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DRAINAGE INVESTIGATIONS

DEVLET SU IŞLERI, TURKEY

A REPORT ON METHODS AND ANALYSIS

Prepared For The

United States Department of State

Agency For International Development

by

William O. Brown

Chief, Drainage Branch

Engineering Division

United States Department of the Interior, Bureau of Reclamation

McCook, Nebraska

PASA NESA-7-65

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
WASHINGTON, D.C. 20240

IN REPLY REFER TO 220

NOV 4-1965

Mr. Russell Dilts
Deputy Director, NESA/TECH
Agency for International Development
Department of State
Washington, D. C. 20523

Dear Mr. Dilts:

In accordance with PASA No. NESA-7-65 and P10/T No. 277-142-2-50007 and amendments, the Bureau of Reclamation provided the services of Mr. William O. Brown, Hydraulic Engineer (General) (Drainage), to the Government of Turkey, General Directorate of State Hydraulic Works. Mr. Brown provided consultive services and technical assistance on drainage in the investigation, planning, and construction of several specific development projects being undertaken by GOT forces. Mr. Brown arrived in Turkey May 15 and departed on August 1.

Enclosed is Mr. Brown's report. Specific recommendations on which follow-up should be provided by USAID/Turkey are as follows:

1. All reports of this nature should be translated immediately and given wide distribution to all interested GOT agencies and their engineers.

2. The effect of existing surface and subsurface drains should be studied for the purpose of gaining investigation and design experience.

3. All field data and computations should be checked for accuracy.

4. More geologic and geomorphological cross sections should be plotted to assist in analyzing subsurface characteristics of proposed and existing projects.

5. Reasons for gathering field data should be thoroughly thought out before gathering the data.

6. Soil logs should show soil structure and estimated permeability.

7. Where pumps are required to lift the subsurface drainage water from the main collector drains, the surface runoff should be diverted from these drains to avoid excessive pumping requirements.

8. After a drainage system has been designed for a particular area, only a portion of it should be constructed and the results studied to determine the adequacy of the entire system.

To support Mr. Brown's recommendations and those previously made in the 1964 report, Drainage Organization and Methods of Investigation, by Thomas Steele, Civil Engineer (Drainage), Bureau of Reclamation, prepared for Devlet Su Isleri (DSI), we are reproducing the Series 520 (tentative), Reclamation Instructions, as an appendix to Mr. Brown's report. We are confident that this material will enhance the value of the report to the Government of Turkey.

This report has our approval.

Sincerely yours,

G. E. Stamm

Assistant Commissioner

Enclosure

INTRODUCTION

Devlet Su Işleri (State Hydraulic Works) has developed a Drainage Branch engaged in the planning, investigation, design, and construction of drainage systems for irrigation and land reclamation. In September 1963, Mr. Thomas L. Steele of the Bureau of Reclamation evaluated the DSI organization, methods, and standards and recommended certain steps to be followed for improvements. In 1965, DSI requested the Agency for International Development (AID) in Ankara, Turkey, for the services of another Drainage Engineering Specialist to follow up on the activities initiated and to make final recommendations. Under PIO/T 277-142-2-50007, AID requested the Bureau of Reclamation of the Interior Department to furnish this engineer, and the writer was selected for the assignment, which extended from May 15 to August 1, 1965.

The assignment involved a detailed study of the data collected on various proposed and existing irrigation projects, review of the techniques used in gathering these data, and discussions of various methods of analyzing the data. Field reviews of a number of drains under construction and a few that had been in operation for a number of years were also carried out.

On July 26-29, a seminar was held at the DSI Office in Ankara. The subject was surface and subsurface drainage investigations, analysis of data, and design of drains. Prior to the seminar questions were submitted by the DSI engineers and at the close of the session these were discussed; where it was possible, direct answers were given.

Appended to this report is a copy of the Bureau of Reclamation Instruction Series 520 (tentative) "Land Drainage Techniques and Standards." This should be very helpful as a guide for the DSI engineers; however, it should not be considered as a final method to the solution of all drainage problems.

SUMMARY

The writer performed a detailed study of the data collected on various proposed and existing irrigation projects, reviewed the techniques used in gathering these data, and discussed various methods of analyzing the data. Field reviews of a number of drains under construction and a few that had been in operation for a number of years were also carried out.

Although the data obtained thus far on ground-water conditions in the Söke Plain are not complete enough to make a detailed study, it is evident that the drainage requirement will be quite extensive. Inspection of the available auger hole logs, permeability results, and observation well hydrographs indicates that much of the area can be drained successfully.

The field investigations of the Silifke Plain high-water table areas are not complete as yet. More data will be necessary before the number of drains required can be determined and this work is to be initiated in the near future. The available soil logs and permeability test results indicate that subsurface drainage is feasible if the cost of providing an adequate outlet can be tolerated.

The field investigations presently being made on lands near Tarsus and Ceyhan were examined and discussions were held with the engineers in charge of the investigations on the recognition and logging of soil structure and its importance in relation to permeability of the soil. A few DSI-constructed surface drains were inspected; obviously, at least portions of the drains were constructed with too little slope.

The engineer in charge of fieldwork must keep in mind at all times the purpose of the investigations. If studies show that subsurface drainage is required, he must acquire sufficient field data to compute the spacing. This will usually consist of the depth to barrier, the average permeability of the overlying soils, and the specific yield.

Mr. Steele's report on organization of a unit for drainage investigations and outlining its responsibilities has not as yet been translated nor have copies been given to the engineers. DSI engineers appear to be competent and dedicated people; undoubtedly they would benefit by reading the report and by being able to refer to it at any time. For this reason it is strongly recommended that the report be translated and copies distributed to each engineer as soon as possible.

Much improvement could be made in the translation and circulation of all existing reports and any that are made in the future. Many questions that were asked of me had previously been answered in previous reports and many recommendations concerning the collection and evaluation of data on existing projects had not been initiated.

The only way the Turkish engineers are going to gain experience is to review the results of completed projects. This will require collecting additional data and continuing measurements of observation wells on existing projects. The data that are collected should be analyzed and incorporated in reports describing the subject area. These reports should then be made available to all interested parties, whether it is the DSI Drainage Branch, DSI Ground Water Branch, or Topraksu.

SÖKE PLAIN

GENERAL

The Söke Plain is one of many areas proposed for irrigation development in the Büyük Menderes River Basin. The Plain extends from near the town of Söke on the east to the Aegean Sea on the west, a distance of approximately 30 kilometers. The plain averages 11 kilometers in width and is bordered on the north and south by mountains. Of the 44,000 hectares studied during the preliminary land classification, 39,000 were placed in an irrigable class.

The Söke Plain is very flat and is subject to much flooding from upland runoff and overflow of the Büyük Menderes River during and shortly after the rainy season. The average yearly precipitation in this area is 100 cm, of which approximately 80 percent occurs during the months of November through March. During this time, particularly from January through March, much of the valley is virtually under water, and because of the very poor surface and subsurface drainage characteristics, the land remains wet quite some time after the rains have stopped. When the land finally becomes dry enough to cultivate, the rainfall during the growing season is so little that, without irrigation of some sort, crop production is extremely limited. It is obvious that flood control and irrigation from storage is necessary for full development of the Söke Plain.

The plan of development of the Büyük Menderes River Basin includes eight major flood control and storage dams upstream from the Söke Plain and two flood control dams near Söke. The Büyük Menderes River will be diked from Aydin (a city located approximately 35 kilometers upstream from Söke) to its outlet, and two major surface drains are proposed for the Söke Plain. One of the drains will be an interceptor type located along the north edge of the valley to carry both upland runoff and ground water; the other, a farm outlet drain, will be located near the center of the plain to carry off the surface water originating on the plain itself either from precipitation or irrigation runoff.

Although some of the land in the Söke area in presently pump-irrigated, either directly from the Büyük Menderes River or from ground water occurring in the road borrow pits or old river channels, this method will be abandoned when full development of the basin is realized. A diversion dam on the Büyük Menderes River south of Söke and a canal system are proposed to divert the necessary water to irrigate the 39,000 hectares of project lands. The water will be supplied by making controlled releases from the aforementioned flood control and storage reservoirs.

INVESTIGATIONS

Very recently topography of the Söke Plain on a scale of 1:5,000 was completed and is now available in printed form. However, during the review of the project a discrepancy between the topographic maps and actual field conditions was discovered. When the ponded floodwaters were traced on the maps according to the recollections of the landowners, it was found that the higher lands were ponded, the lower lands were dry, and there was no indication of a natural dike between the high and low areas. A few valley profiles will be made in the near future as a confirmation of the topography and also for use in locating the aforementioned farm outlet drain.

Although the present report is concerned only with the Söke Plain, the subsurface investigation data discussed in the following paragraphs includes the Aydin Plain, which is upstream between Söke and Aydin. This plain has approximately 89,000 hectares of land classified as irrigable, so that approximately 30 percent of the data can be considered as applicable to the Söke Plain.

The investigations completed thus far include 432 holes augered to depths of 4 meters or until unstable soils were encountered. The holes were placed 1 kilometer apart except

that no holes were augered where village wells were encountered in which the water table could be measured. Auger hole permeability tests were to be made in each hole; however, due to caving of the soils, only 103 tests were successful. All of the auger holes were cased and have been used as observation wells. The water table has been measured at intervals of 1 month in the observation wells and also in all of the village wells since July 1964.

In 1961 the Ground water Division of DSI drilled 15 holes 40 to 300 meters deep during a ground-water supply investigation and copies of the logs have been obtained. Further drilling has been programmed by the Drainage Branch of DSI and will be initiated in the very near future.

Many soil and water samples were taken from the holes augured during the initial drainage investigation and these were analyzed in the laboratory for texture, salinity, alkalinity, permeability, etc.

REQUIRED SUBSURFACE DRAINAGE

Although the data obtained thus far are not complete enough to make a detailed study, it is evident that the drainage requirement will be quite extensive. The present water table is 0.5 to 2 meters below land surface during the growing season and the only natural outlets are the Aegean Sea to the west and the Büyük Menderes River which skirts the extreme southern edge of the Plain. Because much of the irrigable land is so low (1 to 3 meters above sea level), neither the river nor the sea will provide the subsurface relief necessary for irrigation development. In fact, it is quite likely that the river will contribute to the water table during the high flows which occur during and shortly after the rainy season.

Inspection of the available auger hole logs, permeability results, and observation well hydrographs indicates that much of the area can be drained successfully. A few of the soil logs show evidence of barriers at shallow depths but more detailed studies will be made to determine the permeability of these soils and the feasibility of draining the areas in which they occur.

Some of the hydrographs indicate that the water table, even in the presently irrigated areas, fluctuates with the water level in the Büyük Menderes River. If this is true, and studies are presently under way to confirm this, it would indicate that the underlying soils are quite permeable. The geologic profiles may also show that in these areas the river is connected with an aquifer through which this water moves. This will not eliminate the need for drains but should permit increasing the spacing substantially.

Since the land surface is so low it will be necessary to construct a pump outlet for the subsurface drainage water. This will increase the cost of the subsurface drainage system substantially; however, this contingency has been recognized and will be included in the overall feasibility studies.

RECOMMENDATIONS FOR COMPLETING DRAINAGE STUDY

Prior to making a detailed subsurface drainage estimate of the Söke Plain it is recommended that the following additional field data be acquired. Various techniques for acquiring this data and placing it in exhibit form are also suggested.

1. Geologic Cross Sections

   a. Various holes have been located on the 1:25,000 scale map of the area; however, as these holes are drilled the cross sections should be plotted immediately to determine if additional holes are necessary. Also, correlation between cross sections should be made as they are plotted to determine if more data are required between the sections. True land surface elevation should be used and all natural drains shown. If possible, the bottoms of the natural outlets should be probed and the soils encountered shown on the sections.

2. Determination of Aquifer Characteristics

a. During the course of drilling the holes required to obtain the geologic cross sections an aquifer may be encountered that could affect the subsurface drainage requirement. It may be necessary to measure the characteristics of this aquifer; however, rather than make a pump test in each hole immediately after it is drilled, as is now the practice, the completed geologic cross sections should be analyzed first. Then, if it is determined that the aquifer will definitely affect the subsurface drainage requirement it is very possible that only a few pump tests will be necessary. It should be kept in mind that a pump test measures the aquifer characteristics in the zone of influence of the well only and, considering the type of tests to be made, this zone may be very small. Also, careful analyses must be made of the homogeneity and continuity of the aquifer before the measured transmissivity can be applied throughout its entirety. Discontinuous or artesian aquifers may be encountered which will contribute little, if any, subsurface drainage benefit and pump test data acquired in these aquifers would be of no value unless pump drainage is being considered.

3. In-Place Permeability Tests

a. Before making additional in-place permeability tests, geomorphological cross sections using the available soil logs should be plotted perpendicular to the valley at least every 5 kilometers--more may be required. The true land surface elevation should be shown, including all natural subsurface drain outlets. Also, all barriers or possible barriers and the permeability of the soils where measured should be indicated; unstable soils and maximum and average water table elevations during the growing season should be shown. From these cross sections, the number, depth, type, and location of permeability tests still required for analysis of the subsurface drainage requirement can be determined.

SILIFKE PLAIN

The Silifke Plain is on the southern coast of Turkey. It is composed of alluvial soil deposited by the Göksu River as it flows from the mountains on the north to the Mediterranean Sea on the south. The Plain contains approximately 10,000 hectares of irrigable land, of which 5,260 hectares in the northern part of the area are presently irrigated. The southern part of the Plain requires extensive subsurface and surface drainage work and is not being irrigated at this time.

The field investigations of the Silifke high-water table areas are not complete as yet; however, the delineation of the areas seemed to be quite good. More data will be necessary before the number of drains required can be determined and this work is to be initiated in the very near future. The DSI engineers recognize the need for more data and, by using the techniques and guidelines described in the Appendix, should have no trouble in acquiring them.

In general, the available soil logs and permeability test results indicate that subsurface drainage is feasible if the cost of providing an adequate outlet can be tolerated. Much of the subsurface water will have to be pumped into the surface drains or, in some areas, directly into the sea. This is, as was the case in the Söke Plain, recognized and the cost of the pump outlet system will be included in the feasibility studies.

Surface flooding is also a problem in the Silifke Plain, particularly adjacent to a large shallow lake in the southwest part of the Plain. Studies are presently under way to determine the feasibility of constructing an outlet from the lake to the sea. If an outlet can be provided, much land may become available for cultivation that is otherwise flooded when the upland flows raise the lake level. The initial investigations will consist of measuring the difference in elevation between the sea and the lake and also analyzing the quality of the soils in the area to be reclaimed.

OTHER AREAS

After the Silifke Plain had been examined, a few days were taken to observe the field investigations presently being made on lands near Tarsus and also near Ceyhan. Most of the time was used to instruct the engineers in charge of the investigations on the recognition and logging of soil structure and its importance in relation to permeability of the soil. Various techniques were discussed for augering holes with the least disturbance to the soil structure and also methods of breaking up the sealing effect on the walls of the hole.

Also, at this time, a few DSI-constructed surface drains were inspected and it was quite obvious that at least portions of the drains had been constructed with too little slope. Not only do these drains provide little outlet for subsurface water but they also represent a mosquito-breeding haven, as the water in the drains is practically motionless. One drain was inspected that was flowing exceptionally well and it was recommended that the slope of this drain be measured and that future drains be designed accordingly.

An experimental tile drainage system constructed by Topraksu near Tarsus was inspected and the relief provided by the drains was quite evident. The drains were constructed at 30-, 40-, 50-, and 60-meter spacings and it was stated by a Topraksu engineer that the 60-meter spacing seemed to provide the necessary relief. The data acquired from this experiment are to be published and distributed to the various government agencies, including DSI. Since a pump outlet was used to remove the drainage effluent from the collector drain, the amount of water pumped from this field may be valuable information for the design of pump outlets to be used in other projects.

RECOMMENDATIONS FOR IMPROVING QUALITY OF FIELD DATA

Upon reviewing the data that have been collected thus far, it became evident that the quality of the soil logs and techniques of making permeability tests could be improved with little additional effort. The following recommendations were discussed in detail with the DSI engineers:

1. All logs should include the structure of the various soils and a statement of their relative permeability--free draining, slowly permeable, or impeded (barrier). With some experience an engineer will soon be able to estimate the permeability of the soils above the water table by comparing them with the measured permeability of similar soils below the water table.

2. In many of the holes augered in the Söke-Aydin project area, fine-textured soils were found overlying highly permeable sands. The permeability tests made in these holes reflect predominantly the permeability of the sands. It is suggested that in future investigations, when the original hole in this type of formation is completed, another be immediately augered at a distance of 2 to 3 meters away and to a depth approximately 20 centimeters above the sand layer. This hole would be for permeability testing only and a log would normally not be required, as the soil profile should be identical with the first hole (this must be confirmed, however). After the permeability test the hole should be deepened to verify that the sand layer is at least 20 centimeters below the bottom of the hole.

3. After completion of an auger hole in which a permeability test is to be made, the sides of the hole should be brushed or otherwise cleaned to break up any sealing effect caused by the auger. This can often be accomplished by bailing the water out of the hole several times before making the test. If the soil structure indicates that the permeability of the soils is higher than measured it is very likely that the sealing effect has not been broken.

4. Prior to making an auger hole pump-out test it is suggested that the water table be allowed to stabilize at least overnight. This rule can be broken if the soils are highly permeable and no rise of the water in the hole is noted over a period of a few hours; however, it is advisable to make sure that the water table is definitely stable. The auger hole method of measuring permeability was developed on the assumption that no drawdown around the hole occurs during the test. This assumption was found valid if the test measurements were made before the water had risen 0.2 of the depth initially bailed out. If the water table has not become stable prior to the test, the permeability results will not be accurate.

The engineer in charge of the fieldwork must keep in mind at all times the purpose of the investigations. If he is attempting to show that the natural subsurface drainage characteristics of a proposed project are sufficient for irrigation development then he must:

1. Determine that the vertical permeability of the soils overlying the aquifer is great enough to avoid a perched water table.

2. Determine the existence of an aquifer capable of transmitting the additional deep percolation water without causing a detrimental rise in the water table.

3. Determine that the aquifer has an adequate outlet.

If studies show that subsurface drainage is required, then it will be necessary to acquire sufficient field data to compute the spacing. This will usually consist of the depth to barrier, the average permeability of the overlying soils, and the specific yield. The water table build-up due to deep percolation from each irrigation should be measured in the field if valid observation well data are available. It should also be kept in mind that the quality of irrigation water may be such as to require excess applications for leaching purposes. If this is the case, the leaching requirement will be the controlling factor in computing deep percolation losses.

GENERAL COMMENTS

The following comments seem pertinent to the improvement of the DSI Drainage Organization:

Shortly after my arrival in Turkey I began reviewing the various reports that have been written by consultant engineers in the past. Three of them came to my attention immediately:

"A Report on Drainage Problems and Activities" by John G. Sutton for Topraksu. Tour of duty, May 16-July 6, 1963.
"Drainage Organization and Methods of Investigation" by Thomas L. Steele for DSI. Tour of duty, September 16-November 16, 1963.
A report (no title) on "improving aquifer characteristics determinations" by Robert L. Cushman, Ground Water Hydrologist Consultant to the Ground Water Department of DSI. Tour of duty, May 7-August 7, 1964.

Although most of the DSI drainage engineers recalled Mr. Steele's tour of duty and had personal notes concerning various aspects of subsurface drainage investigations recommended by him, to my knowledge, only a very few of them have read his "end-of-tour" report. Five copies of this report had been transmitted to DSI in July 1964, but it has not as yet been translated for the engineers who do not read English nor have copies been given to those who do.

As for the reports made by Messrs. Sutton and Cushman there are, again, to my knowledge no copies available in the DSI Drainage Organization.

It is evident that much improvement could be made in the translation and circulation of the above reports and any that are made in the future. Many questions that were asked of me had previously been answered in Mr. Steele's report and many of his recommendations (and also Mr. Sutton's) concerning the collection and evaluation of data on existing projects had not been initiated prior to my arrival. In Mr. Cushman's report it was recommended strongly that all pump test data be evaluated by a technical man at the site during the pretest and test period. Yet, when I asked for the pump-test results of the Silifke area only the field data were available. No evaluation of the aquifer characteristics had been made either at the test site or in the office.

As has been stated in this and in previous reports, the only way the Turkish engineers are going to gain experience is to review the results of completed projects. This will require collecting additional data and continuing measurements of observation wells on existing projects. This may seem to be an unnecessary expenditure of funds; however, the experience and data obtained will more than offset this cost. The data that are collected should be analyzed, consolidated in charts and/or exhibits, and attached to reports describing the geology, geomorphology, irrigation practices, water table fluctuations, and all other pertinent facts of the subject area. These reports should then be made available to all interested parties whether it is the DSI Drainage Branch, DSI Ground Water Branch, or Topraksu.

The DSI engineers appear to be competent and dedicated people; undoubtedly they would benefit if they could not only read the aforementioned reports but be able to refer to them at any time. For this reason it is strongly recommended that the reports be translated and copies distributed to each engineer as soon as possible.

The following excerpts from Mr. T. L. Steele's end-of-tour report (April 1964) are still applicable to the present situation in the DSI Drainage Organization:

"The frantic pace at which the work is going on leaves insufficient time for either the collection or analysis of data. This situation creates the great danger that drains will be insufficient or improperly located, that project lands will gradually deteriorate as a result, and that production from the irrigated land will decrease rather than increase. This is a situation that has occurred on the Manavgat Project * * * and will be detrimental for Turkey if it occurs on a large scale.

" * * * The Five-year Plan is being pushed forward at a rapid pace by the Government, therefore, all the investigations and reports on individual projects must be made in a very short time to keep ahead of the construction schedule. The result is that only minimum data are available on any project before the layout and design of the drainage system must be completed. One way in which this limitation can be minimized is to continue record keeping through and beyond the initial construction period and to plan for construction of drains to be carried on for several years after irrigation water is applied to the land.

"This is a policy that is suggested for adoption by DSI as being the most economical and technically sound method of drain construction. The science of investigation and analysis of subsurface drainage requirements has not reached the stage where an exact advance prediction can be made of the location, depth, and size of all drains that will be required on any project. A lengthy and expensive study and analysis can minimize the probable error, but the scientific tools with which the drainage engineer must work are not sufficiently accurate to permit predicting drain location and design. The best alternative approach is to maintain a continuing program of record collection and analysis, and to install drains as the need for them becomes apparent. In this manner the most economical drainage system is obtained, since only those drains that are required are built. DSI is advised to adopt this policy and budget for drain construction for several years after a project begins irrigation."

FIELD TRIPS

During this assignment I made a field review of the proposed projects that are in various stages of investigation. I have gone into more detail on the Söke Project than the others because many of the recommendations made on this one project also pertain to the others.

It should be noted here that I worked directly with the engineers performing the fieldwork and many of their specific questions were answered orally in the field. Most of the recommendations given at that time are again covered in either this report or the Appendix, U.S.B.R. Instruction Series 520 (Tentative).

APPENDIX

(Tentative)

UNITED STATES
DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

Reclamation Instruction Series 520

LAND DRAINAGE TECHNIQUES AND STANDARDS

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UNITED STATES
DEPARTMENT OF THE INTERIOR
Bureau of Reclamation

Reclamation Instruction Series 520

LAND DRAINAGE TECHNIQUES AND STANDARDS

Part 522 Introduction

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.1 General. A prime function of successful irrigated agriculture is the development and maintenance of a soil zone in which moisture, air, and salt balance are favorable for plant growth. Plants must have moisture to live, but under normal agricultural conditions production is usually seriously affected when a water table rises into the root zone. The presence of free air in the interstices of the soil in the root zone is as necessary as water for plant growth. The proper balance between soil, moisture, and air is maintained to a considerable extent by adequate drainage. (The term "soil", in this technical guide, 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.)

A simple but comprehensive definition of drainage is "the removal of excess water and salt from the soil." This definition is equally applicable for surface and subsurface drainage in humid and arid regions. A basic function of drainage in arid regions is to maintain the salt balance within the soil, but this function is seldom important in humid areas. Further, adequate drainage may be defined as the amount of drainage necessary for agriculture to be maintained successfully and perpetually. This does not, however, necessarily entail complete and perfect drainage. Such is generally

not feasible, as the cost of preventing occasional damage to crops may not be justified by the amount of the damage. The aspect of economic justification thus enters the picture. The design and construction of a perfect drainage system should not, therefore, be the prime objective. Rather, it should be to design and construct a system in which an optimum integration of soils, crops, irrigation, and drainage has been made. In the Bureau of Reclamation this means the drainage necessary to maintain the productive level of the irrigable class in which specific lands are placed.

Drainage can be either natural or artificial. Most lands have some natural surface and subsurface drainage. When such 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. It usually supplements existing natural systems; that is, natural water courses can be deepened, or where no suitable ones exist, new ones can be constructed. Almost every physical aspect and condition of lands, as well as man's potential agricultural use of them, will affect their ultimate drainage requirement.

.2 Scope. This technical guide reviews the data used in all phases of drainage solutions, suggesting what data are needed, where and how to get them, and how to record, present, analyze, and apply them. It discusses the problem of forecasting drainage requirements,

and suggests drainage design criteria and construction standards. It is not intended to be a textbook on drainage, but rather is directed toward field application of engineering knowledge on the subject. Anyone expecting to find a step-by-step approach which will solve any drainage problem will be disappointed, because good judgment, rather than rote, must be used in the solution of drainage problems. An attempt is made to set up guidelines for use in exercising judgment. Although directed toward the conditions encountered in the Western United States on arid lands under irrigation agriculture, the recommended practices discussed herein should be basically applicable to drainage problems anywhere. Worldwide drainage problems are cast in the same mold. Only the language in which they are discussed and the coin used to pay for their solution are different.

.3 History. Drains have been built since before our recorded history. Some ancient systems were simple, some were elaborate, but very few were entirely successful, and practically none have survived to the present time. Part of man's trouble with drainage has been due to neglect of his drainage systems; part has been due to a lack of understanding of the physical and technical problems involved. In spite of man's basic knowledge of soil physics and hydraulics, drainage engineering is just now emerging from the "build it here and see how it works" stage. It is not yet, however, an exact science and probably never will be. It still is largely a matter of common

sense and judgment. Much drainage construction today is based on generally adequate investigations and more or less scientific approaches. The problem of neglect of maintenance, however, frequently remains.

.4 Importance. The importance of drainage to the irrigation economy of a project, state, or nation has been generally underestimated. The history of irrigation in the United States and in the world generally points up the inescapable conclusion that drainage must be provided where irrigation is practiced. Only on irrigation projects with a rare combination of adequate natural surface and subsurface drainage will excess surface water and accretions to the underground reservoir pass away from the land with sufficient rapidity to prevent a rise of the ground-water table to critical elevations. Where natural drainage is inadequate, and artificial drainage cannot be economically provided, the land cannot be permanently irrigated with success. Numerous projects having original water tables from 20 to 100 feet below the ground surface and with seemingly favorable natural drainage conditions have eventually developed excessively high water tables, leading to waterlogging or salinization, or both.

The outlook on drainage is paradoxical. No one denies that it 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, there can be no irrigated agriculture, but without drainage there can be irrigated agriculture--

of a sort and for a time. Although symptoms of high ground water and salt may not develop for some time aft the beginning of irrigation, the insidious thing about land deterioration due to inadequate drainage is that it will inevitably develop.

.5 Benefits of Drainage. There are many benefits which can be attributed to drainage of agricultural land. Both the direct and indirect benefits may be difficult to evaluate, but the former are more readily recognized and appreciated. It is obvious that a high water table which is near the land surface is detrimental. If it is high in the spring, it hinders the preparation and planting of the land; if high during the growing season, it hinders cultivation; and if high when the crop is at maturity, it hinders harvesting. Any of these conditions are harmful because they reduce production, and a direct benefit would result from a drainage system that would eliminate them.

Another benefit of a well-drained soil is that it will warm up more rapidly early in the growing season with the result of earlier germination and a longer growth period. Four or five times as much heat is required to raise a volume of water a unit of temperature as for an equal volume of soil. If the soil is wet, the heat of the sun spends most of its energy raising the temperature of the water rather than that of the soil, and germination of seed is delayed. At the same time, the water evaporates which further reduces its temperature and that of the soil.

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.

Proper control of salinity and alkalinity can only be done in well-drained soil. Leaching water must be able to pass through the soil profile to dissolve and move excess salts out of the root zone, and this is not possible without drainage. Also, a high water table creates a condition wherein capillarity can move salts to the surface and deposit them there as the capillary water evaporates.

In addition to the foregoing, there are less tangible benefits attributable to drainage, such as reduction or elimination of mosquito breeding grounds, improvement of the appearance of farms by elimination of boggy and weedy areas, and improvement of road systems by elimination of soft, boggy spots and reduction in road maintenance costs. In summary, the benefits of adequate drainage are a longer growing season, increased soil tilth, early and more vigorous plant growth, larger yields and thus greater income, a wider selection of crops, and decreased cost of production.

.6 Drainage Nomenclature. Drainage nomenclature is somewhat complex, being based on various conditions such as the source of water to be

moved, when the drains will be built, and their function. Thus, drains may be either surface or subsurface, open or closed, constructed or deferred (depending on whether they are to be constructed initially with the project or later); they sometimes consist of wells (recharge, relief, or pumped); and they may fall within various functional classifications. Some of the distinctions may seem arbitrary. For example, the distinction between surface and subsurface drainage is based entirely on the location of the water at the time it becomes a drainage problem. The terms "deep" and "shallow" have sometimes been used in lieu of "subsurface" and "surface" but are generally too restrictive, and the latter terms are preferable.

A. Surface Drainage. Surface drainage is the removal of water from the surface of the land. The water may be from excess precipitation, water applied in irrigation, losses from conveyance channels and storage facilities, or water which has seeped from the ground water at a higher elevation. The problem in its removal is essentially one of providing channels to facilitate its removal. Surface drains usually are of the open type, but expensive right-of-way or other special conditions may make it more desirable to make these drains closed.

B. Subsurface Drainage. Subsurface drainage is the removal or control of ground water and the removal or control of salts in water as a vehicle. The source of the water may have been percolation from precipitation or irrigation; leakage from canals, drains, or surface water bodies at higher elevations; or upward

leakage from artesian aquifers. In the broad sense, any drain, whether open or closed, deep or shallow, or any well, which is designed to control the lower ground water is considered a subsurface drain.

C. Deferred Drainage. Deferred drainage is that which is provided after project works have been constructed and irrigation started. The deferral of construction of such drains usually results from 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 since the need for surface drains which are to be constructed as a part of the initial project works is generally more evident, permitting early construction. Bureau of Reclamation policy requires that deferred drainage be included in the project plan and cost estimate. It is not to be ignored or forgotten, but is to be recognized as an inherent part of a project.

D. Open and Closed Drains. As the name implies, open drains are channels with an exposed water surface. Closed drains are buried conduit or pipe. Generally speaking, almost any of the other designations may be applied to either open or closed drains. Drain size and purpose, physical conditions of lands and soils, required drain intensity, and annual cost largely dictate whether drains are open or closed.

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 four types of drains are relief, interceptor, collector, and outlet.

Relief and interceptor drains or wells are works with 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 condition of the ground-water body they control. Generally speaking, relief drains are used to effect a general lowering of ground water over relatively large nearly level areas where the source is percolation from precipitation or irrigation, and where gradients of both the water table and subsurface strata permit little lateral movement of the ground water. Interceptor drains, as the name implies, are used to cut off or intercept ground water which is moving downslope from whatever source. They may be of either open or closed construction. When they are the open type, they may be designed to receive irrigation surface waste and surface runoff from adjacent fields. When they receive significant surface runoff, or flow from surface drains, they should be considered collector drains.

Collector drains receive water from subsurface relief or interceptor drains, and from surface drains carrying irrigation surface waste and storm runoff. Because they must 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 drainage in adjacent or tributary areas. They may be either open or closed, depending on the volume of water to be handled, the available gradient, and whether or not their tributaries are open or closed.

Outlet drains have the principal function of conveying water from other drains to a suitable natural disposal point. They receive flow principally from collector drains and canal wasteways. They are thus the "end" drains, or the lowest constructed drains, of the system and have thus been called "main" drains. They are similar in function to collector drains, except they are not usually required to serve as subsurface drains in the control of ground water to prescribed elevations. They may be a part of either a surface or a subsurface drainage system, or both, and may be either open or closed, but because of the relatively large flows to be handled they are usually of the open type.

F. Inverted, Relief, or Pumped Wells. These special drain types may be used to dispose of surface water, to control ground-water levels, or to relieve piezometric pressures where the local physical conditions are adapted for their use. An explanation of their use and limitations is contained in Chapter 526.3. The following discussions of the various aspects of drainage may be more applicable to horizontal drains but many of the principles also apply to wells.

Reclamation Instruction Series 520

LAND DRAINAGE TECHNIQUES AND STANDARDS

Part 523 Basic Data

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.1 Introduction. Selection of the optimum drainage plan and the design and construction of adequate and potentially successful drainage works all depend to a considerable extent on the basic drainage data which are available. The data requirements for any particular drainage problem vary with the type of problem and the caliber of the investigations or report which is being prepared. In general, data requirements for drainage are comparable to those for any other similar undertaking. The basic data must permit selection of a plan which is firm enough, designs which are representative enough, and cost estimates which are accurate enough for the purposes intended. Inadequate data introduce serious risks.

The basic data for a drainage investigation can be related to soil and water since these are the fundamentals to which drainage applies. The characteristics of water which must be resolved relate to its occurrence, distribution, movement, and quality. The characteristics of soil which must be resolved relate to its formation, its physical and chemical condition, and its ability to transmit water.

This Part 523 will cover the various types of data used in drainage investigations, planning, and design and suggest possible sources of available data. No attempt will be made to differentiate between types and densities of data required for specific purposes.

.2 Topography. Topography is of prime importance in drainage. It frequently determines the general plan that must be used and the location of principal drains. Even before the planning and designing stages of drainage are reached, however, the importance of topographic factors can be recognized. Topography could thus mean the difference between little or no need for artificial drainage facilities and extensive and expensive drainage facilities. Where surface slopes are sufficient, excess precipitation, excess 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 thus provide adequate surface drainage and obviate or reduce the need for any subsurface drainage.

Consider another example of almost flat benchlands cut at frequent intervals by rather deep water courses. Even though surface drainage on the bench may be poorly developed, excess water could percolate into permeable strata and find outlets in the entrenched water courses. Under these conditions, the presence of the natural water courses, a topographic factor, through an area which would normally require extensive drainage, would make a great difference in the drainage plan and the cost of the required works. In short, topography not only influences the planning, layout, and design of drainage systems, but is equally important in determining the artificial drainage requirement or whether, in fact, any artificial drainage is needed.

Topographic maps are essential in any detailed drainage investigation. They show land slopes, length of slopes, location and direction of natural drainage, potential outlets, and other special conditions which may affect drainage. Maps alone will often give a clue 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 involved and the purposes of the investigation. For preliminary study, a scale of 1 inch to 4,000 feet is usually adequate. For smaller areas or for more detailed study, 1 inch to 2,000 feet would be advantageous. Detailed studies of special problem areas and the location and design of specific works may require a scale of 1 inch to 400 feet. Topographic maps should have contour intervals consistent with their scale and the size of the area and the purpose of the map. For preliminary study of large areas with considerable topographic relief, a 5-foot interval would be satisfactory provided the natural drainage pattern is adequately shown. Two-foot intervals are usually sufficient for drainage layout, but for large, nearly level areas a 1-foot interval is usually justified. In addition to relief and natural features, topographic maps should show springs, seeps, wells, and culture such as roads, railroads, culverts, pipe and utility lines, and land subdivision and property lines.

Topographic maps are usually available for a proposed or existing irrigated or cultivated area. These maps may have been

prepared specifically for the purpose of laying out the irrigation system, or they may have been prepared for any related purpose. The Soil Conservation Service, Bureau of Reclamation, and similar Federal and related 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 of smaller scale and wider interval. 1/ Even though the available maps may be inadequate for the required purpose, they may have enough information to reduce significantly the additional surveying required. If adequate topographic maps are not available, a field survey will have to be made.

Aerial photographs of an area are quite useful in drainage studies. They supplement topographic maps in presenting an overall picture of natural and artificial drainageways, and particularly of outlet conditions. Additionally, they will also often reveal the existence and location of drainage problems, such as seeped areas and saline or alkaline spots, and may provide clues to the source of excess water. Department of Agriculture agencies, such as the Soil Conservation Service and Forest Service, and County Agricultural Agents, may have information on the existence of aerial photographs of an area. The State Engineer and State Water Board, or their equivalents, may also have knowledge of maps or photographs.

1/ For a general location of topographic mapped areas in the United States, see "Design of Small Dams," First Edition, U. S. Bureau of Reclamation, 1960, page 79.

Conventional symbols for drainage maps are shown on Figure 1.

.3 Geology. The geology, particularly the geomorphology, of an area is of interest to drainage primarily with regard to its effect on that portion of the earth's surface to a depth of about 30 feet, except when drainage by wells is contemplated and deeper explorations are required. Consideration of geologic processes will be helpful in understanding and analyzing the occurrence and solution of drainage problems, since soils are a product of parent material, topography, climate, vegetative cover, and the weathering process. These factors determine their texture, chemical characteristics, hydraulic properties, etc. Of the items of interest, one is the process of formation of earth-mantle material and associated topography, and another is the existence of artesian aquifers.

In some areas the soil material results from volcanic eruption. Fine ash material is spread over the land surface in the vicinity of the volcano in depths that sometimes reach many feet. The resultant soil derives its fine-grained characteristics from the parent material and the method of formation.

In other areas the soil material 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 transport

and deposition. Glacial formation of lakes, changing of riverbeds, deposition of eskers, moraines, kames, and similar forms, are actions which may have an effect on drainage problems.

Residual soils, formed from disintegration of the underlying parent material, are found in many areas. These soils take their characteristics from the parent material, the weathering process, and a minor amount of reworking 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, with all possible combinations thereof being found from place to place. They consist of outwash from mountains, 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 debouch on the plains, the shape of the land surface is changed. The present topography is the result of those processes over a period of many centuries.

Another material which is found in many areas is the aeolian, or wind-deposited, soil. These soils are fine-grained due to the limited ability of the wind to carry large grains for any distance, and in some places have a great depth. Soils formed in this manner are called loess.

As soils are the result of complicated geologic processes, there are many more geologic soil types than mentioned above, and innumerable combinations of types, and each has a meaning in drainage studies. A broader understanding of the drainage problem may result from a knowledge of the effect of geologic processes.

Geologic identification of artesian aquifers may be important from a drainage standpoint. An artesian aquifer that is under enough pressure so that the water will rise to or near the land surface may contribute to the drainage problem. This water, as well as deep percolation from irrigation and precipitation, must be removed if agriculture is to be successful.

.4 Soil Characteristics. The primary concern of subsurface drainage is with the movement of water through soil (previously defined to include soil, subsoil and substrata). All subsurface drainage planning is based on this phenomenon. All of the soil characteristics of density, porosity, particle size, grain distribution, texture, structure, chemical properties, water-holding capacity, etc., 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 permeability, or hydraulic conductivity. Many attempts have been and are being made to establish a relationship between permeability and one or

more of the readily determined soil characteristics, but only limited correlation has been found. The many characteristics which affect permeability, and their varied interrelationships, are so complex that it is doubtful whether a universal correlation can be established. It may be possible to establish a correlation in limited areas where soils were derived from the same source, were deposited in the same manner, have been affected by the same climatic conditions, and, in general, have enough similarity that many of the interrelationships of the characteristics have been nullified, but this will be the exception. The best information for use in analysis and solution of subsurface drainage problems is knowledge of the soil permeability.

A. Permeability. The facility with which water moves in a soil is a physical property of the soil, which expressed in numerical terms is its permeability. An understanding of and a means of determining this property is essential to understanding and correcting most subsurface drainage problems. Permeability 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 measured and expressed in terms of k in the Darcy equation, and thus is synonymous with hydraulic conductivity. Dimensionally, it is a ratio, $k = \frac{v}{i}$, in which v is velocity of flow and i is the hydraulic gradient. It is usually referred to as the permeability coefficient.

Physical dimensions for the permeability coefficient depend on those used to express the velocity, and no coefficient has been universally accepted as standard. Sometimes the coefficient is measured in cubic feet per square foot per year or per day, but in Bureau land classification and drainage work, cubic inches per square inch per hour is used. These are shortened to feet per year and inches per hour and commonly spoken of as rates. The various coefficients are based on Darcy's law and are, thus, mutually convertible. Figure 2 presents conversion factors for the various permeability units.

The average permeability of a soil profile is much used in determination of subsurface drainage requirements, but great precision in its value is not obtainable. Soils are usually heterogeneous and anisotropic, and seldom are field conditions affecting permeability uniform over an area of appreciable size. Nevertheless, average permeability should be determined as accurately as possible. The method used should be consistent with physical conditions and work requirements. Procedures for the various methods of obtaining permeability are given in Part 524.

r. Texture. The term texture expresses 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 permeability and water retention. In general, the coarse-textured soils have higher

permeabilities and lower water retention than fine-textured soils. Texture is readily measureable by using a gradation analysis to separate the size groups. The following particle size classification was developed by the U. S. Department of Agriculture. It is used by the Bureau of Reclamation in land classification and drainage work because it better relates the agricultural properties of the soils and allows better correlation for projecting permeability than do the Casagrade or Unified Soil Classification systems.

Material	Diameter
Stones	> 10 in.
Cobbles	3 to 10 in.
Coarse gravel	0.5 to 3 in.
Fine gravel	2 mm to 0.5 in.
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 sized particles in the soil mass. The soil texture triangle, Figure 3, is used to convert

quantitative data from detailed mechanical analyses of the separates less than 2 mm in diameter to textural class names of soils. Textural class names of material larger than 2 mm in diameter are taken from the above table. See also Volume V, Part 2, Land Classification, Reclamation Manual, for additional information on texture.

C. Structure. Soil structure is a characteristic that is very useful in evaluating and correlating the permeability 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. The size, shape, and arrangement of the aggregates and the shape 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 degree of distinctness, that is, weak, moderate, or strong, is termed grade of soil structure. The principal types of soil structure with which the drainage engineer will be working are described below. The classes and grades of soil structure are described under each type.

(1) Platy (pl). In this type of structure, the aggregates are arranged in horizontal sheets. The permeability rate varies with the class of structure, usually being highest for medium platy material. The classes of structure are as follows:

very thin platy where thickness of plates is less than 0.04 inch; thin platy where thickness of plates is between 0.04 and 0.08 inch; medium platy where thickness of plates is between 0.2 and 0.4 inch; and very thick platy where thickness of plates is greater than 0.4 inch. Grade of structure is seldom important in platy material because the plates are usually durable and are considered to have strong grade.

(2) Prismatic (pr) or columnar (col). These types of structure are usually found in the upper 4 feet of soil, so are of less importance to the drainage engineer. In these types, the aggregates form prisms with vertical axes longer than the horizontal axes. The shape can be roughly square, pentagonal, or hexagonal. The aggregates may break horizontally along secondary cleavage planes into blocky or very thick plates, but even the broken sections have relatively flat vertical surfaces. In prismatic structure, the aggregates form flat-topped prisms, while in columnar structure they form round-topped prisms.

The highest permeability is usually in the vertical direction and the rate is influenced by the class of structure. The classes of structure are as follows: very fine prismatic or columnar where the macroprisms are less than 0.4 inch wide; fine prismatic or columnar where the macroprisms are 0.4 to 0.8 inch wide; medium prismatic or columnar

where the macroprisms are 0.8 to 2 inches wide; coarse prismatic or columnar where the macroprisms are 2 to 4 inches wide; and very coarse prismatic or columnar where macroprisms are over 4 inches wide. Prismatic and columnar structures are considered to be strong grades of structure.

(3) Blocky (bl). When the term blocky is used alone as a type of structure, it means angular blocky where the aggregates are in dense blocks bounded by planes intersecting at relatively sharp angles. A soil with this structure usually has good permeability in both horizontal and vertical directions and the rate is influenced by both the class and grade. For example, a very corase angular blocky clay loam soil with strong structural grade (which usually means very distinct cleavage planes between blocks) can have in-place permeabilities up to 50 inches per hour. At the other extreme, a very fine angular blocky clay loam soil with a weak structural grade can have in-place permeabilities less than 0.5 inch per hour. The classes are as follows: very fine angular where the blocks are less than 0.2 inch on any side; fine angular blocky where the blocks are between 0.2 and 0.4 inch on any side; medium blocky where the blocks are between 0.4 and 0.8 inch on any side; coarse angular blocky where the blocks are between 0.8 and 2 inches on any side; and very coarse angular blocky where the blocks

are greater than 2 inches on any side. The grade is weak when the disturbed soil material breaks into a mixture of a few complete blocks, many broken blocks, and much unaggregated material. The grade is moderate when the disturbed soil material breaks down into many distinct entire blocks, some broken blocks, and little unaggregated material. The grade is strong when the disturbed soil material consists many largely of entire blocks, few broken blocks, and little or no unaggregated material.

(4) Subangular blocky (sub an bl). In this type of structure, the aggregates are in dense blocks having mixed rounded and plane faces with vertices mostly rounded. As far as permeability is concerned, there appears to be little difference between the angular and subangular blocky structures. 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 (gran). This type of structure is formed of relatively porporous aggregates, spheroidal or polyhedral in shape, and having plane or curved surfaces which have slight or no conformity with the faces of the surrounding aggregates. Soils with this type of structure usually have good permeability both vertically and horizontally. The permeability rate depends upon the class and grade; the coarser the granules and the stronger the grade

the better the in-place permeability. The classes are as follows: very fine granular where the aggregates are less than 0.04 inch on any side; fine granular where aggregates vary between 0.04 and 0.08 inch on any side; medium granular where aggregates vary between 0.08 and 0.20 inch on any side; coarse granular where aggregates vary between 0.2 and 0.4 inch on any side; and very coarse where aggregates are greater than 0.4 inch on any side. The grade can vary from weak to strong.

(6) Crumb (cr). This type of structure is the same as granular except aggregates appear very porous. It has good permeability in both vertical and horizontal directions, and rate depends upon class and grade. Classes are the same size as granular except there are no coarse and very coarse crumb structures. It can have any of the three structural grades.

(7) Massive (mas). 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 no class or grade and practically no permeability.

(8) Single grain (sg gr). 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 permeability. A single-grain soil has no structural class or grade.

D. Specific Yield. Specific yield is defined as the amount of water drained from a volume of saturated soil under the force of gravity and the inherent soil tensions. It is expressed as a percentage in the equation:

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

The amount of water retained in the soil by molecular and capillary forces against the force of gravity is the specific retention and is equivalent to the field capacity of the soil. The specific yield plus the specific retention or field capacity is equal to the porosity, or $S + R = P$.

A quantitative determination of a value for specific yield is difficult because the definition does not specify any time during which the sample is to be exposed to the force of gravity. Experiments have shown that water will continue to drain from a sample of soil for a period of days or even weeks depending on the grain size and distribution of the soil particles. Generally, in a soil of large grain size most of the water will be drained from a saturated sample in a short time, whereas it may take several days for most of the water to drain from a sample of small grain size. Both samples may continue to drain for days or weeks thereafter. Various investigators have used different methods in the laboratory for

determining the specific yield or the field capacity, but the methods have a similarity. Usually they consist of applying a tension of from about 30 centimeters to 340 centimeters (one-third atmosphere) of water on an undisturbed, saturated soil sample and applying it for either an arbitrary period of time or until the sample shows little additional loss in weight. Each method produces results for a different purpose, and the method is devised to provide these results within the limits of permissible error. The use and acceptance of different methods suggests that there is no simple method that will satisfy all requirements.

Figure 4 showing specific yield versus permeability has been prepared from about 9,000 laboratory tests of undisturbed samples which were taken from all types of soils. Both specific yield and permeability determinations were made on the same undisturbed sample, and the results are believed to be within 10 percent of best obtainable values. A value for specific yield within 10 percent is well within the limits of accuracy of the other terms which must be evaluated in drainage work, so the use of Figure 4 to obtain values for specific yield is recommended when permeability is known. Values for permeability must be obtained on undisturbed soil samples or by field tests (see Part 524 for field test procedures) and the latter is recommended.

The specific yield value used in drainage calculations should relate only to the volume of soil that is dewatered by the drain,

so the permeability value for entering the curve should be the average for the saturated profile above the drains. However, the permeability value for use in the drainage calculations is the average weighted permeability for the saturated zone above the barrier.

E. Barrier Zone. The barrier zone, also called the barrier stratum, barrier layer, or simply barrier, is not strictly a soil characteristic, but is a much-used term in drainage work that is closely related to the characteristics of the soil. The barrier zone is a stratum or layer which is slowly permeable and which restricts the movement of ground water. Because strata are found in a generally horizontal attitude as a result of the method of formation, the barrier zone is usually considered a barrier to the vertical movement of water. This is not exclusively the condition because, in areas of unconformity, the barrier may restrict horizontal movement of water.

When water percolating downward under the force of gravity reaches the top of a barrier zone, a saturated condition builds up and differential pressures are created. The water then begins to move laterally in the material above the barrier zone as a result of these pressures. Thus, in ground-water hydraulics the barrier zone limits the depth of material available for the movement of ground water.

F. Salts and Sodium. Usually of no consequence in humid-area drainage, salt and exchangeable sodium are factors which have some effect on

drainage in arid soils. Practically all of these soils contain salt or sodium, or both. Their effect of reducing crop production is most important, but they also have a secondary effect on drainage requirements and drain costs.

