (oz/in^3)	kilograms per cubic meter	1729.994
slugs per cubic foot	kilograms per cubic meter	515.3788

Volume per Area per Unit Time
[1]Hydraulic Conductivity (Permeability)

To convert from	*To*	*Multiply by*
cubic feet per square foot per day	cubic meters per square meter per day ($m^3/(m^2 \bullet d)$)	0.3048
	cubic feet per square foot per minute ($ft^3/(ft^2 \bullet min)$)	0.6944×10^{-3}
	liters per square meter per day ($L/(m^2 \bullet d)$)	304.8
	gallons per square foot per day ($gal/(ft^2 \bullet d)$)	7.480 52
	cubic millimeters per square millimeter per day ($mm^3/(mm^2 \bullet d)$)	304.8
	cubic millimeters per square millimeter per hour ($mm^3/(mm^2 \bullet h)$)	25.4
	cubic inches per square inch per hour ($in^3/(in^2 \bullet h)$)	0.5
gallons per square foot per day	cubic meter per square meter per day ($m^3/(m^2 \bullet d)$)	40.7458×10^{-3}
	liters per square meter per day ($L/(m^2 \bullet d)$)	40.7458
	cubic feet per square foot per day ($ft^3/(ft^2 \bullet d)$)	0.133 68

Volume per Length per Unit Time
[1]Transmissivity

To convert from	*To*	*Multiply by*
cubic feet per foot per day ($ft^3/(ft \cdot d)$)	cubic meters per meter per day ($m^3/(m \cdot d)$)	0.0929
	gallons per foot per day ($gal/(ft \cdot d)$)	7.480 52
	liters per meter per day ($L/(m \cdot d)$)	92.903
gallons per foot per day	cubic meter per meter per day ($m^3/(m \cdot d)$)	0.012 42
	cubic feet per foot per day ($ft^3/(ft \cdot d)$)	0.133 68

[1] Many of these units can be dimensionally simplified. For example, $m^3/(m \cdot d)$ can also be written m^2/d.

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