The expected presence of salt in the irrigation water and soils of arid areas poses the requirement of leaching to maintain a favorable salt balance in the root zone. This means that an equal or greater amount of salt must leave the soil in drainage water than is brought into the soil by irrigation water. It further means that the drainage requirement must contemplate the removal of the leaching water from the substrata. Removal of leaching water, which is in excess of normal deep percolation, may require an increase in the number of drains and thus the cost of the drainage system. In most cases, the deep percolation inherent with standard irrigation practices will maintain a favorable salt balance and an acceptable concentration in the soil solution. Should the investigations disclose the leaching requirement to be in excess of the leaching obtained with deep percolation associated with normal irrigation practices, the drainage requirements would have to be increased accordingly.

Maintenance of a favorable salt balance, as noted above, is a continuous requirement if agricultural production is to be maintained permanently. However, some soils have such high concentrations of

salts prior to irrigation that a heavy initial leaching with water of an acceptable salt concentration is required before agricultural production can begin. To be practical, the drainage provided cannot be more than the land will require for normal irrigation and cropping after initial improvement, which means that during initial improvement the water table will be higher than during normal irrigation.

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

Although the drainage requirement can be based on permeability in a sodic soil, this type of soil is sometimes hard to handle by ordinary construction methods, which increases the cost of construction. This "soupy" material may prevent an open drain from being excavated to grade because the sides continually slough in. Stage construction sometimes may be used to overcome this condition even though considerable time may be required to bring the drain to grade.

Especially for pipe drains, it is difficult to maintain line and grade, and close inspection is required to insure an acceptable installation. One possible solution is to place gravel in the trench until it will support the pipe, while another is to use cradles. The above condition is not exclusively a sodium problem, since it also occurs somewhat in a saturated fine sand or silt, but it is intensified if excessive sodium is present.

There is an additional problem in installing pipe drains in a saturated soil material. When a trench is excavated and backfilled for installation of the pipe, the soil is disturbed. This may cause it to puddle, which further decreases the already low permeability of a sodic soil or decreases the permeability of a nonsodic soil. There are cases where water stands over a pipe drain as a result of this condition rather than percolating into it and being carried away.

Needless to say, every effort must be made to avoid these conditions or to reduce their effects. Many times the sodium condition occurs in small areas in a field rather than covering the entire field. In this case it may be possible to locate the drain at the edge of the sodium area rather than traversing it. The location of the drain will depend on topography, the location of the sodium area within the field, permeabilities of soils adjacent to the area, protection required in the field, and related factors. The drain may be located upslope to intercept ground water before

it reaches the sodium area and deep enough to provide some drainage for the area itself. If it is necessary to traverse a sodium area, the soil should be disturbed as little as possible, and it is best to backfill the trench completely with a permeable gravel.

The following tabulation gives the accepted chemical limits for delineation of saline and sodic soils. It is of interest to the drainage engineer because it may give an indication of construction problems, but it is not conclusive. Conditions in an actual excavation must be correlated with chemical values to provide a basis for conclusions.

Soil	EC x 10^3*	ESP**	pH
Saline	> 4	< 15	< 8.5
Saline-sodic	> 4	> 15	8.5±
Nonsaline-sodic	< 4	> 15	8.5 - 10

*Conductivity in millimhos per centimeter at 25° C.

**Exchangeable sodium percentage.

.5 Surface Runoff. Surface flow must be considered in drainage analysis because this water also must be carried away from agricultural lands. Since all water moves toward the topographic low points, it is normal that in many cases both surface and subsurface flow will occupy the same disposal channel. Design considerations must include capacity for both sources of water.

Surface flow originates from precipitation and from irrigation waste. In general, estimates of flow from precipitation and from irrigation waste will be available to the drainage engineer for use in drain design. Where such estimates are not provided, the following simplified methods can be used to obtain quantitative values for these factors.

A. Precipitation. Normal precipitation in arid regions is usually a small part of the inflow to an irrigated area, and is thus of little concern in drainage investigations. It becomes increasingly important as irrigation reaches into semihumid areas. Excessive precipitation, however, usually in the form of storm rainfall, may be of primary concern in any irrigated area and must be considered in several aspects of drainage. It may be a factor in the subsurface drainage requirement, and is usually the controlling factor in surface drainage. Where grades are relatively steep or soils are easily erodible, handling storm-water runoff can be a difficult and expensive problem.

Precipitation records seldom have to be collected or compiled primarily for drainage investigations. Usually they will be available, but, if not, precipitation data can be obtained from records of the U.S. Weather Bureau for the area under consideration. Additional records may be available from other local sources. These records should be analyzed if the generalized data in the following paragraph do not appear to be applicable.

B. Storm Flow. Storm flow depends on topography, soils, vegetal cover, land use, and the hydrometeorological characteristics of the area. Surface drains should be designed to handle considerably less than the potential maximum flow. In general, 5- to 15-year flood frequencies should be used. Floods of lower frequency should be used where relatively expensive structures are involved, or where damage from them may dictate the need for a more conservative design. As the consequences of inadequate channel capacity are not usually too severe, refinement in estimating is not warranted.

Notwithstanding the many formulas and analytical methods available, the most practical way of estimating surface drainage requirements for storm runoff is by study of existing channels and culverts. The flood capacity or degree of protection used for farm and county roads and irrigation laterals, is about the same as that ordinarily considered adequate for surface drains. Thus provision of storm runoff capacity comparable to that for existing facilities is usually about right. If existing facilities have or once had capacities less than required for 5-year storms, they will show signs of inadequacy or will have been replaced.

Where there are too few existing culverts or drainage channels to permit comparison, some type of analytical method must be used. The McMath formula gives results which are generally considered fairly reliable:

$$Q = C\ i\ S^{1/5} A^{4/5}$$

where:

Q = flood discharge in cfs,

C = a coefficient representing the basin characteristics,

i = rate of rainfall in inches per hour for the time of concentration and frequency,

S = fall of main channel in feet per 1,000 feet, and

A = area of basin in acres.

Values of C will vary from 0.20 for low runoff conditions to 0.75 for high runoff conditions, depending principally on vegetation, soils, and topography. It increases as the vegetative cover becomes less dense, as the soil becomes heavier, and as the slope of the ground becomes greater. Of these three basin factors, vegetation and soil have the greater effect on C. A single characteristic, such as a rock surface, may at times have a controlling effect on C. In the usual case, however, no one characteristic will predominate, and all three must be considered before selecting a value for C. Arbitrarily weighing their relative importance as vegetation 40 percent, soils 40 percent, and topography 20 percent, will allow selection of appropriate factors for each, which can be added for a value of C.

Table 1

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 varies widely in the western United States. Many data are available and elaborate methods have been developed for studies where the need for refinement is indicated. Estimating storm runoff for surface drainage does not require such procedures. The U.S. Weather Bureau has prepared rainfall intensity-frequency data 2/ which can be used to good advantage. Figure 5 shows 5-year 1-hour rainfall for the western United States. 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 the one used. The time of concentration for a particular area depends principally on the length and slope of its main channel. It can be estimated with sufficient accuracy by the nomograph of Figure 6.

2/"Rainfall Frequency Atlas of the United States," Technical Paper No. 40, U.S. Department of Commerce, Weather Bureau, May 1961.

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

(1) Find value of C for physical conditions of the area from Table 1.

(2) Estimate the time of concentration (X) from Figure 6.

(3) Select a value (b) for 5-year, 1-hour rainfall from Figure 5 for the area under study.

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

For a time of concentration of 1 hour or greater:

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

For a time of concentration of 30 minutes:

$$y = 0.80\ b, \text{ in which}$$

y = 5-year, any-hour rainfall depth, inches,

b = 5-year, 1-hour rainfall depth, inches, and

X = required rainfall duration (time of concentration) hours. X must be greater than 1 hour.

(5) Convert y 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 $\frac{y}{X}$.

(7) Solve for Q by use of the McMath formula. Figure 7 gives the 1/5 and 4/5 powers of numbers.

C. Farm Waste. Farm surface waste from irrigation varies with many factors including soil textures, land slopes, length of irrigation run, and irrigation efficiency. Under best conditions with good management, it is possible to irrigate without any waste water leaving the irrigated area, but this is the exception rather than the rule. A deep sandy soil with practically flat slopes and short runs is most easily managed to have no farm waste, 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 irrigation operations in an area show that it is not needed.

Since 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 the 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. These criteria are usually shown on curves similar to Figures 8 and 9 , or curves can be prepared from the criteria. The canal and lateral capacity curves are based on the soil, climatic, cropping pattern, and similar factors for the particular project, and they 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.

A topographic map on which the irrigated land and the drain are located will permit determination, for any point on the drain, of the total irrigated acreage whose farm waste must pass through that point on the drain. The lateral capacity for that acreage can be taken from the curve. By applying a factor to that capacity, which factor will vary with those items enumerated above, a drain capacity allowance for farm waste can be obtained. For the average irrigation project a factor of about 15 percent can be used.

For example, assume that the topographic map shows that there are 800 irrigable acres which slope toward the point on the drain in question. From Figure 8, a lateral capacity of 20 cfs is found for 800 acres. The drain capacity for farm waste at the point would be 15 percent of 20 cfs, or 3.0 cfs.

Figure 1
Par. 523.0.2

SYMBOLS FOR DRAINAGE MAPS

CANAL OR LATERAL

PROPOSED CANAL

MH# MANHOLE

CLOSED DRAIN

PROPOSED CLOSED DRAIN

OPEN SUBSURFACE DRAIN

PROPOSED OPEN SUBSURFACE DRAIN

OPEN SURFACE DRAIN

PROPOSED OPEN SURFACE DRAIN

NATURAL DRAIN

LIMIT OF GROUND-WATER INVESTIGATION

1820 GROUND-SURFACE CONTOUR

4225 OTHER CONTOURS (Heavy Solid Contours can be used when surface contour does not appear on same drawing.)

PROPOSED	COMPLETED		PROPOSED	COMPLETED	
○	●	CASED HOLE	◎	◉	IRRIGATION WELL
⊖	⊕	UNCASED HOLE	△	▲	DRAINAGE WELL
□	■	TEST PIT	⊡D	⊡I	OTHER WELLS (Use letter to designate type)

All wells, holes, and test pits should have identification number

SYMBOLS AND CORRESPONDING COLORS FOR GROUND WATER OR BARRIER DEPTH

RED	FIRST INTERVAL BELOW GROUND SURFACE
BLUE	SECOND INTERVAL BELOW GROUND SURFACE
ORANGE	THIRD INTERVAL BELOW GROUND SURFACE
GREEN	FOURTH INTERVAL BELOW GROUND SURFACE
NO COLOR	DEPTHS GREATER THAN LAST INTERVAL USED

The depths for each interval should be shown on all maps

CONVENTIONAL SIGNS III

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40-D-5063

Figure 2
Par. 523.0.4B

CONVERSION TABLE
COEFFICIENTS OF PERMEABILITY

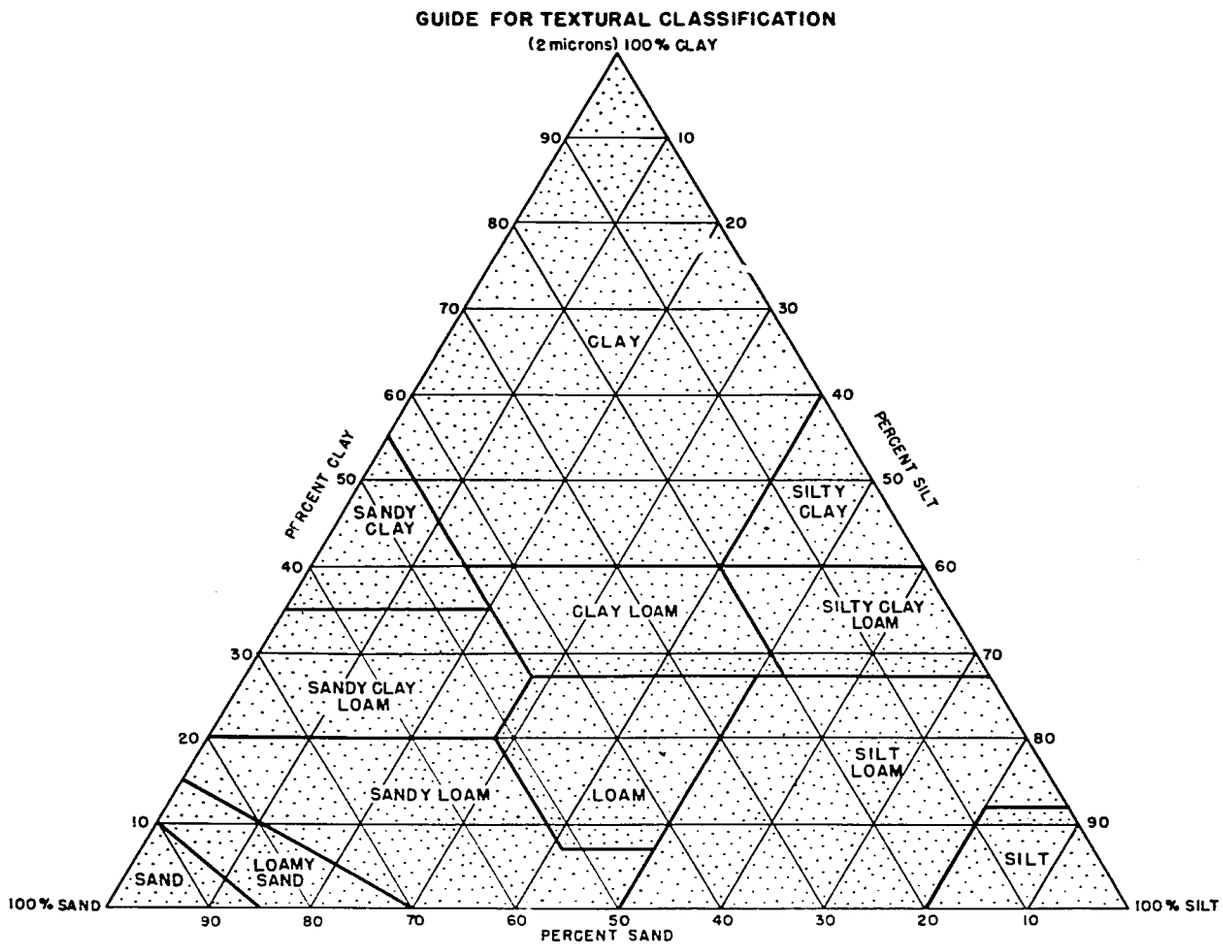
$ft.^3/ft.^2/yr.$ ①	$ft.^3/ft.^2/dy.$ ②	$ft.^3/ft.^2/hr.$ ③	$ft.^3/ft.^2/sec.$ ④	$in.^3/in.^2/dy.$ ⑤	$in.^3/in.^2/hr.$ ⑥	$gals./ft.^2/dy.$ ⑦	$cm.^3/cm.^2/hr.$ ⑧
	①÷365	②÷24	③÷3600	②×12	⑤÷24	②×7.4805	③×30.48006
1	.00274	.000114	.0000000317	.0329	.00137	.0205	.00348
365	1	.04167	.0000116	12	.50	7.481	1.27
8760	24	1	.000278	288	12	179.53	30.48
31,536,000	86,400	3600	1	1,036,800	43,200	646,315	109,728
30.416	.0833	.00347	.000000964	1	.04167	.623	.106
730	2.0	.0833	.0000235	24	1	14.96	2.54
48.80	.1337	.00557	.00000154	1.604	.0668	1	.1698
287.402	.787	.0328	.00000911	9.450	.3937	5.89	1

Examples:

1. The permeability of a soil has been determined to be 15 gals./$ft.^2$/dy. What is this in $in.^3/in.^2/hr$? - Find value of 1 in Col. ⑦ and move horizontally to value for $in.^3/in.^2/hr.$ in Col. ⑥. Multiply value in Col. ⑥ (.0668) by 15 = 1.002 $in.^3/in.^2/hr.$

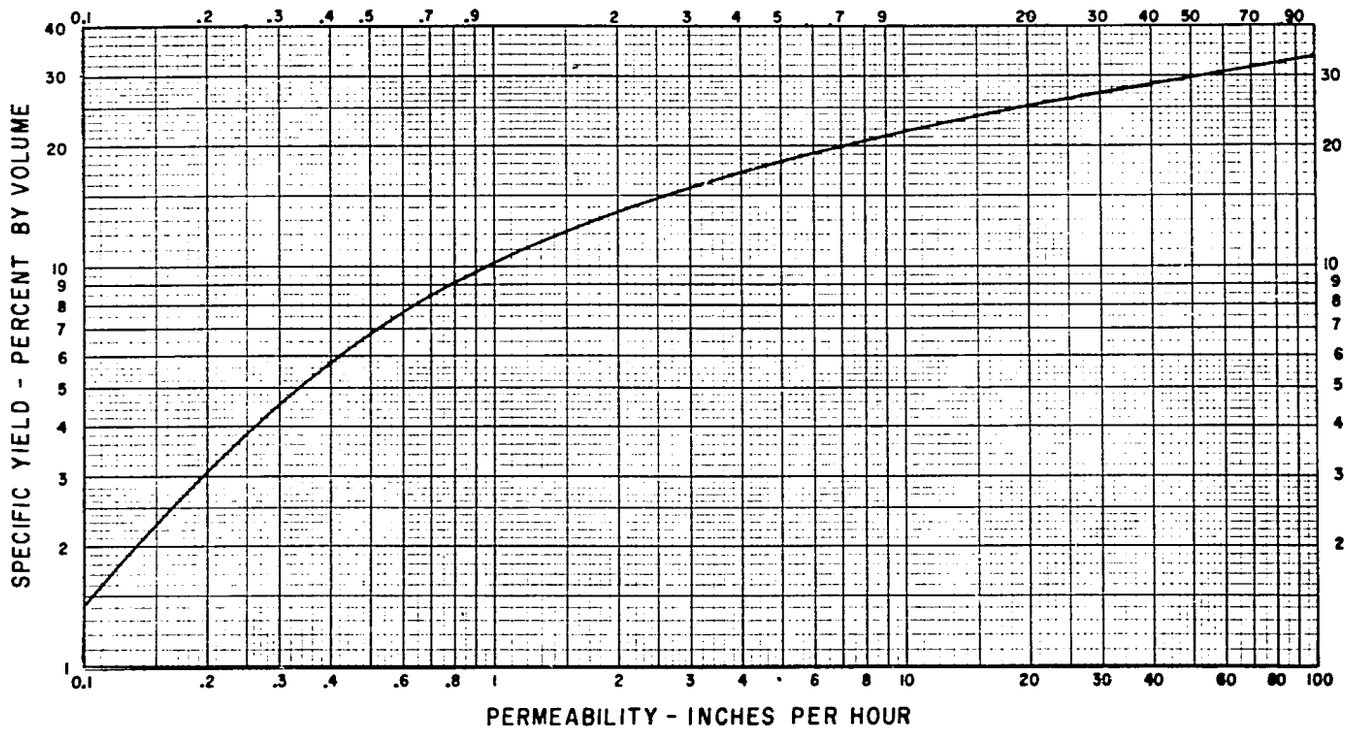
2. The permeability of a soil has been determined to be 4000 $ft.^3/ft.^2/yr.$ What is this in gals./$ft.^2$/dy.? - Find value of 1 in Col. ①. and move horizontally to value for gals/$ft.^2$/dy. in Col. ⑦. Multiply value in Col. ⑦ (.0205) by 4000 = 82.0 gals./$ft.^2$/dy.

103-D-690



Source: U.S. Department of Agriculture, Handbook No. 18 August 1951

Figure 3
Par. 523.0.4B



CURVE SHOWING GENERAL RELATIONSHIP BETWEEN SPECIFIC YIELD AND PERMEABILITY

103-D-693

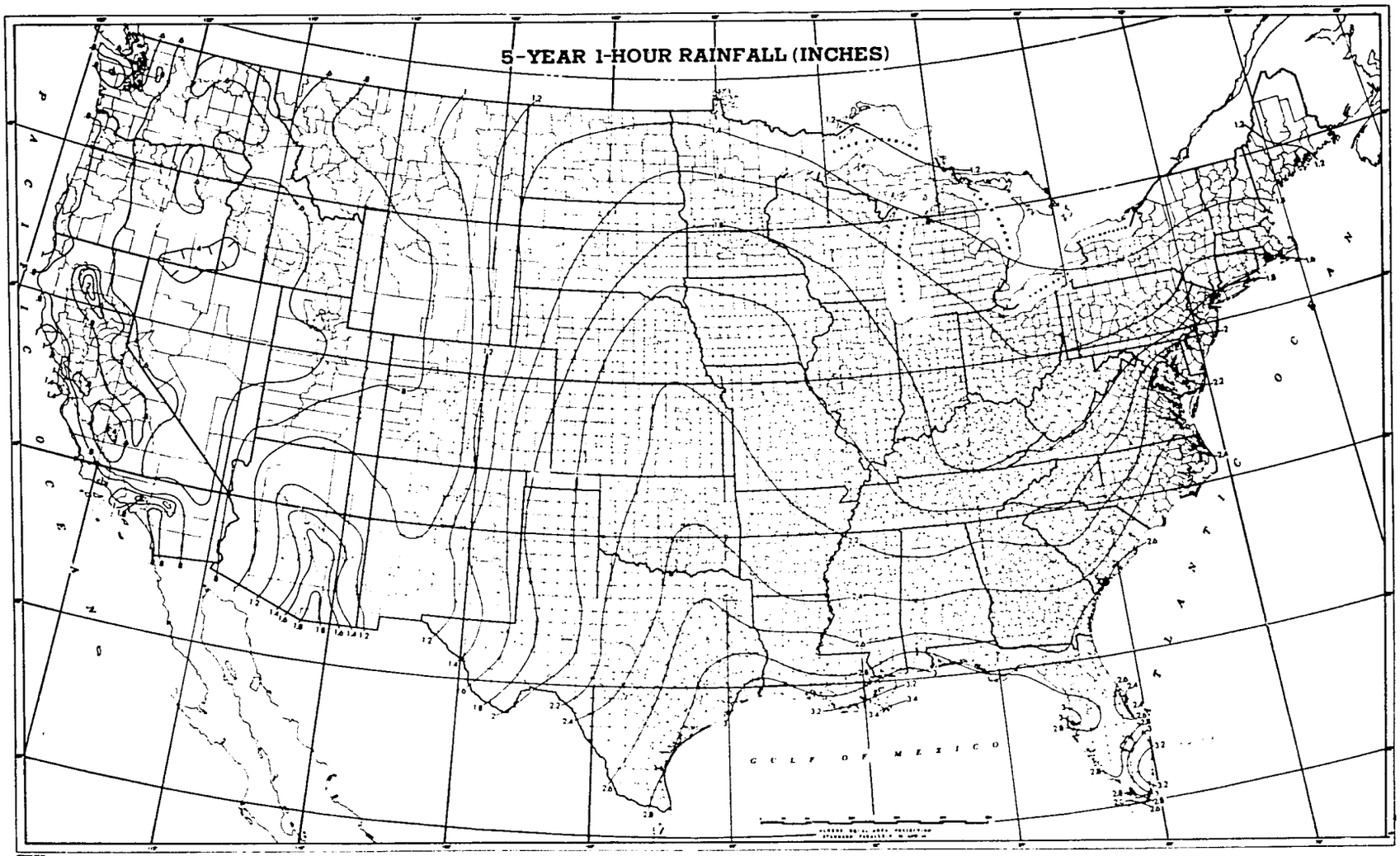
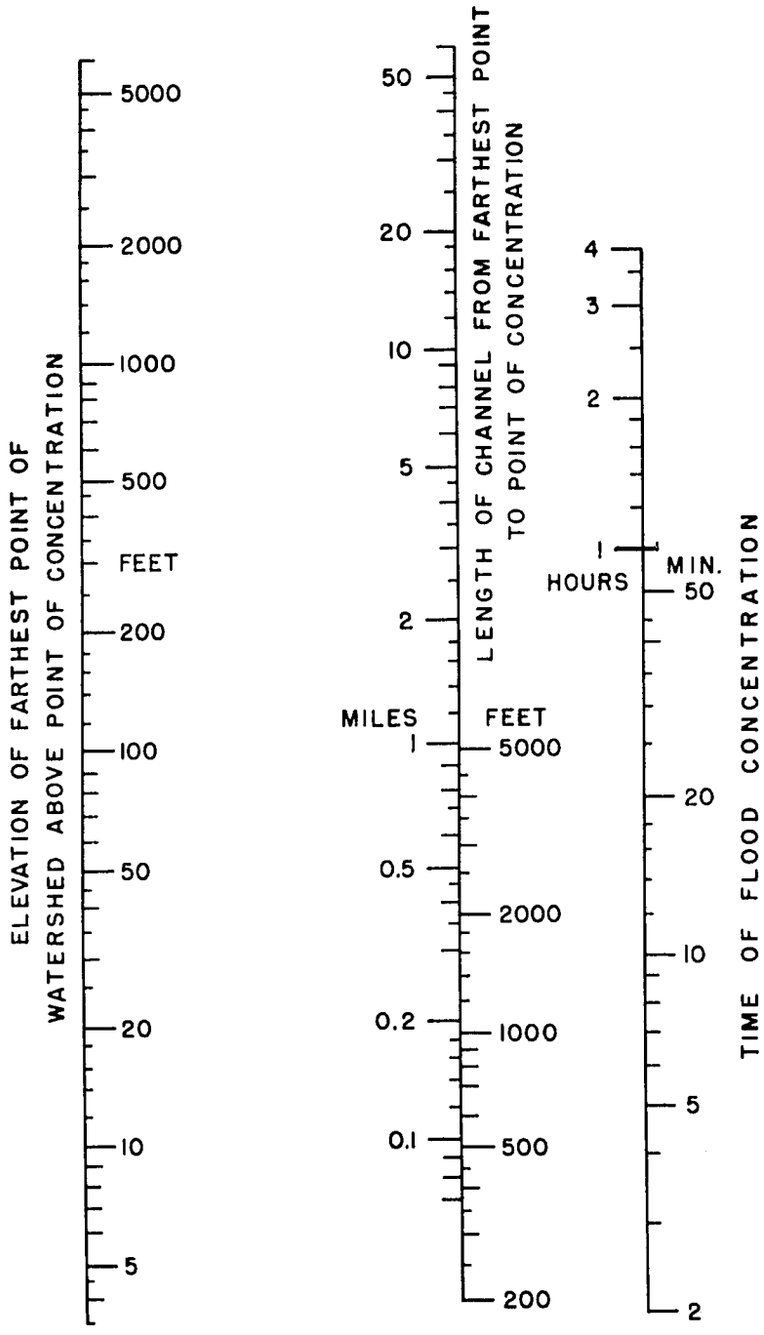


Figure 5
Par. 523.0.5B

Figure 6
Par. 523.0.5B



TIME OF FLOOD CONCENTRATION

103-D-692

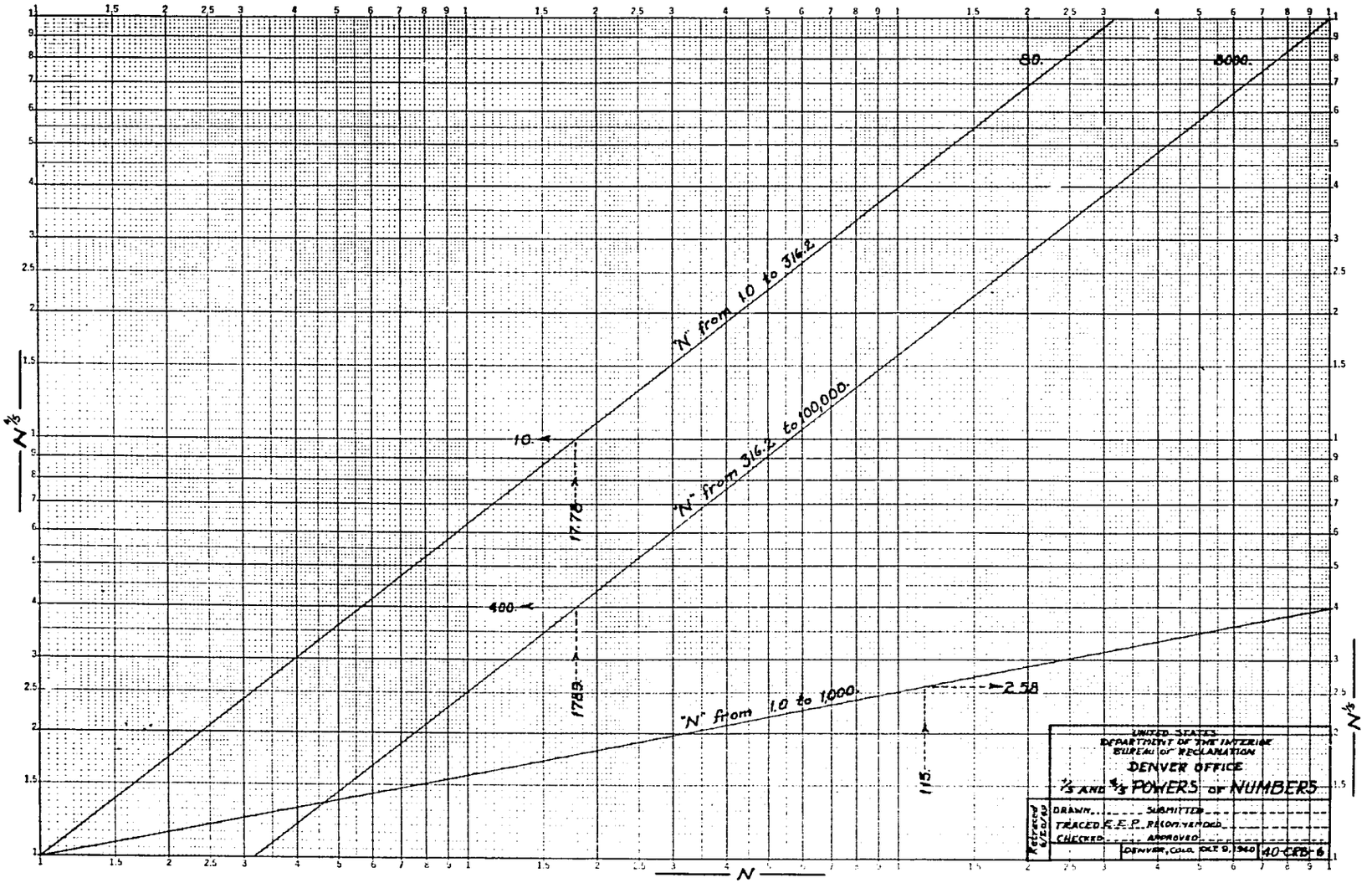
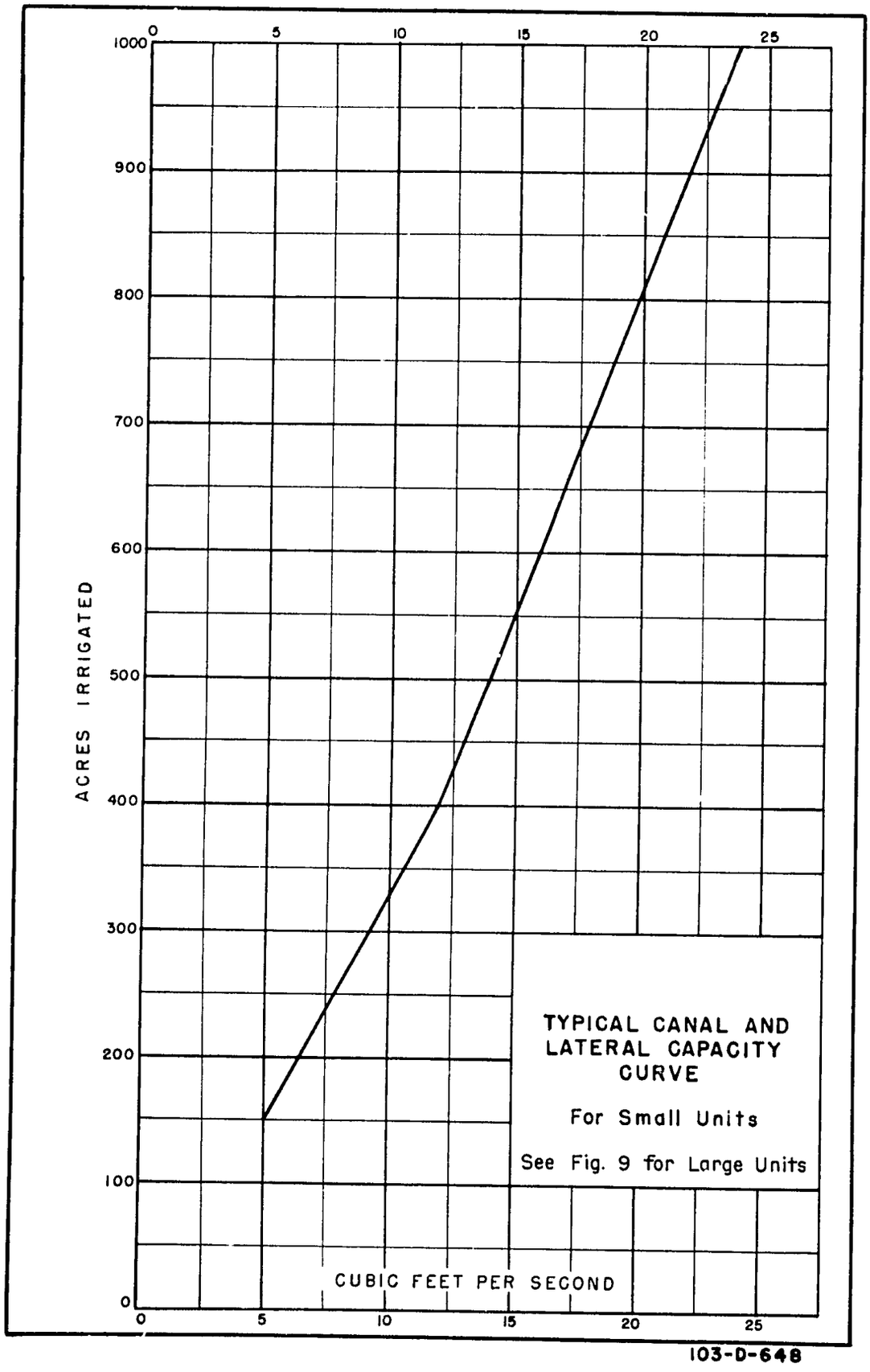


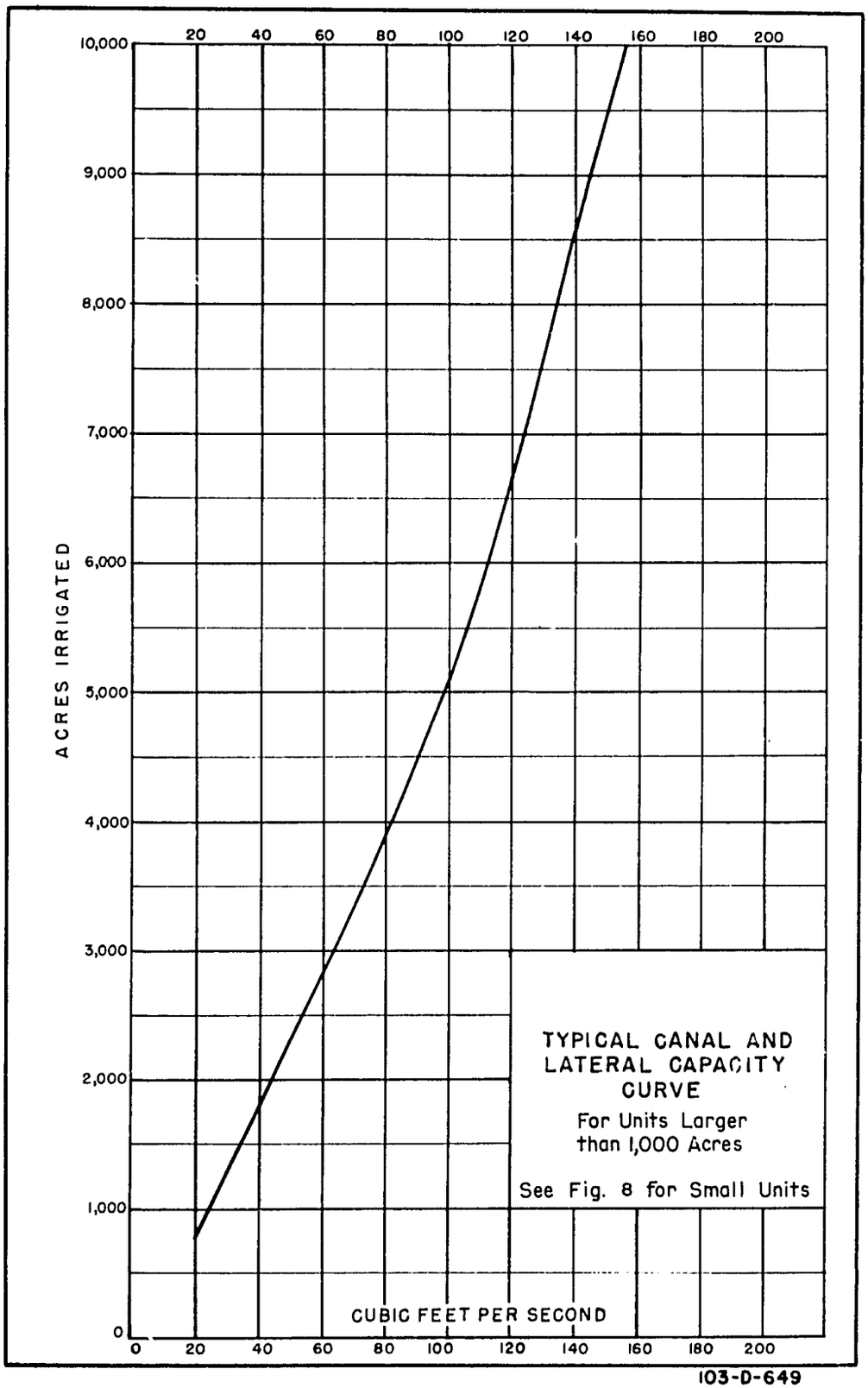
Figure 7
Par. 523.0.5B

Figure 8
Par. 523.0.5C



103-D-648

Figure 9
Par. 523.0.5C



103-D-649

Reclamation Instruction Series 520

LAND DRAINAGE TECHNIQUES AND STANDARDS

Part 524 Field and Laboratory Procedures

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Figure 1 Drainage Investigations Test Hole Record Book

2 Typical Sub-area Map--Drainage Investigations

3 U.S. Geological Survey Well Numbering System

4 Devices for Measuring Depth of Water in a Well

.1 Objective. A number of methods for conducting in-place permeability tests below a water table have been developed. The two that have been found most adaptable are the auger-hole and piezometer tests. It is not essential that the tools and methods described herein be used, since others will serve equally as well. The objective is to obtain the rate of change of the water level in a hole, or the difference of water-level elevation with time, and any procedure or tool that accomplishes this objective is satisfactory.

AUGER-HOLE TEST FOR PERMEABILITY

.2 Introduction. The auger-hole permeability test measures the average horizontal permeability of the soil profile from the static water table to the bottom of the hole when an impermeable layer is at the bottom of the hole, or to a few inches below the bottom of the hole when an impermeable layer is at some distance below the bottom of the hole.

A number of workers have described the auger-hole test. Maasland and Haskew discussed it in great detail,1/ and those interested in the analytical details should consult the paper they prepared. Only a procedure for making field measurements and calculating permeability will be described here.

1/Maasland, M., and Haskew, H. C., 1957, "The Auger Hole Method of Measuring the Hydraulic Conductivity of Soil and its Application to Tile Drainage Problems," 3rd Congress, International Commission of Irrigation and Drainage. R. S., Questions 8:8.69-8.14.

.3 Equipment. Equipment requirements are somewhat flexible, but the following items have been used successfully:

4-inch-diameter auger with three 5-foot handle extensions

Recorder board, recording sheets, and float apparatus

Tripod

Measuring rod or tape

Hole brush

Bailer

Stopwatch

Inside calipers

Computation sheets and clip board

Burlap

Perforated casing or wire-wound well screen when and as required

Mirror or strong flashlight

Windshield

A 4-inch-diameter auger is preferred. It can be either the Dutch type, Orchard type, or Durango type (Figure 1). The Orchard or Durango type is generally best for use in lighter textured (sandy) materials, and the Dutch type in heavier, stickier soils. Samples from the Durango-type auger are less disturbed than those from the other two types and can be more easily examined for soil structure.

The recorder board, recording sheet, and float apparatus are preferred instead of an electric sounder or other measuring equipment. (See Figures 2 & 3.) The float and recorder board is preferred because it

is inexpensive and easy to construct, is simple to operate, and provides a permanent record. The board most commonly used is 5 feet long and 6 inches wide. For carrying convenience, it can be cut and hinged 1 foot 3 inches from each end, which cuts its length to 2 feet 6 inches. Nylon rollers similar to those used on drapery floor guides, or small grooved wheel casters, are positioned at both ends of the board to prevent the string from dragging. The string rides in a slot extending the length of the board. ¶ The float should be less than 3 inches in diameter and weighted at the bottom so it will drop fast. It should also be sufficiently buoyant so there will be no lag in the pointer as the water table rises in the hole. The float should have sloping shoulders so it will be less likely to catch on pebbles or roots on the sides of the open hole or on the joints and perforations when casing is used. The pointer is made of aluminum or other lightweight material. It is attached to the float string and used on the recorder board to show the position of the float. The counterweight used to keep the float string tight should be only slightly lighter than the float. The recorder sheets are 5-foot paper strips graduated in hundredths of a foot, and are easily made by sawing 2-inch lengths from rolls of 10 by 10 cross-section paper. Some operators prefer to mark the graduations on the board and use blank strips of paper for the recorder sheets. Either method provides a permanent record of the time and recovery of the water table for each test.

Any rigidly constructed tripod can be used. Planetable tripods furnish a rigid support and a fast method of setting up and leveling the measuring board. A planetable head can be attached to the recorder board.

A 15-foot measuring rod graduated in tenths of a foot can be made, or a tape with a weight on the bottom can be used. Or, to minimize equipment, the three 5-foot extensions for the auger can be marked and used as a measuring rod.

A hole brush can be made from a 10- by 3-inch floor scrub brush cut in two and bolted together back-to-back around a coupling to fit the pipe on the 4-inch auger.

A bailer can be made from a 3-foot length of 3-1/2-inch downspout, with a rubber or metal foot-valve at one end and a handle at the other end. Bailers longer than 3 feet will be difficult to get in and out if the hole is not straight. The hole in the foot-valve should be large to allow water to enter as rapidly as possible. The bailer should be weighted at the bottom so it will drop fast when more than one bail is required to empty the hole.

Any standard second and minute stopwatch is satisfactory when the float apparatus is used. All readings should be made from a single reference time which is the beginning of bailing, and all time during a test should be accounted for.

An ordinary pair of inside calipers can be used to determine the diameter of the hole. To prevent the points of the legs from gouging

the walls of the auger hole, small flat plates are welded to the legs. A rod screwed into the top of the calipers is used to determine the hole diameter at depth. The average hole diameter is used in the calculations. The diameter cannot be checked below the water table with ordinary inside calipers because the water surface reflects the light and prevents a visual determination of the contact of the calipers with the sides of the hole. For this reason, the average hole diameter is determined by the average of measurements made about 1 foot below the ground surface and just above the water table.

Computation sheets should be made up, using the example shown in Figure 4.

The burlap is used to prevent muck from entering at the bottom of the hole. A piece about 2 feet square is required for each hole.

Perforated casing or wire-wound well screen is necessary in unstable soils. It should have the same or slightly larger outside diameter as the auger hole, so as it is pushed into the ground there is a definite contact between the casing and the periphery of the hole. Commercial well screen with at least 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 should be satisfactory. About sixty 1/8- by 1-inch hacksaw perforations per foot will give 4 to 5 percent perforations and have negligible effect on limiting the amount of water entering the casing for most agricultural soils tested.

A mirror or strong flashlight is used to examine the sides of the auger hole and facilitate measurements with the calipers.

.4 Procedures. The most efficient team for performing the auger-hole field test for permeability consists of three men. One man operates the recorder board, one puts the float in the hole and operates the stopwatch, and the third operates the bailer. After the water level in the hole has become static, an experienced three-man team can perform the entire test in 10 to 15 minues in most soils, the time depending upon the permeability of the soil being tested.

The hole should be augered as straight as possible to the required depth, which in turn depends on the depth to water table, the depth to a slowly permeable zone, and the soil strata to be tested. If the soil is homogeneous throughout the profile to be tested, the hole can be excavated to the total depth to be tested. When the soil is heterogeneous, it is sometimes desirable to make several tests at varying depths. If the material is highly permeable throughout the profile to be tested, it is best to stop the hole about 2 or 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 brushed to break up any sealing effect caused by the auger. The burlap is then forced to the bottom of the hole and tamped lightly to prevent any materials from entering the bottom. The water table is then allowed to reach its static level in the hole. Careful measurements are made of the depth to the static water table from the ground surface, the total depth of hole, and the distance from the static water table to the bottom of the hole.

To begin the test, the tripod with the recorder board, recording sheets, 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 and, when it becomes steady at the static water-table level, the pointer is set at zero on the recording sheet and the counterweight positioned so the full change of water-table level can be recorded. The float is then removed and the water is bailed from the hole as quickly as possible to minimize the amount of water which returns before the readings are started. For best results most of the water should be bailed from the hole so the test can be completed before the water level rises to half of 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, the float is placed in the hole as quickly as possible. (When a very rapid rise of the water level in the hole is experienced, it is sometimes advantageous to leave the float in the hole and below the bailer during the bailing process. This minimizes the amount of water returning into the hole before the first reading can be made.) The stopwatch is started at the moment of withdrawing the first bailer and should be run continuously until the test has been completed.

When using the recorder board and float mechanism, it is most convenient to use equal time intervals, starting from the initial tick mark. As the intervals come up on the stopwatch, the operator says "Mark," Each time the recorder hears the word "Mark," he puts a small tick mark

opposite the pointer. Measurements are continued until the 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 board of 0.8 Y_o has been reached, Y_o being 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 sheet. Any irregularities in the record can be quickly observed on the recorder sheet, and if readings are highly irregular, the test should be rerun after a static water table has been reestablished. Only the period covering the equally spaced tick marks is used in the computations. There will usually be one irregular spacing at the beginning while the float is settling down. As the water rises and the hole fills, the marks will no longer be equally spaced, but will become closer with each reading. The beginning of the shorter spacings usually compares fairly well with the 0.8 to 0.75 Y_o calculation.

.5 Calculations. Upon completion of the auger-hole field test for permeability, the time intervals and the corresponding distances between tick marks on the recorder board sheet are transferred to the computation sheet. Sample computations are shown in Figure 4. Care should be taken in selecting consistent consecutive time intervals and water-table rises to be used in determining $\overline{y_n}$, $\overline{\Delta y}$, and $\overline{\Delta t}$ ($\overline{y_n}$ is the average distance from static water table to the water surface in the hole during the test period; $\overline{\Delta y}$ is the average incremental rise

during incremental time intervals; and $\overline{\Delta t}$ is the average incremental time interval between ticks, usually a constant when the float apparatus is used).

The C values determined from the charts of Figures 5 and 6 are for conditions where the barrier is considered to be at infinity and at zero distance, respectively, below the bottom of the hole. The way Maasland and Haskew (1957) plotted C against the dimensionless parameter $\bar{y}_n/r$ makes the determination of C easy for a wide range of values of H/r and $\bar{y}_n/r$. For the usual case where there is no barrier above the bottom of the hole, Figure 5 should be used. If the hole penetrates into a slowly permeable zone below a permeable zone, Figure 6 should be used to obtain the C value, with H being 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 permeability can be determined by multiplying the C factor by $\overline{\Delta y}/\overline{\Delta t}$. The permeability will be in feet per day, and by dividing by 2 the permeability in inches per hour can be obtained.

.6 <u>Limitations of the Auger-hole Test</u>. While the auger-hole test furnishes reliable permeability data for most conditions, the results are entirely unreliable under artesian conditions; that is, when the hole penetrates a permeable zone under pressure underlying an impermeable zone. Another condition which usually makes the test difficult to perform and gives unreliable data as well, is when there are small sand lenses between less permeable layers. Water flowing into the hole through the lenses falls

on the float, causing erratic readings. Another condition where the test cannot be used is when the water table is at or above the ground surface. Surface water or water running through permeable surface layers will cause erroneous rise increments. A depth of more than 20 feet to water table, while not a limitation as far as obtaining reliable data is concerned, entails considerable more work in making the test.

Comparatively high permeability rates, in the magnitude of 10 inches per hour or more, make the auger-hole test hard to perform because the bailer cannot remove the water as fast as it enters. At the other extreme, auger-hole tests in soils with permeability rates in the range of 0.001 to 0.01 inch per hour usually give such erratic readings that accurate values cannot be obtained. However, such results can be valuable in analysis and determination of drainage requirements even though exact values are not obtained. The knowledge that permeabilities are very high or very low can be quite useful from a practical standpoint.

The auger-hole test cannot always be performed in rocky or coarse gravel material, because it is usually not practical to auger or dig a hole of uniform size through these materials. Casing can sometimes be used to stabilize the walls of the hole in case a test is needed in these materials. In general, however, most agricultural soils being investigated for subsurface drainage systems can be tested by the auger-hole method if a water table exists near enough to the surface.

.7 Step Tests in Layered Soils. Sometimes it is valuable to know the permeability of individual strata within a soil profile or to know the variation in permeability with depth. This can be found by an auger-hole step test although the piezometer test, described later, may be more adaptable for testing permeability at greater depths.

The auger-hole step test consists simply of a series of auger-hole tests at the same hole location but at different depths. The hole is augered to the first depth and the auger-hole test is then run. This depth will be to within 3 or 4 inches of the bottom of the first stratum if the objective is to obtain the permeability of each stratum, or to any selected depth if the variation of permeability in the profile is required. The hole is then augered to within a few inches of the bottom of the next stratum, or to the next selected depth, and a second auger-hole test is run. This procedure is continued until the desired depth is reached. The permeability value calculated for each step will be the average value from the water table to the depth of the hole. The permeability for the individual stratum, or for the portion of the profile between the depth of one test and the depth of the next test, is found from the formula (Figure 7):

$$k_{xn}\, d_n = k_n\, D_n - k_{n-1}\, D_{n-1}$$

where:

k_{xn} = permeability to be determined,

k_n = permeability obtained in the nth test run,

k_{n-1} = permeability obtained in the (n-1) test run,

d_n = thickness of the nth stratum,

D_n = total depth of the nth test from the static water level,

D_{n-1} = total depth from the static water level for the (n-1) test, and

n = number of the test run or stratum.

PIEZOMETER TEST FOR PERMEABILITY

.8 Introduction. The piezometer test measures the horizontal permeability of thin layers in soils below a water table. In subsurface drainage investigations, its primary application is to provide data for determining which layer below a proposed drain depth is the effective barrier layer. An indication of the location of this layer can be determined from permeability data obtained from disturbed (remolded) samples taken at a change of structure, texture, or density; or from observation of dense shale, sandstone, or bedrock. However, at times there may be some doubt as to which layer should be considered the barrier. When permeability tests on disturbed samples are not available, when they give inconclusive results, or when more reliable determinations are desired, the piezometer test should be used.

.9 Equipment. Suggested equipment required for the piezometer test is as follows:

Casing of minimum 1-inch ID, thin-walled electrical conduit of suitable length for depths to 10 feet and black iron pipe with smooth inside walls for depths greater than 10 feet

Ship auger which fits inside the casing (see Figure 1)

Pipe-driving hammer, consisting of a piece of 2-inch iron pipe that fits over the casing, with a 10-pound weight fixed to the pipe

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

Stopwatch

Recorder board, recording sheets, and float apparatus or an electrical sounder. The float is made similar to the one used for the auger hole test but of smaller size to fit into the smaller diameter casing. The counterweight must be adjusted accordingly.

Computation sheets and clipboard

Measuring tape or rod

Windshield

Casing puller

.10 Procedures. A three-man team has been found best for making the piezometer field test for permeability, but it can be made by two experienced men.

The test layer should be at least 12 inches thick so that a 4-inch length of uncased hole, or cavity, can be placed in the middle of it. This is especially important if there is a marked difference in the texture, structure, or density of the layers above and below the test layer. After the layer to be tested has been selected, the topsoil is removed from the

ground surface, and a hole is augered to within 2 feet of the layer to be tested. Some operators prefer to auger 6 to 12 inches, then drive the casing, repeating this process for the entire length of hole. This is a slower method, and experience does not seem to warrant its use. Other operators jet the casing to within 2 to 3 feet of the test layer and then auger and drive the casing the remaining distance. This requires additional equipment, and in a waterlogged field jetting equipment usually cannot be moved in. The augering and driving procedure is always used for the last 2 feet to insure a good seal and also to minimize soil disturbance. The casing is stopped at the depth selected for the top of the 4-inch-long cavity, and the cavity is augered below the casing.

The size and shape of the cavity are important in the test, so care should be taken to make it the predetermined length and diameter. If the soil in the test layer is unstable and the cavity will not remain open during the test, screens should be made that can be pushed down inside the casing. For a 1-inch ID casing and a 4-inch cavity, the screen should be 5 inches long and 15/16 inch OD, with a rigid point welded on the bottom. A pole about 3/4 inch in diameter can be used to push the screen to the bottom of the cavity. If a small bent nail or hook is placed on the opposite end of the pole, the screen can sometimes be reclaimed at the end of the test by hooking the nail into the screen and pulling it up into the casing. The cavity is cleaned by pumping or bailing water and sediment out of the hole until the discharge is clear.

After the static water table has been established, the recorder board and float apparatus are set up, and the float dropped down the casing. See Figure 2 for the equipment setup. When the float comes to rest, the pointer is set at zero on the recorder sheet. The float is then removed from the hole and the water pumped or bailed out. A small foot-valve for attachment to the end of the suction line on the pitcher pump can be made similar to larger commercial types, or a bailer similar to that used in the auger-hole test can be made from small conduit. On stopping pumping or bailing, the float is immediately dropped down the casing, and when it starts to rise, a tick mark is made on the recorder sheet and at the same time the stopwatch is started. In using the recorder board and float, it is easier to select a convenient time interval between observations and corresponding tick marks on the recorder sheet. When an electrical sounder is used, it is more convenient to select an increment of equal water-level rise which will give a convenient, though variable, time interval. It is not essential to remove all of the water from the piezometer because measurements can be obtained and used anywhere between the static water-table level and the initial bailed-out level, but use of three or four readings during the first half of the rise will give more consistent results.

When the piezometer test is used to determine the barrier layer, tests must be made in two or more layers. This can be done by first making the test in the top layer and then augering and driving the same casing progressively to the next layers to be tested. The barrier layer is not necessarily the

layer with the lowest permeability, but rather the layer that has a marked decrease in permeability as compared with the weighted permeability of the more permeable layers above it. (See Paragraph 526.6.7.)

.11 Calculations. After completion of the piezometer test, the permeability is calculated from the equation developed by Kirkham (1945), which is as follows:

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

where:

k = permeability (inches per hour),

Y_1 = distance from static water level to level at time t_1 (inches),

Y_2 = distance from static water level to level at time t_2 (inches),

D = diameter of casing (inches),

t_2-t_1 = time (seconds) in which water level changes from Y_1 to Y_2, and

A = a constant for a given flow geometry (inches).

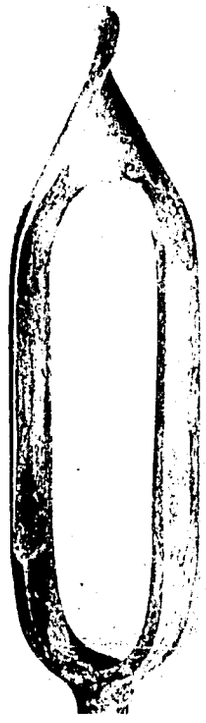
The factor A may be taken from the curve shown in Figure 8. The curve is valid when d and s are both large compared to w, (d = distance from the static water level to bottom of piezometer; s = distance below bottom of cavity to top of the next zone; and w = length of cavity). According to Luthin and Kirkham (1949), when s = 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.

In addition to the curve, a sample calculation sheet is shown on Figure 8.

.12 Limitations of the Piezometer Test. One of the principal limitations in the use of the piezometer test for permeability is that it cannot be used in gravel or coarse sand material due to installation and sealing difficulties. Even when the hole can be augered in these materials, rocks on the sides of the hole might dent or rip the casing. Also, when the casing bottoms in coarse gravel, it is impossible to obtain a satisfactory seal.

Twenty feet is about the practical limit in depth, both for installation and water removal. Duplicate tests in soils of very low permeability, in the range of 0.001 to 0.01 inch per hour, are always in a low range, but they can vary as much as 100 percent. This much variation is of little consequence in this range. Test layers less than about 10 to 12 inches thick, between more permeable materials, will not give reliable results, probably because of the influence of the more permeable materials. The size of the casing is a matter of preference, as long as it is 1 inch or more in diameter. Pipe diameters of 4 inches or more are difficult to install at depths over 10 feet.

Figure 1
Par. 524.1.3



(A) Dutch (Open) Type

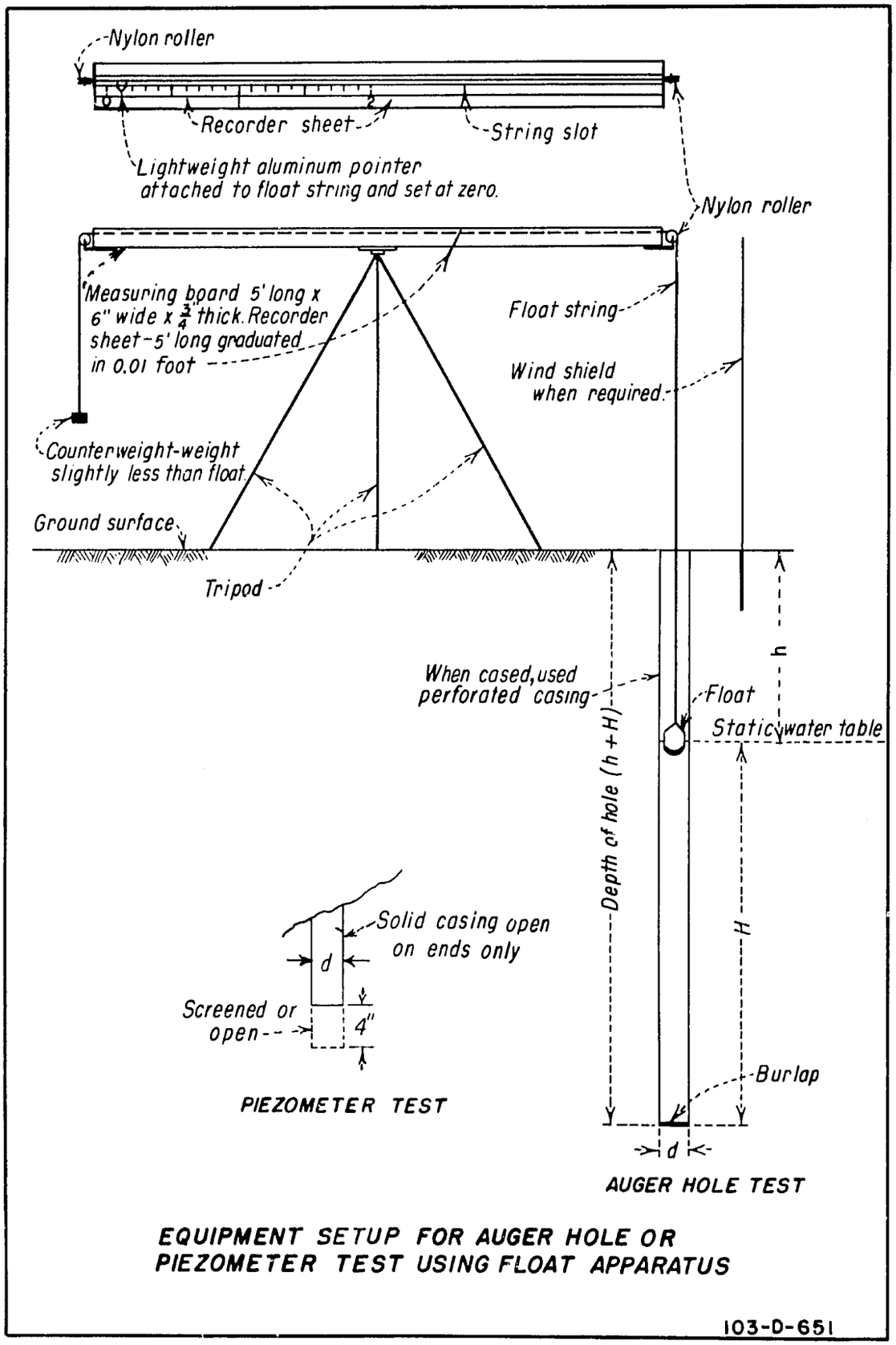
(B) Orchard Type

(C) Durango Type

(D) Ship or Helical Type

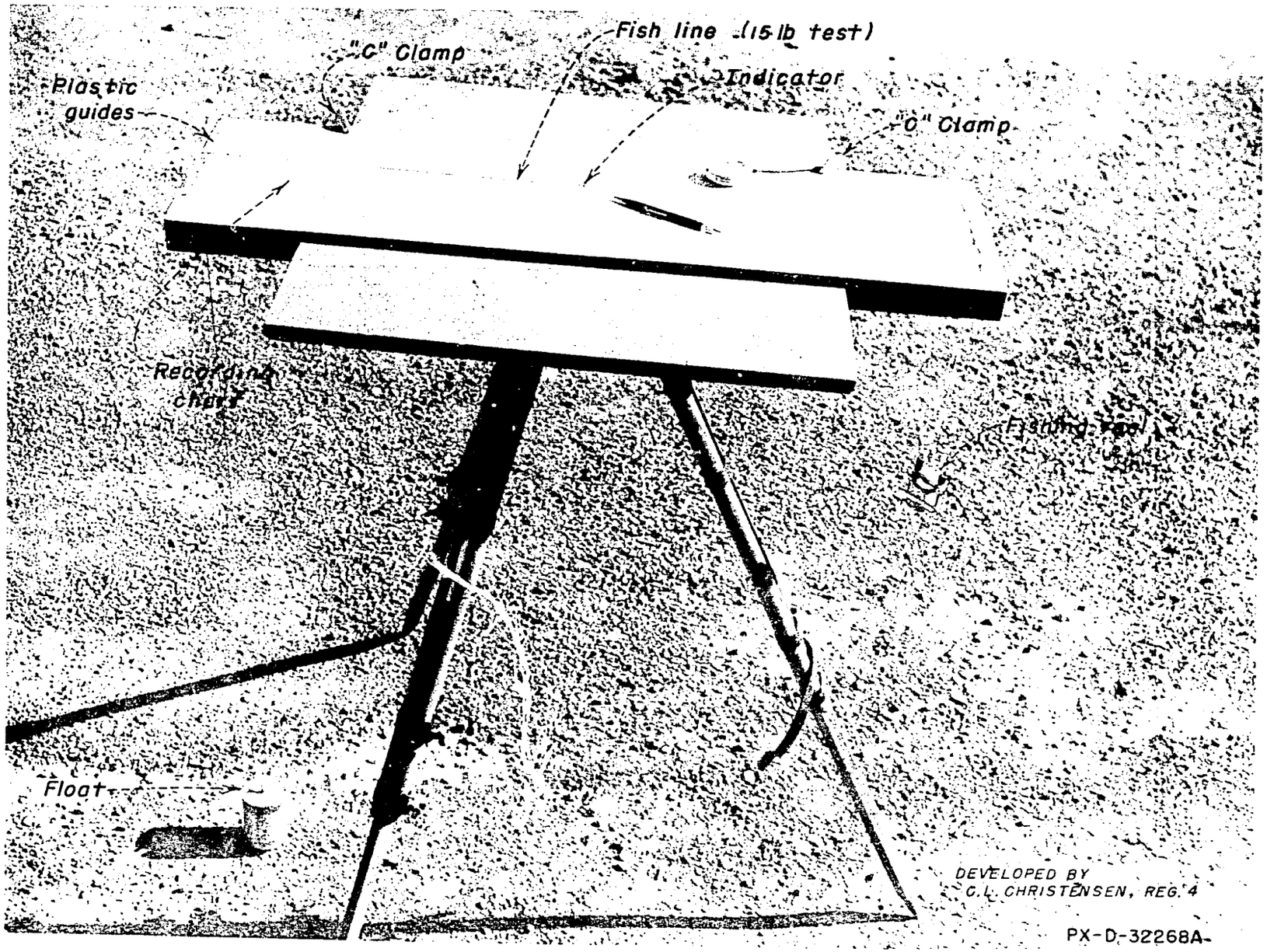
SOIL AUGERS

Figure 2
Par. 524.1.3



EQUIPMENT SETUP FOR AUGER HOLE OR PIEZOMETER TEST USING FLOAT APPARATUS

103-D-651



PHOTOGRAPH OF EQUIPMENT SETUP FOR AUGER HOLE OR PIEZOMETER TEST

Figure 3
Par. 524.1.3

Figure 4
Par. 524.1.3

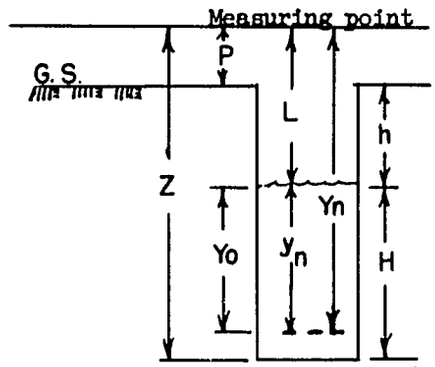
Hole No. E-4 Location Sample Farm

Observer A. P. Brown Date October 8, 1958

Hole: Cased /X/ Uncased / / Hole Dia. 4 inches Radius 0.167 feet

Equipment: Float apparatus

Log	Description of material
SURFACE	
1–9 SL	0' - 9' Lt. Brown Sandy Loam, friable, nonsticky, granular. Becomes wet at about 5', slightly compacted below 6'. Appears to have good permeability.
	WT = 4.80'
9 BOTTOM	Barrier at 11 ft



1. Measuring point to ground sur. = P = 4.10 ft
2. " " to static water sur. = L = 8.90 ft
3. " " to bot. of hole = Z = Ini. 13.10 ft
   Final 13.10 ft
4. Ground surface to static W.S. = h = 4.80 ft
5. Static W.S. to bottom of hole = H = 4.20 ft
6. Depth of hole h + H = 9.00 ft
7. Initial drawdown in hole = Y_o = 3.15 ft
8. 0.8 Y_o = 2.52 ft

	Time, Sec		Δt	Yn		$y_n = (Y_n - L)$		
	Ini.	Final	Diff	Ini.	Final	Ini.	Final	Δy
	0	13	13	0	12.05	0	3.15	-
	13	23	10	12.05	11.94	3.15	3.04	0.11
	23	33	10	11.94	11.83	3.04	2.93	0.11
	33	43	10	11.83	11.72	2.93	2.82	0.11
	43	53	10	11.72	11.60	2.82	2.70	0.12
	53	63	10	11.60	11.49	2.70	2.59	0.11
.8Yo	63	73	10	11.49	11.39	2.59	2.49	0.10
	73	83	10	11.39	11.30	2.49	2.40	0.09
	83	93	10	11.30	11.21	2.40	2.31	0.09

$\overline{y_n} = \frac{3.15 + 2.49}{2} = 2.82$ feet

$\overline{\Delta t} = 10$ seconds

$\overline{\Delta y} = \frac{.66}{6} = .11$ feet

$H/r = \frac{4.20}{0.167} = 25.15$

$\overline{y_n}/r = \frac{2.82}{0.167} = 16.89$

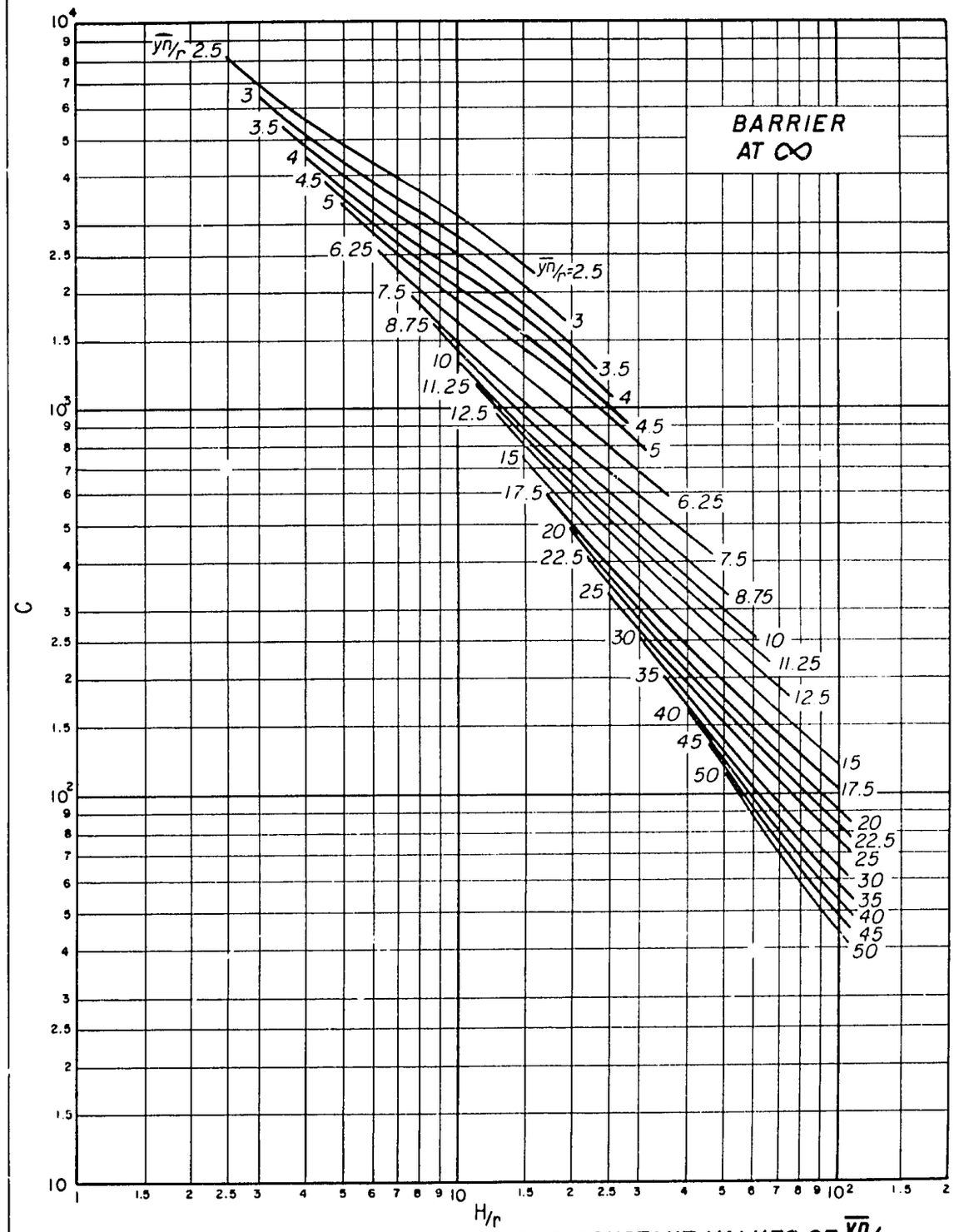
C (from chart) = 390

$k = C \times \frac{\overline{\Delta y}}{\overline{\Delta t}} = 390 \times \frac{.11}{10} = 4.30$ ft/day

$k = \frac{\text{ft/day}}{2} = 2.15$ in/hr

DATA AND COMPUTATION SHEET FOR AUGER HOLE TEST

103-D-650

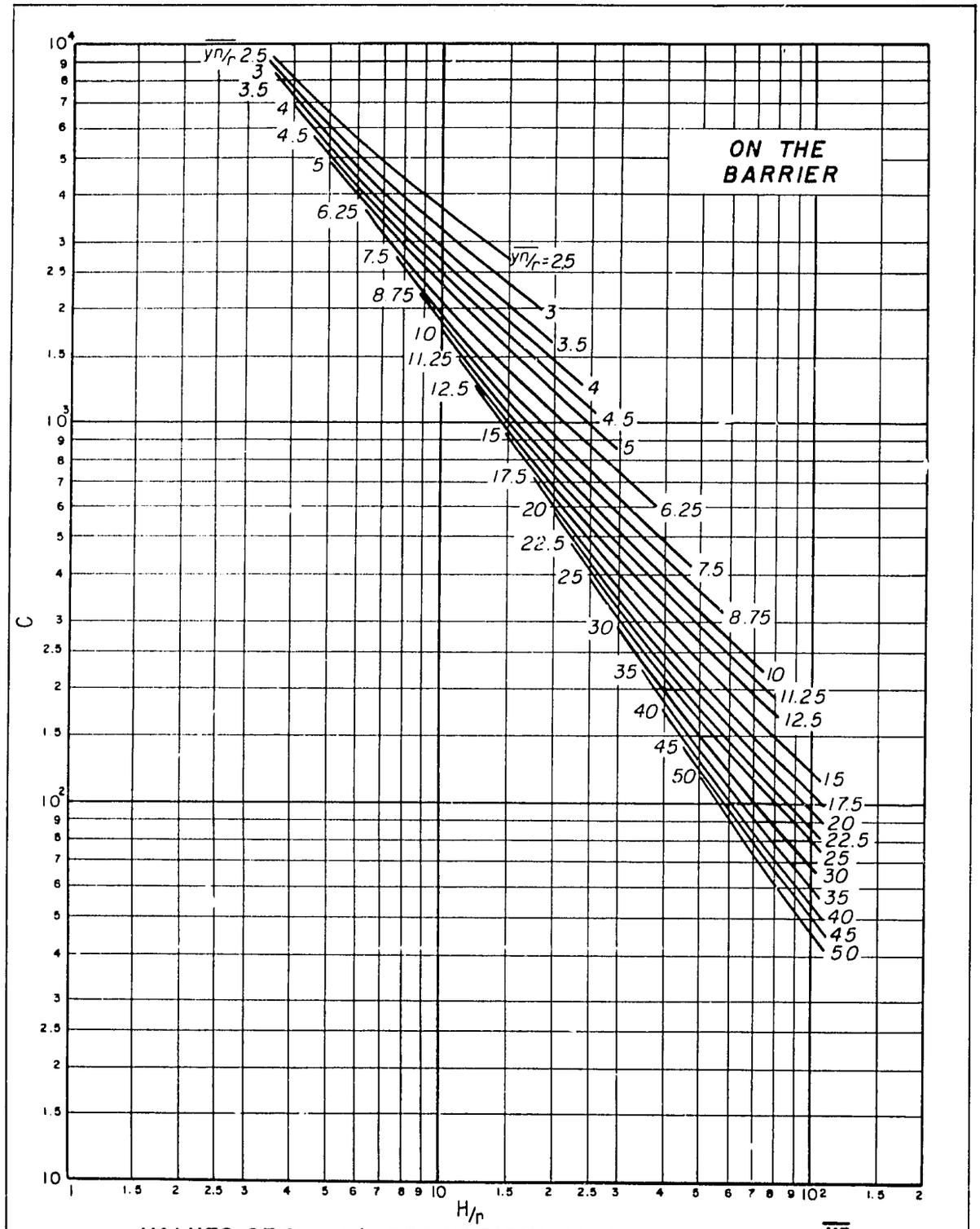


VALUES OF C vs H/r FOR VARIOUS CONSTANT VALUES OF $\overline{yn}/r$

FROM MAASLAND AND HASKEW (1957)

103-D-653

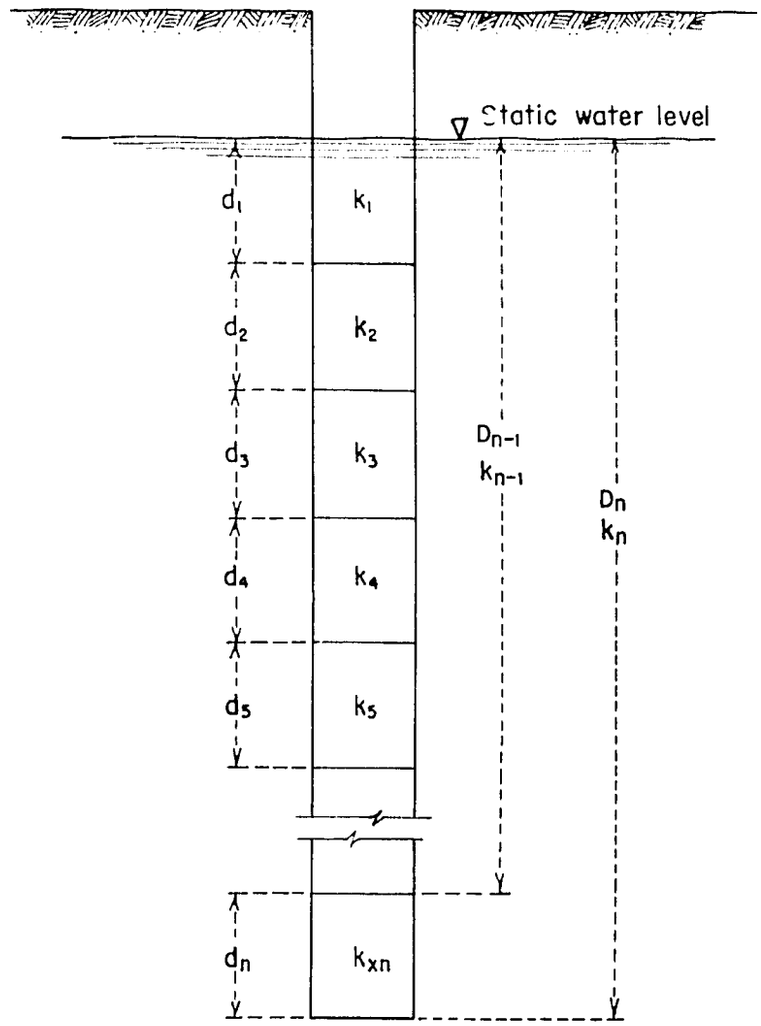
Figure 6
Par. 524.1.5



VALUES OF C vs H/r FOR VARIOUS CONSTANT VALUES OF $\overline{y_n}/r$

F[illegible] .IA SLAND AND HASKEW (1957)

103-D-652

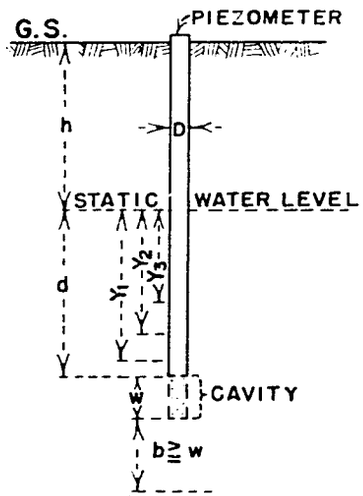


$$k_{xn} d_n = k_n D_n - k_{n-1} D_{n-1}$$

AUGER HOLE STEP TEST

103-D-694

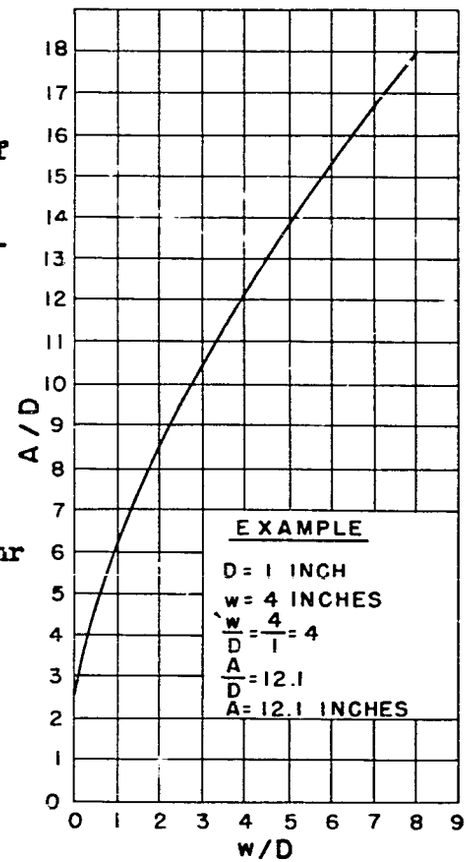
Location: Hole C-2--Sample Farm
Observer: A. P. Brown Date: October 9, 1958



h = 86.40 Ground surface to static water level (inches)
D = 1.0 Inside diameter piezometer and cavity (inches)
d = 93.60 Static water level to bottom of piezometer (inches)
w = 4.0 Length of cavity (inches)
A = 12.1 Constant for a given flow geometry taken from curve (inches)
k = Permeability (inches per hour)
b = Depth to texture change
Y_1, Y_2, Y_3 = Distance from static water level to water level at times t_1, t_2, t_3 (inches)
(t_2-t_1) = Time for water level to change from Y_1 to Y_2 (seconds)

$$k = \frac{3{,}600\,\pi\,(D/2)^2 \log_e (Y_1/Y_2)}{A\,(t_2-t_1)} \text{ in. per hr}$$

Time (sec)		Y (inches)		A	t_2-t_1	Y_1/Y_2	$\log_e Y_1/Y_2$	$3{,}600\,\pi \times (D/2)^2$	k
Ini	Final	Ini	Final						
0	30	86.00	77.90	12.1	30	1.104	0.099	2827.44	0.77
30	60	77.90	70.25	12.1	30	1.109	0.104	2827.44	0.81
60	90	70.25	63.00	12.1	30	1.115	0.107	2827.44	0.85
90	120	63.00	57.27	12.1	30	1.100	0.095	2827.44	0.74
120	150	57.27	51.64	12.1	30	1.109	0.104	2827.44	0.81
						Average for 5 readings =			0.80



A as a function of D and w.
REDRAWN FROM LUTHIN & KIRKHAM (1949)

DATA AND COMPUTATION SHEET FOR PIEZOMETER TEST

103-D-680

Figure 8
Par. 524.1.11

CHAPTER 2 IN-PLACE PERMEABILITY TESTS ABOVE A WATER TABLE

.1 Objective. The two methods that have been adapted for use in investigations are the shallow well pump-in test and the ring-permeameter test. These tests are used to determine the permeability of soils above a water table, and the values are used in predicting the subsurface drainage requirements if and when a water table builds up. In order to reduce as far as possible any extraneous effects on permeability, the water used in the tests must be free of sediment, should be warmer than the soil, and should be of the same chemical composition as the irrigation water that is applied or is going to be applied to the land.

SHALLOW WELL PUMP-IN TEST FOR PERMEABILITY

.2 Introduction. The shallow well pump-in test for permeability is also called the well-permeameter test, and it 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 permeability determined by this test is a composite rate for the full depth of the hole being tested, but reflects, primarily, the permeability of the more permeable layers.

.3 Equipment. Equipment required for the shallow well pump-in test is as follows:

4-inch soil auger

Brush, floor scrub brush cut in two and bolted together around a 1/2-inch pipe coupling

Side scratcher, consisting of a 3-1/2-inch-diamter by 3-inch-long cylinder with small nails protruding about 1/8 inch. A 1/2-inch coupling is placed in the center of the cylinder so the scratcher can use the same pipe as the auger.

Casing, perforated

Casing, with only lower 6 inches perforated

Uniform sand for use in lieu of casing

Burlap to be placed in the bottom of the hole to keep the hole clean when perforated casing is used

Water supply tank truck of at least 350-gallon capacity with gasoline-powered water pump

25 feet of 3/4-inch garden hose for rapid filling of head tank from supply tank

Head tank, 50-gallon minimum, carefully calibrated in cubic feet with zero marking at the top. This tank should have fittings so that two tanks can be connected when required.

Wooden platform to keep head tank off the ground to prevent rusting

A 1-inch-diameter pipe 4 feet long, to be driven into the ground to keep tank in position when nearly empty

Large graduate for final filling of tanks

Constant level float valve which must fit inside the casing, made from cut-down carburetor

Rod threaded to fit threads on top of the carburetor, used to regulate depth the carburetor is lowered into the hole

Sufficient 3/8- to 1/2-inch ID flexible rubber tubing to connect tank to carburetor

Plexiglass cover, 12 by 12 inches by 1/2 inch, 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

Steel fence posts with post driver; four required per site (needed only when site must be fenced)

Wire for fencing site--approximately 85 feet

Thermometer which can be lowered into hole, Centigrade preferred

Watch

10-foot steel tape

Clipboard and computation sheet

16-inch tiling spade

The constant level float valve suggested for use in the Shallow Well Pump-In Test and in the Ring Permeameter Test, described later, is an automobile carburetor. The reasons for suggesting this piece of

equipment are that it is universally available in junkyards and that it has been successfully used in many tests. However, the carburetor has limited capacity and is not adequate for use in material with a permeability over about 2 inches per hour. A valve of greater capacity and of general availability is the type that is used on stock watering tanks, and there are others that will work equally well. The only requirements are that the valve have the required capacity and control the water level within 0.05 foot plus or minus.

.4 Procedure. A two-man team can efficiently install the equipment and conduct the shallow well pump-in test for permeability. The hole for the test should be hand augered, with as little disturbance to the sides as possible. A complete log, including texture, structure, mottling, color, density, and compaction, should be obtained for use as a guide in interpreting results. Upon completion of the hole to the desired depth, it should be carefully brushed to break up any slight compaction caused by the auger and to remove any loose material that might be on the sides. In the medium to finer textured soils with moisture at about field capacity, it is usually necessary to use the side scratcher. This scratcher moved up and down in the hole will break any hard seal on the periphery which the brush could not break. In coarse-textured soils, a thin-walled casing should be installed, with perforations extending from the bottom up to the predetermined controlled water level. For a 4-inch casing, 60 uniformly spaced perforations per foot, 1/8 inch wide

by 1 inch long, have proved satisfactory. This many perforations will somewhat weaken the thin-walled casing, so a commercial well screen is preferred if available.

Another method of preserving the sides of the hole is to fill it with pervious sand to within about 6 inches of the predetermined controlled water level. A casing with the lower 6 inches perforated should then be installed from the sand to the ground surface. This upper casing is essential as a means of preventing fine material on the sides of the dry hole from dropping onto the sand and sealing the surface. The sand should be clean and of uniform size, passing a No. 4 sieve and retained on the No. 8 sieve. However, this method is less satisfactory than using perforated casing as above.

The carburetor float apparatus should be installed and approximately positioned. The carburetor is then connected with tubing to the calibrated supply tank, which is on an anchored platform beside the hole. The 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 supply tank is opened and the height of the carburetor is carefully adjusted so that the water level will be held at the desired depth. The use of the plexiglass cover to keep material out of the hole and to hold the carburetor float adjusting rod facilitates observation of the carburetor during the test. The time and the reading on the tank gage are recorded when everything is operating satisfactorily. The tank

should be checked and refilled when necessary. A record is kept of the time, tank gage readings, and volume of water added, each time the site is visited. When water temperature fluctuations exceed 2° C, viscosity corrections should be applied. Figure 1 shows the equipment setup for this test.

The test should be continued until the material around the hole has become saturated and the flow from the tank is relatively constant. The permeability should be computed after each visit, and when a relatively constant value, with viscosity correction, has been reached for a 24-hour period, it can be assumed that the periphery of the hole is saturated, that no tension exists at the periphery, and that no forces other than gravity affect the flow. When this condition has been reached, the test is completed and the equipment can be removed.

.5 Computations. A sample computation sheet for the shallow well pump-in test is shown on Figure 2. Figures 3 and 4 show alinement charts used in the computations.

.6 Limitations of the Shallow Well Pump-in Test. One of the principal limitations of this test for permeability is that 2 to 6 days and considerable equipment are required to conduct it. Also, a relatively large amount of water is required, especially if the material has a permeability over 2 to 3 inches per hour. Water having about the same salt content and sodium percentage as the irrigation water should be used in this test. Another limitation is that the hole cannot be augered to accurate dimensions in rocky material or coarse gravels.

Also, according to comparison of electric analog test results and comparisons with values from the auger-hole test, the h/r ratio must be equal to or greater than 10.

In general, the pump-in test will give a lower permeability rate for the same soil than the auger-hole test, the magnitude of the variation apparently depending upon the soil texture and structure. Water flowing into the material from the hole moves some fines from the periphery back into the material. These fines sometimes form a seal which reduces the permeability. It is therefore desirable to bail the hole out after a steady state has been reached and wash the fines back into the hole. The test is then run for an additional 6 to 24 hours to check the permeability.

RING-PERMEAMETER TEST

.7 Introduction. In most drainage studies, the lateral permeability of the soil is required for drain spacing determinations. Usually it is assumed that the vertical permeability is sufficient to permit deep percolation from irrigation and rainfall to reach the saturated zone in which it moves horizontally. Sometimes, however, there are slowly permeable layers that interfere with percolation and cause perched water tables. Thus, a means of determining the vertical permeability of such a suspected tight layer is sometimes desirable.

The ring-permeameter test is a somewhat specialized method of obtaining vertical permeability of a critical zone in place. The

test is based, generally, on Darcy's law for movement of liquids through saturated material. The test is slow, but the results are uniformly dependable and can be obtained at reasonable cost. Tensiometers and piezometers are used to confirm attainment of saturated conditions, absence of a perched water table, and fulfillment of the requirements of Darcy's law.

If the soil has structure or root holes, the field test will give permeabilities somewhat higher than the laboratory tests on disturbed samples. If a significant correlation between field and laboratory permeability rates can be obtained in a large area, reliance can be placed on the far cheaper laboratory method, and additional expensive field tests are not required.

.8 Equipment. Equipment required for the ring-permeameter method is as follows:

14-gage steel welded-seam cylinder with reinforcing band on top and sharpened bottom edge (with seam ground down flush), 18 inches ID by 20 inches high

20-inch-diameter field driving disc with 17-3/4-inch-diameter center ring to fit inside the 18-inch cylinder, with a 4-foot length of 1-inch pipe welded in the center for a hammer guide

50- to 75-pound driving hammer (heavy steel cylinder with hole in the center and pipe welded to center which fits over the 1-inch pipe on driving disc)

Water-supply tank truck of at least 350-gallon capacity

Gasoline-powered water pump to fill tank truck

25 feet of 3/4-inch garden hose, used to fill tank from water truck

50-gallon head tank with manometer carefully calibrated in cubic inches with zero marking at top

Wooden platform to support head tank to keep it from rusting

1-inch pipe, 4 feet long, driven into the ground to keep the tank upright

Sufficient 3/8-inch ID rubber tubing to connect tank to carburetor

Constant level valve made from cut-down carburetor

Threaded bolt which fastens to the steel cylinder and supports the adjustable rod which holds the carburetor at the desired elevation

Adjustable rod to hold the carburetor at the desired elevation

Large graduate for final filling of tanks

1/2-inch ID piezometers 18 inches long, rigid copper tubing (two needed for each site)

Small driving hammer to fit over 1/2-inch ID piezometer pipe

7/16-inch wood auger for cleaning out piezometers

Fine sand to fill cavity in piezometers

Bentonite to seal tensiometers and piezometers

Mercury manometer tensiometers (two needed for each site)

Mercury for manometer tensiometers

Distilled water to fill tensiometers initially; distilled water is desirable but unnecessary after initial filling

Small ear syringe to fill tensiometers and remove air after they are filled

1-inch wood auger for installing tensiometers

Thermometer, centigrade preferred

10-foot steel tape

24-inch carpenter's level

White chalk

Clawhammer

Wire-cutting pliers

Clipboard and reference sheets

16-inch tiling spade

Short-handled, square-tipped shovel to clean out hole

Bucket with rope for removing soil from hole

10-foot ladder

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

Cover for the 18-inch cylinder to reduce evaporation and keep out debris

Steel fence posts with post driver; four required per site (needed only when site must be fenced)

Wire for fencing site, approximately 85 feet.

.9 Procedure. A two-man team can efficiently install equipment and conduct the ring-permeameter test. One laborer to help dig the hole will speed up the installation. After the site has been selected and the zone of critical permeability determined, a 4- by 4-foot hole is excavated to

within 3 inches of the zone to be tested. The last 3 inches are excavated when the equipment is ready to be installed, taking care not to walk in the 18-inch area to be tested. This 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 on it. The cylinder is marked with chalk 6 inches from the bottom edge and driven into the soil with the field driving disc and hammer until the chalk mark is at the soil surface. 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 wall is tamped lightly to prevent channeling along the sides. About 1 inch 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 readings in the tensiometers.

Next, the two 18-inch piezometers are marked 9 inches from the sharpened bottom and installed on opposite sides of the cylinder about 3 to 4 inches from it. They are installed by driving 2 or 3 inches with a driver and then augering out the core, continuing this process until the 9-inch mark is at ground level. Care should be taken that the piezometers do not run or come up with the auger during installation. A 4-inch cavity is then augered below the piezometer and filled with clean, fine sand. As an additional means of preventing channeling along the

sides, a 1 to 1 bentonite-soil mixture is tamped around the piezometer. The piezometer is then filled with water and checked to see that it is functioning properly. If the water falls in the piezometer, the installation is satisfactory, and a small can is placed over the piezometer to keep out dirt and water during the rest of the installation. If the water does not fall, the piezometer should be flushed with a pitcher or stirrup pump and reaugered if flushing does not clear it.

The two calibrated and tested tensiometers are then installed on opposite sides of the cylinder and 3 to 4 inches from it on a line at right angles to that of the piezometers. The calibration and testing for these should be done in the laboratory, so that the tensiometers are ready to install when taken to the field. Instructions for calibrating and testing them can be obtained from the manufacturer of the tensiometers. During the testing, 100 on the scale is set at zero tension, so that pressures caused by rising water table can be observed. The holes for the tensiometers are excavated with a 1-inch soil auger to a depth of 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 a 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 1 inch of the soil surface. A 1 to 1 soil-bentonite mixture is then added to prevent channeling. Extreme caution should always be exercised when using bentonite to assure that none of it drops into the piezometers or into the testing ring. Mercury is then placed in the tensiometer cup

and the tubes filled with water. A small ear syringe is used to remove air from the tensiometer tube.

The carburetor float apparatus should be installed and adjusted to hold a constant 6-inch head in the cylinder. The carburetor is connected to the head tank with the rubber tubing. The tank should always be anchored and the gage should face away from the sun. The cylinder is then filled with water to the 6-inch mark, and the valve of the carburetor opened. The hole outside the cylinder should also be filled to a depth of 6 inches. When all adjustments have been made and the tensiometers are full, the time and water content of the tank are recorded. The hole outside the cylinder should be kept approximately full to the 6-inch depth during the entire test period. It is desirable, but not essential, to use an extra tank and carburetor for this purpose, but if this is not available the hole outside of the cylinder can be filled to a 6-inch depth each time the site is visited. Figure 5 shows the equipment setup for this test.

The head tank should be checked two or three times a day, depending upon the percolation and permeability rates, and filled as necessary. Each time the site is visited, a record should be made of the time, the volume of water in the tank, the gage readings of the tensiometers and piezometers, and the temperature. When the tensiometer gages read approximately 100 (zero tension), no water shows in the piezometer, and water is moving through the 6-inch test layer at a constant rate, it can be assumed that the requirements of Darcy's law have been met.

Tensiometers vary in different soils and it is not always possible to get the 100 reading. If they stabilize at readings between 100 and 105, they are probably indicating saturated conditions for that particular soil. Also, it is not necessary for both tensiometers to have the same reading as long as they both read in this range.

If the saturated front should reach a zone less permeable than the layer being tested before the requirements of Darcy's law are met, a mound of water will build up into the test zone. When this happens, the hydraulic gradient will be less than unity, and the base of the soil column being tested will be at greater than atmospheric pressure. This condition will be shown by both piezometers and tensiometers. At the time the piezometers show that a mound has reached the bottom of the cylinder, the test will no longer give a true permeability value. When this happens, the tests 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 permeability, it is sometimes possible to lower the water-table mound by augering a number of holes around the outside periphery of the cylinder approximately 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 must be moved down 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 so it can be examined for root holes, cracks, and possible channeling.

.10 Computations. Permeability computations for the ring-permeameter test are very simple. The formula used is a form of the Darcy flow equation:

$$k = \frac{VL}{tAH}$$

where:

k = permeability in inches per hour,

V = volume of water passed through the soil in cubic inches,

A = cross-sectional area of the test cylinder in square inches,

t = time in hours,

L = length of the soil column in inches, and

H = height of the water level above the base of the ring, in inches.

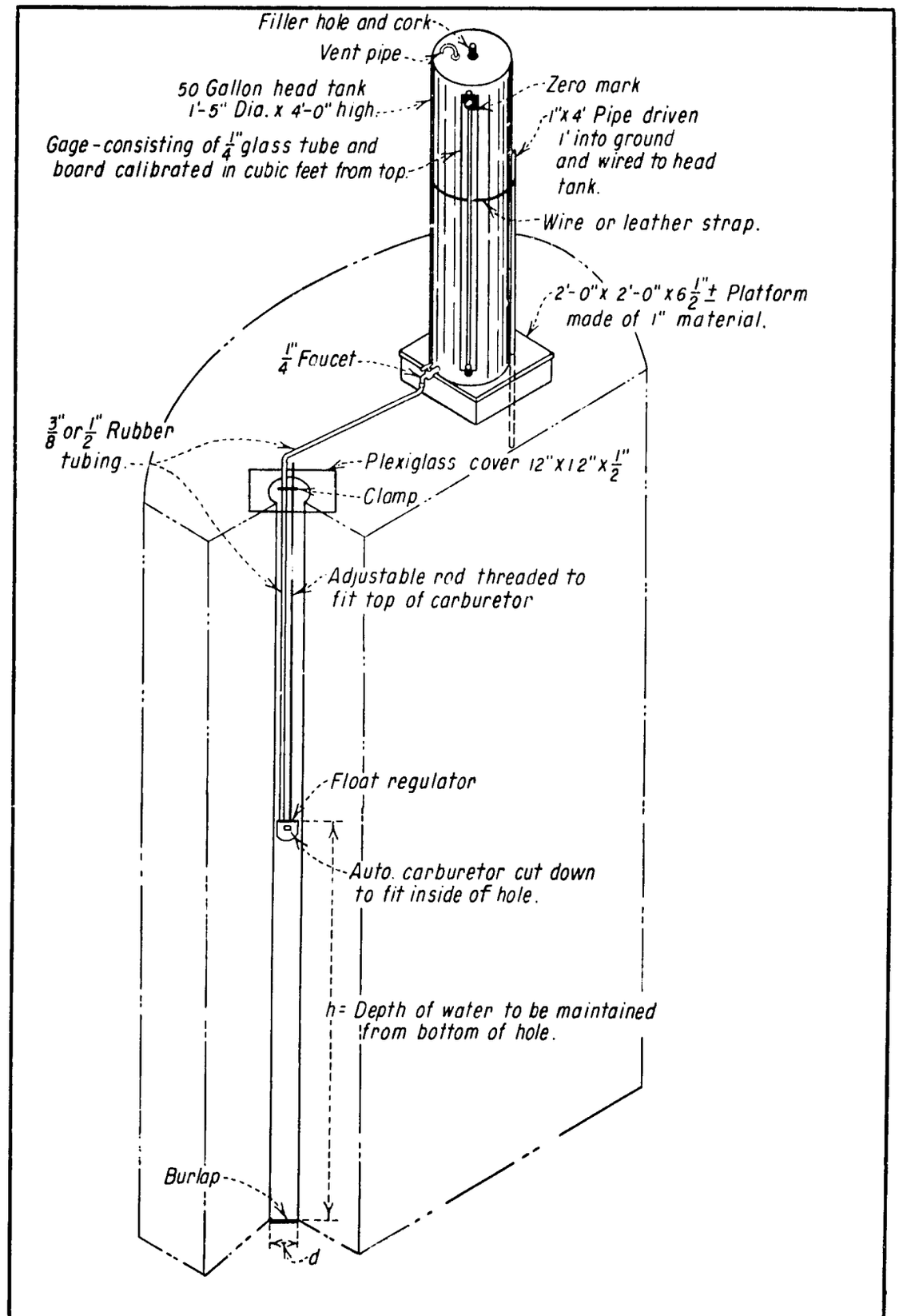
A sample data sheet and computations are shown in Figure 6.

When fluctuations in the water temperature exceed 2° C, viscosity corrections should be made. This usually results in more uniform permeability values, as can be illustrated by using the temperature readings shown in the sample data sheet, Figure 6.

.11 Limitations of the Ring-Permeameter Test. The principal limitation in this test is that the material directly below the test zone must

have equal or greater permeability than the test zone. Also, it must extend to a sufficient depth below the test zone that a steady state flow is reached for at least three consecutive readings before any water mound builds up to the bottom of the cylinder. Another limitation is where there are progressively tighter soils below the test zone. With this condition, a steady state flow is never reached, and the permeability apparently becomes less as the test proceeds. Another condition which results in unreliable data is when the test zone is immediately above a thick, very permeable material. A fairly steady state flow can be obtained, but the tensiometers will never indicate zero tensions below the test zone, and thus the requirements of Darcy's law are not met. As in most in-place methods, the test cannot be used in rocky or coarse gravel materials because the cylinder cannot be driven into such materials, and even if the cylinder could be installed there would probably be channeling along the inside periphery of the ring.

Figure 1
Par. 524.2.3



103-D-655 **EQUIPMENT SETUP FOR SHALLOW WELL PUMP-IN TEST**

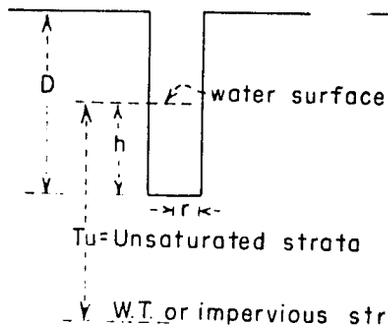
Figure 2
Par. 524.2.5

Location: Hole C-3--Sample Farm

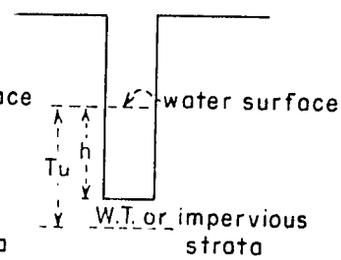
Observer: A. P. Brown Date: October 8, 1958

D = 6.0 total depth of hole (feet)
r = 0.167 radius of hole (feet)
W.T. or impervious strata = 7.0 depth below ground surface (feet)
Tu = 4.5 depth of W.T. or impervious strata from surface of water maintained (feet)
h = 3.5 depth of water maintained from bottom of hole (feet)

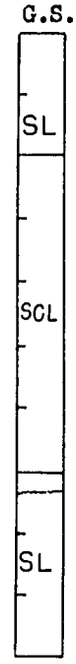
Condition I
Tu ≥ 3h



Condition II
3h>Tu ≥ h



Log of hole



0' - 2' Lt. Br. Sandy Loam, friable, non-sticky.

2' - 7' Lt. Grayish Brown Sandy Clan Loam, friable, slight stickiness, damp at about 7'. Fair permeability. Slight compaction at 6' to 7'.

W.T. 7.2'

7' - 10' Lt. Brown Sandy Loam, friable, good permeability, nonsticky.

Initial Date	Initial Time	Final Date	Final Time	Time, min	Tank reading, cu ft Initial	Tank reading, cu ft Final	Q, cubic ft/min	Temp of water, °C	Viscosity of water, Centipoise	Adj Q, /1 cubic ft/min	Perm, in/hr
10- 8-58	0800	10- 8	1100	180	0	6.12	0.034				
10- 8-58	1100	10- 8	1400	180	0	5.97	0.033				
10- 8-58	1400	10- 8	1800	240	0	6.00	0.025	Note: Connected two barrels			
10- 8-58	1800	10- 9	0530	690	0	12.41	0.018		for greater capacity		
10- 9-58	0530	10- 9	1130	360	0	6.82	0.019	16	1.1111	0.019	0.90
10- 9-58	1130	10- 9	1800	390	0	7.65	0.020	19	1.0299	0.019	0.90
10- 9-58	1800	10-10	0530	690	0	12.10	0.018	13	1.2028	0.020	0.95
10-10-58	0530	10-10	1130	360	0	6.63	0.018	15	1.1404	0.019	0.90

Remarks: No trouble with apparatus, assumed test satisfactory and results reliable.

Calculation: $\frac{h}{r} = \frac{3.5}{0.167} = 20.96$ $\frac{h}{Tu} = \frac{3.5}{4.5} = 0.78$

Q (average after stabilization) = 0.019 cubic foot per minute
3h (or 3 x 3.5) > Tu (4.5) > h (3.5), so use Condition II

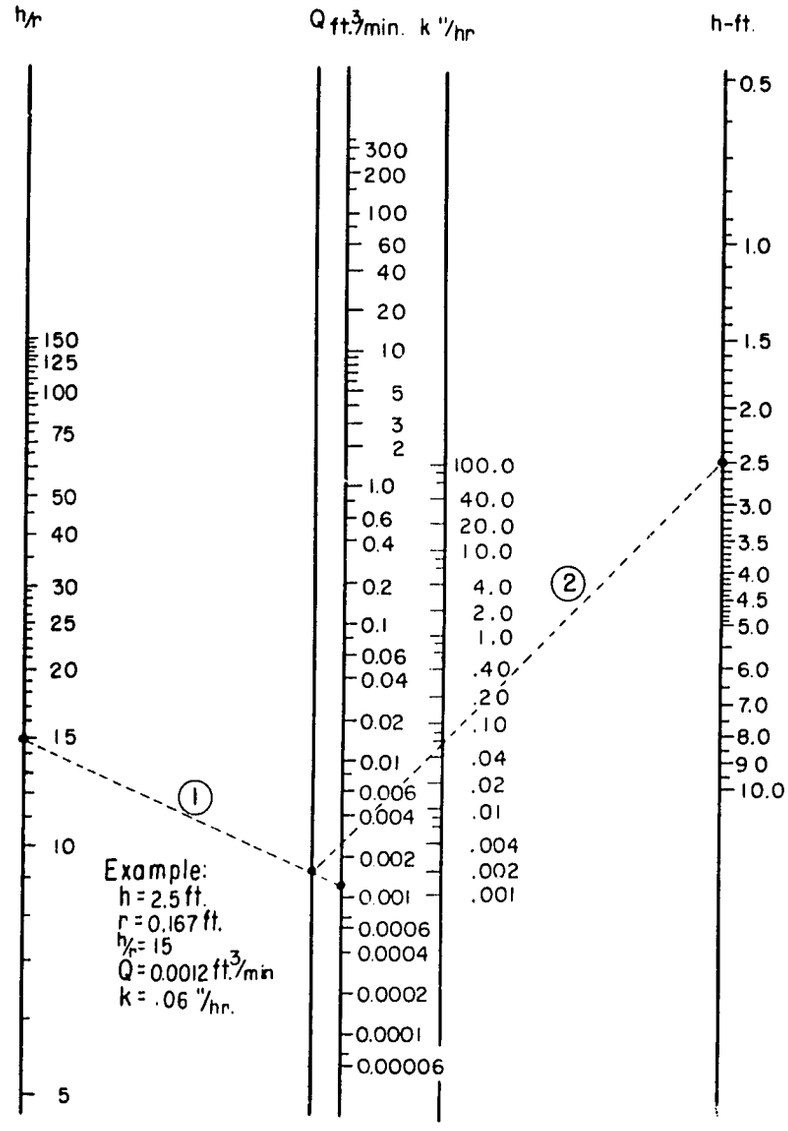
From Nomograph: k = 0.90 inch per hour

/1 See Figure 6 for adjustment procedure.

DATA AND COMPUTATION SHEET FOR SHALLOW WELL PUMP-IN PERMEABILITY TEST

103-D-647

SHALLOW WELL PUMP-IN PERMEABILTY TESTS



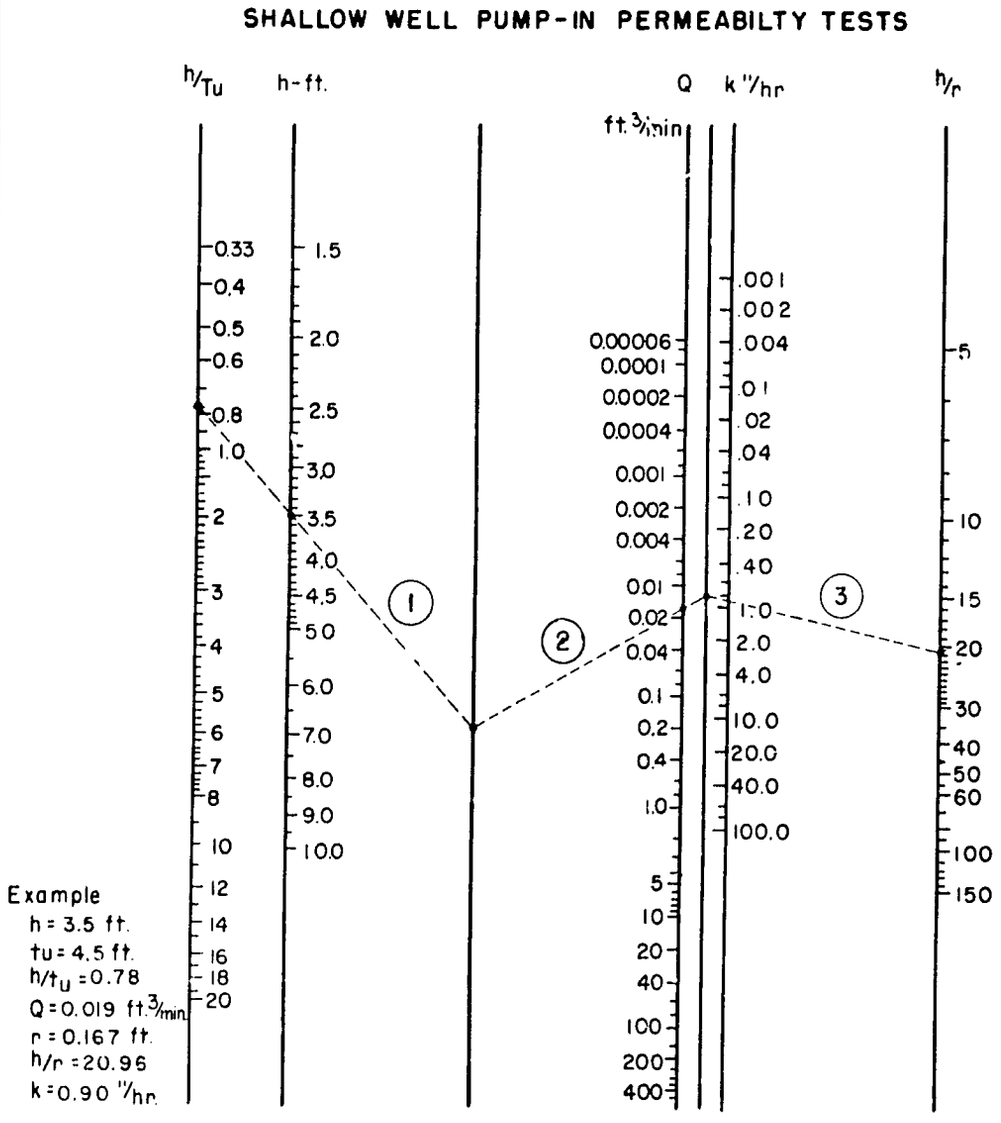
CONDITION I
$T_u \geq 3h$

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

ALINEMENT CHART FOR DETERMINING PERMEABILITY FROM SHALLOW WELL PUMP-IN TEST DATA

103-D-656

Figure 4
Par. 524.2.5

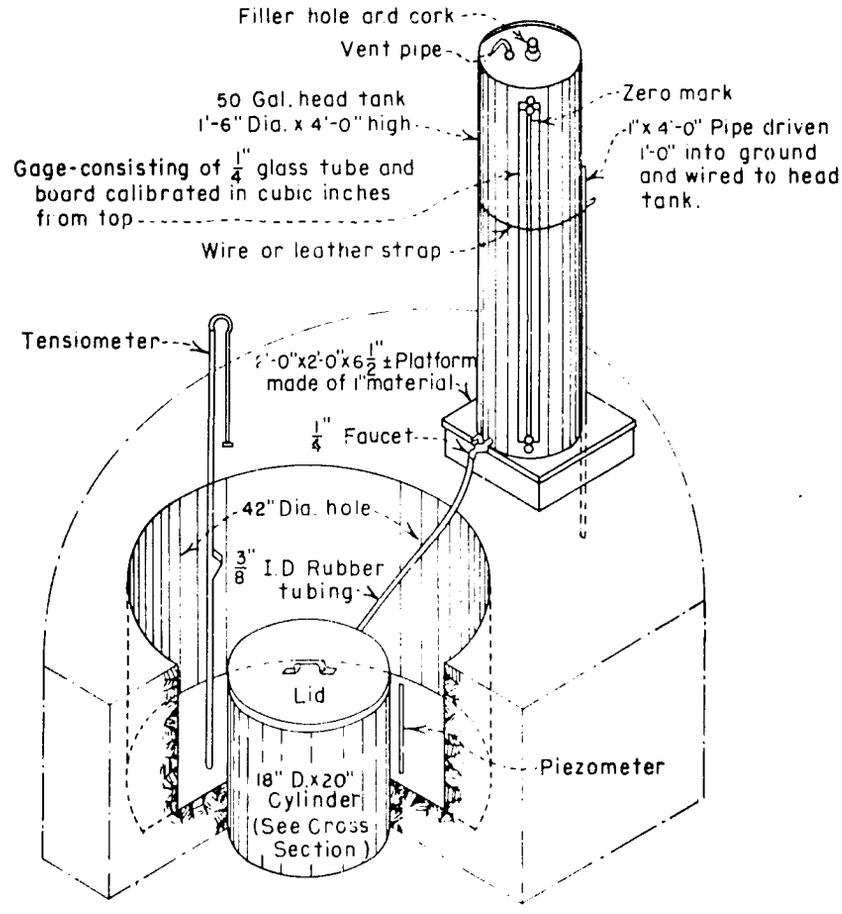


CONDITION II
$3h \geq Tu \geq h$

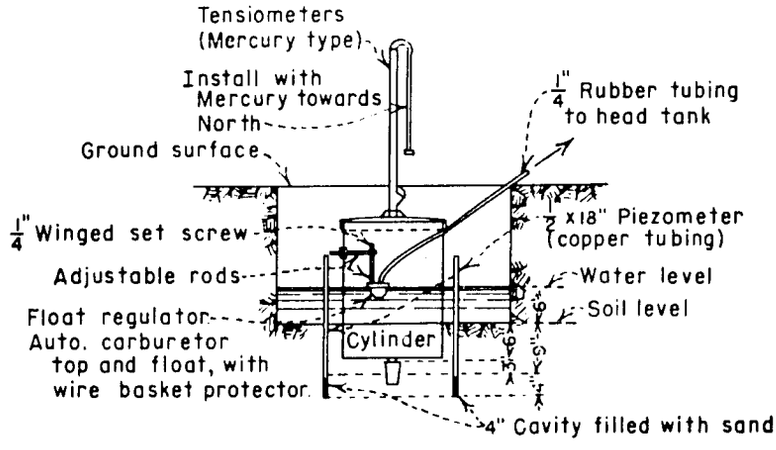
$$k = 720\left[\frac{3 \log_e \frac{h}{r} Q}{\pi h(h+2t_u)}\right]$$

ALINEMENT CHART FOR DETERMINING PERMEABILITY FROM SHALLOW WELL PUMP-IN TEST DATA

103-D-657



ISOMETRIC VIEW
(Equipment set-up for field permeability tests)



CROSS SECTION
(Thru 42" dia. hole and 18" cylinder. Showing tensiometers, piezometers and other equipment in place.)

EQUIPMENT SETUP FOR PERMEAMETER TEST

103-D-658

Initial Date	Initial Time	Final Date	Final Time	Time hrs	Tank reading, cu in Initial	Tank reading, cu in Final	Volume, cu in V	Q, cubic in/hr	Temp/1 of water, °C	Viscosity of water, Centipoises	Adj. Q, cubic in/hr	Perm, in/hr	Tensiometers N-side	Tensiometers S-side	Piezometers E-side	Piezometers W-side
10-13-58	0800	10-13	1212	4.20	0	362	362	86.2	17	1.0828	84.0		145	153	dry	dry
10-13-58	1212	10-13	1630	4.30	362	722	360	83.8	19	1.0299	77.7		138	142	dry	dry
10-13-58	1630	10-14	0725	14.92	722	1742	1020	68.3	13	1.2028	74.0		135	138	dry	dry
10-14-58	0725	10-14	1235	5.17	1742	2110	368	71.2	16	1.1111	71.2		131	133	dry	dry
10-14-58	1235	10-14	1635	4.00	2110	2398	288	72.0	18	1.0559	68.5		122	127	dry	dry
10-14-58	1635	10-15	0750	15.25	2398	3344	946	62.0	14	1.1709	65.4		117	117	dry	dry
10-15-58	0750	10-15	1215	4.42	0	281	281	63.6	16	1.1111	63.6	0.12	111	113	dry	dry
10-15-58	1215	10-15	1710	4.92	281	586	305	62.0	19	1.0299	57.5/2	0.11	108	109	dry	dry
10-15-58	1710	10-16	0735	14.42	586	1383	797	55.3	12	1.2363	61.6	0.12	103	105	dry	dry
10-16-58	0735	10-16	1210	4.58	1383	1661	278	60.7	15	1.1404	62.3	0.12	105	104	dry	dry
10-16-58	1210	10-16	1650	4.67	1661	1962	301	64.4	18	1.0559	61.3	0.12	102	102	dry	dry
10-16-58	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" - 48"

Calculations: $k = \frac{VL}{tAH} = \frac{QL}{AH}$

$Q = 61.2$ 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

DATA SHEET FOR RING PERMEAMETER TEST

103-D-659

.1 Permeability from Undisturbed Soil Samples. (Tests to determine permeabilities may be conducted in the laboratory on undisturbed soil samples.) An undisturbed sample is one taken from the earth with as little disturbance as possible. Different methods are used for taking undisturbed samples, but all attempt to provide for removal of a certain size of earth sample without disturbing the relation of the grains to each other by compression, expansion, or lateral displacement. A properly performed test on such a sample should give a permeability value reasonably consistent with the accuracy of the value from an in-place test. However, there are economic limitations to the use of this type of sample in an overall drainage study. A properly obtained undisturbed sample is usually about 4 to 6 inches long, but for solution of drainage problems it is necessary to know the permeability through at least a 10-foot depth over the study area. This means that, in a heterogeneous profile, many samples must be taken in the field and tested in the laboratory to get the desired information, which is more costly than obtaining an equal amount of data by in-place tests.

The lateral permeability of many soils is greater than the vertical, and may be many times as great. This is a result of the manner in which the soil was deposited in horizontal layers when it was formed. The

formation may also have been such that the grains of soil were arranged in one predominant direction, which also affects the permeability in different directions. Although movement of ground water to a drain is a component of lateral and vertical movement, it is primarily lateral. The permeability value for use in the solution of drainage problems is usually the component value of lateral and vertical permeabilities that applies to the particular problem, but in some instances the vertical permeability alone is of critical importance.

Either horizontal or vertical undisturbed soil samples can be taken, but the cost is relatively high. Horizontal samples taken at depths greater than a few feet are especially costly. Undisturbed samples taken in both directions can be used to obtain usable permeability values, but the in-place tests come closer to providing the proper value as well as averaging the value over a larger volume of material. Methods of taking undisturbed samples and laboratory methods of determining permeability are described in Series 510, Land Classification Techniques and Standards.

.2 Permeability 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 a hole by an auger and broken up in a machine before the test is run. The permeability values obtained by this procedure have a doubtful relation to the true permeability value of the soil in its natural state.

Attempts have been made to establish correlations between permeabilities of disturbed samples and those of undisturbed samples or of in-place tests, but no relation has been found except in specific instances. Sands and gravels have about the same permeability in the disturbed and undisturbed states, except that the compression factor, due to the overburden on materials at a depth of about 10 feet or more, tends to reduce the permeability. This factor must be reproduced in the laboratory in order to duplicate the in-place values at depth, but the variation is small and errors are insignificant if proper laboratory procedures are used. For this reason, permeability values obtained from tests on disturbed samples of sands and gravels can be used with confidence.

Another use for disturbed permeabilities is in the detection of the presence of excessive exchangeable sodium. Excessive amounts of exchangeable sodium will cause a very low permeability value to be obtained from a test on a disturbed sample of all but coarsest material. This is not the only reason for a low permeability, but a low value is sufficiently indicative that it would prompt additional tests to establish the presence or absence of excessive exchangeable sodium. The harmful effects of sodium are explained in Paragraph 523.0.4F.

Obtaining and testing disturbed samples is quickly and cheaply done compared with undisturbed samples and in-place tests. Many times disturbed samples are taken in connection with land classification or other studies in a project area and little additional work or cost

is involved in making permeability tests. Permeability tests on disturbed samples are worthwhile for the specific purposes and cases discussed above and as indicators, but they generally are not adequate for designing and locating drains. The best method for the latter purposes is the use of in-place tests and the next-best method is undisturbed sampling and testing. Methods of taking disturbed samples and laboratory methods of determining permeability on them are found in Series 510, Land Classification Techniques and Standards.

CHAPTER 4 OBSERVATION HOLES

.1 Introduction. Observation holes for drainage studies are needed to furnish information concerning the character of the soil materials and to provide a means whereby the location, fluctuations, and pressures of ground-water bodies may be periodically observed. Observation holes for ground-water information are for two purposes--to measure the static water level and to measure the pressure of the water at a given point in an aquifer. Observation holes for the latter purpose are called piezometers.

.2 Location of Observation Holes. Selection of the location for holes should be done in the field with careful consideration and judgment, and each location should then be placed on a map. The objective of the study must be kept in mind and locations chosen which will meet the objective. In general, observation holes are for the purpose of locating and observing the areal water table. Ponds, lakes, canals and laterals, rivers, and similar water-holding structures lose water by seepage, forming mounds of water which do not reflect the general areal ground-water table conditions. This is also the case under a field for a few days after it has been irrigated, under swales and ephemeral streams, under a road borrow pit or other depression in the land surface after a rain, and, in fact, under any temporary or

permanent water body on the land surface. When possible, holes should be located to eliminate the effect of these water bodies on the general water table. If the hole cannot be located elsewhere, 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 insure 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, they 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 be located parallel and perpendicular to surface slopes, and a line of holes should be located above and below any significant break in surface slope. However, it is often convenient to locate observation holes on a grid system along land subdivision lines because fence lines and roads follow this pattern. This pattern is more satisfactory if the land slopes also follow the land lines, but it can be used in any case.

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 location should follow the same criteria as stated for open observation holes.

.3 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. If the materials are unconsolidated and the hole is not deep, a hand auger may be used. In general, a power auger should be used if a large number of holes are required, if the material is compacted, if sand and gravel are encountered, or if the holes are over about 10 feet deep.

The hole should be put down to its final depth and about 4 inches of sand or gravel poured into it before installing the casing. The annular space around the casing should then be filled with gravel to the top perforation. At this point, a 1 to 1 bentonite-soil mixture should be tamped around the pipe and mounded at the land surface. This will prevent water in the field from flowing directly into the gravel and the casing. Before the gravel pack is installed, the hole should be flushed with clear water which then is pumped from the hole. Ground water that runs into the hole after flushing should also be pumped out before the gravel envelope is installed.

Well drilling equipment can also be used to install observation holes but is an expensive method. Drilling may be necessary if holes are needed of greater depth than can be augered. Drilled holes may be necessary to locate and identify deep artesian aquifers.

The depth of an observation hole should usually be below the lowest expected water level. A careful log of each hole should be

made showing texture, structure, color, moisture, changes in density, etc. Sufficient samples of the materials should be taken for mechanical analyses to insure that accurate texture appraisals are being made. When a sodic environment is suspected, some samples should also be taken for exchangeable sodium analyses.

.4 Casing. Holes that will stand as long as needed do not require casing. Generally, most holes will be in material that will not stand without casing, or will be needed for a period of time that will not permit taking the chance of them standing without casing. Many types of material can be used for casing and the type chosen will depend on the cost and availability of the material and the degree of permanence required. The cheapest material is probably thin metal stovepipe or roof-gutter downspout pipe. Standard pipe or also well casing is ordinarily used, but other available materials are asbestos-cement, bituminous impregnated fiber, and plastic pipe. All casing for observation holes must be perforated. A satisfactory method is to perforate at about 6-inch vertical intervals, with the perforations alternating between opposite sides of the pipe. Perforations should extend from the bottom of the pipe to within 4 feet of the ground surface. Perforations may be made by any convenient means such as hacksaw, chisel, machine drill, punch, acetylene torch, or any other available tool that makes a hole large enough for water to enter but small enough to prevent the outside soil material or gravel pack from entering in any quantity. Generally a slot about 1/8 inch in width will be satisfactory.

The casing should be extended about 12 to 18 inches above ground surface so that it will be visible from a distance. An additional precaution is to paint the protruding portion of the pipe yellow, orange, or some color that contrasts with the natural surroundings during most seasons of the year. This not only makes the hole easy to locate for measuring, but also makes it easy for the farmer to locate the hole and casing in a cultivated field. When the casing is not protected by a fence or similar permanent structure, a 4- by 4-inch wood post or a steel post, painted orange or yellow, should be installed near the casing. The hole number can be painted on a wood post, stamped on a steel post with dies, or stamped on aluminum ribbon. If this post extends above the ground about 4 feet, there is very little chance that the farmer will miss seeing it.

An alternate method that can be used if it is considered inadvisable to leave a rigid pipe and post standing in a field is to attach a rubber hose to the top of the casing. The casing is cut off about 6 inches below the ground surface and a rubber hose about 2 feet long slipped over the top of the casing. Use of hose results in fewer ruined observation holes and less damage to equipment.

.5 Capping the Casing. Casing should be capped if there is a possibility of malicious damage. Many times people will drop rocks or sticks into an open pipe and clog it up until the water level cannot be measured. A perforation should be made in the cap or in the pipe just below the cap. This will prevent pressure or vacuum from building up in a pipe

with an airtight cap during fluctuations in the water table. The cap should be tightened with wrenches so that it will be more than handtight.

.6 Piezometers. The piezometer is a device in which the pressure can be measured at any point in an aquifer. This is an important device since pressure differentials exist in a moving ground-water body. Differential elevations of the water table, as measured in observation holes, give only information on thickness of unconfined water bodies and gradient of their phreatic water surface. 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 source; in determining ground-water-flow patterns; and in determining upward leakage from a confined aquifer. In such studies groups of two or more piezometers are employed to measure the hydrostatic pressure in separate saturated soil strata or at a given depth. Piezometers should never be used to determine water table elevations.

.7 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 3/8-inch inside diameter and up to 4-inch diameter can be used. However, 1- to 2-inch-diameter pipe has been found the easiest to install at depths over 10 feet.

There are many methods of installing piezometers. For shallow depths under 5 feet the alternate augering and driving method as described in Paragraph 524.1.10 provides a good seal. For depths over 5 feet a hole can be augered to within 18 to 24 inches of the proposed bottom, the pipe placed in the hole, and the alternate augering and driving method used for the last 18 to 24 inches. A cavity about 4 inches long and with the diameter the same as the inside pipe diameter is augered below the pipe in both methods to provide an easy access for the water to enter the pipe. 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 2- to 3-foot length of pipe with an inside diameter slightly larger than the outside diameter of the pipe to be driven. A 10- to 20-pound weight is welded over one end to give the driver sufficient weight. A standard wood auger with close clearance inside the piezometer, altered so that it can be used with extensions, provides a suitable inside auger. When using the alternate augering and driving method, the hole is augered about 6 inches below the pipe each time and the pipe is then driven down to the bottom of the hole. The cavity at the bottom should be filled with sand to assure that the piezometer continues to function properly.

When holes are too deep for the alternate augering and driving procedure, which is usually a depth of about 15 feet or greater, or are to be used for installation of multiple piezometers, they are

drilled or augered to full depth. Holes for piezometers over about 15 feet in depth may have to be drilled with a power auger. Before a pipe is installed, about 4 inches of sand or gravel is poured into the hole. The pipe is then installed and another 2 to 3 inches of sand poured around it. The annular space around the pipe is then sealed with grout or a 1 to 1 bentonite-soil slurry to eliminate vertical water movement around the pipe. This seal should be a minimum of 2 feet thick vertically when grout is used and the bentonite-soil slurry should be a minimum of 5 feet thick. 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. Any remaining annular space between the pipe and the hole can be filled with any material available.

In unstable material an outside casing must be used to hold the hole open. After the pipe has been installed, the sealer put in, and the hole filled, the outer casing can be pulled.

Testing to assure that the piezometer is functioning properly should not be done within 24 hours of installation. Then, water is pumped from or poured into the pipe and the time and height that the water level rises or falls is observed. If the rise or fall is rapid, considering the soil at the bottom of 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 augered out again. 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 as explained in Paragraph 524.4.5.

.8 Records of Observation Holes. A permanent record should be made of all pertinent data obtained from the installation of an observation hole. This record should include such things as the location of the hole; its depth; the kind, diameter, and length of casing installed; the perforated length; a log of the hole showing the texture of the material encountered; the elevation of 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); the elevation of the natural ground surface; and the periodic measurements of depth to water. The Bureau of Reclamation has a field book for recording this information, Form 7-268, Drainage Investigations Test Hole Record Book. Figure 1 shows the blank form used in this book, which provides for the log of a hole not more than 15 feet in depth. Form 7-259, Boring Records, can be used for holes up to 80 feet in depth. This latter form does not include all the items of information on Form 7-268, but the additional information can be included under "remarks." Where cooperative programs with the Geological Survey are carried on, it may be preferable to use USGS Forms No. 9-185, 9-194, or 9-195 for recording information on the hole and for recording water level measurements. These forms are available from the Geological Survey.

.9 Numbering System for Observation Holes. It is important to establish a numbering system for observation holes that will make them easy to find in the field or to locate on a map. Two systems have proved satisfactory--the coordinate system and a system developed by the Geological Survey.

In the coordinate system the study area is located on a map and N-S and 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 simpler and there is less likelihood of 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 visualized to be in the first quadrant of a rectangular coordinate system. A well that is 2 miles east and 3 miles north of the intersection of the zero lines, or the point of origin, would be number 2E-3N. Wells do not have to be even miles from the point of origin but can be located by decimals such as 1.2E-2.13N or by proportional parts of a mile such as 2-1/4E-3-1/8N. The system not only locates the wells on maps and in the field, but their location with respect to each other is immediately known when their numbers are known. The system operates best in an area which has had a land survey, but this is not essential. It might be convenient to locate the point of origin at the intersection of two highways that traverse the area. In this case there would be wells in all four quadrants that would have numbers with combinations of E, W, N, and S. This is not necessarily objectionable, but it introduces an opportunity for error when plotting by confusing the cardinal directions. Figure 2 shows an example of the coordinate system.

The U.S. Geological Survey method is based on a land subdivision system which utilizes township, range, section, and four lower case letters. "The first numeral of a well number denotes the township, the second the range, and the third the section. The lower case 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 lower case letters." 1/ Figure 3 shows an example of the USGS numbering system.

.10 Measuring Devices for Depth to Water. There are several devices for measuring the depth to water in an observation hole. Probably the most widely used is the chalked tape. An ordinary steel tape is chalked for the first 2 or 3 feet with carpenter's chalk or ordinary blackboard chalk. When immersed in water, the chalk will change color and the point to which it penetrates the water surface is easily read. The tape is lowered into the hole until it reaches water and then lowered more until an even foot mark is held at the

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 U.S. public land surveys.

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. It may require more than one try to get the end of the tape properly submerged, but it can be quickly done if the approximate depth to water is known.

Another method in wide use employs a steel tape with a "popper" on the end of the tape. A popper can be made from a 1-inch steel bar about 2 inches long; a fastener is welded on one end of the bar so that it can be fastened to the end of the steel tape, and the other end of the bar is hollowed out. The objective is to provide an air space that is open on the bottom and enclosed on all other sides, so there are many materials that can be used and many ways that a popper can be made. In operation, the popper is lowered into the hole and a distinct "pop" can be heard when the popper meets the water surface. After a little experience it is possible to locate the water surface within 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 zero on the tape is added to the reading to obtain the depth to the water surface from the measuring point.

A graduated "rule" or "dipstick" made of about 1/2-inch-thick by 1-inch-wide hardwood is useful for measuring water levels closer than about 8 feet to the surface. The stick 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. As with all nonelectric measuring devices except the popper, caution should be exercised, particularly when measuring in small diameter pipes, to avoid displacement of a sufficient volume of water with the device to give significantly erroneous measurements. This method is particularly efficient in investigations where there are numerous observation wells comparatively close together.

Several commercial measuring devices are available for measuring the depth to water in a well or observation hole. Most are based on completing an electrical circuit through the water in the well. Some employ 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. Most employ flashlight batteries for power, and contact with water is evidenced 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. If a situation is found where a chalked tape, popper, or rod cannot be used, a commercial device may serve the purpose. (See Figure 4.)

.11 Plugged Observation Holes. After a series of measurements, it may be noted that the water level no longer fluctuates in certain holes, or that the fluctuation departs from its former pattern or the position of the water table or magnitude of fluctuation in nearby holes. Such holes may have become plugged by an accumulation of silt. Whether plugging has occurred can be ascertained 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 that the hole is plugged. 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 flow upward out of the hole, between the casing and the plastic hose, and flush out the material that forms the plug, or permit bailing it. Other methods may be equally successful.

GROUND WATER READINGS
(Hole on opposite page)

_____District

Line No._____

Hole No._____

Subtract Tape Reading from_____ 16—45074-1

Date	Tape reading	Elevation water surface	Condition surrounding area	Reader	Remarks

Form 7-268

LOG OF HOLE

Party ..... Date .....

..... Level Book Ref. .....

Location. Section ..... Township ..... Range .....

.....

.....

Kind { Cased Boring ..... / Uncased Boring ..... / Open Test Pit ..... / Domestic Well .....

PROPERTY OWNER

.....

.....

Size of Hole ..... Casing .....

Elev. Top of Pipe .....

Elev. Platform .....

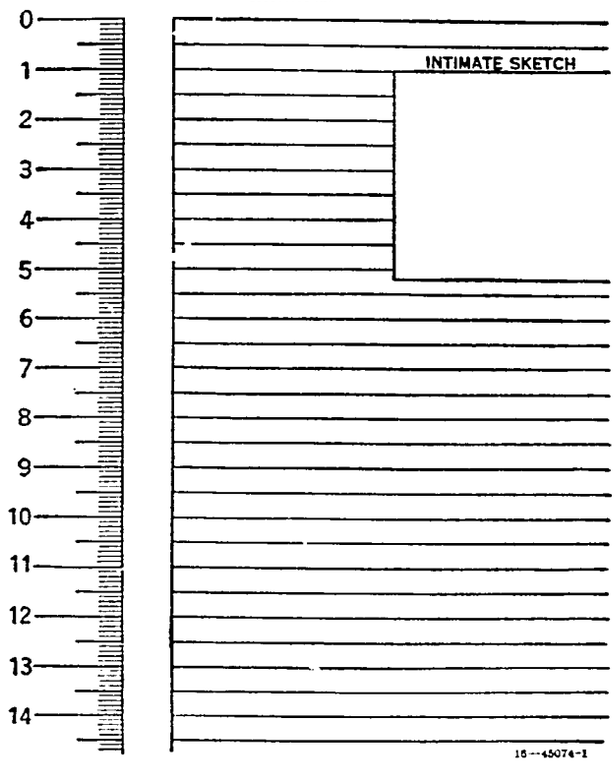
Elev. Datum Hub .....

Elev. Ground Surface .....

Depth, Ground to Water—

when first encountered .....

..... min.
..... hr. afterward .....



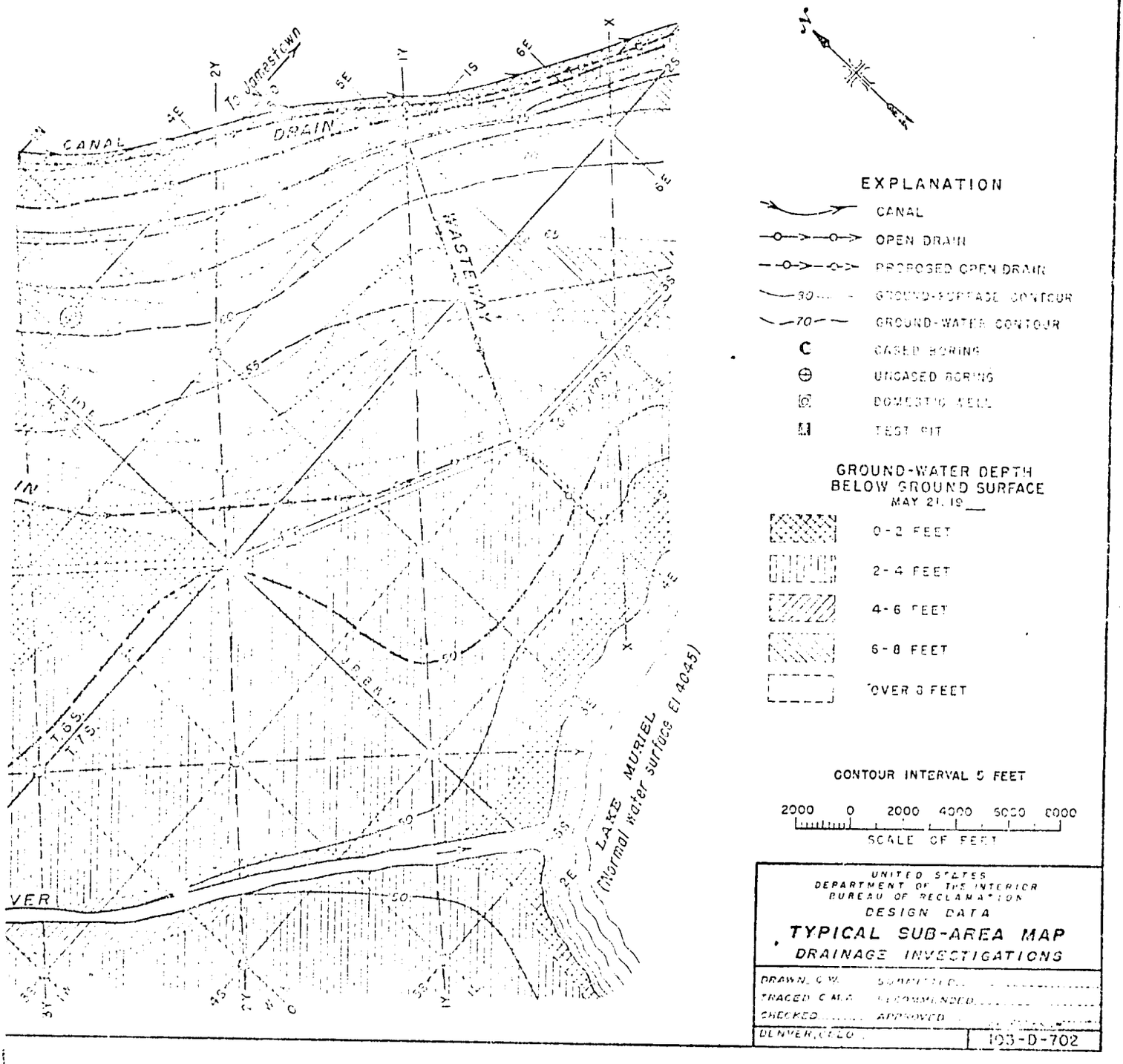
16—45074-1

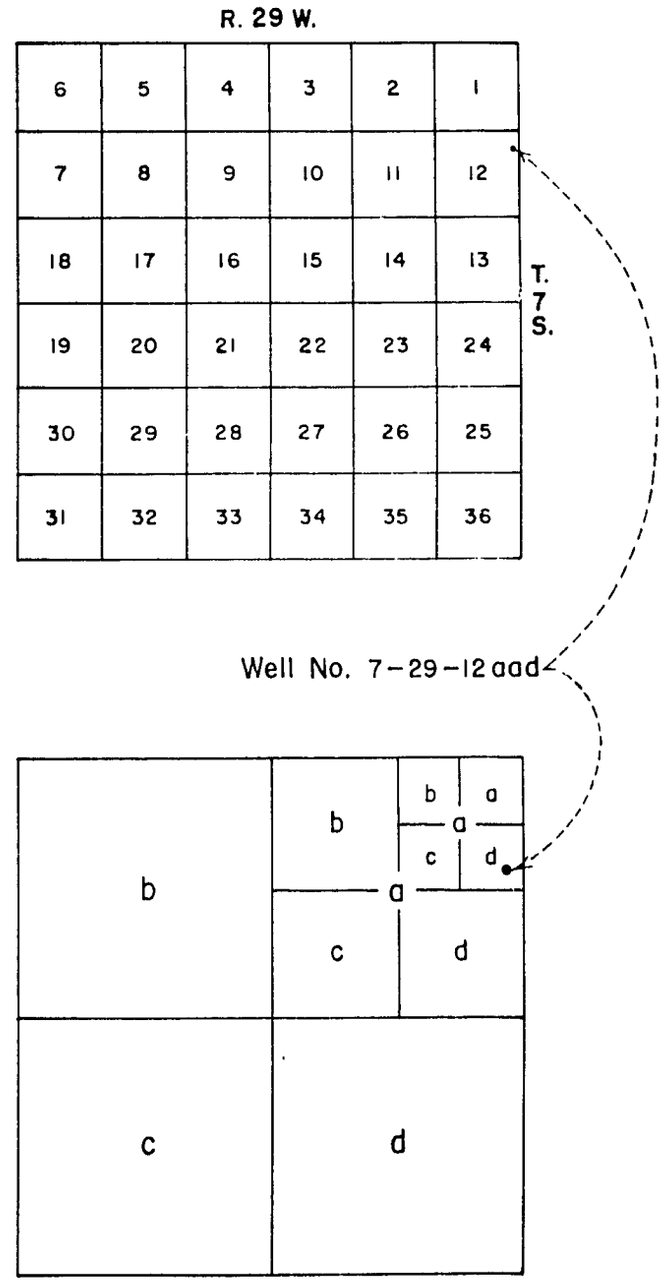
DRAINAGE INVESTIGATIONS TEST HOLE RECORD BOOK

Figure 1
Par. 524.4.8

MAIN
PROPOSED
PROPOSED
NADIR
To Gallsburg
T.5S.
T.6S.
R.9E.
R.8E.
TAYLOR
Temporary
FORMERLY X-0-3961

Figure 2
Par. 534.4.9



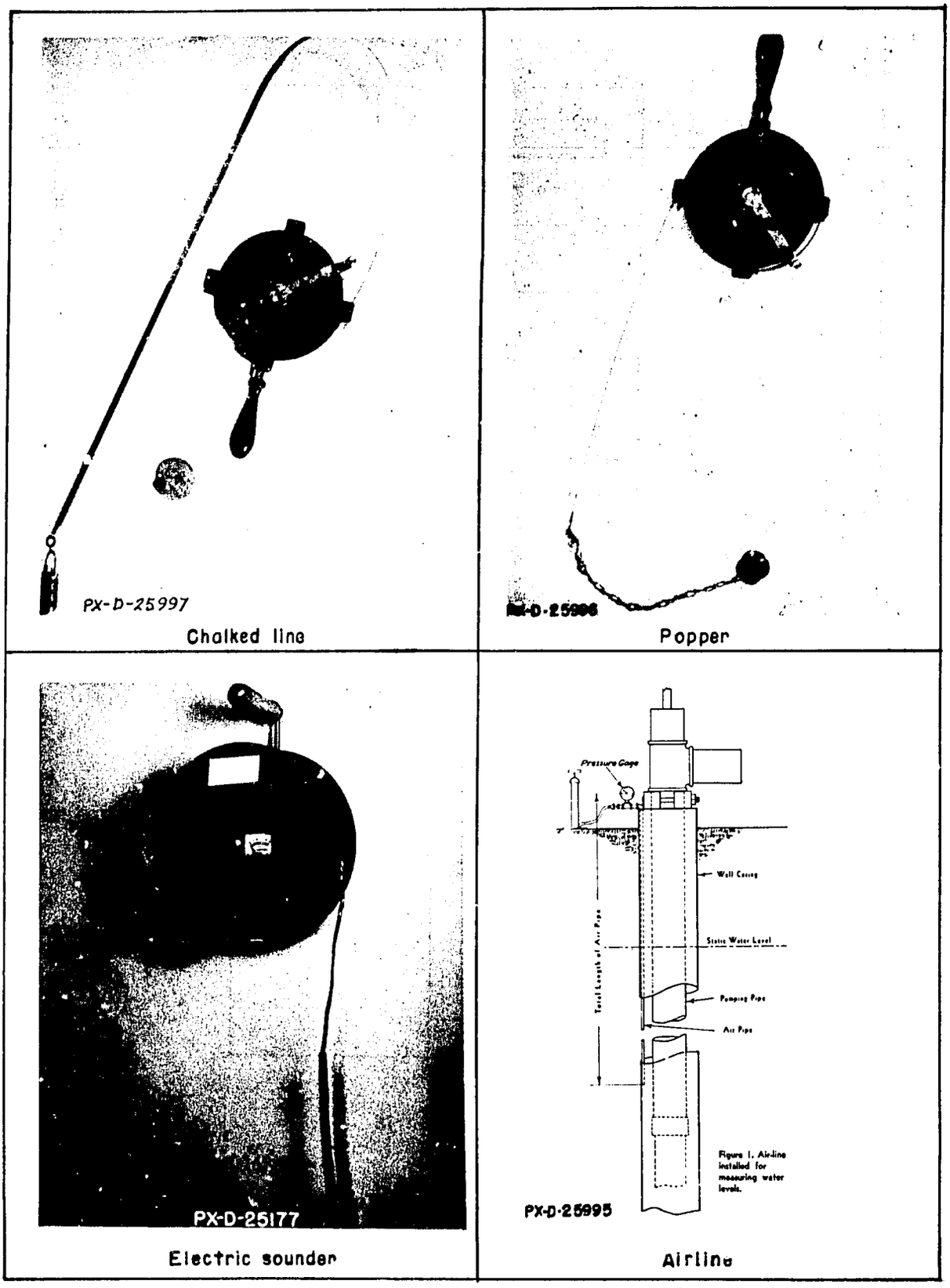


Section 12

U.S. GEOLOGICAL SURVEY WELL NUMBERING SYSTEM

103-D-696

Figure 4
Par. 524.4.10



DEVICES FOR MEASURING DEPTH TO WATER IN A WELL

Reclamation Instruction Series 520

LAND DRAINAGE TECHNIQUES AND STANDARDS

Part 525 Drainage Investigations

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525.0.10 Ground-water Accretions to Drains

525.0.11 Design Discharge for Determining Pipe Size

525.0.12 Outlet Conditions

525.0.13 Drain Location

525.0.14 Drain Numbering

525.0.15 Existing Structures

525.0.16 Economic Considerations of Drainage Problems

Figure 1 Typical Profiles of Test Well Lines

2 Typical Ground-water Table Conditions

3 Typical Depth-to-water Map

4 Typical Hydrographs of Water-table Elevations

5 Conditions in Section 32 T1N R7W--Courtland Canal (Nebraska)

.1 Scope of the Investigation. There are many types of drainage problems, and the investigations must be varied as required to solve each particular problem. There is a place of common beginning, however, and the first step is a clear understanding of the purpose of the investigation, the type and level of report to be prepared or solution required, the minimum data required for the specific type of report or plan, and the best way of obtaining such data. After becoming acquainted with the area and the data available about it, the scope of the investigation can be established. This scope will represent a balance between the available data, the amount and types of additional data required as dictated by the accuracy and completeness expected of the final report or plan, and the time and manpower available for the investigation. Thus, the scope of investigation, and the resultant plan and/or report, will be less detailed for a reconnaissance investigation than for an investigation leading to construction. When making an investigation of the former type, the requirements of the latter type should be kept in mind and all work done should fit into a pattern that can be expanded into a more complete study.

Each project or piece of construction is also an economic problem in which it must be determined that drain construction is justified. It is not the responsibility of the drainage engineer to determine

whether a soil is worth draining, although he may collaborate in the decision. His job is to devise an effective drainage system at the least cost. In this technical guide only the engineering investigation will be covered. The Bureau of Reclamation method of ecnomic analysis can be found in Series 110, Project Planning.

Some drainage problems are simple, and their solution is quite apparent; and for others a limited investigation will afford a solution. In general, however, a drainage study involves a complex relationship of soils, water, crops, and irrigation practices which must be thoroughly evaluated.

.2 Factors in an Investigation. The main factors to be considered in any drainage investigation are topography, soils, ground water, and sources of water. In any drainage investigation the answers to the following questions must be determined: (1) Is there or will there be excess water? (2) is an adequate outlet available for excess water? (3) what is the source of the excess water? (4) can the soil be adequately drained? (5) how much water must be removed? and (6) what type of drainage system will give the best results?

.3 Review of Existing Data. One of the first steps in any drainage investigation is to collect, review, and analyze all pertinent existing data. Data on geology, soils, topography, well logs, water levels and fluctuations, precipitation and surface flow, and similar items are pertinent. Analysis of these data will determine their adequacy and establish the amount and kind of additional data required.

.4 Field Reconnaissance. The field reconnaissance is one of the most important steps in the investigation. The information and impressions obtained will be valuable in programing additional investigations. If possible, in making a field reconnaissance the investigator should be accompanied by someone familiar with the area, and the investigation should be sufficiently thorough to acquaint the investigator with the following items:

(1) Location and capacity of natural waterways.

(2) Location and condition of outlets.

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

(4) Local irrigation practices, such as method of water application, efficiency of irrigation, leveling, grades, etc.

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

(6) Present cropping practices and crop conditions and a notation of any trend toward future change in those practices.

(7) Type, location, spacing, depth, and effectiveness of any drains in the area or adjacent areas. This is one of the most important items in any investigation, since existing drains in similar areas constitute the soundest foundation from which to determine additional drainage requirements in the area or the drainage requirement of a similar area.

(8) High water marks or other information which may be used in evaluating flood flows.

(9) Obvious topographic features which might affect the location of drains.

(10) Indications of salinity or alkalinity.

(11) Discussions with local people, particularly in a cultivated or irrigated area, may be valuable. Such information as types of crops grown, crop yields, irrigation practices and extent and effects of local floods can be obtained.

.5 Subsurface Investigation. Information on subsurface conditions is basic in a drainage investigation. Required information includes: (1) soil characteristics of permeability, texture, and structure; and (2) the thickness, position, and continuity of the various strata. When few data are available, it will be necessary to locate and install observation holes as described in Chapter 524.4. The number and spacing of holes will depend on the scope of the investigation, size and shape of the area, etc., so the spacing may range from tens of feet in a small problem area, to hundreds of feet in a construction investigation, to miles in a reconnaissance investigation.

Mathematical analysis of subsurface conditions to establish natural flow, drainage requirements, or drain location requires a value for depth to barrier or knowledge that barrier depth is great enough that it has an insignificant effect on the analysis. In areas with existing domestic or other wells, the information may be obtained from a review of the logs, but if it is not otherwise available, enough holes must be installed to make the information available. As stated in Paragraph 526.4.3F, for

very deep barriers the depth for use in the spacing method should be limited by a maximum value of one-fourth the drain spacing, so this provides one limiting value for depth of holes. In some cases, it has been found that the effect of depth to barrier beyond about 30 to 40 feet has little effect on drain spacing, so some holes should be carried to this depth. Since drains are frequently installed to depths of about 10 feet, all holes for a subsurface drainage study should be carried to at least this depth. As a starting point for an investigation of subsurface conditions in an area for which few data are available, the plans can reasonably be based on the following depth distribution:

1 hole in 10 carried to the barrier

3 holes in 10 carried to 30 to 40 feet

6 holes in 10 carried to 10 feet

The adequacy of this distribution should be reappraised as enough initial information becomes available to permit preliminary calculations of the effect of depth to barrier on drain spacing.

In order to analyze the effect of subsurface characteristics on drain location, depth, and spacing, a series of profiles should be plotted which show the location, extent, and slope of the various strata. These features can then be analyzed in relation to slope of the ground surface and to the existing or projected ground-water conditions. A sample set of profiles is shown in Figure 1. Where important soil strata, either fine-textured, slowly permeable material or coarse-textured, highly permeable material, are continuous over a large area, it usually is helpful to plot a contour map of the surface of the

stratum. Such a contour map is extremely helpful in planning a drainage system for an area underlain at depths of 4 to 10 feet by these materials. If this map and the ground-water map are drawn on transparent paper, they can be overlain on the base map showing ground surface elevations, canal and drain locations, and other pertinent data. Study of this overlay is sometimes very helpful in locating new drains. In case depth to barrier and depth to ground water are plotted on transparent paper and used as overlays, the same coloring system should be used for each to simplify interpretation.

.6 Identifying the Barrier Zone. By definition, as used in the Bureau of Reclamation, a barrier zone is a stratum which has a permeability one-fifth or less of the weighted permeability of the strata above it. Although this is an arbitrary standard, it has worked out satisfactorily in practice and can be used until more is known about the movement of ground water in heterogeneous soils.

The weighted permeability of soil strata is obtained by the following formula:

$$k_w = \frac{t_1 k_1 + t_2 k_2 + \cdot \cdot \cdot \cdot \cdot t_n k_n}{t_1 + t_2 + \cdot \cdot \cdot t_n}$$

where:

t = thickness of the individual stratum, and

k = in-place permeability of each stratum.

.7 Unconformable Contact Zone. During the formation of the earth's mantle, there were many places where wind and water erosion left the earth's surface in an undulating pattern. This is due, among other things, to

the harder, more dense areas being more resistant to erosion than the less dense areas. The more dense areas are also more resistant to internal water movement and have comparatively low permeabilities. Later in geologic time, some of these areas were covered with alluvium and may, at the present time, have a fairly regular surface topography and be adaptable to irrigation. Under irrigation, any excess water will percolate downward through the surface material but will be held behind the dikes formed by the undulating lower formation. This phenomenon is many times responsible for wet spots that cannot be explained otherwise.

It is quite possible that the normal observation hole system will not reveal the presence of this unfavorable subsurface condition. In areas underlain by shale and in areas where deep cuts may reveal an undulating stratum of impermeable material, a more detailed investigation should be made. In these areas it is necessary to locate and map these barriers through more closely spaced holes and to provide a drainage system which will cut through the dikes and drain the perched water bodies behind them.

.8 Water-source Studies. 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, or a combination of these sources. The source of the damaging water must be known in order that the proper protective measures can be taken. If the source of

the water causing a drainage problem is precipitation, the solution may involve more adequate surface drains; if overirrigation, the solution may involve education on water use in addition to drains (but it must be remembered that practically all arid soils require some irrigation in excess of consumptive use, for salt control); if seepage, the solution may involve canal lining; if hydrostatic pressure, the solution may involve relief wells. All of these solutions will generally be in combination with relief or interceptor drains.

A. Precipitation. The precipitation record obtained in the study of rainfall-runoff relations should be analyzed both from the standpoint of its effect on surface runoff and its effect on the ground-water table. The distribution of precipitation should be related to the fluctuations in water-table elevations, and long-term records of precipitation should be related to long-term hydrographs of water levels where possible. A coincidence of the fluctuations of these factors would indicate that precipitation is dominant as a water source.

B. Irrigation. Owing to the common practice of applying excessive amounts of irrigation water, drainage problems can frequently be traced to the irrigation practices of an area. In determining whether excess irrigation water is the source of the drainage problem, the following points should be investigated: (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, and (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 be related to soil types and crop needs, and ideally only enough water should be applied to furnish crop needs and to maintain a salt balance.

C. Seepage. Seepage is a major source of ground water in many drainage problem areas. Most seepage stems from irrigation development works, such as canals, laterals, reservoirs, or the irrigation of higher lying lands, although in some cases seepage may stem from rainfall or snowmelt on 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 below possible sources is an indication of a high water table and possible subsurface seepage. Other methods of detecting seepage involve the use of dyes, salts, observation holes, and piezometers.

D. Hydrostatic Pressure. In some areas, it may be found that hydrostatic pressure in underlying aquifers is a source of damaging water. Hydrostatic or artesian pressures are found where a slowly permeable layer overlies a saturated permeable layer whose intake source is at a higher elevation. Water may be forced upward by the hydrostatic pressure through the slowly permeable layer or through fractures or displacements in this layer. Damaging amounts of artesian water may be present in areas where old artesian wells may be leaking below

the ground surface or are allowed to run freely without proper facilities to dispose of the surface flow.

.9 Ground-water Studies. Studies of the water table produce much interesting and necessary information regarding a drainage problem. In areas where a high water table has already developed, such information is essential for an understanding of the problem; and in areas where one is expected, it is necessary for making preventive preparations. The water-table investigation provides data on the position, extent, and fluctuations of the water table, the direction of movement of the ground water, and an indication of water sources and areas of discharge. The investigation is made through installation of observation holes and piezometers (see Chapter 524.4) and analysis of the readings of periodic measurements.

Depth-to-water measurements in observation holes and piezometers are made at a frequency dictated by the particular problem under study. This frequency may vary from daily readings to quarterly readings, but, in general, the readings should be made at least monthly. The objective of the readings is to establish a record of the water-table fluctuations over a course of time that will reflect all factors that affect the water table. At least one full annual cycle is necessary to reflect all the factors, so this is the minimum that should be available before undertaking the location and design of a drainage system.

Data on water-table observations are meaningless and useless without an analysis to interpret their significance. The mere gathering of data is a needless expense unless it is followed by plotting of the data in a

form for study and an interpretation of the results. Interpretation begins with the person gathering the data who must remain alert to abrupt changes in previous conditions and must attempt to account for them. A few notes in the fieldbook can spare confusion during analysis.

In many cases the use of automatic recorders at selected locations provides records for use in conjunction with other measurements. In many cases the use of recorders will permit longer time intervals between measurements.

Drawings which are useful in analyzing ground-water problems are water-table maps, depth-to-water maps, water-table profiles, piezometric profiles, and hydrographs. Methods of preparing and using these drawings are as follows:

A. Water-table Maps. Locate on an areal map all points at which ground-water elevations were taken. Mark the elevations at these points and prepare a contour map of the water table (see Figure 2.) Note that the measurements of water-table elevation must be made in the shortest possible time span because these elevations fluctuate and readings taken one day at one location cannot be related to a reading taken several weeks later at another location. The inclusive dates during which the elevations were read must be noted on the map.

These maps show the direction of water movement by the shape and position of the contour lines; an indication of the areas of recharge and discharge; and give some indication of the relative permeability by the distance between contour lines.

B. Depth-to-water Maps. One method of preparing these maps is by overlaying a water-table map over a topographic map. Mark each intersection of contours and enter the difference in their elevations at the point. Using these values, prepare a contour map which will show the depth to water below the ground surface at any point. Another method of preparing a depth-to-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. (See Figure 3, and Figure 2 of Chapter 524.4.) These maps indicate the problem areas, which can be viewed more clearly if the area between contours is colored in accordance with the legend in Figure 1 of Chapter 523.0.

C. Depth-to-barrier Maps. A depth-to-barrier map can be prepared in a manner similar to preparation of a depth-to-water map if sufficient data are available on the location of the barrier. This map is useful in making drain locations and for information needed in making calculations for drainage requirements.

D. Water-table Profiles. A profile is prepared along the line of a series of observation holes. The base profile is prepared by plotting the location and depth of the observation holes, the ground surface elevation, 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. If different colored pencils are used for readings taken at different times of the year, a visual comparison can be made of fluctuations in the water table along the profile.

A profile is more valuable 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 information can be plotted at its proper location. If soil textures are available, it may be possible to make tentative connections between holes to obtain an idea regarding textural variations. The elevation of the barrier in each hole should also be plotted on the profile as this information will be of assistance in drain location and calculations for drainage requirements.

E. Piezometric Profiles. Readings from several clusters of piezometers can be plotted on a profile drawn through the clusters. The water elevation or pressure reading in each piezometer is plotted at the elevation of the lower end of the pipe from which the reading was taken. Lines drawn through points of equal pressure form a contour pressure map. Lines drawn from high pressure points through lower pressure points and perpendicular to the lines of equal pressure 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.

F. Hydrographs. Drawings may be made showing the elevation of the water table with respect to time for any single observation hole, well, or piezometer. Such a drawing clearly shows any fluctuations in the water table as well as trends in water-table movement. (See Figure 4.) 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 5 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 graph.

.10 Ground-water Accretions to Drains. In the 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, and this body moves slowly from a higher to a lower elevation. Over a period of centuries, the underground basin fills with water until it spills into a natural outlet such as a spring or a stream. A continuation of the cycle causes a rise in the water table during periods of high precipitation and deep percolation, followed by an increase in flow at the natural outlet. A period of low precipitation causes a reverse condition. 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 still higher, and the discharge at the natural outlet increases. If water is added

at a faster rate than it can travel to the outlet and be discharged, new outlets may be reached that will increase the discharge, or the water table may continue to rise in search of outlets. When, through this process, the water table nears the land surface, the agricultural production that was initiated by the irrigation is adversely affected and artificial outlets, in the form of drains, must be installed. The drains perform the function of keeping the water table from encroaching the root zone to the extent that crop production is reduced. A depth to water table of 3 to 5 feet is generally satisfactory with the exact design value depending on local conditions including type of crops grown. After drains are installed and irrigation continues, the natural ground-water outlets continue to function and to discharge water at a greater rate than during their original stability. Thus, the drains do not have to remove all of the deep percolation, since a portion of it passes through the natural outlets.

Data obtained by observation of an operating drainage system are the best for use in determining the design capacity for a new system, provided soils, cropping pattern, climate, water management, and other conditions are relatively similar. Every effort should be made to obtain such information before a value for design capacity is decided upon. Discharge measurements made on drains at various locations provide valuable information on the general limits of design capacities for drains which do not carry surface runoff or farm waste. Nearby drains on projects with similar conditions are probably best to use for comparison,

but it may also be possible to use data from other projects by analyzing the differences in conditions. Incidentally, data on subsurface drain discharge are relatively sparse. Drainage engineers should make every effort to obtain additional data, together with information on area served by the drain, irrigation pattern, probable deep percolation, and soil characteristics, and to publish or otherwise disseminate the information. Actual discharge measurements in different locations indicate that an average design capacity of about 1 cubic foot per second (cfs) per mile of drain or 1 cfs per 300 acres normally will be sufficient. The discharge measurements vary, however, from about 0.5 to 3.0 cfs per mile of drain and from 150 to 500 acres per cfs.

Where experience data are not available, the following formula can be used to approximate the quantity of water that will enter spaced relief drains from deep percolation when the drains are above the barrier:

$$q_p = 0.0000727 \frac{ykD}{L}$$

where:

q_p = cfs per foot of drain from deep percolation,

y = maximum distance water table is to be permitted to rise above the drain invert, feet,

k = weighted average permeability of soil profile between maximum water table and the barrier, feet per day,

D = distance of drain above barrier plus $\frac{y}{2}$, feet, and

L = drain spacing, feet.

The terms in this equation relate to the terms and sketches shown in Figure 1 of Chapter 526.4.

For the case where the drain is on the barrier, the applicable formula is:

$$q_p = 0.0000463 \frac{kH^2}{L}$$

where:

q_p = cfs per foot of drain drom deep percolation,

k = weighted average permeability of soil profile between maximum water table and the drain, feet per day,

H = maximum distance water table is to be permitted to rise above the drain invert, feet, and

L = drain spacing, feet.

The terms in this equation relate to the terms and sketches shown in Figure 2 of Chapter 526.4.

Subsurface water flowing into an area from upslope sources can be evaluated quantitatively by use of the basic equation $q_u = kiA$, where q_u is the unit flow. An application of this principle is described in Paragraph 526.6.6A. The slope, i, is obtained from a ground-water table contour map along a line which is normal to the contours, since flow is in this direction. The permeability, k, is the average weighted permeability of the saturated strata above the barrier. Generally, the maximum water-table height would be used to obtain the saturated depth through which the weighted permeability should be obtained. This same depth would be used to obtain the area, A, through which flow would occur.

With this depth the area can be obtained for a 1-foot width. The plane along which the area must be obtained is parallel to the contour or normal to the direction of flow.

The value of q_u in the above equation is for the total amount of moving water within the saturated profile above the barrier, but a 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. The equation then becomes:

$$q_u = kiA \frac{y}{y+d}$$

where:

q_u = cfs per foot of drain from underflow sources,

k = permeability in feet per second,

i = slope of water table, feet per foot,

A = saturated area of flow in a 1-foot width, square feet,

y = height of maximum water surface immediately above proposed drain, feet, and

d = distance from drain invert to barrier, feet.

The flow determined in this manner may originate from one or several upslope sources, depending on the circumstances. One source could be underflow from upslope irrigated farm land; another could be seepage from canals at higher elevations; a third could be seepage from streams, lakes, or other water bodies. It may be necessary to evaluate contributions from individual sources, or it may be that a single computation

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where:

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k = weighted average permeability of soil profile between maximum water table and the drain, feet per day,

H = maximum distance water table is to be permitted to rise above the drain invert, feet, and

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where:

q_u = cfs per foot of drain from underflow sources,

k = permeability in feet per second,

i = slope of water table, feet per foot,

A = saturated area of flow in a 1-foot width, square feet,

y = height of maximum water surface immediately above proposed drain, feet, and

d = distance from drain invert to barrier, feet.

The flow determined in this manner may originate from one or several upslope sources, depending on the circumstances. One source could be underflow from upslope irrigated farm land; another could be seepage from canals at higher elevations; a third could be seepage from streams, lakes, or other water bodies. It may be necessary to evaluate contributions from individual sources, or it may be that a single computation

for q_u will suffice. In making this single computation, the situation must be carefully considered in order to obtain either an average value or limiting high and low values of q_u. The water-table contours will not be regular and parallel and they will fluctuate throughout the year. It is important that records be available for at least a year in order that an evaluation of the proper values of i and A can be made.

If it becomes necessary to evaluate the ground-water contribution of a surface water body, such as a stream, pond, or lake, it may be done by evaluating the factors of surface and subsurface inflow, precipitation, transpiration and evaporation, imported and exporated water, surface outflow, and the change in surface storage, to obtain a value for the seepage loss. It is rarely necessary to make this evaluation, which is fortunate because it is difficult to obtain better than approximate values.

Contributions to ground water by seepage from canals can be obtained by a ponding test wherein the seepage loss can be measured by changes in volume, corrected as necessary by transpiration and evaporation losses, or it can be approximated by the Moritz formula, which is:

$$S = 0.2C \frac{Q}{V}$$

where:

S = loss in cfs per mile of canal,

Q = discharge of canal in cfs,

V = mean velocity of flow in feet per second, and

C = depth of water in feet lost through the wetted area in 24 hours.

Values of C are as follows:

Kind of material	Loss per day in feet of depth
Concrete lining	0.33
Cemented gravel and hardpan with sandy loam	0.34
Clay and clay loam	0.41
Sandy loam	0.66
Volcanic ash	0.68
Volcanic ash with some sand	0.98
Sand and volcanic ash or clay	1.20
Sandy soil with some rock	1.68
Sandy and gravelly soil	2.20

It is often necessary to know the amount of ground-water contribution by canal seepage in order to evaluate the benefits of reducing this contribution. When q_u has been computed under the condition that canal losses are also included, it may be possible to analyze the effect of canal losses on the drainage requirement when the amount of canal loss is known. In this way a determination can be made of the effect of canal lining on the drainage requirement, and a cost comparison made of canal lining versus drain construction. This does not necessarily imply that the drainage requirement will be eliminated by lining of canals, but that it will be reduced and possibly eliminated. Lining of a canal does not permit the assumption that seepage is eliminated because even the best lining usually permits some seepage--as can be seen by the Moritz value for C for concrete lining. The effect of canal

lining on the drainage requirement will depend on the capability of the formation to convey water in relation to the amount remaining to be conveyed.

The accretion for which drains should be designed is the sum of the individual components, or:

$$q = q_p + q_u$$

where:

q = cubic feet per unit of time per foot 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 to seepage from surface water bodies.

.11 Design Discharge for Determining Pipe Size. The discharge, q, determined above for each foot of pipe can be used in the formula $Q = qL$ where Q is the discharge in cubic feet per second at the end of L feet of pipe. This formula for Q will be applicable for a length of pipe L which serves an area that is irrigated practically simultaneously--probably within about 2 days. If q is the maximum rate of discharge per unit length of pipe, the formula obviously gives the discharge only for the time that the water table is highest. At any other time the rate of discharge will be less than maximum. The water-table height, and the resultant value of q, will fluctuate mainly because of the intermittent application of irrigation water, since the q values for canal seepage, underflow, etc., will be practically constant.

Now take the case of a collector drain which receives water from a group of drains serving an area which requires about 10 days to

irrigate. Each of the branch drains will be delivering water to the collector at a rate Q which will depend 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 least recently will have the lowest discharge. Those drains serving parcels intermediately irrigated will be discharging at a rate 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.

There are few data on which to base a rationalization of the reduction in flow received by collector drains as described above. If the reduction is due exclusively to the alternate application of irrigation water to various land parcels, then the relation of drain discharge to area should follow a similar relation of water application as shown on the canal and lateral capacity curve in Figures 8 and 9 of Chapter 523.0. This relation has not been satisfactorily established for general use, so, meanwhile, the required capacity of collector drains with only two or three tributaries should be based on the summation of maximum discharges delivered by these tributary drains. In case of a collector drain that has more than three tributaries, it is permissible to make a reduction in required capacity for pipe design of not to exceed 25 percent of the quantity calculated above. Any adjustment must be made on a judgment basis and should only be done for larger systems where it is

obvious that the summed total quantities will not accrue simultaneously to the collector drain.

.12 Outlet Conditions. One of the first considerations in all drainage planning is to determine the adequacy of the outlet for the system of drains. If the outlet is not adequate, it must be made so or pumping of the discharge must be planned. Either of these measures may affect the feasibility of drainage.

The investigations necessary to determine the adequacy of outlets depend upon the characteristics of the stream or area which is to serve as the outlet or disposal area. Where drainage systems are to discharge into rivers, creeks, lakes, or other water bodies which are affected by high water, it is necessary to determine the elevation, frequency, and duration of the high water as nearly as possible, and to analyze this effect on the drainage system. These elevations will limit the elevation of the lower end of the hydraulic gradient 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, in which case the invert elevation must be raised. Under usual circumstances, this means that the drained lands must be about 10 feet or more above the outlet elevation if the lands are to be economically drained.

These high water conditions can be obtained from gage records, if available, from observation of marks on the banks of streams or lakes,

and from discussions with local people. The adequacy of natural outlets can be found by computing the estimated runoff from the entire area which they serve and checking their capacity.

There may be exceptional cases where the effluent from surface drains may be disposed of in sumps where it will percolate into the ground and join the ground-water body. This is possible only where the ground-water body itself has an outlet into a stream or other drainage pattern, or into an area where it will not become a problem that will require subsurface drains. The infiltration rate in these sumps must be high enough to support disposal of the necessary quantities, and must remain at this capacity for the time necessary to make the method economic.

.13 Drain Location. There is no fixed rule or set of rules to direct the drainage engineer in locating every drain. Each location presents an individual problem which is solved by analyzing the conditions involved. Wherever possible, outlet drains and collectors should be located in natural drainageways, while relief and interceptor drains must be located where they will produce the best results. The location and spacing of drains requires careful study and a great deal of common sense on the part of the drainage engineer. As the tentative drain locations are decided upon, they should be drawn on the map of the area. After this, their centerlines should be located on the ground. Frequently, the actual locations on the ground will make it obvious that a change in location or alinement needs to be made. In these cases, the locations on the ground should be changed as required and the map locations changed

to correspond. The centerline on the map should be scaled and stations marked for future reference. After the centerline has been established on the ground, it should be drilled at intervals 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 as required. Stationing should start at the mouth of the outlet and proceed upstream. In some instances it may be necessary to use a transit to establish centerlines, but quite often, where the location is open, the line may be staked out by eye with the use of range poles. This method will usually give as accurate alinement as is necessary. In the consideration of ditch locations, allowance should be made for sufficient right-of-way to permit proper construction.

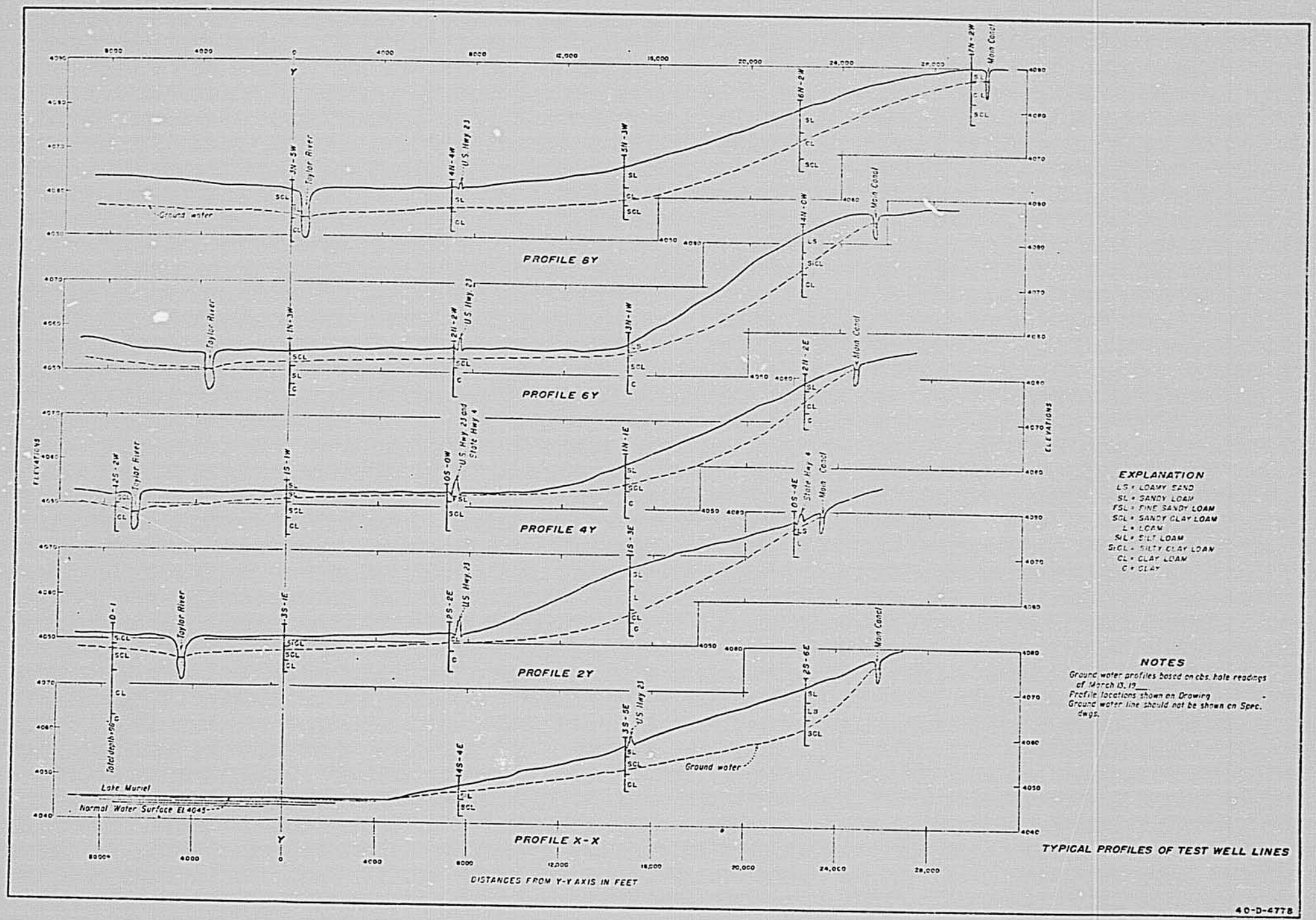
.14 Drain Numbering. After the drains have been laid out and staked, they should be given an identifying number. No single numbering method that fits all drain layouts has been devised. One method of numbering that is adaptable to many situations is to locate the mouth of the outlet with respect to land subdivisions, and the junction of tributaries with respect to the outlet. If the mouth of the outlet is in Section 3, Township 7 North, Range 10 West, the number of the outlet drain would become 3-7N-10W. Letters for the cardinal directions are not necessary if there is no possibility of confusion. If more than one outlet drain discharges in Section 3, the first would become 3A, the second 3B, and so on. If the first branch is located 3,200 feet up the outlet from its mouth, the number of the tributary drain would be 3-7N-10W;3.2. If

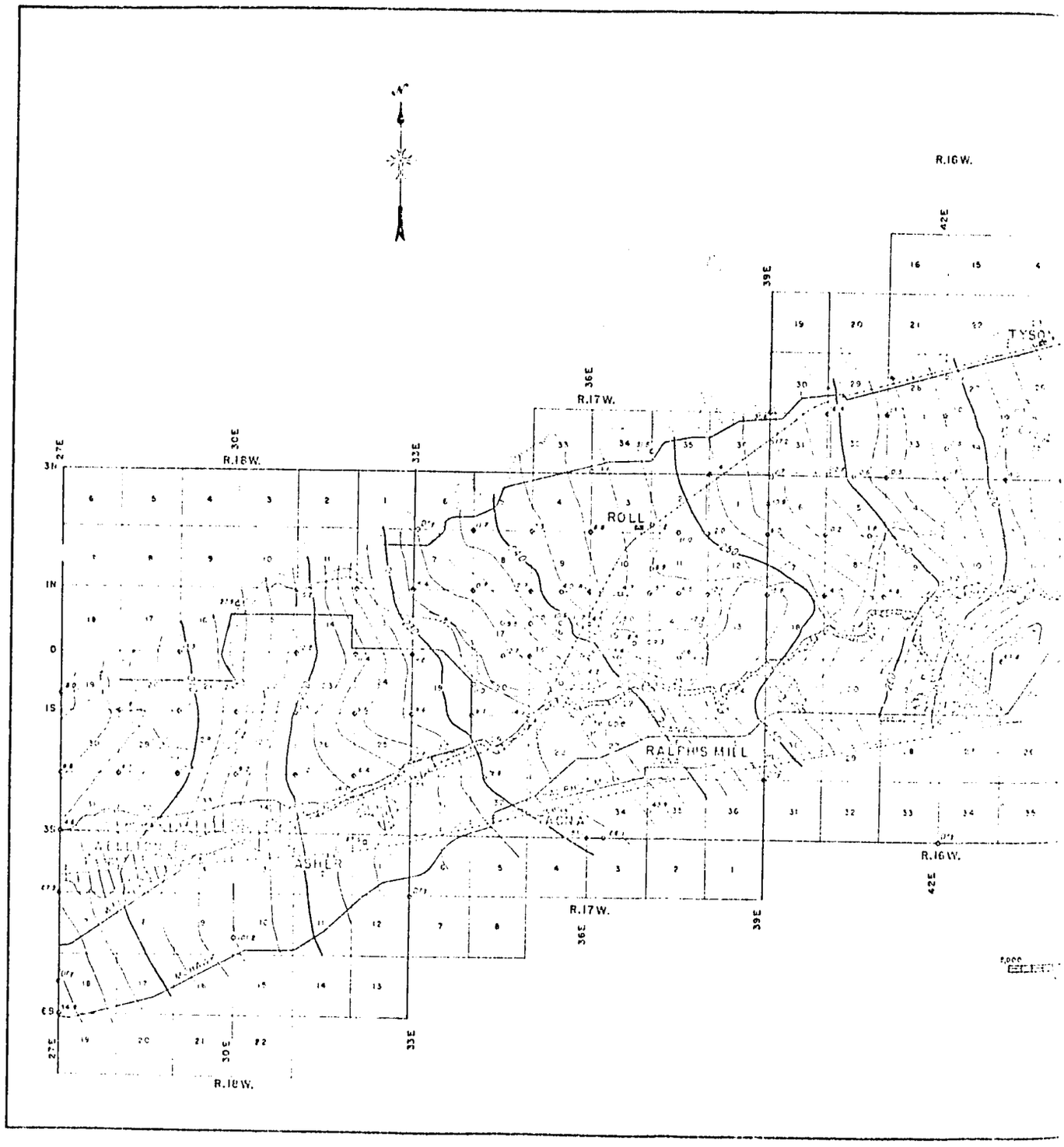
a tributary drain from both sides intersects the outlet drain at this point, the one on the right (looking upstream) would be numbered 3.2R and the one on the left, 3.2L. Junctions up the tributary drain would be numbered the same way, by adding to the above number the distance to the upper junction from the lower junction in units and decimals of 1,000 feet. This can be continued as far as required until the highest drain is numbered. It should be noted that R and L do not conform to hydraulic practice of assigning right and left when looking downstream, but to drain surveying practice of starting the stationing at the lower end and proceeding upstream. In case the method is not adaptable for any particular situation, some other numbering method will have to be devised since all drains should have a number which will aid in locating them both on maps and in the field.

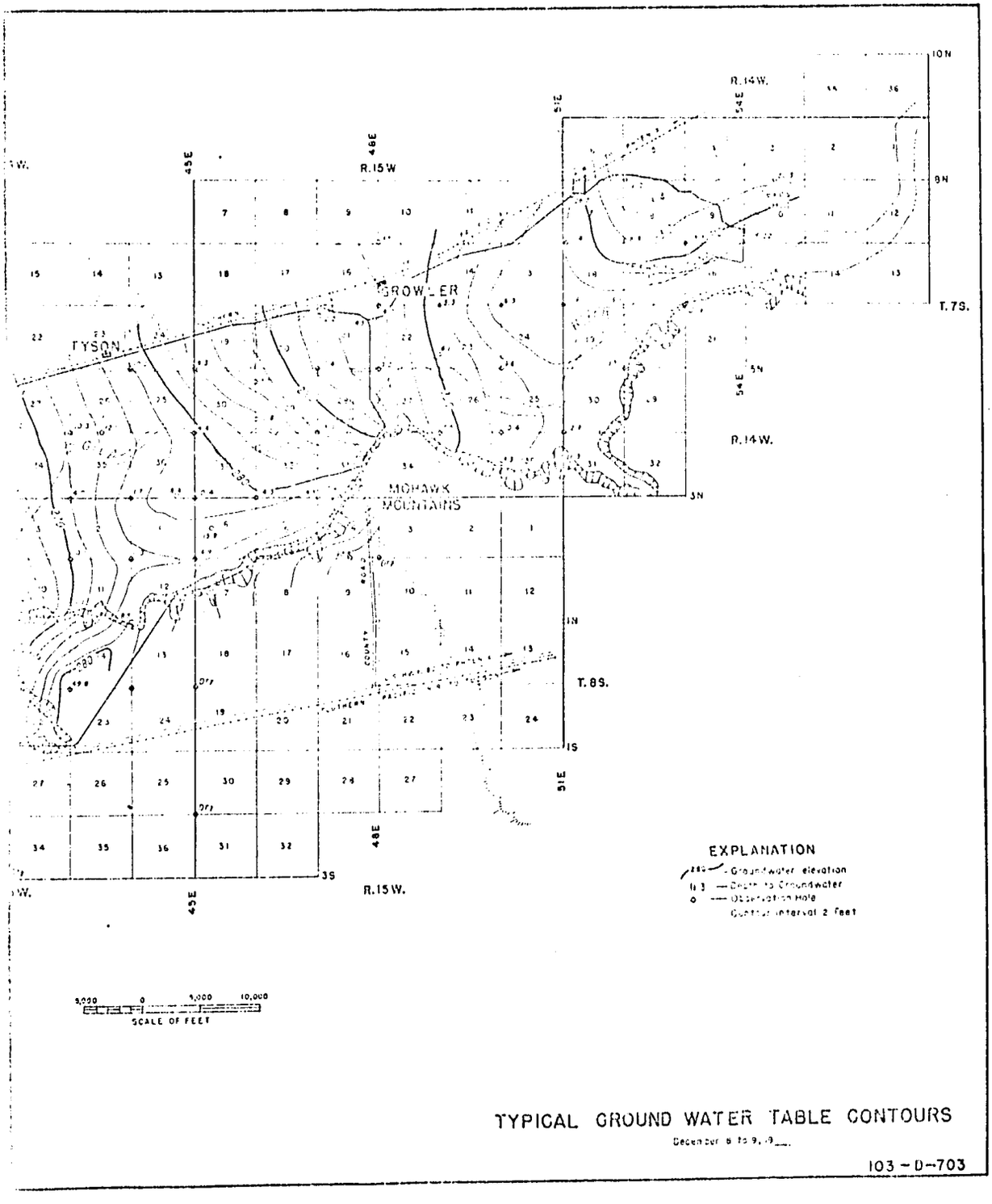
.15 Existing Structures. The location, elevations, and capacities of all bridges and culverts through which the proposed drain will pass should be determined. Bridge footings should be investigated and the elevations of road or railroad fills determined. The location of all utility lines and buildings which could have an effect upon the construction work should be noted and appropriate descriptions of structures and the conditions obtained.

.16 Economic Considerations of Drainage Problems. Many times a cost-versus-benefits analysis is required to determine whether an area should be drained; whether an irrigation system should be lined rather than a drainage system built to carry off canal seepage; or whether drainage

should be by pumps, by open or closed drains, or by a combination of all these. The drainage engineer is responsible for making cost estimates for all alternative methods, but analysis of benefits is not his responsibility. He should be alert at all times to alternative methods of obtaining results and should strive toward the engineering solution of problems at the lowest cost. When a direct comparison of costs is not adequate for an economic solution, a benefit analysis should be made by an economist.


PROFILE 8Y
PROFILE 6Y
PROFILE 4Y
PROFILE 2Y
PROFILE X-X
Taylor River
U.S. Hwy 23
State Hwy 4
Main Canal
Ground water
Lake Muriel
Normal Water Surface El 4045
Total depth-86'
ELEVATIONS
DISTANCES FROM Y-Y AXIS IN FEET
EXPLANATION
LS = LOAMY SAND
SL = SANDY LOAM
FSL = FINE SANDY LOAM
SCL = SANDY CLAY LOAM
L = LOAM
SiL = SILT LOAM
SiCL = SILTY CLAY LOAM
CL = CLAY LOAM
C = CLAY
NOTES
Ground water profiles based on obs. hole readings of March 13, 19__
Profile locations shown on Drawing
Ground water line should not be shown on Spec. dwgs.
TYPICAL PROFILES OF TEST WELL LINES
40-D-4778


R.16W.
42E
39E
36E
R.17W.
30E
R.18W.
33E
27E
TYSON
ROLL
RALPH'S MILL
ASHER
R.16W.
R.17W.
R.18W.



TYPICAL GROUND WATER TABLE CONTOURS

December 8 to 9, 19__

103 - D-703

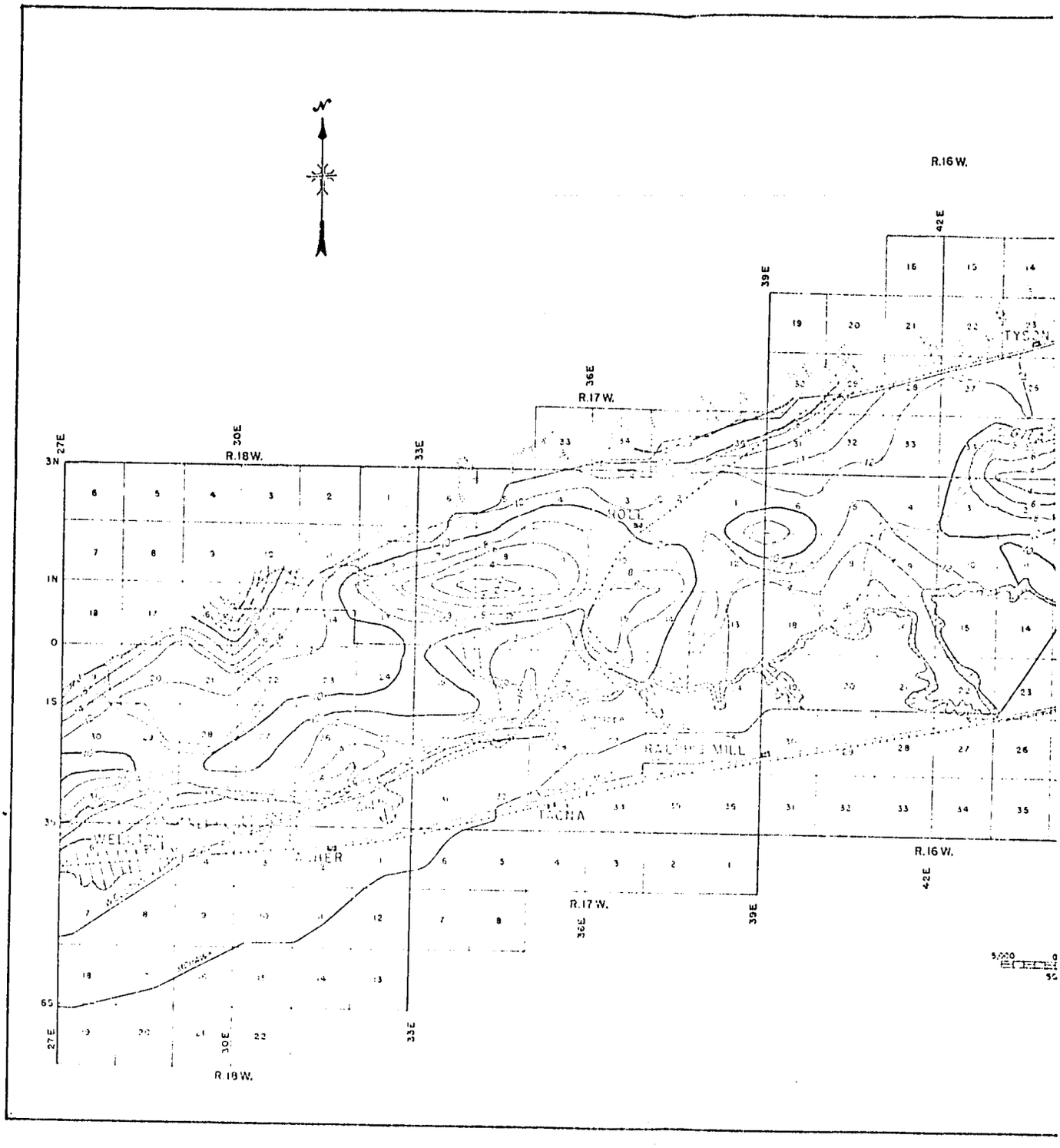
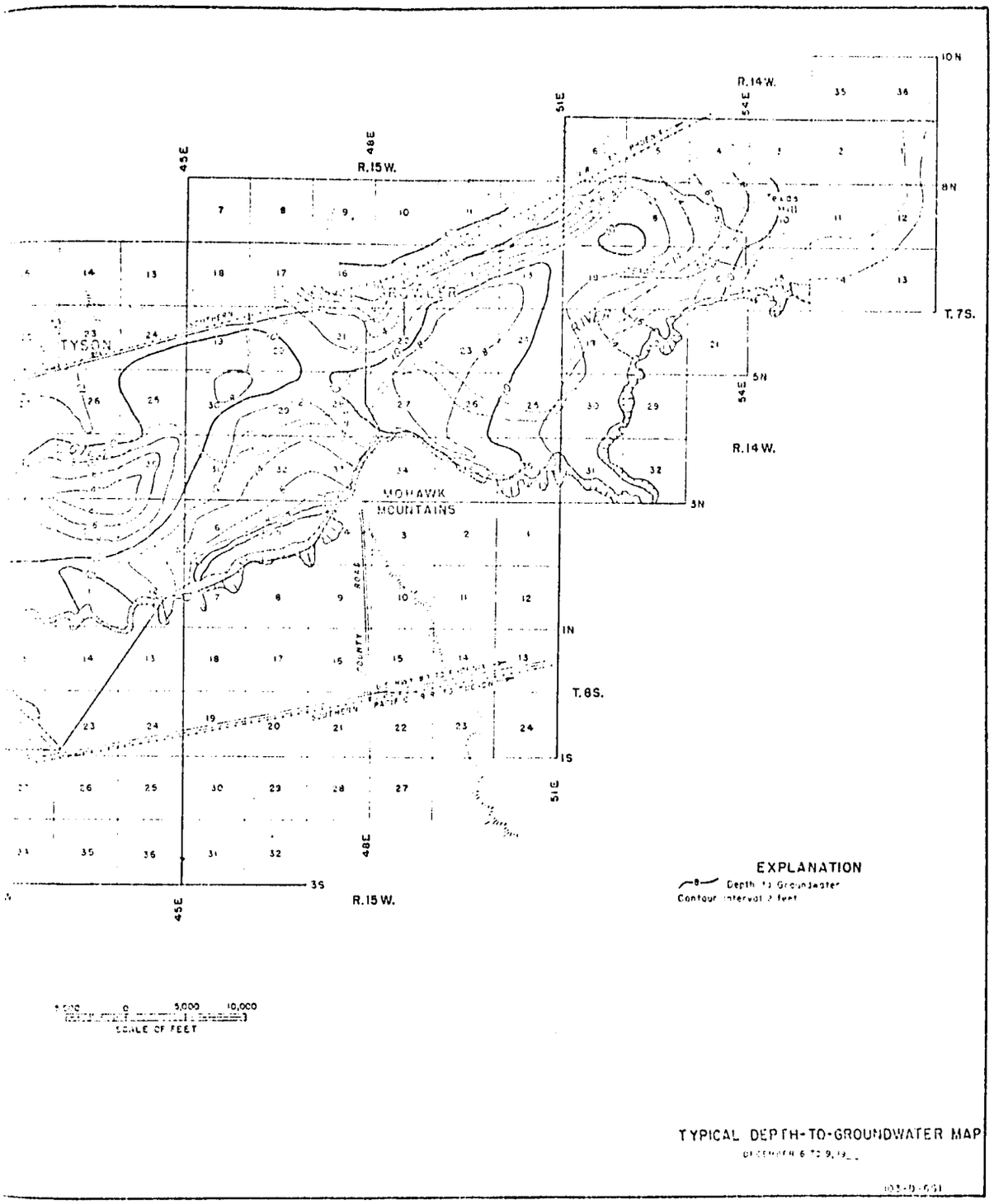

R.16 W.
R.17 W.
R.18 W.
3N
1N
0
1S
6S
27E
30E
33E
36E
39E
42E
TYSON
HOLL
RALPHS MILL
TACNA
WELLMAN
5,000

Figure 3
Par. 525.0.9B



TYPICAL DEPTH-TO-GROUNDWATER MAP

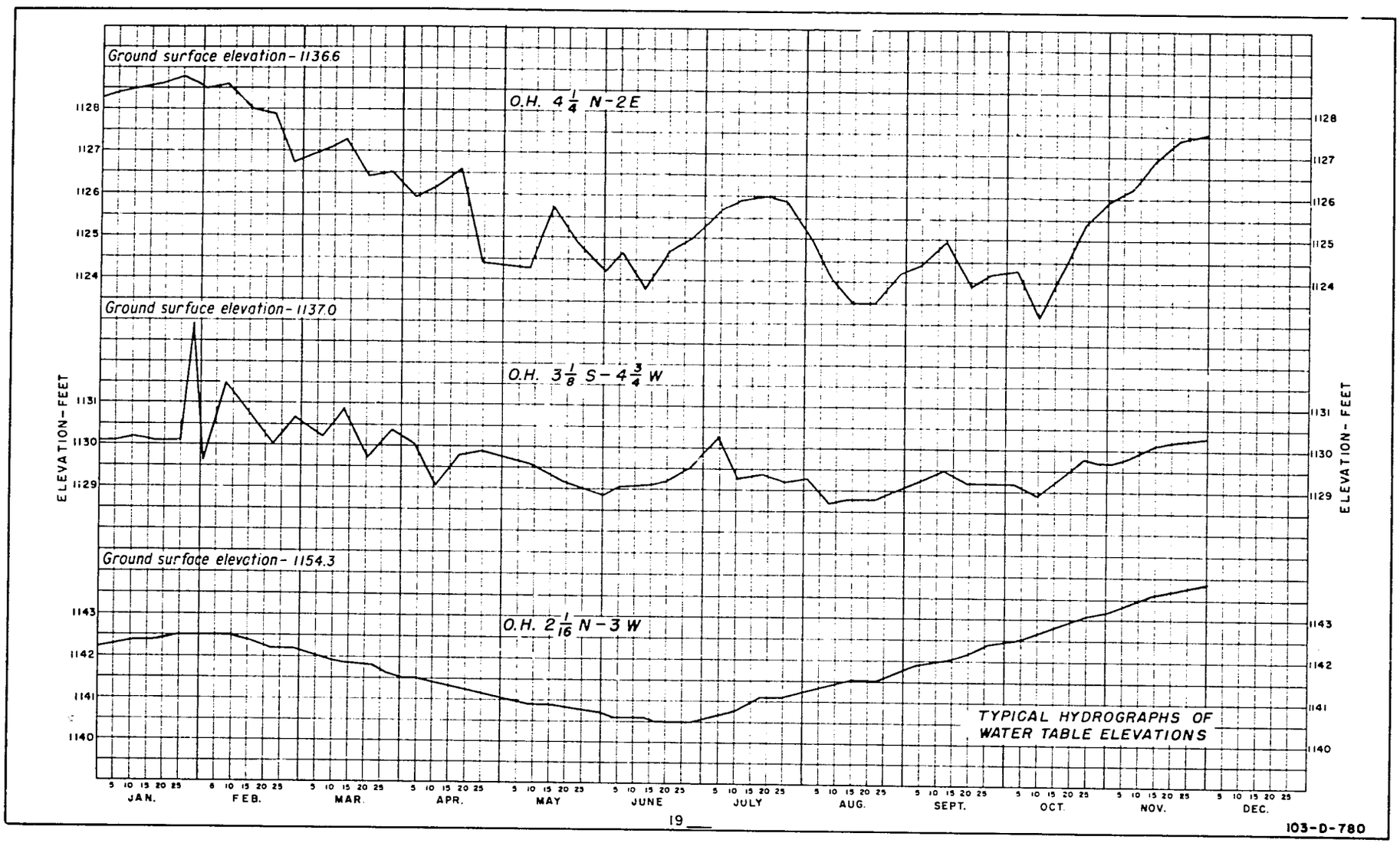
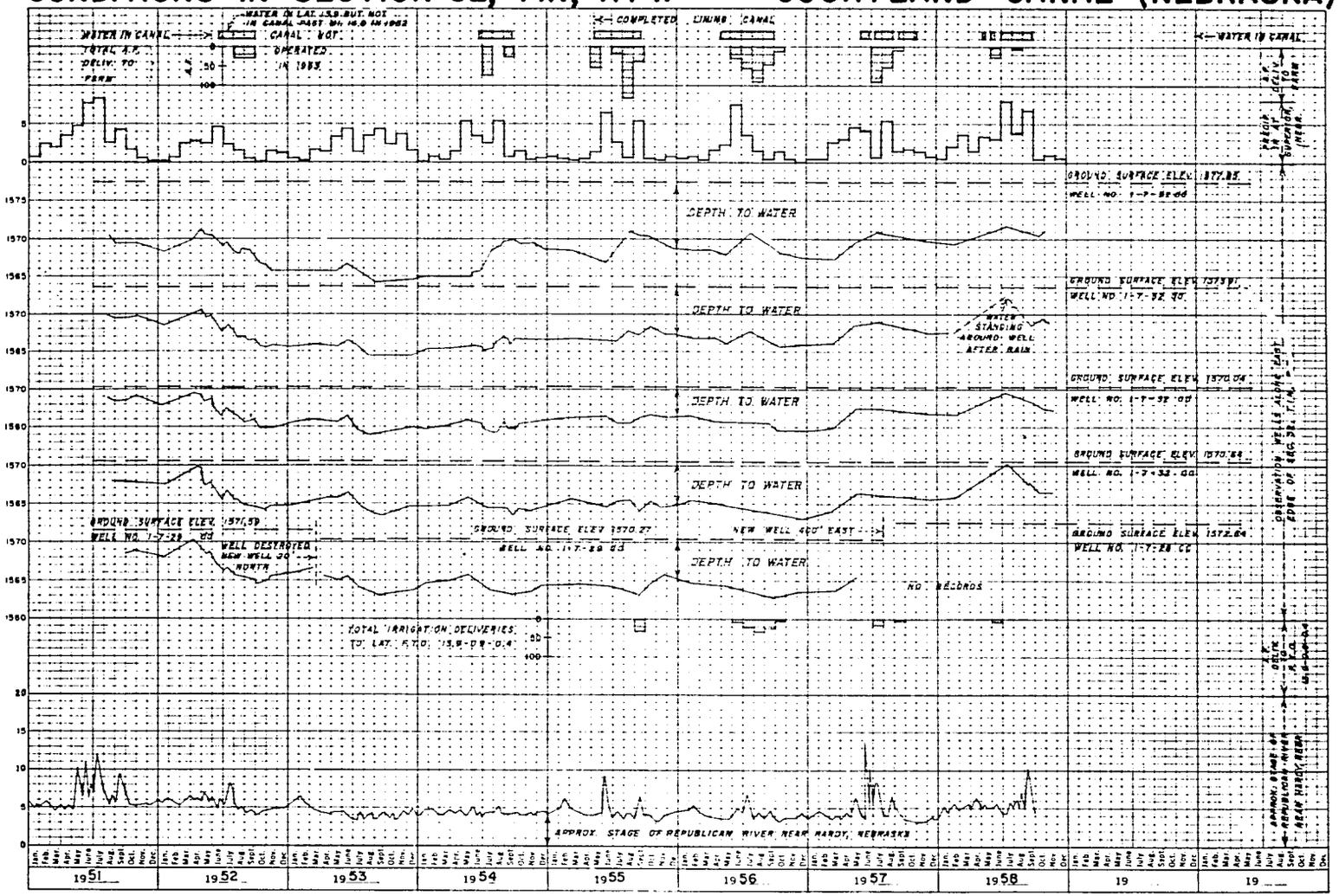


Figure 4
Par. 525.0.9F

CONDITIONS IN SECTION 32, T1N, R7W - COURTLAND CANAL (NEBRASKA)



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Reclamation Instruction Series 520

LAND DRAINAGE TECHNIQUES AND STANDARDS

Part 526 Design Criteria

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.1 Introduction. There are two general types of open drains--shallow drains and deep drains. Shallow drains are normally used for the removal of irrigation surface waste and of storm water. This type of drain provides very little underdrainage and is considered simply a waste-water ditch or storm channel. Deep drains are used not only to provide underdrainage but also to act as outlets for a closed drain system or for the shallow, surface drainage system. The design criteria are approximately the same for either type of drain.

The theory and details of open channel design are thoroughly presented in many text books on hydraulics, so only those criteria that pertain to design of drains are presented here. Figure 1 shows a typical plan and profile of an open drain.

.2 Open Channel Flow. The area of a drain section for any flow is found from the equation $A = Q/v$. The velocity, v, based on Manning's formula, can be taken from Bureau of Reclamation "Hydraulic and Excavation Tables" using a coefficient of roughness value of $n = 0.030$. Where tables are not available, the Manning formula can be used with the same n value. This formula is

$$v = \frac{1.486}{n} r^{2/3} s^{1/2}$$

where:

v = velocity, feet per second,

r = hycraulic radius, feet, and

s = slope of the drain, feet per foot.

The Hydraulic and Excavation Tables do not contain values of A and r for small, V-shaped, surface drainage ditches, so these values can be found in Figure 2.

.3 Velocities. Permissible maximum velocities are as follows:

Soil	Velocity, fps
Stiff clay	4
Sandy loam	2.5
Light sandy	1.5

In doubtful soil textures it may be necessary to make a tractive force analysis 1/ to determine the probable 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.

Most areas subject to waterlogging are relatively flat, with only a limited elevation above the water surface in outlet channels where drainage water must be discharged. Minimum gradients, rather than maximum, are usually the controlling factor. In some cases, even with minimum grades throughout the drainage system, it is necessary to provide pumping plants to lift drainage water into a river or other outlet channel. The maximum grade or slope possible under given topographic conditions is desirable, provided the velocity is kept below that which would cause significant erosion. Where surface slopes are steep, structures will have to be provided to control velocities.

1/"Progress Report on Results of Studies on Design of Stable Channels," Hydraulic Laboratory Report No. Hyd-352, Bureau of Reclamation, Denver, Colorado, June 1952.

.4 Depth of Drain. The depth of a shallow drain for carrying surface water is controlled only by the quantity of water is is required to carry, while the depth of a subsurface drain for controlling the ground-water table is affected by the spacing of the drains and limitations of construction equipment as well as by the quantity of water it must carry. The most difficult design case is that of a drain required to receive water from tributary drains, to carry flood flow and farm waste, and to pick up ground water throughout its reach. First, it must be deep enough that the surface of the water flowing in it will be below the water table. This will permit the drain to pick up ground water, and the greater the depth the greater the area of influence the drain will have. Second, the depth must be great enough that tributary drains can discharge into it. The water surface elevation in the collector drain must not be higher than that in the tributary drain. Third, an allowance of capacity must be made for carrying flood flows. This is usually no problem in a complete open drain system, because when the first two items are satisfied there is excess capacity that will usually handle most flood flows. It may be that the flood flow will raise the water level in the drain to a point higher than the ground-water elevation, which prohibits the drain from picking up ground water, but this is a temporary condition and is not harmful. In certain areas where flash floods may be frequent and the soils are highly erosive, it may be economical to provide separate drainage and flood water systems.

When the tributary drains are closed drains, the invert of the open collector drain should be below the invert of the closed drain a

distance sufficient to provide for the depth of normal flow (subsurface water plus farm waste) in the open drain, plus an additional distance which will allow for some flood water in the open drain without backing up in the closed drain and will allow for some sloughing of the ditch banks between cleanings. Here again an occasional rise in water level due to larger floods is not detrimental. This additional distance should be 18 inches if economically and physically possible, but can be as low as 6 inches if banks are stable, or if the open drain depth would otherwise be unreasonable.

In general, subsurface drains should have a depth in the range of 8 to 10 feet to provide the best economic balance between cost of drain and drain spacing. On occasion, they may be shallower or deeper, depending on local conditions, the most important of which is the location of the underlying permeable and impermeable strata.

.5 Drain Section. In common with any open channel designed to carry water, the drain cross section should approach, as nearly as possible, a trapezoidal section. A semicircular canal has the highest hydraulic efficiency, but physical construction difficulty makes the trapezoidal shape the most economic section.

Slope of the sides depends on the type of material through which the drain is excavated. The side slopes should be greater than the angle of repose of the saturated material, at least as far up the slope as will be wet when the drain is operating. Above the saturation line the slopes can relate to the angle of repose of the dry material. In

material in which drains are usually constructed, side slopes will be between 1-1/2:1 and 2:1, but in some materials they may be 1-1/4:1 or as much as 3:1.

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 both the excavating and maintenance equipment available for use. If a dragline is to be used, there is no advantage in calling for a width less than that of a bucket which is about 3 feet. Figure 3 shows the recommended relation between roadways, spoil banks, and berms for drains of different sizes.

In cases where large base widths are required for flood control purposes, the use of pilot channels in the bottom of the drain should be considered. A pilot channel, which may be considered a "drain within a drain," is a small ditch of sufficient capacity to accommodate normal subsurface drainage flows only. Such channels serve to stabilize the bottom of the large drain by keeping it dry, thus confining aquatic growths and sloughing to the pilot drain and thereby appreciably lowering maintenance costs.

.6 Tributary Drain Intersections. Open tributary drains should enter the larger drain with their water surfaces at the same elevation. If the tributary drain carries more than about 15 cfs, it must be curved downstream on the lower end to make the flow lines of the two streams more

nearly parallel at the point of juncture. This is not required for tributaries of less than 15 cfs flow, but it will improve the flow characteristics and reduce maintenance costs even on small streams.

.7 Surface Water 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 4 shows an acceptable method for installing a surface water pipe inlet to an open drain.

.8 Transition Sections. When changes occur in the channel depths or bottom widths, the changes should not be made abruptly but over a distance of 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 side drains. It is simpler to change either the depth or bottom width instead of both; however, in some designs it will be necessary to change both.

.9 Design Capacities. Surface drain channels should be designed for storm flow only (see Subparagraph 523.0.5B) and no allowance need be made for irrigation waste. This is because the value for storm flow is so much greater than the value for irrigation waste that the addition of the smaller amount to the larger is an unnecessary refinement. In general, the storm flow should be that obtained from the 5-year frequency storm unless there is information available which will justify variation from this value. The minimum capacity of surface drains will be 3 to 5 cfs,

as determined by available construction equipment. Ponding in the fields of storm flows should be considered in surface drain capacity estimates, but ponding on arable land should not be permitted for periods exceeding 48 hours.

Capacities of open interceptor and relief drains intended primarily for control of ground-water levels should be sufficient to carry (1) the estimated ground-water accretion plus (2) the estimated farm waste, with the water surface at or below the required effective drainage depth. Storm water from fields, which may enter these drains through the regular farm waste inlets, will not be considered in these designs unless stability is a problem as neither the quantity nor the duration of flow would normally adversely affect the efficienty of the drain.

Capacities for open collector drains should be sufficient to carry normal flow of ground-water accretions and irrigation surface waste, plus the estimated storm flow, plus the quantities delivered to the collector drains by relief and interceptor drains.

Capacities for open outlet drains should be sufficient to carry the flows from the collector drains.

Wasteways from canals are sometimes turned into 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.

.10 Structures. Open drain structures consist of inlets to the drain; drops and chutes; and road, railroad, and canal crossings. Actual structural

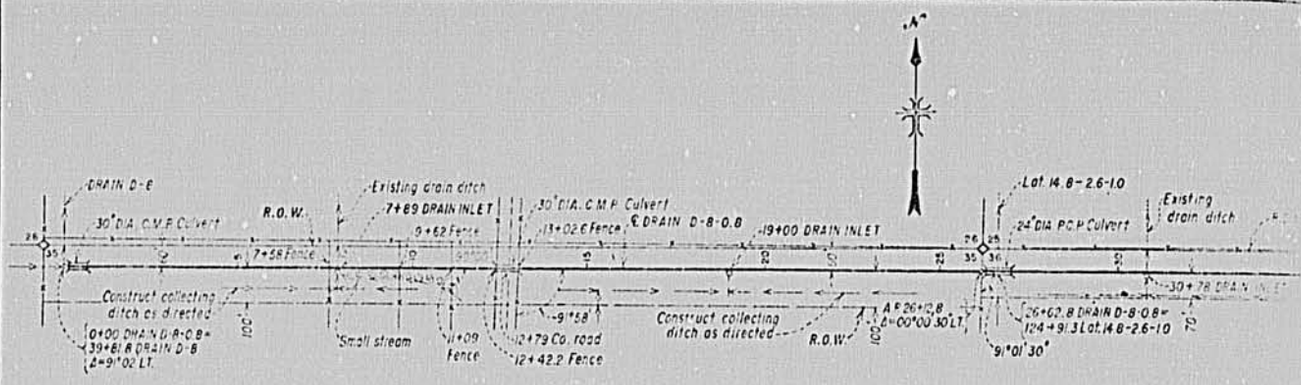
design should be made in accordance with Bureau policy and standards as contained in Design Standards No. 3, Canals and Related Structures.

Inlets should be of corrugated-metal pipe with a design coefficient of roughness, n, of 0.021. The pipe can be galvanized, asphalt dipped, or asbestos bonded, depending on the corrosivity of the soil. This can be best determined by experience in the area with highway culverts, existing drainage structures, or similar means. The minimum pipe size should be 18 inches to minimize operation and maintenance costs, the velocity in the pipe should not exceed 10 feet per second, and the minimum pipe slope should be 0.01. The outlet end should extend 12 inches beyond the edge of the normal water surface in the drain so that water from the pipe will not fall on, and erode, the bank, and this end should be at least 18 inches above normal water surface (see Figure 4). 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 1 foot above the pipe. No collars are required.

Conventional chute structures may be used where appropriate. Drop structures should be used as follows:

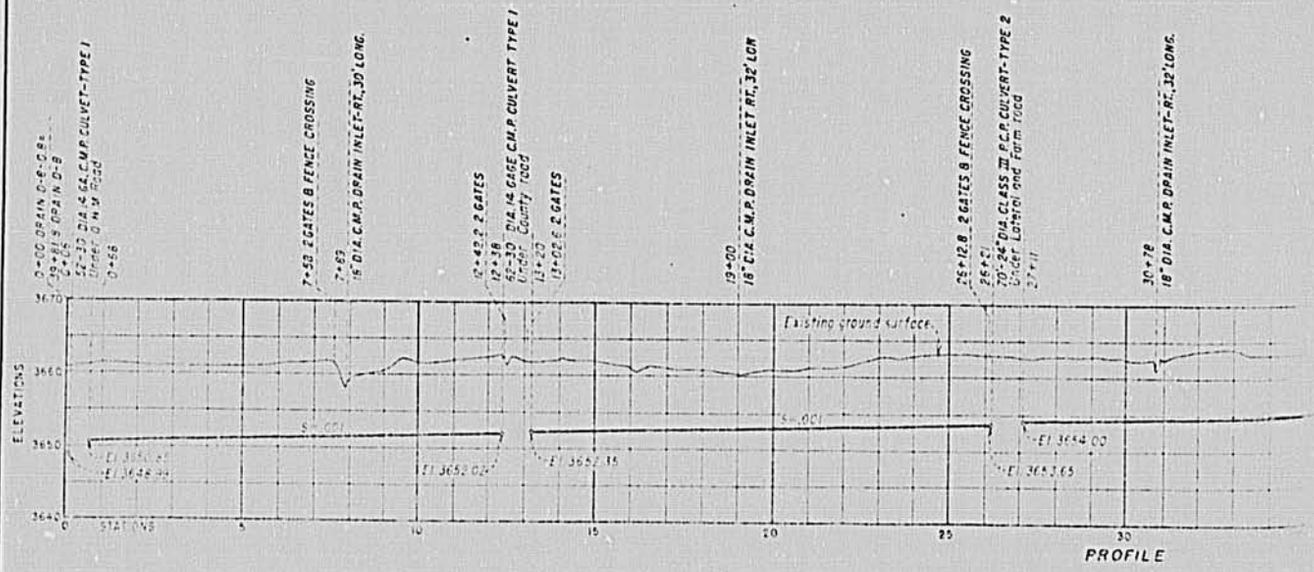
Drop differential in feet	Structure
0 to 2.0	No structure
2.0 to 5.0	Cascade drop with sheet piling
5.0 and over	Baffled apron

Crossings can be of either metal or concrete pipe depending on the importance of the crossing, which is measured by the loss that would result from its failure. In aggressive soils and with aggressive water, pipe should be protected for an economic life. Crossings of major highways, railroads, and canals should be designed for flows from a 25-year storm; for crossings of less importance 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 farm ditches. Circular pipe culverts should be placed with a maximum of 50 percent of the diameter below grade line. Pipe-arch corrugated-metal culverts, if justified, can be placed with about 20 percent of the "rise" value below grade line. The pipe should extend beyond the toe of the fill. Collars should be placed on the pipe as necessary. (See Figure 4.)



PLAN

200 0 200 400
SCALE OF FEET



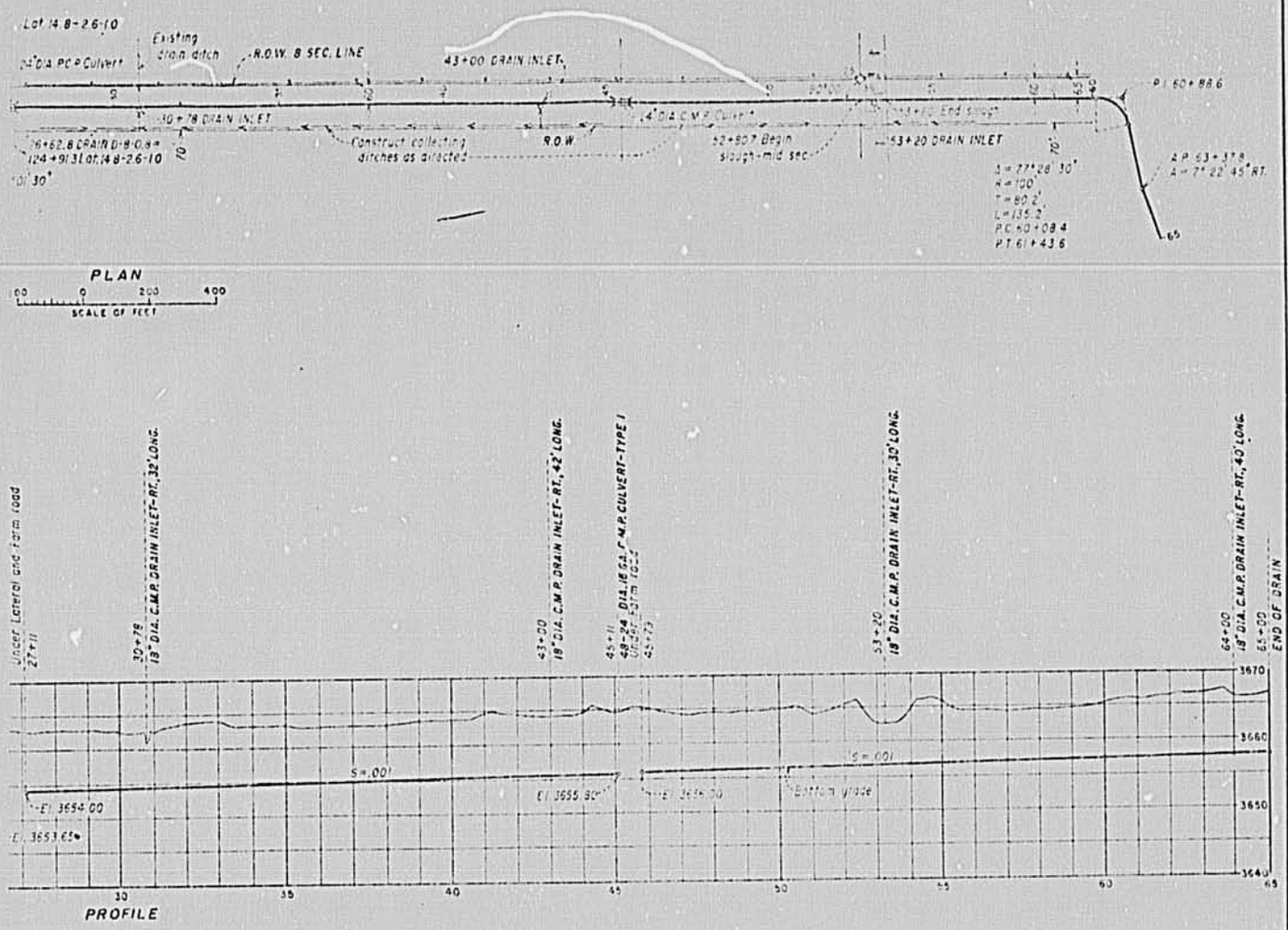
PROFILE

HYDRAULIC PROPERTIES

n=.030

REACH	b	s	NORMAL Q	SIGNIFICANT Q	d	A	r	V
0+66 TO 13+20	4.0	.001	1.5		.9	4.82	.065	1.14
				.33	2.3	7.15	.140	.93
13+20 TO 30+78	4.0	.001	4.0		.7	3.55	.54	.13
				.23	1.9	5.51	.20	.77
30+78 TO 53+20	4.0	.001	2.5		.6	2.93	.48	.85
				.18	1.7	4.15	.10	.61
53+20 TO 65+82	4.0	.001	1.0		.3	.94	.26	.75
				.11	1.3	2.24	.83	.42

Figure 1
Par. 5.6.1.1



	A	r	V
	4.92	0.66	1.14
3	7.15	1.40	1.93
7	3.55	0.54	1.13
9	10.3	1.20	1.77
	2.94	.48	.85
7	11.15	1.10	1.61
3	1.34	.26	.75
3	7.74	.89	1.42

GENERAL NOTES FOR DRAINS

For typical drain sections see Dwg. 103-D-664
For culvert and drain inlet details, see Dwg. 103-D-635
Normal Q in the hydraulic properties table is the estimated sub-surface accretions plus irr. 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.
Where angle points are shown in plan, construct alinment on a 50' radius curve, where feasible.
Center lines of drains may be shifted within the right-of-way as directed.
Where directed, gravel shall be placed on drain banks. Thickness of gravel to be 6" for Δ of 45° or less and 12" for Δ of greater than 45°

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DESIGN DATA

TYPICAL OPEN DRAIN
PLAN AND PROFILE

DRAWN ..... SUBMITTED .....
TRACED E.E.R. ..... RECOMMENDED .....
CHECKED ..... APPROVED .....
DENVER, COLORADO | 103-D-663

1½ : 1 SIDE SLOPES

DEPTH (feet)	0.0		0.2		0.4		0.6		0.8	
	A	r	A	r	A	r	A	r	A	r
0							0.54	0.13	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 (feet)	0.0		0.2		0.4		0.6		0.8	
	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

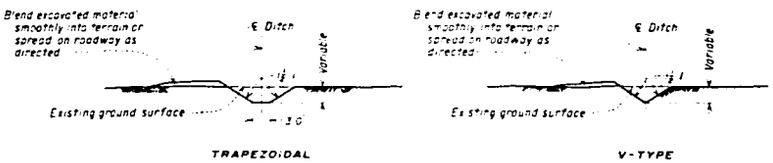
DEPTH (feet)	0.0		0.2		0.4		0.6		0.8	
	A	r	A	r	A	r	A	r	A	r
0							0.90	0.28	1.60	0.37
1	2.50	0.46	3.60	0.56	4.90	0.65	6.40	0.74	8.10	0.84
2	10.00	0.93	12.10	1.02	14.40	1.11	16.90	1.21	19.60	1.30
3	22.50	1.39	25.60	1.49	28.90	1.58	32.40	1.67	36.10	1.76
4	40.00	1.86	44.10	1.95	48.40	2.04	52.90	2.13	57.60	2.23

3 : 1 SIDE SLOPES

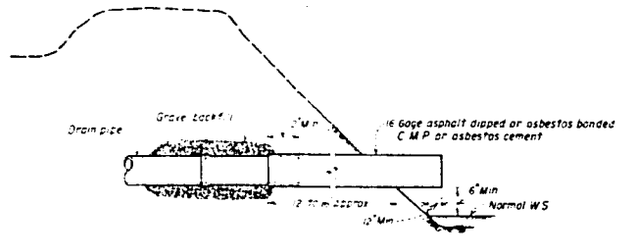
DEPTH (feet)	0.0		0.2		0.4		0.6		0.8	
	A	r	A	r	A	r	A	r	A	r
0							1.08	0.28	1.92	0.38
1	3.00	0.47	4.32	0.57	5.88	0.66	7.68	0.76	9.72	0.85
2	12.00	0.95	14.52	1.04	17.28	1.14	20.28	1.23	23.52	1.33
3	27.00	1.42	30.72	1.52	34.68	1.61	38.88	1.71	43.32	1.80
4	48.00	1.89	52.92	1.99	58.08	2.09	63.48	2.18	69.12	2.27

CROSS-SECTIONAL AREA AND HYDRAULIC RADIUS FOR "V" DITCHES

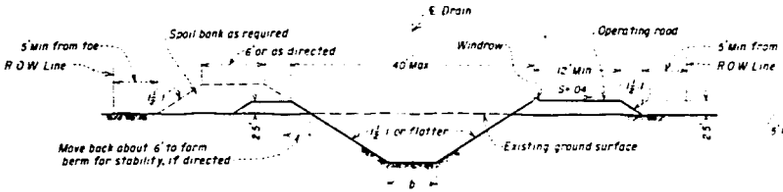
103-D-682



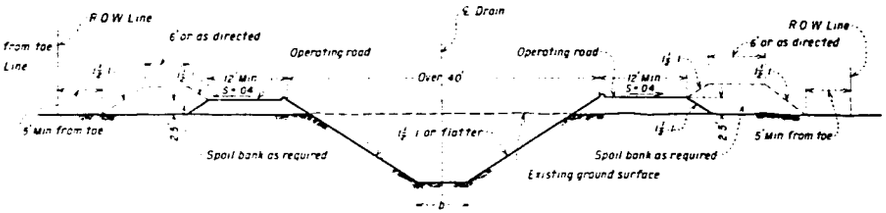
TYPICAL SURFACE DRAIN AND COLLECTING DITCH SECTIONS



ELEVATION CLOSED DRAIN OUTLET



OPERATING ROAD - ONE SIDE ONLY

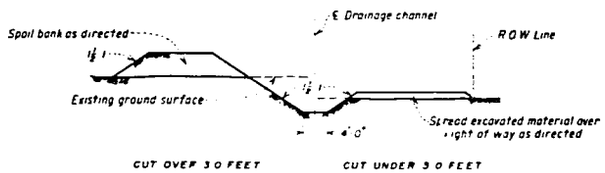


OPERATING ROAD - BOTH SIDES

TYPICAL OPEN DRAIN SECTIONS

NOTE

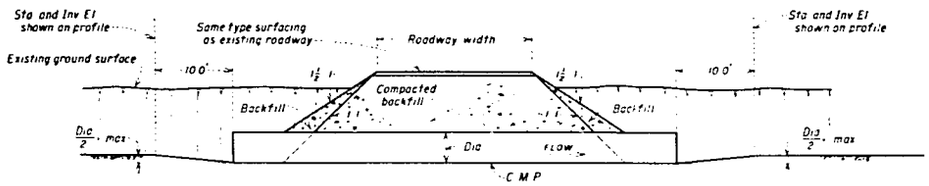
See plan and profile drawings for alinements, grades, dimensions, and hydraulic properties



TYPICAL CROSS DRAINAGE CHANNEL SECTION

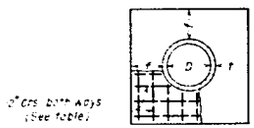
TYPICAL DRAIN, AND COLLECTING DITCH SECTIONS

103-D-664



TYPE 1 IN-LINE CULVERT

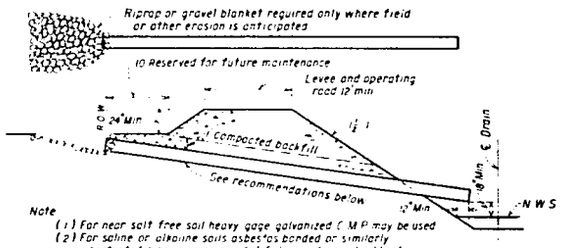
Note Pipe lengths shown on profiles are based upon roadway widths



CONCRETE CUTOFF COLLAR

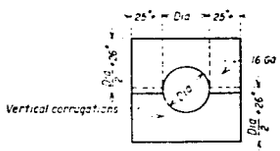
TABLE OF DIMS AND REINF

D	t	SCHEDULE	COLLAR REINF
18"	18"	[illegible]	#3
21"	18"	[illegible]	#3
24"	18"	[illegible]	#3
30"	21"	[illegible]	#3
36"	24"	[illegible]	#3
42"	[illegible]	[illegible]	#4
48"	[illegible]	[illegible]	#4
[illegible]	[illegible]	[illegible]	#4
60"	[illegible]	[illegible]	#4

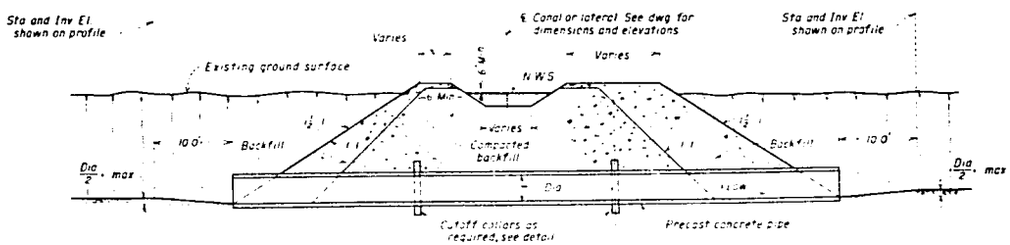


Note
(1) For near salt free soil heavy gage galvanized C M P may be used
(2) For saline or alkaline soils asbestos bonded or similarly protected pipe is recommended Galvanized pipe should not be used
(3) Minimum pipe diameter is 18"

TYPICAL SECTION OF DRAIN INLET



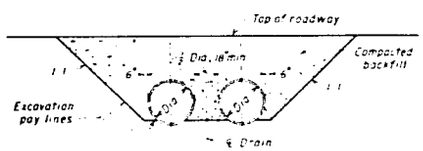
METAL CUTOFF COLLAR



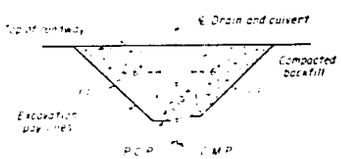
TYPE 2 IN-LINE CULVERT

NOTES

For locations and dimensions of culverts and drain inlets, see Plan and Profile Dwgs of Drains
For protection details at ends of culverts, use depth for the storm Q to determine type and extent of protection
For typical sections of drains, see Dwg 103-D-6F4



TYPICAL SECTION OF DOUBLE BARREL CULVERT



TYPICAL SECTION OF SINGLE BARREL CULVERT

TYPICAL CULVERTS AND DRAIN INLETS

103-D-683

Figure 4
Par. 526.1.7

.1 Introduction. Closed drains are used when they are lower in annual cost 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. There is also an aesthetic value between open and closed drains, but this may be difficult to establish.

In general, closed drains should only be used to collect and remove ground water, but in certain special instances they may have to be used to carry storm water or excess irrigation water as well. The disadvantages of discharging surface water into a closed drain are that (1) in the small sizes closed drains are easily clogged with debris and (2) the sizes must be large and costly to adequately carry storm flow.

.2 Pipe for Drains. Closed drains consist of buried pipe with openings 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 or concrete but can be plastic or any other material that will perform the function and resist deterioration.

Some types of pipe are manufactured with holes or some similar special provision for water entry, but these are usually too expensive for general use. Ordinary clay and concrete drainpipe is laid with 1/8-inch openings between pipe lengths and water enters the line through

these openings. When drain lines are laid under canals, and sometimes under railroads or highways, the opening is closed and sealed by one of the methods used in laying sewer pipe to prevent possible piping into the drain line and resultant damage to the overlying structure.

Drainpipe is manufactured with plain, tongue-and-groove, or bell-and-spigot ends. With the latter two types, adjoining sections interlock and so are easier to lay and hold to grade and alinement than sections with plain ends. However, in the tongue-and-groove type of construction, the openings between sections can be dangerously reduced or even completely eliminated when the sections are laid. Since this defeats the drainage objective, tongue-and-groove ends are not desirable.

.3 Pipe Specifications. Unreinforced concrete pipe specifications for closed drains may be either ASTM C14, C412, C118, or C444, latest revisions. In addition to the requirements of these specifications, the following requirements must be met:

A. A minimum of 7-1/2 sacks of Type V cement per cubic yard of concrete.

B. A minimum of 72 hours steam curing between 110° F and 140° F, or 7 days moisture curing, with the entire concrete surface continuously moistened during the period of either type of curing.

C. Maximum concrete absorption of 6.5 percent--5-hour boiling test.

D. Pipe shall be air dried for not less than 30 days prior to placing in ground.

E. Calcium chloride shall not be used in the concrete for concrete pipe.

These additional requirements are considered to be necessary to produce pipe which will have a long life when placed in the ground in continuous contact with water. When concrete pipe is used for manholes or when reinforced concrete pipe is used under railroads or in other special locations, the concrete is to be made with Type V cement.

Clay pipe specifications for closed drains may be either ASTM C4, C13, or C200.

.4 Outlets. Deep open drains or natural drainageways normally provide the outlets for closed drain systems; however, it is sometimes necessary to discharge closed drains into a sump and dispose of the drainage water by pumping into shallow surface drains. A thorough study of outlet conditions and requirements is an important consideration in planning a closed drainage system which will function satisfactorily.

.5 Depth of Closed Drains. The depth of closed drains is a major consideration, since the success or failure of the entire drainage system may depend upon this factor. The depth will usually be dependent 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. Since the primary function of a drain line is to collect and remove underground water, the pipe should be placed, if possible, in a relatively coarse-textured stratum. If the drain line must be placed in a stratum of low permeability for short reaches, it is doubly important that the gravel envelope be of full thickness and of proper gradation to encourage entrance of

drainage water into the pipe. (See Paragraph 526.2.7.) Generally, 6 feet is the minimum depth at which the pipe should be placed and 8 feet or deeper is preferable.

.6 Grade and Alinement. The proper installation and functioning of a drain line requires rigid control of grades and alinement. The minimum grade for a closed drain line should be 1 foot of fall per 1,000 feet of line. Steeper grades are desirable. The steeper the grade the less likelihood of silt clogging the drain and the less exacting is the control required during installation. See Paragraph 526.2.10 for further discussion of alinement.

.7 Gravel Envelope. Since closed drains may be located in all kinds of material, it is good practice to lay the pipe in a gravel envelope. Such an envelope is used to stabilize the base material and to provide a permeable path for water to move into the pipe openings from the base material. In addition to the gravel envelope, the joints between plain-end pipe sections should be covered at the top and along the sides with asphalt building paper to prevent the finer particles of the envelope material from falling through the gap openings under the action of gravity. A gravel envelope less than 4 inches thick around the pipe probably would be sufficient, but the physical difficulty of placing the envelope material uniformly to a small thickness makes it more economic to specify a 4-inch thickness.

The best gravel envelope for agricultural drains should be fairly well graded between the coarsest and finest particles. It should not contain material larger than 1-1/2 inches in diameter because larger

sizes might crack the pipe during backfilling operations, and it should not contain significant quantities of material smaller than the No. 30 mesh screen because the smaller sizes tend to become self-clogging. The requirement for grading is not of great importance but, if two sources are available and other things are equal, the source with the better graded material should be selected. Figure 1 shows the excavation yardage for various widths and depths of trenches, and the gravel envelope yardage for various trench widths and pipe sizes.

.8 Gap Width, Length of Pipe Sections, and Permeability of Gravel Envelope. In designing a closed drain, it is assumed that the pipe will accept the drainage water when it arrives at the drain line and carry it away without a buildup of pressure within the pipe. Unless these assumptions are met, the lines will not function as they should, and the land will not be effectively drained. To meet the first assumption requires consideration of the relationship among the permeability of the gravel envelope, the length of pipe sections, and the gap between sections; and to meet the second requires the pipe size and slope to be sufficient to carry the water away after it enters the pipe. Design for the second assumption is explained in Paragraph 526.2.14.

The theoretical relationship among rate of flow, permeability of the gravel envelope, and the head lost during convergence of flow to the gap openings between lengths of pipe has been worked out by W. T. Moody 1/ of the Bureau of Reclamation. The relationship is valid for

1/"Effect of Gap Width on Flow into Draintile," a memorandum by W. T. Moody, June 7, 1960.

all conditions of the closed drain line from empty to flowing full, but is not valid if the line is under pressure. Moody concluded that increasing gap width is a very ineffective way of increasing rate of inflow. Examination of his results shows that increasing the permeability of the gravel envelope is more effective in increasing the rate of inflow.

The curves and equations on Figure 2 provide a means of computing the relations for any particular set of conditions. The curves in Figures 3 through 9 were computed on the basis of certain conditions and are recommended for use in design. There is one figure for each pipe diameter and a curve for each pipe length. The figures are plotted with the design inflow to the pipeline against the permeability of the gravel envelope. The conditions are that the gap opening is one-eighth inch (which is considered ample to pass the required amount of water and small enough to prevent any significant amount of gravel from moving through the crack opening); that the value of H in Figure 2 is equal to b + nb which means that the free water surface is at the top of the gravel envelope (which is considered to be the maximum head allowable for entry of water into the pipe); and that the gravel envelope is 4 inches thick (which means that nb = 4 inches). One curve is shown on Figure 3 that was computed for a 1/4-inch gap opening. This shows the relative value of doubling the gap opening and points up the conclusion that this is an ineffective way of increasing the rate of inflow.

Use of the curves in design can be made in several ways. The rate of design inflow will always be known before using these curves, so it will be a starting point. Then if a certain length of pipe is more readily available than others, the minimum required permeability of gravel envelope can be ascertained. Or if the gravel to be used is known, and its permeability determined, the maximum permissible pipe length can be found. When the point representing the design inflow and the permeability of the gravel envelope is plotted on one of more figures, any condition represented by curves to the right of that point will meet the requirement. It should be noted that there are cases where a drain line is placed in a permeable base material stratum. In these cases, the base material should be tested to see if its permeability meets the requirements. If it does, there is no need to import gravel envelope material since the excavated material will serv the purpose.

As an example, assume that a 4-inch closed drain is to be installed where the design inflow is 0.00014 cfs per foot or about 0.74 cfs per mile. A sample of sand and gravel material from a pit in the vicinity of the work has a size range from No. 200 screen to 3-inch screen. The sizes above 2-inch must be removed for construction use because they might crack the pipe when they are placed during installation, so they should also be removed before making a permeability test of the material. The permeability of the disturbed material, made in the laboratory on a representative sample of size 2-inch and below, is 32 inches per hour.

On Figure 3 for 4-inch pipe, it will be noted that a 2-foot pipe length with a 4-inch gravel envelope will meet the requirement, while on Figure 4 for 6-inch pipe, a 3-foot pipe length, or shorter, with a 4-inch gravel envelope will also meet the requirement.

If, in the above example, the permeability of the pit-run gravel envelope material had been only 20 inches per hour, it would be noted on Figure 3 for 4-inch pipe that a pipe length of less than 2 feet would be required. A pipe diameter of 6 inches and a 2-foot pipe length would be satisfactory as shown on Figure 4. Since short pipe lengths are difficult to procure for some plants, except at an increase in cost, it would be possible to use 6-inch pipe to satisfy the design requirements. However, there is another possibility which should be investigated since it would also meet the requirements and the cost might be less than using 6-inch pipe. That possibility is to increase the permeability of the gravel envelope material by screening out some of the finer fractions. If the permeability can be increased to 24 inches per hour, a 2-foot length of 4-inch pipe will meet the requirements. The permeability of the pit-run gravel envelope material should be checked in the laboratory with successive fine sizes screened out to determine the sizes that must be eliminated to provide the required permeability, and the cost of screening balanced against the difference in installed cost of the 4- and 6-inch pipe plus the cost of the additional envelope material required with the 6-inch pipe.

It can be seen that there are many possible combinations of pipe diameter and length and gravel envelope permeability that will satisfy the inflow requirements. All reasonable possibilities should be investigated to determine the most satisfactory and least expensive combination.

Assuming, in the above example, that the excavation and backfill quantities are the same for both 4-inch and 6-inch pipe, the following cost comparison can be made using contract costs:

For furnishing and laying 6-inch standard strength pipe, per foot--$0.65

For furnishing and laying 4-inch standard strength pipe, per foot--$0.48

For furnishing and placing graded filter material around open jointed pipe drains, per cubic yard--$4.25

From Figure 1, the quantity of gravel envelope material per foot is:

For 6-inch pipe--0.049 cubic yard

For 4-inch pipe--0.038 cubic yard

For 100 feet of drain line, the cost would be:

4-inch pipe =	$ 48.00	6-inch pipe =	$ 65.00
Gravel =	16.15	Gravel	20.83
	$ 64.15		$ 85.83

Some amount up to the difference of $21.68 could be spent in either improving the permeability of the gravel envelope material or to pay for shorter lengths of 4-inch pipe. For the difference of 1.1 cubic yards of gravel required per 100 feet of drain line, as much as

$19.71 per cubic yard could be spent to improve its permeability, or $0.21 per foot more could be paid for shorter lengths of 4-inch pipe before equalling the cost of installing 6-inch pipe. Since the allowable expenditures are obviously larger than would be required to provide the alternate solution, provision should be made to use the smaller diameter pipe.

.9 Stability of Drainpipe Bed. The ideal, and highly desirable, condition for installing drainpipe is to have a dry, stable trench in which the pipe is laid. Most of the time this is not feasible, because the need for the drain does not become apparent until after the groundwater table has risen higher than the bottom of the prospective drain. Many times, saturation makes the material so unstable that the drainpipe cannot be laid without special measures to insure maintenance of line and grade. It is important that stability be obtained, because, if a single pipe gets badly out of line or grade, the entire system upgrade from this pipe will lose its effectiveness.

The most economical and simplest method of stabilization is to add gravel to the subgrade. This may require overexcavation in some cases, while in others the coarse gravel will work itself down into the fine, unstable material. Usually, excavation to the depth required for placement of the gravel envelope material will be sufficient. Any sand and gravel mixture, including the gravel envelope material, is suitable for stabilizing material.

If, for some reason, the addition of gravel will not stabilize the bed of the trench, it may be necessary to use "cradles" consisting

of wooden stringers on which the pipe is laid. Under extreme conditions, the cradles may have to be supported on piles or long stakes driven to a supporting zone. Figures 10 and 11 show standard designs of wooden cradles for plain-end or tongue-and-groove and bell-end pipe.

.10 Manholes. Manholes should be placed at the junction of two or more drainpipe lines, and at convenient intervals of about 1,320 feet on tangents. They should also be placed at all changes in alinement, but the requirement is not essential in special circumstances. Where alinement changes would require several manholes at close intervals, such as where the line must circle a topographic knoll, the line can either be laid on a curve or standard pipe bends can be used at appropriate intervals. This procedure is permissible only where there is reasonable certainty that operation and maintenance problems will not be increased and that the position of the line can be firmly established for future location. Reductions in pipe diameter should be made at a manhole if convenient, but can be made along the line by use of a standard increaser. A manhole is not required at grade changes.

Manholes should extend about 2 feet above the natural ground surface to make them easier to find in the field, and should be placed in fence rows or other out-of-the-way places if at all possible. Neither a manhole nor a cleanout is required at the upper end of the line, but this end must be plugged. The location of the plugged end should be recorded both in field books and on as-built drawings.

General practice has been to provide a drop within all manholes between the invert elevation of the influent and effluent pipes. The

purpose of the drop is to compensate for the head losses within the manhole. This practice is satisfactory but is not necessary and sometimes creates an expensive and critical situation in level country where grade has to be obtained by increasing depth. A more rational method is to place the top of the inlet and outlet pipes at the same elevation because (1) 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; or (2) if design data do not require a size change at the manhole, neither pipe will be at capacity and the slight head loss requirement will be available in the unused capacity.

The base of the manhole should be about 18 inches below the bottom of the effluent pipe to form a trap to catch material which may enter the line. The last job on completion of construction of a new drain should be to clean out all traps and set the manhole covers. Traps should also be cleaned periodically as a maintenance item.

Figure 12 shows a standard design for a manhole made with concrete pipe. Manholes may also be constructed of asphalt-dipped or asbestos-bonded corrugated-metal pipe (CMP) where a saving in cost can be realized, and where salinity of the soil and water is low.

.11 Surface Inlets. In general, it is not considered good practice to admit surface water into a closed drain and this should be avoided if at all possible. In some cases, however, it may be necessary to dispose of small amounts of surface water in this manner.

The topography may be such that the open ditch can discharge directly into the closed drain, but more often the ditch will discharge

into the top of a buried 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 is installation of a trashrack which will prevent entry of large rocks, russian thistle, etc. The trashracks should be self-cleaning if possible.

.12 Outlet Structures. The outlet end of a closed drain, if not properly protected, will be undercut by the action of the discharging water. If allowed to continue, this undercutting will cause the short lengths of drainpipe to shift out of proper grade and alinement and create an expensive maintenance problem if not completely block the outlet of the drain. Protection against this undercutting and tipping of pipe sections should be provided by placing a long length (12 to 16 feet) of heavy gage, asphalt-dipped or asbestos-bonded CMP or asbestos-cement pipe at the outlet end of closed drains. (Figure 3 of Chapter 526.1 shows a closed drain outlet.)

.13 Strength of Drainpipe. Since closed drains in irrigated areas are usually placed at a considerable depth below the ground surface, the ability of the pipe to carry the load of the backfill is an important consideration. Both concrete and clay pipe are made in different strengths, so designs for the proper strength pipe are not only necessary to insure the permanence of the line but also to permit use of the most economical pipe.

Figure 13 gives the loads of drainpipe per linear foot caused by backfilling with various materials and for varying depths and widths of trenches. The loads given are not exact because they will vary

slightly with the diameter of the pipe, but they are within the limits of accuracy of the other items that affect the load and are satisfactory for use in design. They are based on the Marston formula which can be used if it is considered that greater accuracy is needed. Note that the trench widths given are measured at the top of the pipe. This value is used whether the trench sides are vertical or sloping. A nomograph for solving the Marston formula is given in Figure 14.

Figure 15 gives the allowable crushing strength of various pipe that is laid in a gravel envelope. If not laid in a gravel envelope, only 75 percent of these values is to be used as the allowable crushing strength. The tabular values assume a Class C bedding will be attained with a gravel envelope, while a Class D bedding is assumed without a gravel envelope. If a different class of bedding is provided, the tabular values can be revised accordingly.

The following procedure can be used to determine the strength of pipe that is required for a particular installation:

A. Knowing the unit weight of soil, depth of trench, and width of trench at the top of the pipe, use Figure 13 or 14 to determine the load per linear foot to be expected on the pipe.

B. Knowing the diameter and type of pipe to be used, use Figure 15 to determine the quality of pipe required to support the load to be imposed.

As an example, assume preliminary design shows that a 10-inch pipe is required and that the backfill over the pipe will be 8.5 feet. For a 10-inch pipe with 4-inch gravel envelope, a 24-inch width ditch

should be satisfactory, but this ground is not expected to be stable so a ditch width at the top of the pipe of 2 feet 6 inches is provided for. The backfill material will be saturated topsoil weighing 110 pounds per cubic foot.

From Figure 13:

8-foot cover	1,295 lb/lin ft
9-foot cover	1,380 lb/lin ft
so 8.5-foot cover	1,338 lb/lin ft
$1{,}338 \times \frac{110}{100} =$	1,472 lb/lin ft

From Figure 15 it is found that the allowable crushing strength of any of the conduit listed except standard clay or concrete draintile will exceed the required strength.

.14 Size of Pipe. Using the formula for ground-water accretion given in Paragraph 525.0.10, the drain line is designed to run full. No pipe less than 4 inches in inside diameter is permitted, and this size may be used only in the upper reaches of a line that cannot possibly have a future requirement for extensions or branches, and only if the line is enclosed in a gravel blanket; otherwise, a 6-inch diameter is the minimum size that should be used.

Pipe sizes are determined from calculations involving the required discharge and the hydraulic gradient of the drain line. Using the discharge decided upon and knowing the gradient of the line, pipe size can be determined from the curve shown in Figure 16. This curve is based on Manning's formula (see Paragraph 526.1.2) using n = 0.015. Figure 17 shows a typical profile as well as sample design data for a closed drain.

.15 Capacity of Closed Drains. The capacity of closed drains ordinarily need be sufficient to carry ground-water accretion only. Closed collector and outlet drains must, of course, also carry the flows delivered to them by higher closed drains. In the rare case where open drains discharge into closed drains, the closed drains must also have capacity for the surface and subsurface flows carried by the open drain. In studies involving capacities, areas, and velocities, the information in Figure 18 is useful for closed drains flowing full and for comparison when the drain is flowing partially full.

DRAIN TRENCH YARDAGE
FOR VARIOUS DEPTHS AND WIDTHS

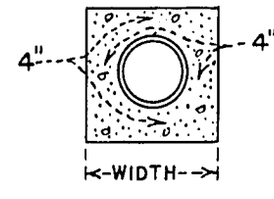
DEPTH (feet)	CUBIC YARDS PER LIN. FT. FOR WIDTH			
	24"	27"	30"	36"
0.1	.0074	.0083	.0093	.0111
0.2	.0148	.0166	.0185	.0222
0.3	.0222	.0250	.0278	.0333
0.4	.0296	.0333	.0370	.0444
0.5	.0370	.0416	.0463	.0555
0.6	.0444	.0500	.0556	.0666
0.7	.0518	.0583	.0648	.0777
0.8	.0592	.0666	.0741	.0888
0.9	.0666	.0750	.0833	.0999
1.0	.074	.083	.093	.111
2.0	.148	.167	.185	.222
3.0	.222	.250	.278	.333
4.0	.296	.333	.370	.444
5.0	.370	.417	.463	.556
6.0	.444	.500	.556	.667
7.0	.518	.583	.648	.778
8.0	.592	.666	.741	.889
9.0	.666	.750	.833	1.000
10.0	.740	.833	.926	1.111
11.0	.814	.916	1.019	1.222
12.0	.888	1.000	1.111	1.333
13.0	.962	1.083	1.204	1.444
14.0	1.036	1.166	1.296	1.556
15.0	1.110	1.249	1.389	1.667

GRAVEL ENVELOPE YARDAGE*

CUBIC YARDS PER LINEAR FOOT FOR VARIOUS PIPE SIZES								
4"	6"	8"	10"	12"	15"	18"	21"	24"
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



CLOSED DRAIN YARDAGES

103-D-684

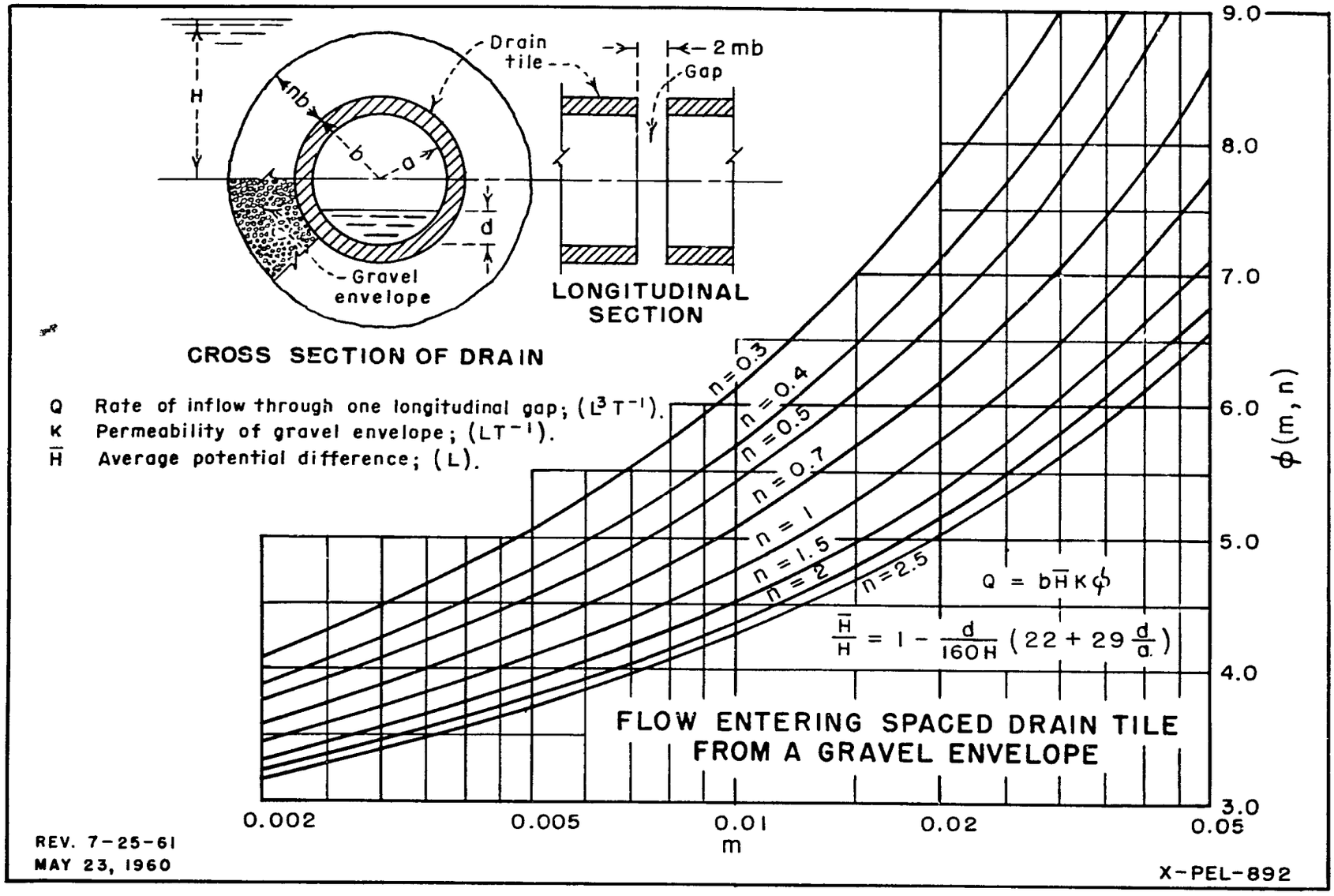
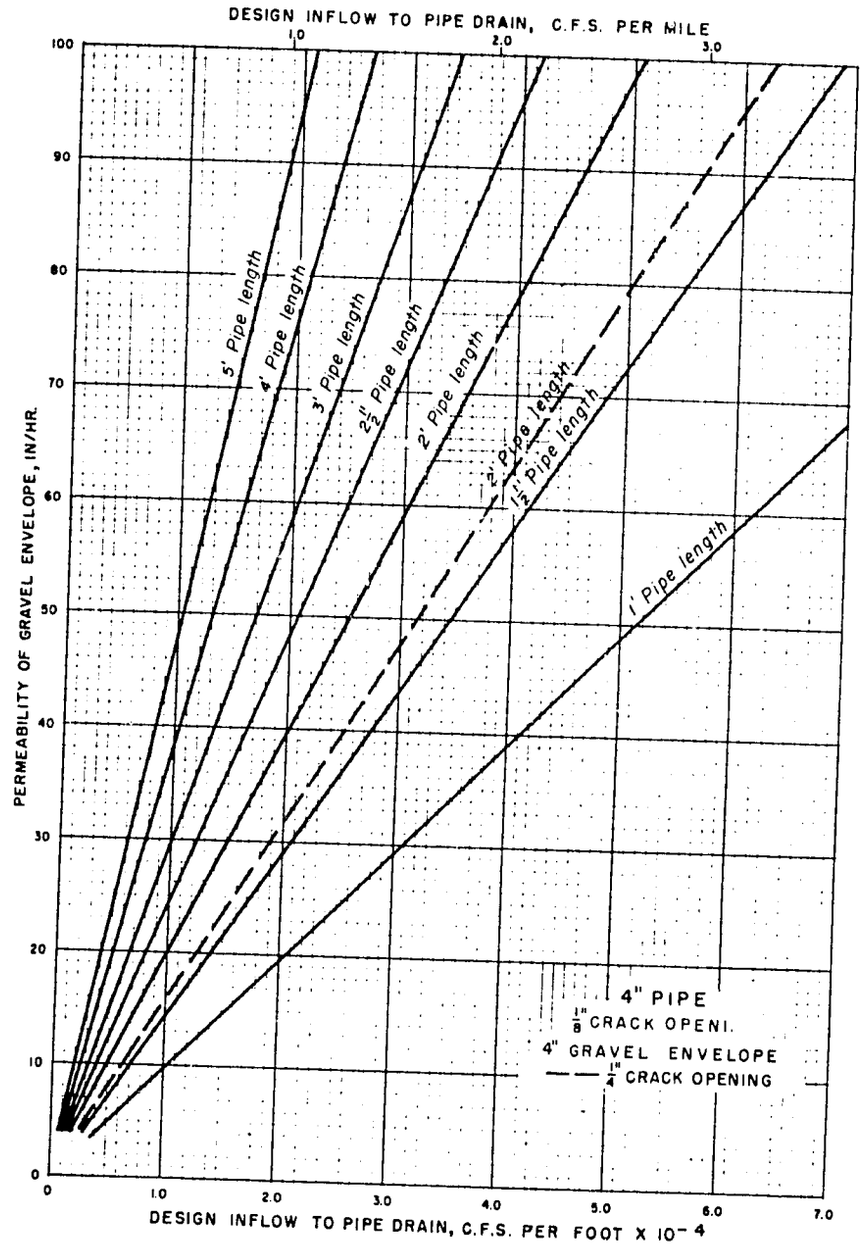


Figure 2
Par. 526.2.8

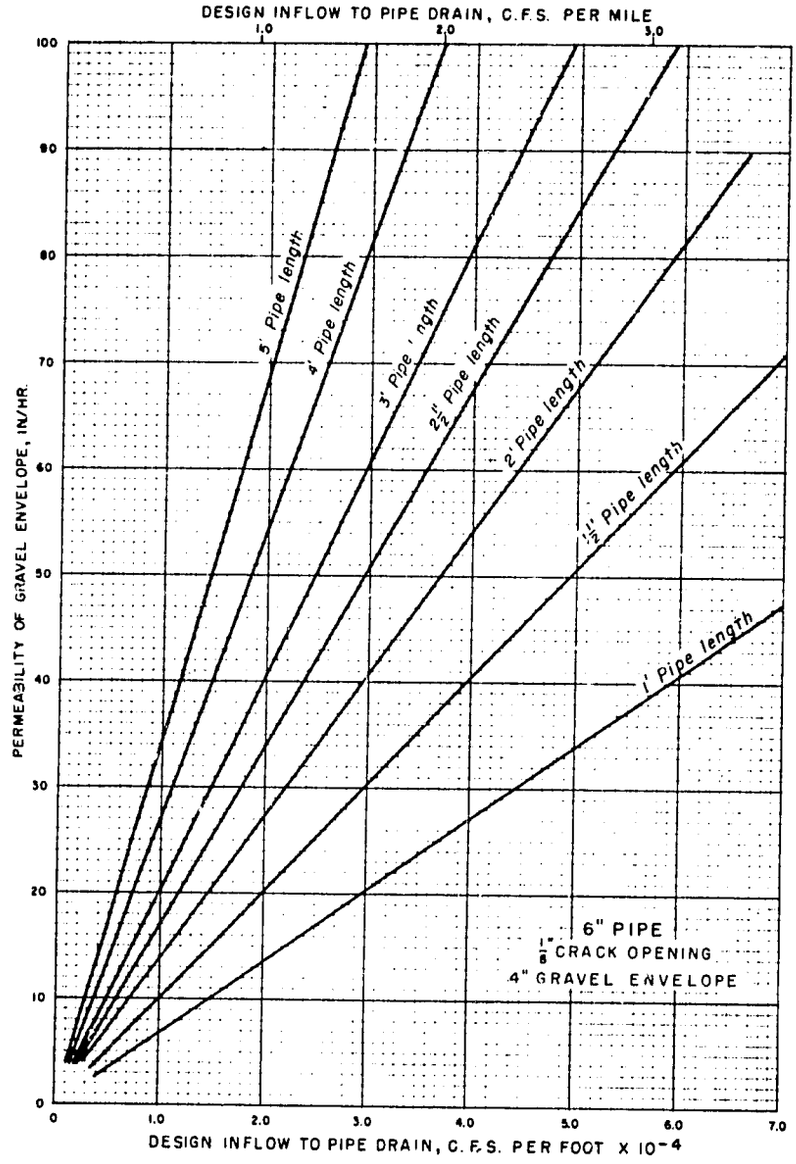
Figure 3
Par. 526.2.8



PIPE - GRAVEL ENVELOPE RELATIONSHIP FOR AGRICULTURAL DRAINS

103-D-768

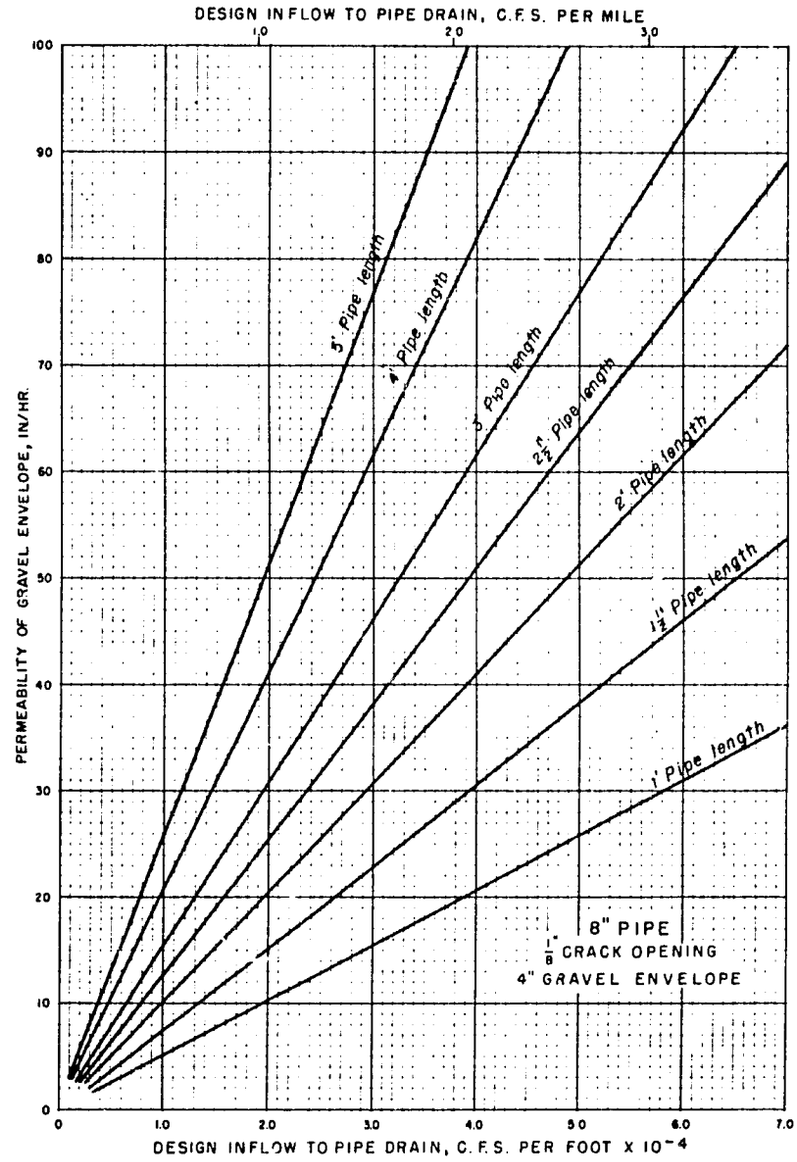
Figure 4
Par. 526.2.8



PIPE - GRAVEL ENVELOPE RELATIONSHIP FOR AGRICULTURAL DRAINS

103-D-769

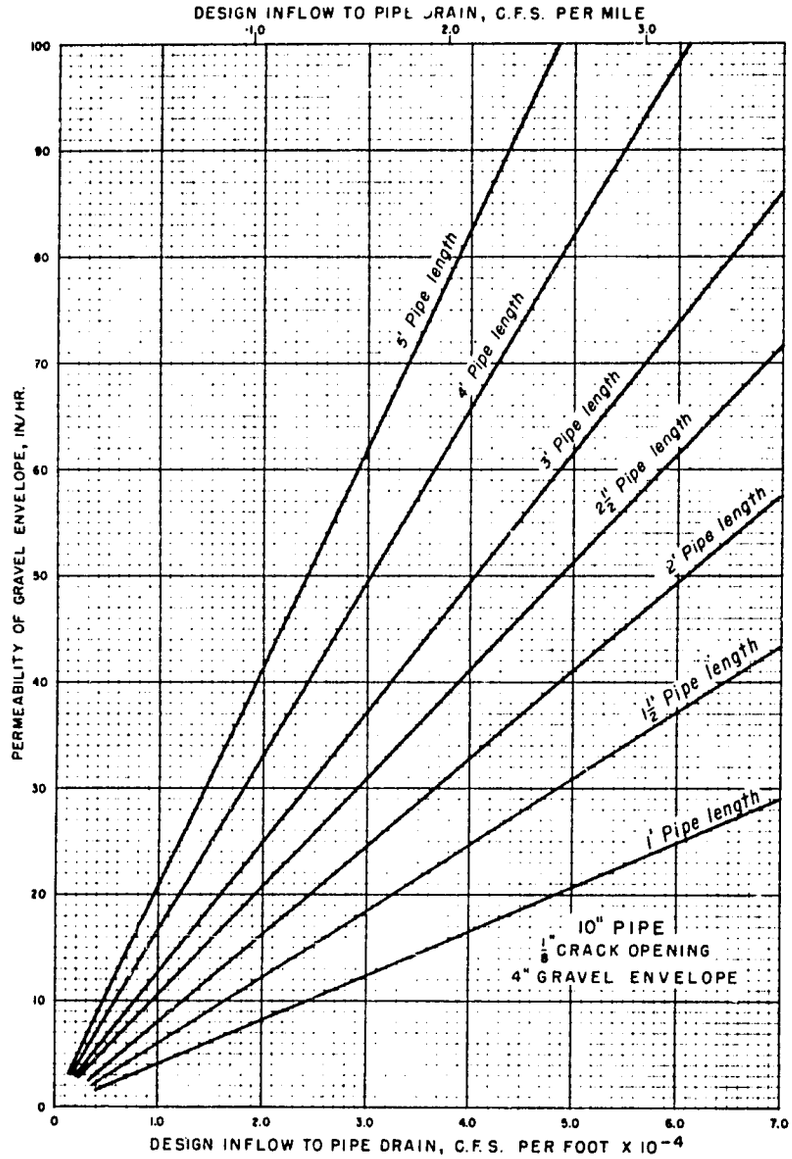
Figure 5
Par. 526.2.8



PIPE - GRAVEL ENVELOPE RELATIONSHIP FOR AGRICULTURAL DRAINS

103-D-770

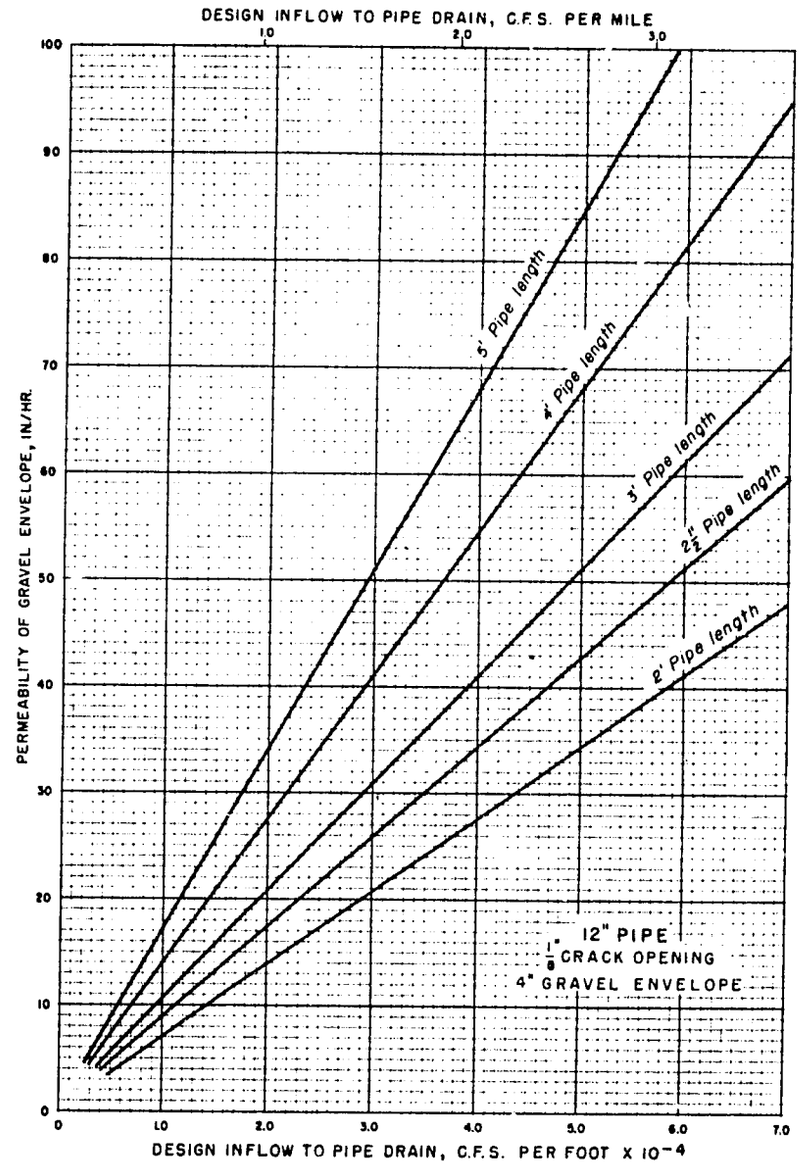
Figure 6
Par. 526.2.8



PIPE - GRAVEL ENVELOPE RELATIONSHIP
AGRICULTURAL DRAINS

103-D-771

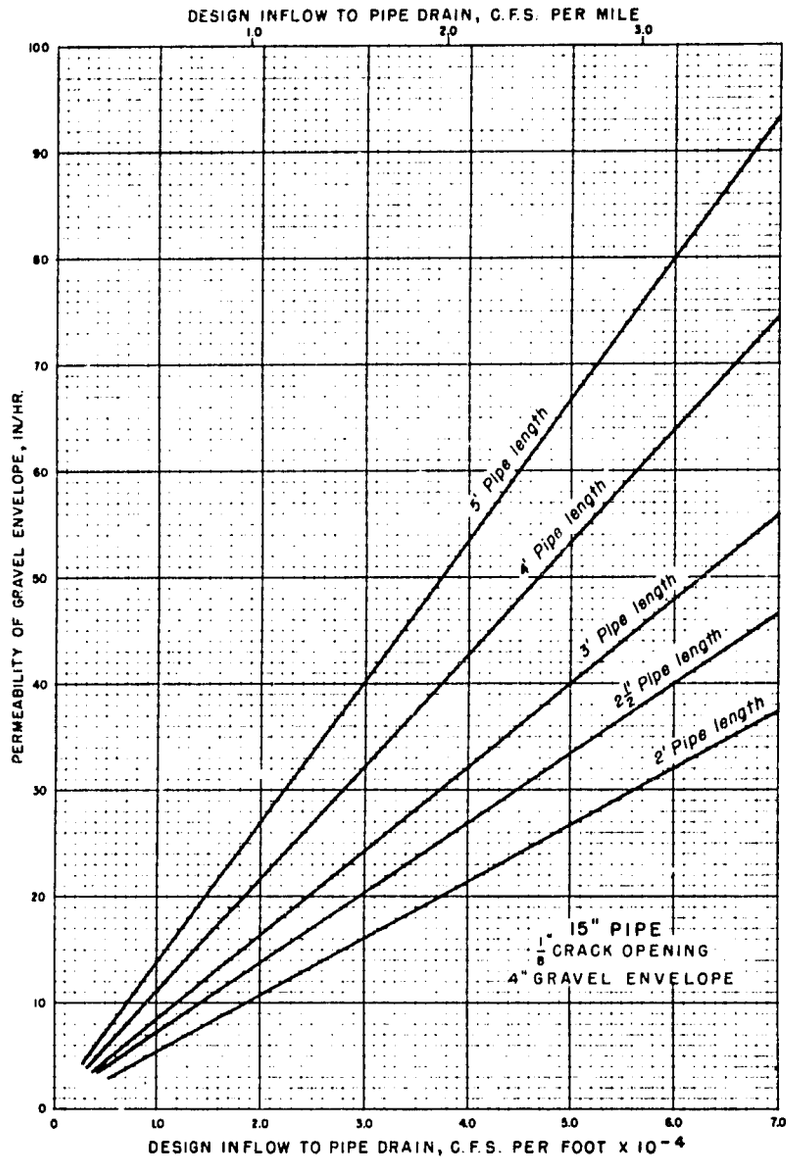
Figure 7
Par. 526.2.8



PIPE - GRAVEL ENVELOPE RELATIONSHIP FOR AGRICULTURAL DRAINS

103-D-772

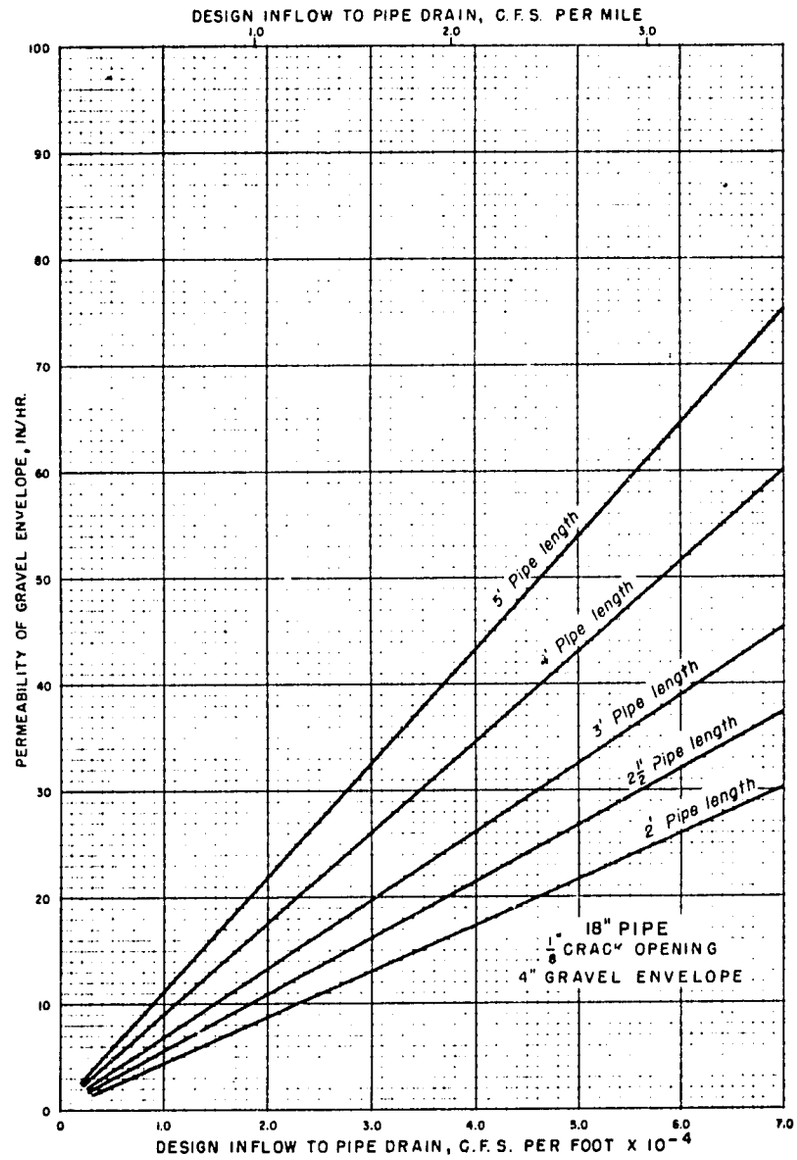
Figure 8
Par. 526.2.8



PIPE - GRAVEL ENVELOPE RELATIONSHIP FOR AGRICULTURAL DRAINS

103-D-773

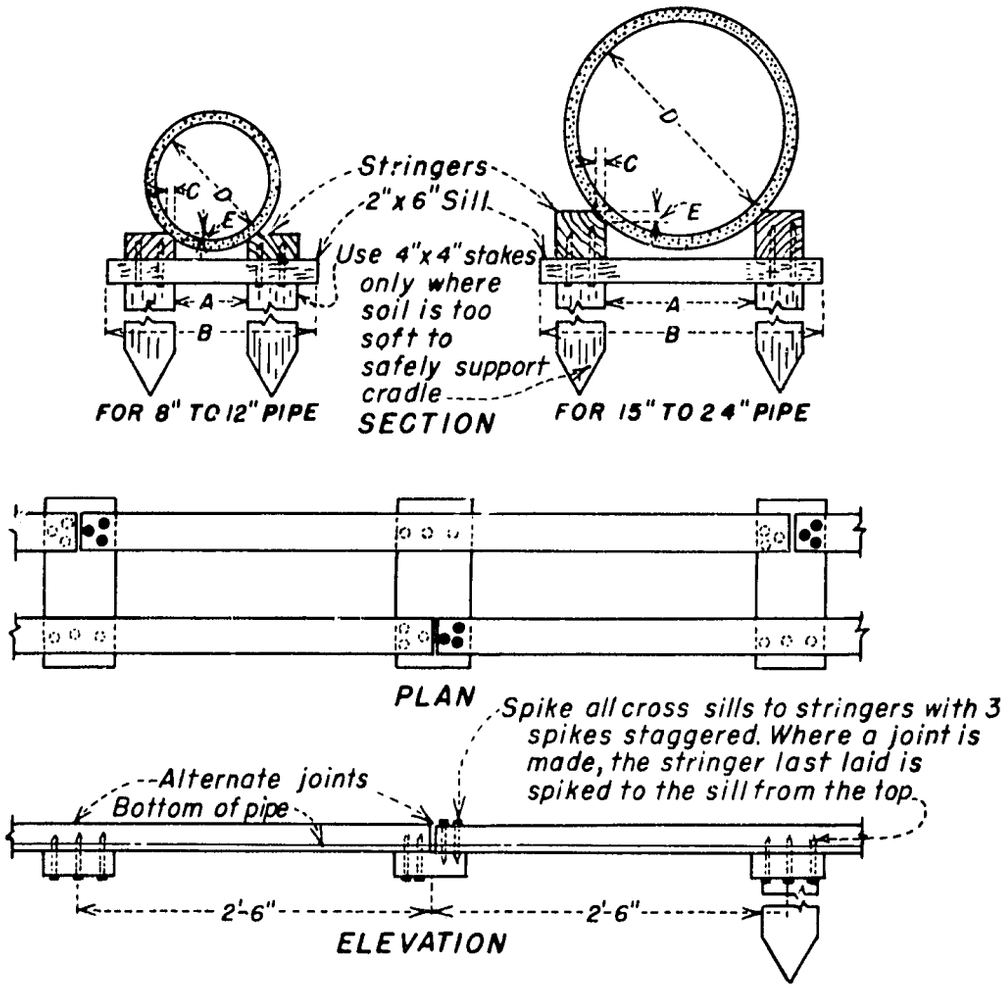
Figure 9
Par. 526.2.8



PIPE - GRAVEL ENVELOPE RELATIONSHIP FOR AGRICULTURAL DRAINS

103-D-774

Figure 10
Par. 526.2.9

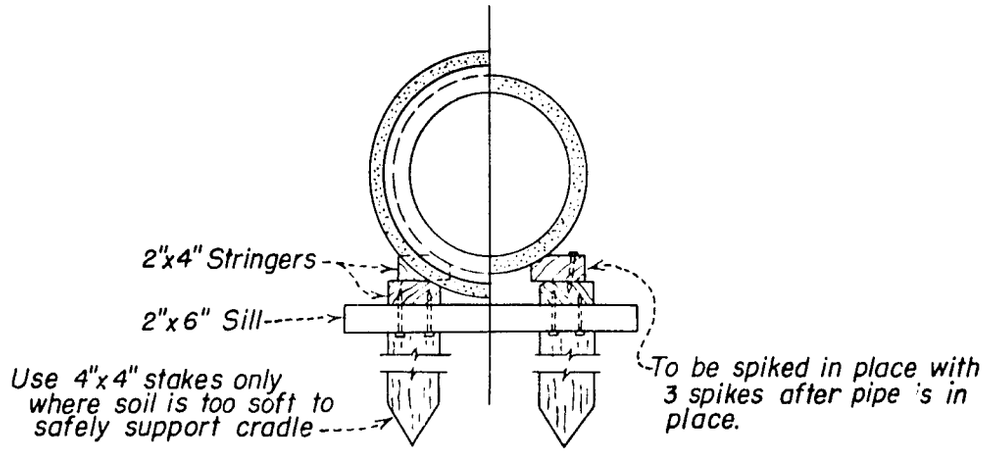


DIMENSION TABLE

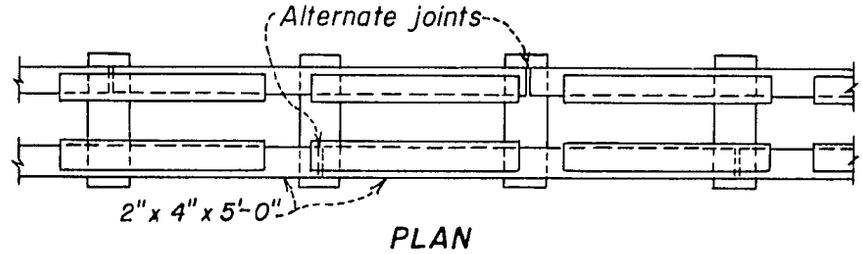
SIZE OF PIPE D	SIZE OF PIPE D	STRINGERS-ROUGH				STRINGER-S1S			
		A	B	C	E	A	B	C	E
8"	2"x4"	$5\frac{1}{4}$"	16"	$\frac{3}{4}$"	$\frac{3}{4}$"	4"	15"	$\frac{7}{8}$"	$\frac{1}{2}$"
10"	2"x4"	$6\frac{1}{4}$"	18"	$\frac{3}{4}$"	$\frac{5}{8}$"	5"	16"	$\frac{7}{8}$"	$\frac{1}{2}$"
12"	2"x4"	$6\frac{1}{4}$"	18"	1"	$\frac{5}{8}$"	$5\frac{1}{4}$"	16"	$1\frac{1}{8}$"	$\frac{5}{8}$"
15"	4"x4"	12"	24"	$\frac{7}{8}$"	1"	$11\frac{1}{4}$"	22"	$\frac{7}{8}$"	$\frac{7}{8}$"
18"	4"x4"	$13\frac{3}{8}$"	26"	$\frac{7}{8}$"	$\frac{7}{8}$"	$12\frac{5}{8}$"	24"	$\frac{7}{8}$"	$\frac{7}{8}$"
20"	4"x4"	14"	26"	$1\frac{1}{8}$"	1"	$13\frac{1}{8}$"	24"	$1\frac{1}{8}$"	1"
24"	4"x4"	$15\frac{5}{8}$"	28"	$1\frac{1}{8}$"	1"	$14\frac{3}{4}$"	26"	$1\frac{1}{8}$"	$\frac{7}{8}$"

TYPICAL WOODEN CRADLE FOR PLAIN END OR TONGUE AND GROOVE DRAIN PIPE

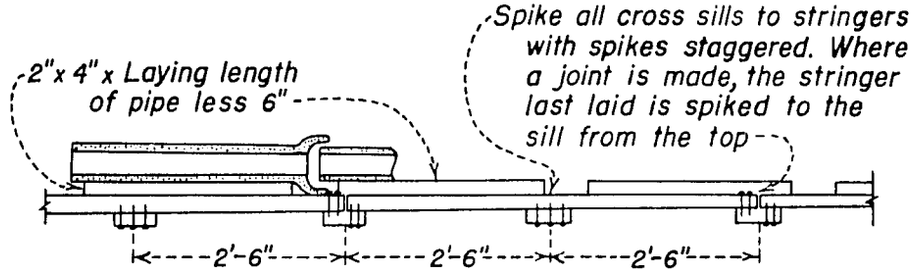
103-D-685



SECTION



PLAN

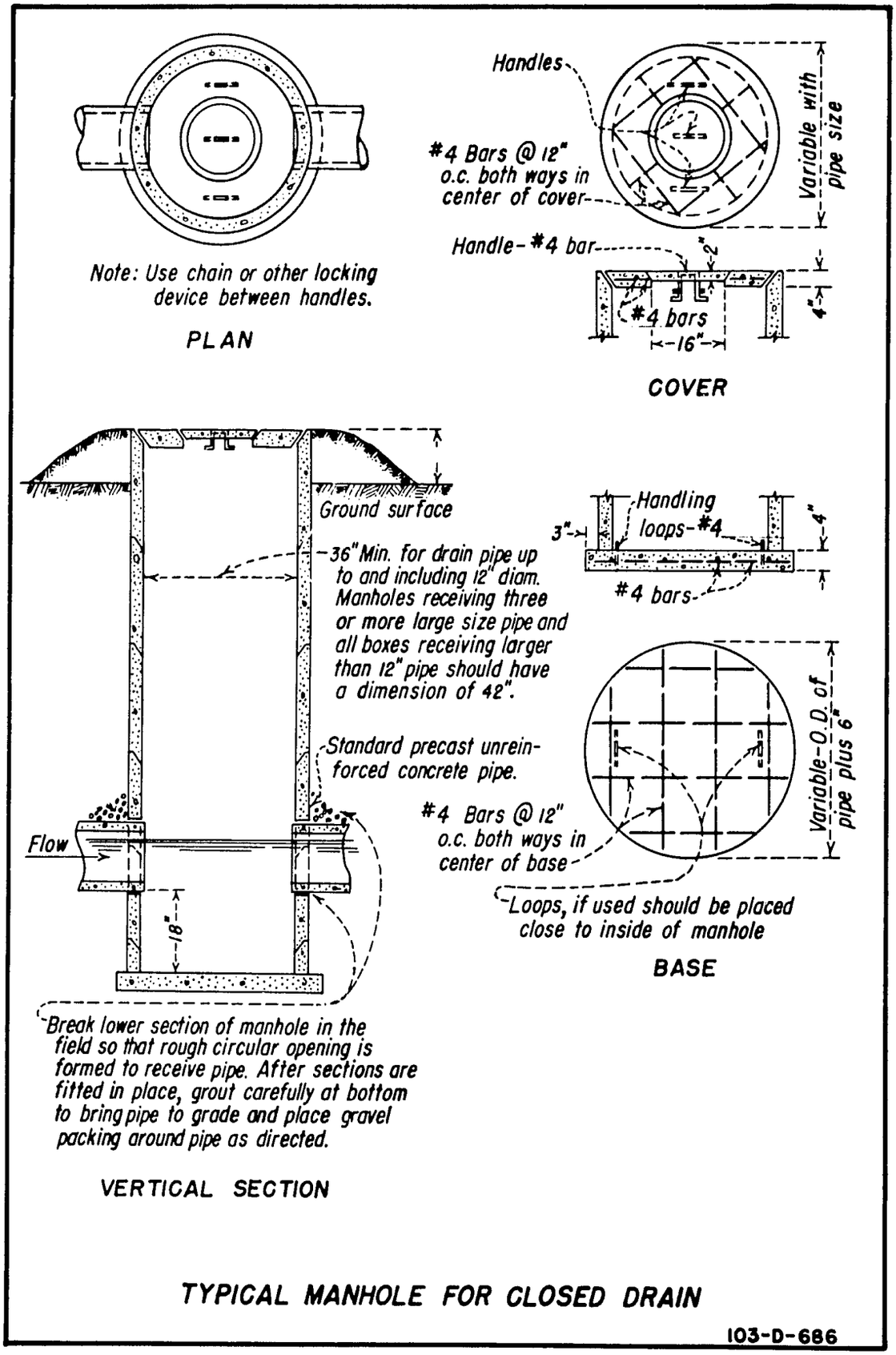


ELEVATION

TYPICAL WOODEN CRADLE FOR BELL AND SPIGOT PIPE

103-D-665

Figure 12
Par. 526.2.10



TYPICAL MANHOLE FOR CLOSED DRAIN

103-D-686

SATURATED TOPSOIL

WEIGHING 100 POUNDS PER CUBIC FOOT*

DEPTH OF BACKFILL OVER TOP OF PIPE (feet)	TRENCH WIDTH AT TOP OF PIPE (feet)								
	1'-6"	1'-9"	2'-0"	2'-3"	2'-6"	2'-9"	3'-0"	3'-6"	4'-0"
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
11	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

WEIGHING 100 POUNDS PER CUBIC FOOT*

DEPTH OF BACKFILL OVER TOP OF PIPE (feet)	TRENCH WIDTH AT TOP OF PIPE (feet)								
	1'-6"	1'-9"	2'-0"	2'-3"	2'-6"	2'-9"	3'-0"	3'-6"	4'-0"
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#/FT.^{3} MULTIPLY LOAD SHOWN BY 0.9. FOR BACKFILL WEIGHING 110#/FT.^{3} MULTIPLY LOAD SHOWN BY 1.1, ETC.

1/ Based on the Marston formula for loads in trenches:

$W = C w B^2$

where

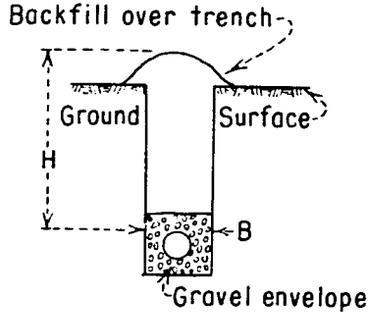
W = Load on pipe in #/ linear ft.

C = Coefficient of load on pipe.

w = Weight of fill in #/ft.^{3}

B = Width of ditch at top of pipe in feet.

H = Height of fill above top of pipe in feet



LOADS ON DRAIN PIPE PER LINEAR FOOT CAUSED BY BACKFILLING WITH VARIOUS MATERIALS 1/

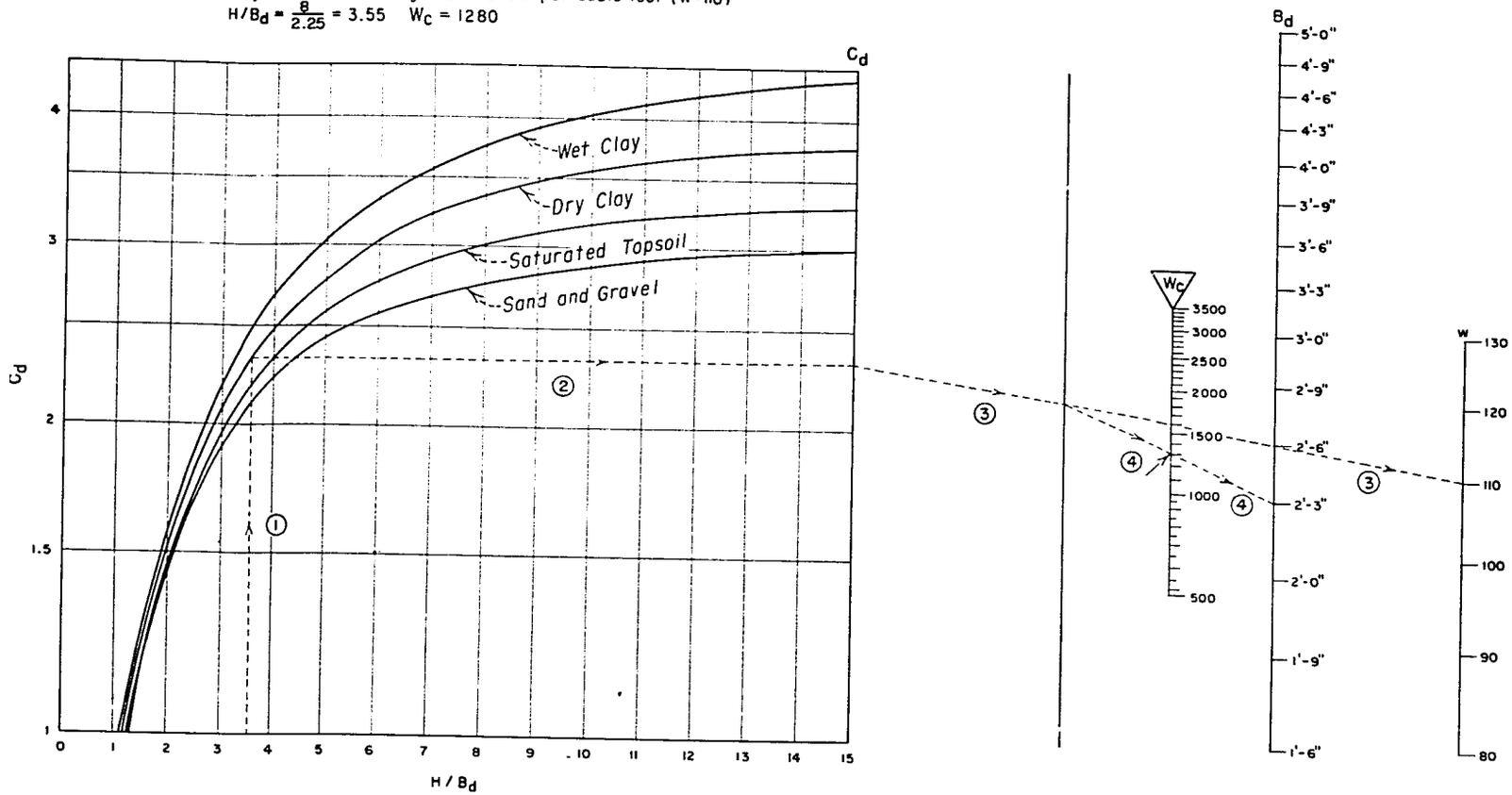
103-D-689

Figure 13
Par. 526.2.13

EXAMPLE

A drain pipe line is to be placed in a trench 2'-3" wide at the top of the pipe ($B_d = 2'\text{-}3''$) and covered 8' over the top of the pipe ($H = 8'$). The material is dry clay with a unit weight of 110 lbs. per cubic foot ($w = 110$)

$H/B_d = \frac{8}{2.25} = 3.55 \quad W_c = 1280$



EXPLANATION

$W_c = C_d \; w \; B_d^{\,2}$

where W_c = The vertical external load on a closed conduit due to fill materials, pounds per linear foot

C_d = Load coefficient depending upon the type of backfill materal

w = The unit weight of fill material, pounds per cubic foot

B_d = Horizontal breadth of ditch at top of pipe, feet.

also H = Height of fill over top of conduit, feet

LOADS ON PIPE IN TRENCHES

BASED ON MARSTON FORMULA

103-D-775

ALLOWABLE CRUSHING STRENGTH OF PIPE USED FOR DRAINS IN GRAVEL ENVELOPE

IN POUNDS PER LINEAR FOOT

	CLAY SEWER PIPE		CONC. SEWER PIPE [3]		CLAY DRAIN TILE [4]			CONC. DRAIN TILE [5]		CONC. PIPE FOR [7] IRRIG. & DRAINAGE
DIA.	STD. [1] STRENGTH	EXTRA [2] STRENGTH	STD. ** STRENGTH	EXTRA STRENGTH	STANDARD STRENGTH	EXTRA QUALITY	HEAVY DUTY	STANDARD QUALITY	EXTRA * QUALITY	
4	1500	—	1500	3000	1200	1650	2100	1200	1650	1800
5	—	—	—	—	1200	1650	2100	1200	1650	1875
6	1650	3000	1650	3000	1200	1650	2100	1200	1650	1950
8	1950	3000	1950	3000	1200	1650	2250	1200	1650	2025
10	2100	3000	2100	3000	1200	1650	2325	1200	1650	2100
12	2250	3375	2250	3375	1200	1650	2550	1200	1650	2250
14	—	—	—	—	1260	1650	2775	—	1650	2400
15	2625	4125	2620	4175	1305	1725	2970	—	1650	2475
16	—	—	—	—	—	1800	3150	—	1650	2550
18	3000	4950	3000	4950	—	1950	3510	—	1800	2700
20	—	—	—	—	—	—	—	—	1950	2775
21	3300	5775	3300	5775	—	2175	4020	—	2100	2850
24	3600	6600	3600	6000	—	2500	4500	—	—	3000
27	4125	7050	—	—	—	2700	5000	—	—	—
30	4800	7500	—	—	—	3000	5385	—	—	—
33	5250	8250	—	—	—	—	—	—	—	—
36	5850	9000	—	—	—	—	—	—	—	—

* Also special quality
** Also perforated concrete pipe [6]

CURRENT SPECIFICATION NO.

[1] ASTM C13-57T
[2] ASTM C200-57T
[3] ASTM C14-59
[4] ASTM C4-59T
[5] ASTM C412-58T
[6] ASTM C444-59T
[7] ASTM C118-59

NOTE:

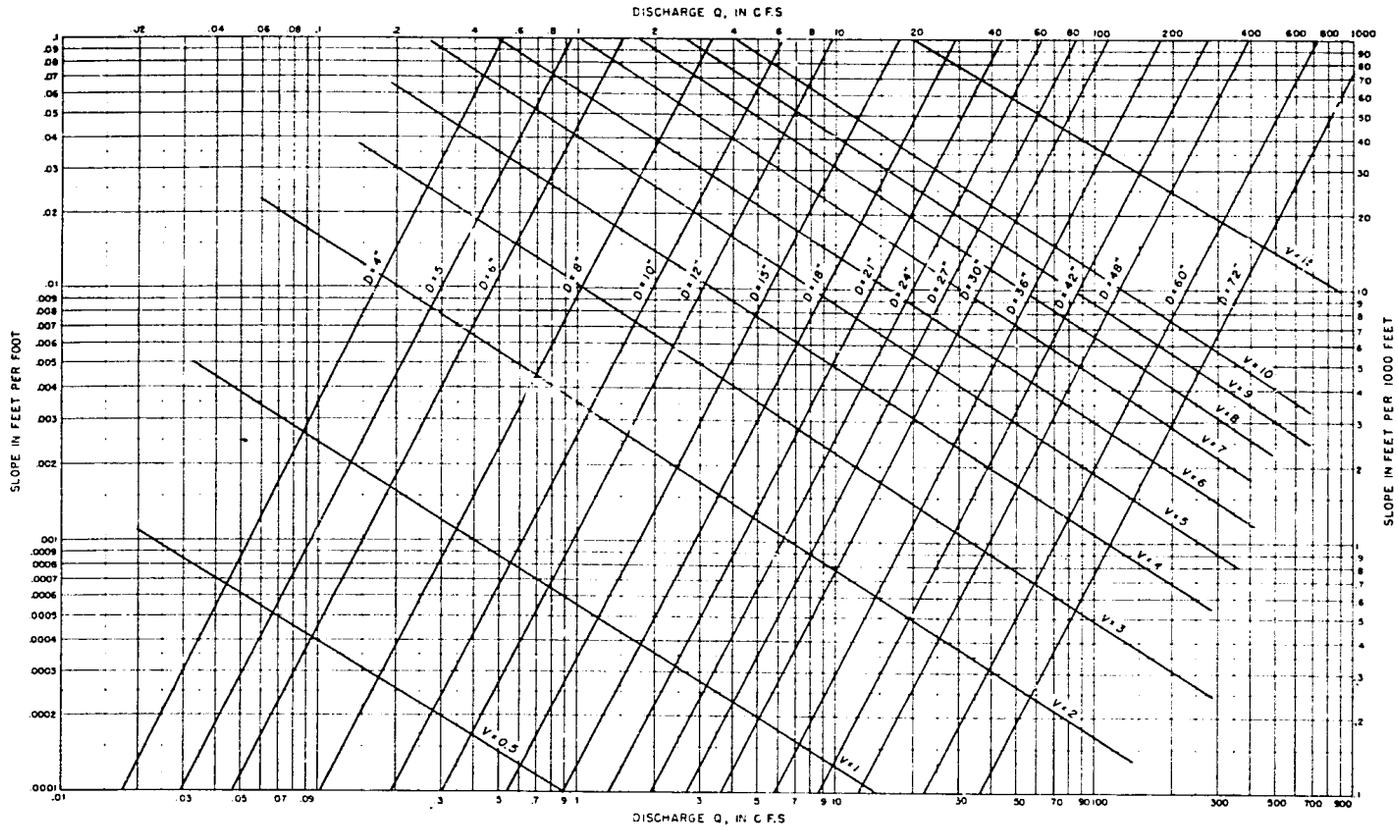
When the crushing strength of pipe listed will not meet an unusual load condition, reinforced concrete sewer or culvert pipe should be considered. See Federal Specification No. SS-P-371, Type II and ASTM C76-57T.
When concrete pipe is used, it should be manufactured with Type V cement.

Values are 150% of the three-edge bearing strength values.

103-D-778

Figure 15
Par. 526.2.13

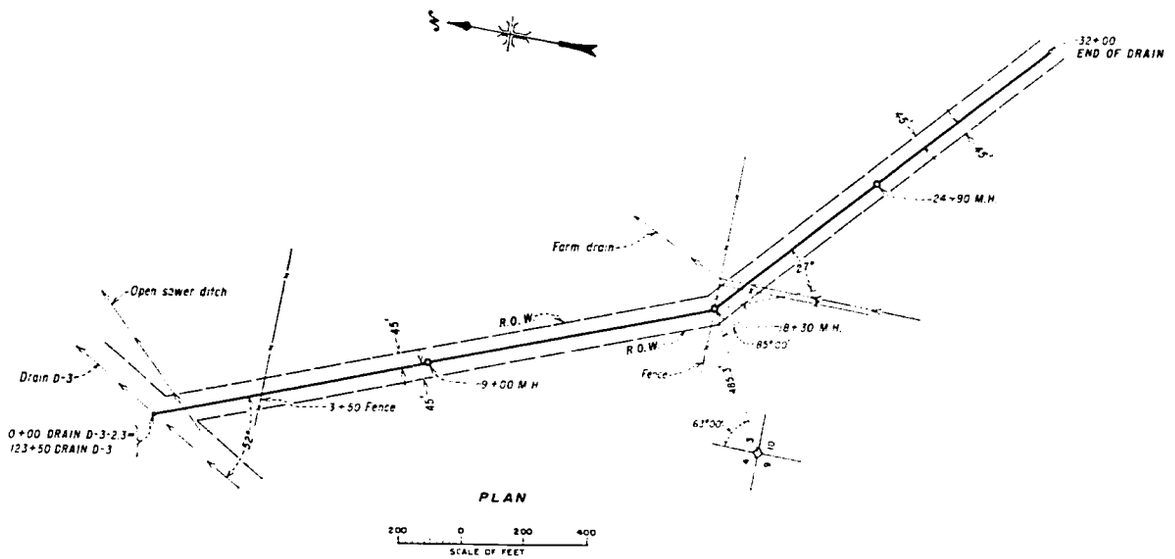
Figure 16
Par. 526.2.14

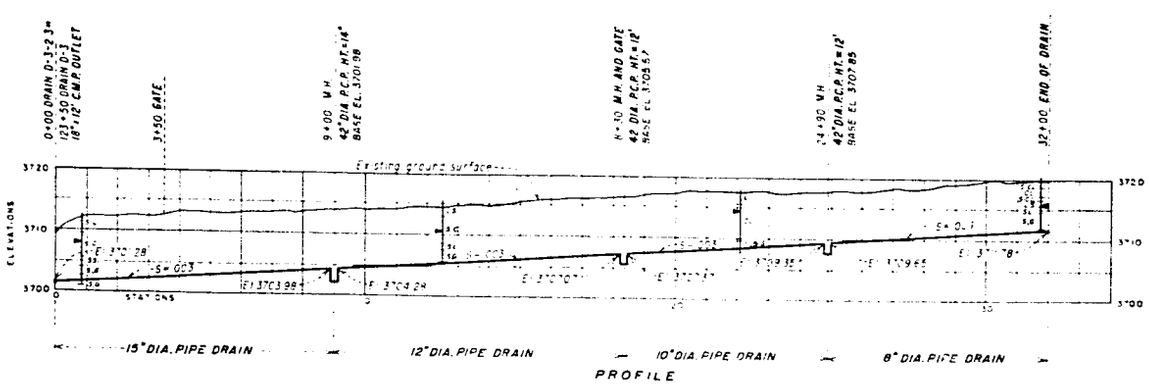


n = 0.015
v = velocity in feet per second
D = pipe diameter

FLOW IN DRAIN LINES

103-D-666





NOTE

For details of manholes, timber cradles and outlet pipes see Dwg. 103-D-664, 665, 585, and 696

Pipe drains shall have uncemented joints.

LOG SYMBOLS

Si — Silt
C — Clay
L — Loam
S — Sand
G — Gravel
Water table elevation on May 21, 19__

HYDRAULIC PROPERTIES

REACH	D.A	S	n	V	Q PIPE FULL	Q EST. FLOW
0+00 to 9+00	15"	0.03	0.016	7.46	7.4	4.0
9+00 to 18+30	12"	0.03	.0	5.5	4.9	3.0
18+30 to 24+90	10"	0.07	.0	9	7.4	2.85
24+90 to 32+00	8"	.0	.0	.66	2.48	2.42

EXPLANATION

M.H. = Manhole
H = Height of manhole

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

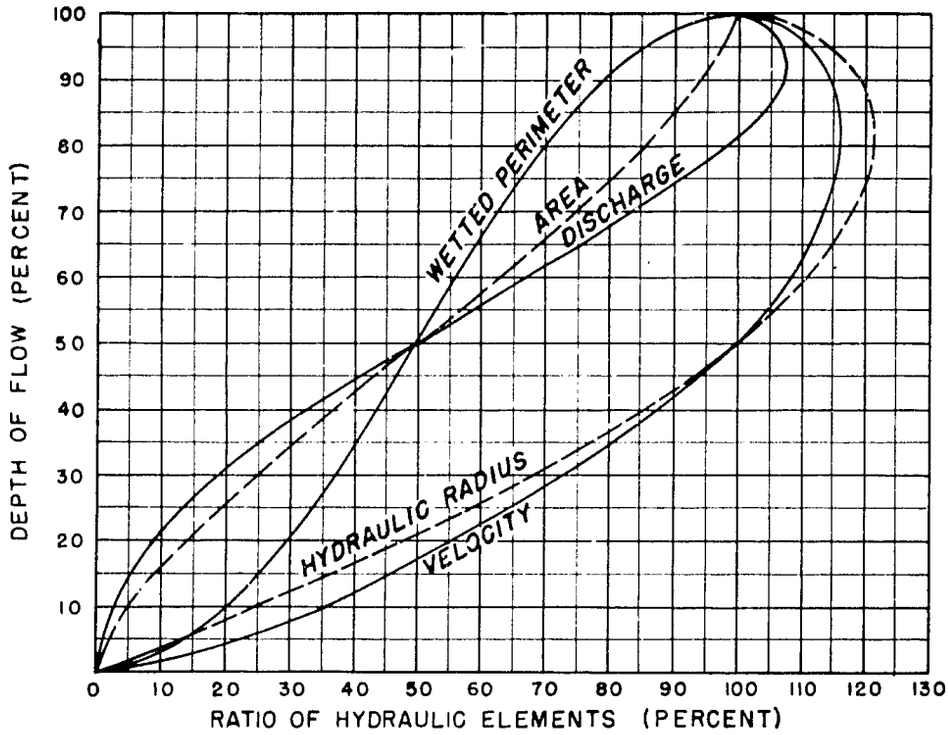
TYPICAL CLOSED DRAIN
PLAN AND PROFILE

DRAWN — SUBMITTED
TRACED — RECOMMENDED
CHECKED — APPROVED

DENVER, COLORADO. 103-D-667

Figure 17
Par. 526.2.14

Figure 18
Par. 526.2.15



SIZE OF PIPE (INCHES)	DIAMETER (FEET)	AREA (SQUARE FEET)	HYDRAULIC RADIUS (FLOWING FULL OR HALF FULL) (FEET)
8	0.667	0.349	0.167
10	0.833	0.545	0.208
12	1.000	0.785	0.250
15	1.250	1.227	0.312
18	1.500	1.767	0.375
21	1.750	2.405	0.437
24	2.000	3.142	0.500
27	2.250	3.976	0.556
30	2.500	4.909	0.625
33	2.750	5.940	0.687
36	3.000	7.068	0.750

HYDRAULIC PROPERTIES OF DRAIN PIPE

103-D-687

.1 Introduction. In addition to open and closed relief and interceptor drains, there are certain other types of drains that are used under particular circumstances. These are relief wells, inverted wells, and pumped wells. Detailed instructions for investigating, planning, and installing wells are given in Series 530, Ground-Water Engineering Techniques and Standards.

.2 Relief Wells. It may be found that an area is underlain by an artesian aquifer whose top confining bed is fractured or otherwise sufficiently permeable that the artesian pressure moves water up toward the land surface and maintains a high water table. Removal of this water by a normal drainage system might require spacings too close to be economical, and the artesian aquifer may be too deep to intercept by a deep drain. In very rare cases where conditions are extremely favorable, it may be found that relief wells drilled into the artesian aquifer and outletting in the bottom of a deep open drain will relieve the artesian pressure sufficiently to lower the ground-water table a safe distance below the ground surface. Ordinarily, however, a well of this type does not relieve enough head from an artesian aquifer that its effect extends for a sufficient lateral distance. Since relief wells are successful only in rare cases, the investigation must be carefully and thoroughly carried out to insure success. Artesian

pressures must be located and identified, and pressure reductions must be verified and their extent defined before undertaking any relief well program.

.3 Inverted or Recharge Wells. An inverted or recharge well is one in which water flows into the earth instead of coming from it. When used in connection with drainage, the inverted well constitutes only the outlet or point of disposal of water from a drainage system. It does not replace the system.

The condition under which these wells may be used is when a permeable zone underlies the area to be drained but is separated from it by a slowly permeable zone or aquiclude. For practical use, the permeable zone must be able to accept the required quantities of water either by storage or by carrying the water to a natural outlet. Examples are extensively fractured basalts or cavernous limestones. Coarse sands and gravels may have limited usefulness if they have good outlet conditions.

The well is constructed in a normal manner, but provision must be made to remove all sediment from drainage water before it enters the inverted well. Sediment will clog the aquifer in the vicinity of the screen and will gradually reduce the effectiveness of the well. The life of the well is in inverse ratio to the amount of sediment the water carries. 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 sea water is becoming more important.

These studies have been made on filtered water, chlorinated water, deaerated water, and water under pressure. Filtration seems to be most important, but the results of studies now underway may provide greater clarification of the related effects.

.4 Pumped Wells. Under certain conditions, pumped deep wells may offer the most efficient solution for a drainage problem. In some cases these pumped wells may provide all necessary drainage, while in others the wells may furnish supplemental drainage for certain critical areas only. These wells may discharge directly into the irrigation system, with reuse of the water for irrigation, or they may discharge into a waste ditch or drainage channel. Drainage by pumping is feasible only for localities having extensive underlying aquifers of good thickness in which wells will have large areas of influence under nominal drawdown. The latter condition insures a maximum effect from each well, thus reducing the total number of wells required to protect a given area. Pumped relief wells in artesian areas may prove especially effective. By pumping the artesian pressures can be lowered over a widespread area and be an effective drainage tool in an area where open or closed drains would be uneconomic or ineffective. Investigations for pumped drainage wells are carried out in accordance with the procedures in Series 530, Ground-Water Engineering Techniques and Standards. Power costs may be 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.

.1 Introduction. In nearly flat and extensive regions where the area water table is now or is expected to be high, the necessary measures to control its height within acceptable limits will require the installation of a series of parallel relief drains. The proper spacing of these drains is a very important yet sometimes rather nebulous determination, particularly in new areas which have not been irrigated and where no drains have been constructed. Proper spacing of drains is dependent upon a number of closely interrelated conditions, the interaction of which can be highly variable. Such things as depth of drain; depth to a slowly permeable barrier; soil permeability, specific yield, and required depth of soil aeration for plant growth; irrigation application and resultant deep percolation; length of irrigation season, number of irrigations; climatic conditions; and in some cases irrigation water quality all affect the spacing of drains. In predicting the proper spacing of drains for use in estimating the drainage requirement and drain costs for planning purposes, every effort should be made to obtain information from satisfactorily operating systems in the vicinity or in areas of similar soil, topographic, climatic, and other characteristics. Lacking such information, it becomes necessary to resort to the use of mathematical formulas to support judgment in predicting spacing requirements.

Various methods have been developed for estimating drain spacing. Most of these are empirical methods developed to meet the specific characteristics of a particular area, or are based on assumptions of steady-state flow conditions. The very nature of the precipitation or irrigation pattern in either a humid or irrigated area dictates that the storage and discharge of ground water follow a transient or nonsteady-state flow regimen. The Bureau of Reclamation has developed formulas and charts (Figures 1 and 2) based on transient-flow conditions which relate the behavior of the water table, under varying physical soil characteristics, to time and drain spacing. Verification of the applicability of these formulas and charts is shown by the close correlation obtained between actual spacing and drawdown values from constructed drains and the corresponding values predicted through use of the charts and measured soil characteristics.

A method of determining drain spacing has been developed using these charts in conjunction with the measured soil characteristics and irrigation regimen of any specific area. Although this method was developed for use in a relatively flat area, it has been determined that it is also applicable for areas having slopes amenable to normal surface irrigation.

.2 Background of the Method. A study of ground-water hydrographs in an irrigated area shows that, in general, the water table rises during the irrigation season, reaching its highest elevation after the last irrigation of the season, or, in an area of year-around 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 starts rising again with the beginning of irrigation the following year. In addition to this general fluctuation through the year, there is a definite rise immediately after each recharge to the ground water from precipitation or irrigation with a corresponding lowering of the water table in the time interval preceding the next recharge.

If annual discharge from an area does not equal or exceed annual recharge, the trend of this general cyclic water-table fluctuation will be progressively upward from year to year. When the annual discharge and recharge become equal, the highest level and range of the cyclic annual water-table fluctuation become reasonably constant from year to year. This condition is defined as "dynamic equilibrium."

Figures 3A and 3B are ground-water hydrographs which show how these conditions developed under irrigation in two specific areas. Figure 3A shows the upward cyclic trend and the leveling off and stabilization of the cyclic fluctuation when, on an annual basis, outflow became equal to inflow or when dynamic equilibrium was reached. At this location dynamic equilibrium was reached with the maximum water-table elevation at a point sufficiently below ground level to preclude the need for artificial drainage. In other words, at this location the soil characteristics are such that natural drainage provides adequate protection under the irrigation practices in this area. Figure 3B shows a similar upward trend of the water table in another

area. However, at this location the maximum 1956 water-table elevation and the continued upward trend indicated the imminent approach of a damaging water-table condition the following year. On the basis of this information, a drain was constructed in the area in the Spring of 1957. The effect of this drain in producing dynamic equilibrium at a safe water-table level is quite noticeable.

The method of determining drain spacing shown herein takes into account the transient regimen of the ground-water recharge and discharge. It is designed to give a spacing which produces dynamic equilibrium conditions at a specified water-table height under the specific soil, irrigation, crop, and climatic characteristics of the area under consideration.

.3 Data Required. The curves of Figures 1 and 2 show graphically the relation between the dimensionless parameters y/y_o versus $\frac{k\ D\ t}{SL^2}$ and $\frac{Z}{H}$ versus $\frac{k\ H\ t}{SL^2}$ based on the transient-flow theory. They represent the solution, at the midpoint between drains, for the cases where the drains are located above a barrier and on a barrier, respectively.

The definitions of the various items in the parameters are shown in graphic form on the sketches at the left of Figures 1 and 2, and each item is discussed in the following paragraphs.

A. y_o and H. These items represent the water-table height above the drain at the beginning of each individual drain-out period, or the time zero for each drain-out period. As used in the drain-spacing method, they represent the water-table height immediately after an instantaneous water-table buildup caused by deep percolation from

precipitation or irrigation, or the height at the beginning of each new drain-out period during the lowering process of the non-irrigation season. Their maximum value is based on the requirements of aerated root zone for the crops and climatic conditions of each specific area.

B. y and Z. These items represent the water-table height above the drain at the end of each individual drain-out period. They represent the height to which the midpoint water-table elevation has fallen during specific time periods due to the specific soil characteristics and drain spacing.

C. Permeability, k. This item represents the weighted average permeability in the flow zone between the midpoint water-table height and a slowly permeable zone considered to be a barrier insofar as ground-water flow to the drains is concerned. The mathematical model upon which the solution of the transient-flow theory is based assumes a homogeneous, isotropic soil material in this zone. It is well known that such a condition seldom, if ever, exists. However, the use of a k value representing the average from a number of individual locations within the area under consideration has given good correlation between measured and computed drain spacing and water-table fluctuations, and is considered satisfactory for use in the drain-spacing calculations.

D. Specific Yield, S. The specific yield of a soil material is the amount of ground water that will drain out of a saturated soil under the forces of gravity. It 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 recharged into or discharged out of the system. On the basis of considerable data, a general relationship has been developed between permeability and specific yield. This relationship is shown on Figure 4 of Chapter 523.0, and can be used in estimating specific-yield values in the drain-spacing calculations.

Since the fluctuation of the water table in a drained, irrigated area takes place in the zone between the drains and the maximum allowable water-table height (y_o or H), it is reasonable to assume that the average specific yield in this zone will adequately reflect water-table fluctuations. The use of Figure 4 of Chapter 523.0 in estimating specific yield, therefore, requires that the permeability in this zone be known.

The specific-yield value, when used in the dimensionless parameters of Figures 1 and 2, accounts for the amount of drain-out as represented by water-table lowering. To determine the buildup in water-table depth from each increment of recharge, the depth of each recharge is divided by the specific yield.

E. Time, t. This item represents the drain-out time periods between irrigations or at specified intervals during the nonirrigation season during which the water table is being lowered by the drains. In an irrigated area these time periods between irrigations have generally been established. If this information is not available

or is required in an area proposed for irrigation, it can be derived in the manner described in Paragraph 526.6.4.

It has been determined that the spacing calculation should consider the longnonirrigation season as two approximately equal time periods.

F. Flow Depth, D. This item represents the average flow depth transmitting water to the drain. As shown on Figure 1, it 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 any specified time ($D = d + y_o/2$).

It will be noted that the curves of Figures 1 and 2 are for the cases where the drains are located above a barrier and on a barrier, repsectively. The theoretical derivation for the case where the drains are located above a barrier was based on the assumption that the drains are at a shallow depth compared with the depth to barrier; i.e., $d \gg$ maximum y_o. This poses a question regarding cases where the drains are above the barrier but d is not large compared with maximum y_o. A study of the results of the verifications of the applicability of Figures 1 and 2 indicated that where $d/y_o \leq 0.10$ the spacing computations should be made on the basis of the case where the drains are located on the barrier. The study also showed that for conditions where $d/y_o \geq 0.80$ the spacing computations should be made on the basis of the case where the drains are located above a barrier. Unfortunately, none of the data available for these comparisons produced a d/y_o value within the

range between 0.10 and 0.80. Therefore, there still remains a question as to which case should be used when the d/y_o value falls within this range. It should be possible, however, to predict a satisfactory drain spacing for this range through a judicial use of both cases.

Research results have indicated that a barrier at a depth greater than L/4 does not appreciably affect drain spacing. Therefore, values of d greater than L/4 should not be used.

G. Drain Spacing, L. This item represents the distance between parallel drains. The spacing is not calculated directly in this method, but values must be assumed and a solution made to determine whether, with the prescribed physical conditions, the successive intermittent water-table buildup and lowering will offset each other on an annual basis, or produce a state of dynamic equilibrium.

Convergence. When ground water flows toward a drain, the area of flow must converge, or become smaller, as the drain is approached. This causes a head loss in the system which must be accounted for in the spacing computations.

The curve of Figure 1 does not account for this convergence loss, and the spacing derived through the use of this curve is somewhat too wide. The following formula can be used to estimate the amount the spacing, derived through the use of Figure 1, should be reduced.

$$\text{Correction} = D \log_e \frac{D}{4r}$$

where:

D = average flow depth (feet), and

r = outside radius of drainpipe plus gravel envelope thickness, or one-half bottom width for open drains (feet).

The curve of Figure 2 is based on a solution in which convergence is accounted for in the initial water-table profile. Hence, no correction for convergence is required in drain spacing derived through the use of Figure 2.

.4 Method of Use. The method of using the data described in Paragraph 526.4.3, above, in producing dynamic equilibrium conditions is briefly described in this paragraph. A more detailed description is given in the example computations of Paragraph 526.4.5.

Starting with an assumed drain spacing, L, and the assumption that the water table reaches the maximum allowable height, y_o, above the drain immediately after the application of the last irrigation of each season, the position of the water table at the midpoint between drains is calculated through the lowering process of the nonirrigation season (even in areas of year-around cropping there is a slack period sometime during the year), and then through the buildup and drain out from each irrigation of the following irrigation season. If dynamic equilibrium conditions have been produced by the assumed drain spacing, the water table at the end of the following irrigation season will again return to the maximum allowable height, y_o. Obviously, this is a hunt-and-try process in which a drain spacing is assumed and a solution made to see if, with

the prescribed physical conditions, the successive intermittent water-table buildup and drain out will offset each other on an annual basis.

Normally, it is necessary to make only two drain-spacing assumptions before the dynamic equilibrium producing spacing can be verified. A straight-line relation between two assumed spacings and their resulting values of y_o after a complete annual cycle will, if the original assumptions are reasonably close to the proper spacing, permit determination of the proper spacing.

.5 Drain Above the Barrier Layer. The following example is given to illustrate the methods of computation for this case. The following conditions are assumed:

A. The depth from the barrier to the drain, d, is 22 feet.

B. The depth of the drain is 8 feet.

C. The root zone requirement is 4 feet. This gives a maximum allowable water-table height above the drain of 4 feet.

D. The weighted average permeability in the zone between the barrier and the maximum allowable water-table height is 5 inches per hour, or 10 feet per day.

E. The permeability is uniform with depth. Therefore, the permeability in the zone between the maximum allowable water-table height and the drains is also 5 inches per hour. From Figure 4 of Chapter 523.0, the corresponding value of specific yield is 18 percent.

F. The deep percolation from each irrigation (also assumed to be the same from a spring snowmelt) is 1 inch, or 0.083 foot. The water-

table buildup from each increment of recharge is the deep percolation divided by the specific yield, or 0.083 ÷ 0.18 = 0.46 foot.

G. The approximate dates of the snowmelt and the irrigation applications are as follows:

	Date	Time period between irrigations, days
Snowmelt	April 22	
First irrigation	June 6	45
Second irrigation	July 1	25
Third irrigation	July 21	20
Fourth irrigation	August 4	14
Fifth irrigation	August 18	14
Sixth irrigation	September 1	14
		132

Therefore, the nonirrigation period when no additional water is added is 233 days (365 - 132).

H. A drain spacing of 1,450 feet results from the relation of assumed spacing and final water-table heights from two prior calculations.

With the assumption that the water table reaches the maximum allowable height immediately after the application of the last irrigation of each season, the computations are started at this point in time. The first step in applying the method is to compute the $\frac{k\ D\ t}{SL^2}$ value for the first time interval. With this value and the curve in Figure 1, it is possible to find the value of y/y_o. Knowing the initial y_o, we can compute y, or the height to which the midpoint water table falls during this time period. This process is repeated for each successive time interval, obtaining for each the water-table height attained as a net result of each successive recharge and drain out.

The following table shows the computations which verify the spacing:

Computation of Water-table Fluctuation with Drain Above the Barrier Layer and a Drain Spacing of 1,450 Feet

Irrig. No.	t, days	Buildup per irrigation, feet	y_o, feet	D, feet	$\frac{k D t}{S L^2}$	$\frac{y}{y_o}$	y
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
6							
	117		4.00	24.00	0.0735	0.566	2.26
	116		2.26	23.13	.0714	.580	1.31
SM		0.46					
	45		1.77	22.89	.0272	.877	1.55
1		.46					
	25		2.01	23.01	.0152	.958	1.92
2		.46					
	20		2.38	23.19	.0123	.978	2.33
3		.46					
	14		2.79	23.40	.0087	.985	2.75
4		.46					
	14		3.21	23.61	.0087	.985	3.16
5		.46					
	14		3.62	23.81	.0088	.985	3.56
6		.46					
			4.02				

Explanation of each column:

Column (1). Number of each successive increment of recharge such as snowmelt (SM), rain, or irrigation.

Column (2). Length of drain-out period (time between successive increments of recharge or between incremental drain-out 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 drain out (Column (8) of preceding period plus Column (3) of current period).

Column (5). Average depth of flow, $d + y_o/2$ (d should be limited to L/4).

Column (6). A calculated value representing the flow conditions during any particular drain-out period $[(\frac{k}{SL^2}) \times (\text{Column } 2) \times (\text{Column } 5)]$.

Column (7). Taken from the curve in Figure 1.

Column (8). Midpoint water-table height above drain at end of each drain-out period [(Column 4) x (Column 7)].

The spacing of 1,450 feet results in a maximum water table about 4.02 feet above the drain and is satisfactory. As stated in Paragraph 526.4.3G, this solution does not account for convergence. The reduction in spacing due to convergence is obtained by use of the equation in the reference paragraph, as follows:

D = average depth of flow; in this example $D = \frac{(24.0 + 22.89)}{2} = 23.4$ feet, and

r = outside radius of drain plus gravel envelope; in this example, outside radius of 6-inch ID pipe plus 4-inch envelope = 0.63 foot.

The correction is therefore:

$$C = D \log_e \frac{D}{4r}$$

$$= 23.4 \log_e \frac{23.4}{4(0.63)} = 52 \text{ feet.}$$

The corrected spacing then would be 1,450 - 52 = 1,398 feet, or about 1,400 feet. Figure 4 illustrates the water-table fluctuation produced as a result of the conditions of this example and this drain spacing.

.6 Drain on the Barrier Layer. The following example is given to illustrate the methods of computation for this case. All conditions, except for barrier depth, are the same as in the preceding example.

The assumption of drain spacing and subsequent computations of water-table heights are similar to those of the previous example except that the curve of Figure 2 is used with a slightly modified computing form. The cut-and-try process is necessary, but, as in the previous example, the correct spacing can be determined after two trial computations.

The following table shows the computations which verify the spacing:

Computation of Water-table Fluctuation with Drain on the Barrier Layer and a Drain Spacing of 470 Feet

Irrig. No.	t, days	Buildup per irrigation, feet	H, feet	$\frac{k\,H\,t}{SL^2}$	$\frac{Z}{H}$	Z, feet
6						
	117		4.00	0.1180	0.655	2.62
	116		2.62	.0770	.745	1.95
SM		0.46				
	45		2.41	.0273	.890	2.14
1		.46				
	25		2.60	.0163	.932	2.42
2		.46				
	20		2.88	.0145	.940	2.71
3		.46				
	14		3.17	.0112	.953	3.02
4		.46				
	14		3.48	.0123	.949	3.30
5		.46				
	14		3.76	.0132	.945	3.55
6		.46				
			4.01			

As stated in Paragraph 526.4.3G, no correction for convergence is necessary. The correct spacing then would be 470 feet.

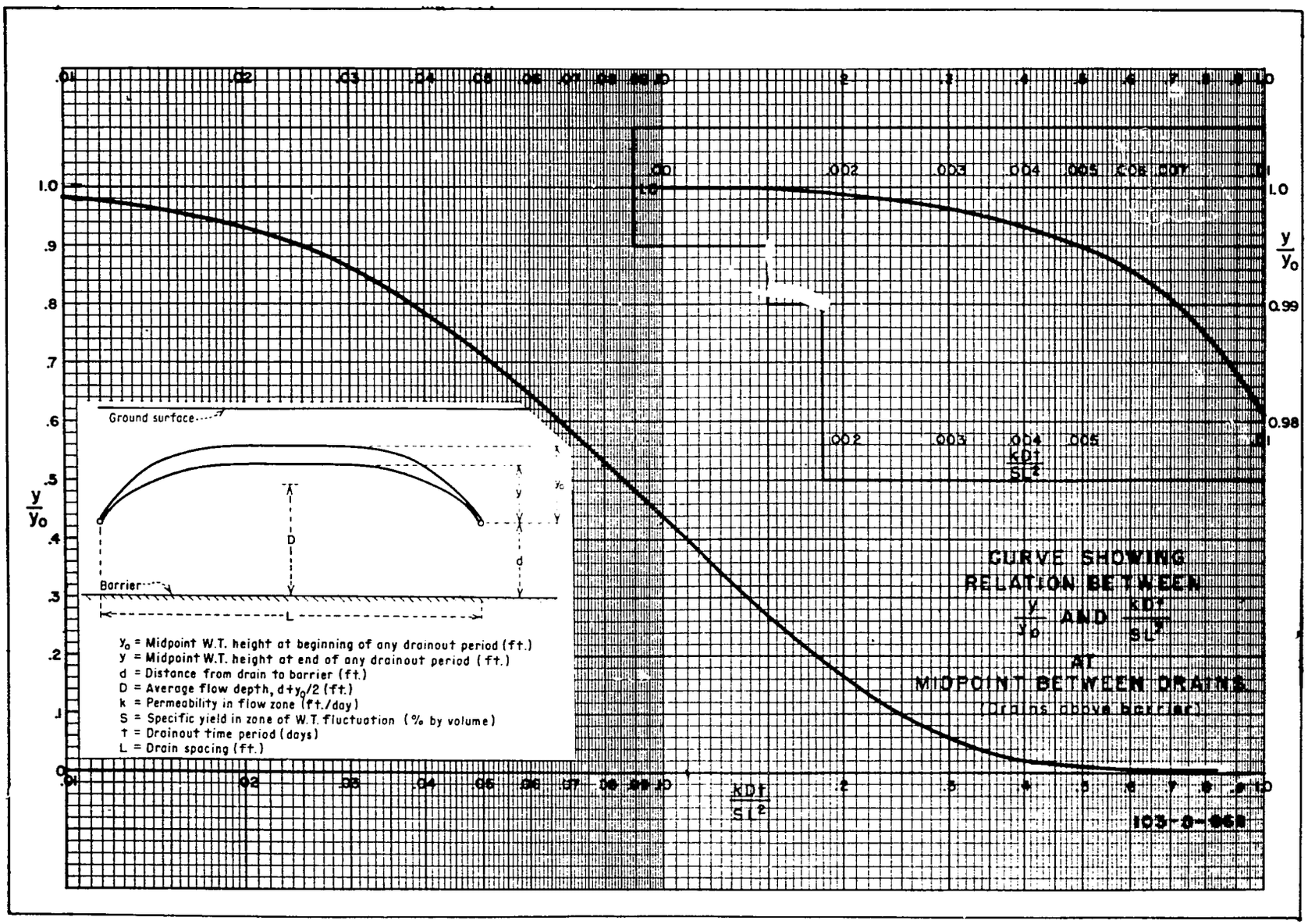


Figure 1
Par. 526.4.1

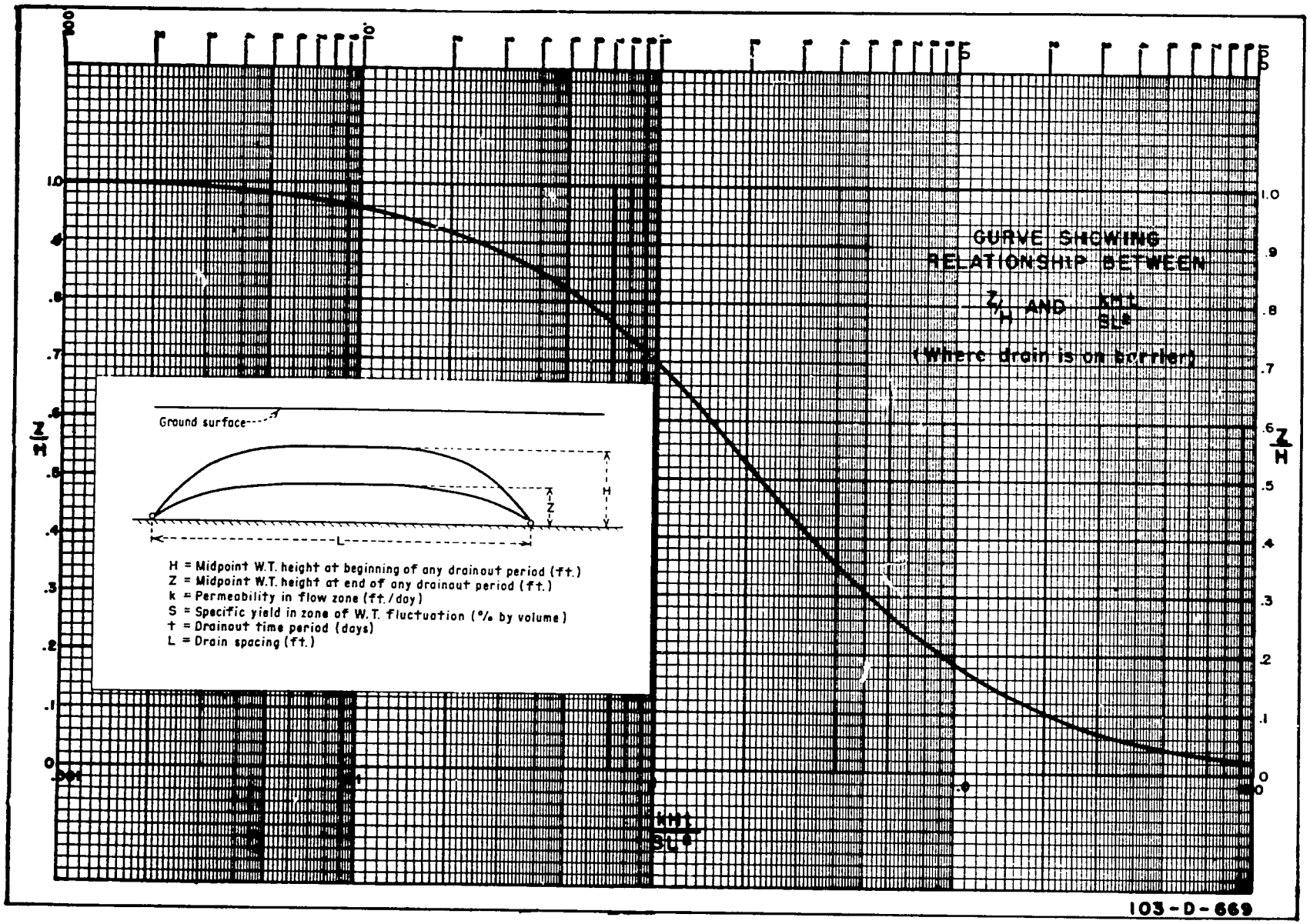
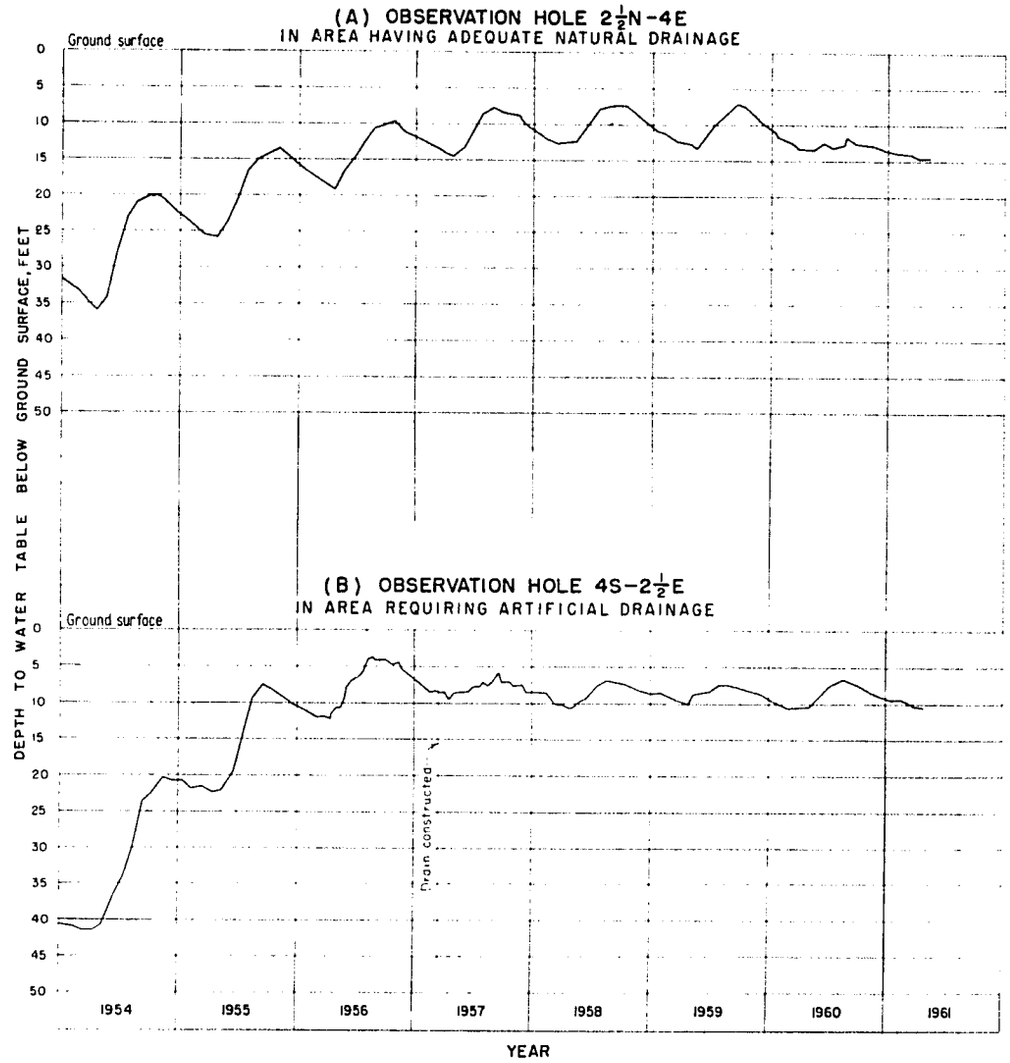


Figure 2
Par. 526.4.1

Figure 3
Par. 526.4.2



GROUND WATER HYDROGRAPHS

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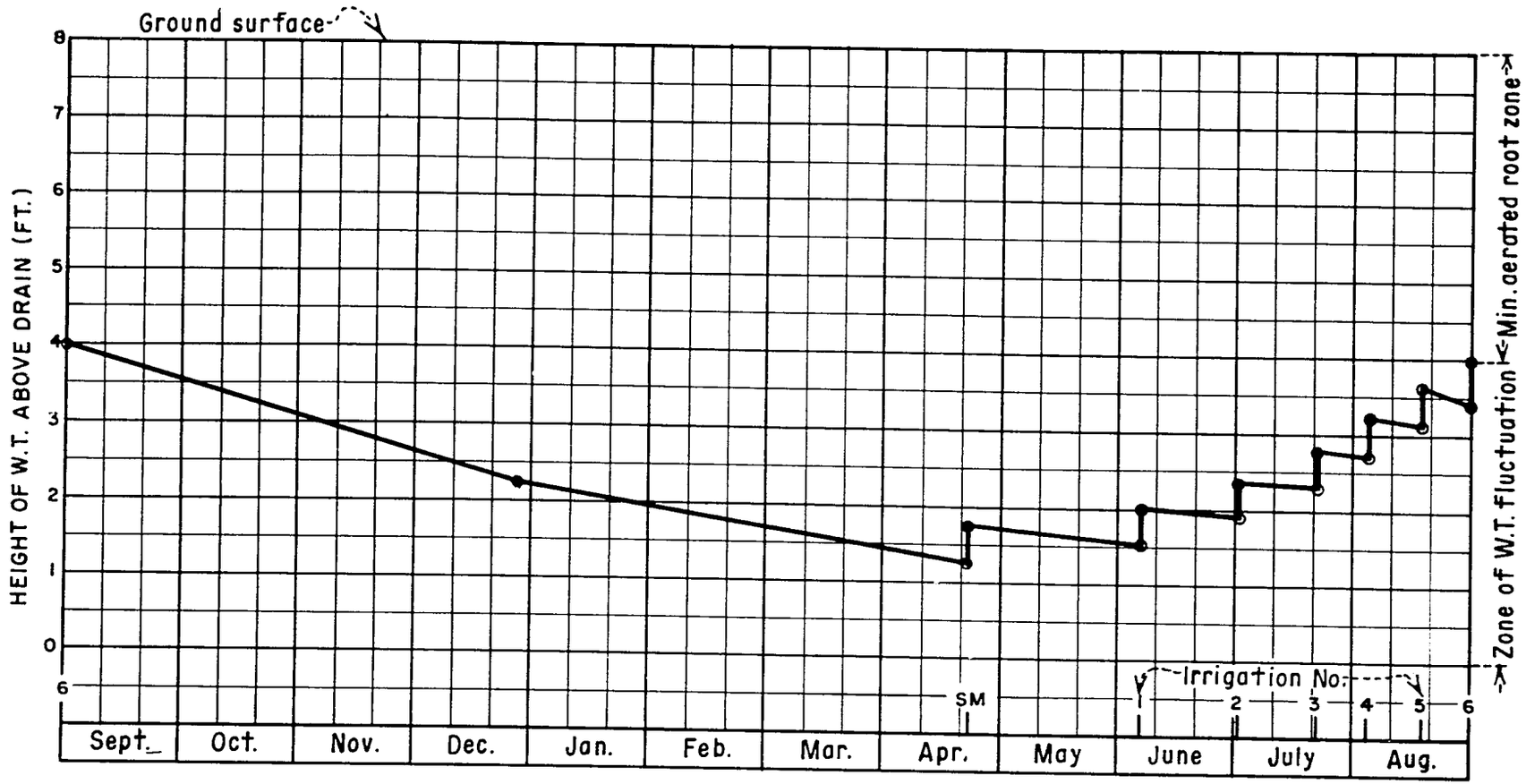
Figure 4
Par. 526.4.5



WATER TABLE FLUCTUATION FOR DRAIN ABOVE THE BARRIER LAYER

103-D-776

CHAPTER 5 INTERCEPTOR DRAIN LOCATION

(To be issued at a later date.)

.1 Introduction. As a means of illustrating the application of the various methods and procedures of drainage investigations, the following paragraphs will describe an investigation of a Sample Farm that developed a waterlogged condition after about 3 years of irrigation. Using the test data, a drainage system is planned for this farm. Figure 1 shows the layout, surface topography, and irrigation facilities of the Sample Farm.

.2 Investigation Procedure. The first step in the investigation was to lay out a grid system covering the waterlogged area. A 400-foot grid was arbitrarily chosen because it will provide sufficient points for making a detailed ground-water contour map and for providing adequate permeability data. If additional holes should be required, they can be readily located from the grid points. The grid was located to include any suspected source of seepage from canals. Ground surface elevations were determined at each grid point, and elevations were taken at the bottom and 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 10 feet and to 20 feet at the 800-foot grid points. The depth to the water table was measured and 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. Figure 2 shows the water-table conditions at the time of the investigation.

Based upon water-table location and soil profile data, three general types of conditions were recognized, each requiring a different combination of permeability test methods. Figure 3 shows the location of the test sites and the combination of permeability methods required at each site. Typical soil profiles in Subareas A, B, and C are shown in Figure 4. The water table in Subarea A was about 7 feet from the surface at the time of the investigation, but the farmer reported that it rose to within 1 or 2 feet during the period of heaviest irrigation. These conditions indicated a need for horizontal permeabilities under saturated conditions in the 2- to 7-foot sandy clay loam zone. As this zone was now dry, the shallow well pump-in test was indicated. Below 7 feet in the sandy loam zone, the horizontal permeability under saturated conditions could be determined by the auger-hole test. Therefore, three additional 6-foot holes were augered at Grid Points D-1, C-3, and B-4 and pump-in tests were run. The sides of the original 10-foot holes at these points were brushed, the holes covered, and the water table allowed to stabilize, after which auger-hole tests were run.

The water table in Subarea B was at about 4.8 feet, and there was a 2-1/2-foot clay layer beginning at about 4 feet which might cause a perched water table to develop during the irrigation season. As a check on this, the vertical permeability of the clay layer was needed. This

required use of the permeameter test. The tests were run at D-2 and at E-3. During the test the water table at E-3 rose into the 6-inch test zone and the test had to be abandoned. As the clay layer appeared homogeneous and isotropic at E-2, the piezometer test was substituted for the permeameter test. This gave a value for horizontal permeability, but in view of the homogeneity of the clay, the vertical permeability could be assumed to be in about the same range as the horizontal.

Since the 4- to 9-foot profile in Subarea C was homogeneous and the water table was at 4.8 feet, the auger-hole test was used for determining the permeability of this zone and the piezometer test for determining the permeability of the clay loam and clay zones below 13.5 feet.

Points on the 800-foot grid were used to determine the probable barrier layer. This required testing the permeability of various tight layers below the water table and the prospective drain depth. At these depths, the auger-hole test was not practical, so the piezometer test was used and tests were run at C-2, C-4, E-0, E-2, and G-4. Figure 5 shows the location of all test sites and the permeability data from all in-place tests.

.3 Moisture-holding Capacity in the Root Zone. Another physical soil property needed in analysis is the moisture-holding capacity of the soil within the root zone. That part of the moisture-holding capacity which can be used by plants is termed the plant available moisture and is the amount held in a given soil between field capacity and the wilting point.

The three subareas of the Sample Farm shown on Figure 3 were analyzed with respect to moisture-holding capacity within an assumed 4-foot root zone, and Subarea C was found to be the critical subarea. In Subarea C the plant available moisture was determined to be 1.16 inches in the first foot, 1.25 inches in the second foot, 1.45 inches in the third foot, and 1.45 inches in the fourth foot. These values may be available from previous land classification studies made in the area; from agricultural bulletins published by Federal or state agencies; or they may be obtained in the laboratory by methods described in Series 510, Land Classification Techniques and Standards.

Theoretically, all of this moisture is available for plant growth, and rapid crop growth during periods of maximum transpiration will be maintained if the moisture is replaced to field capacity at the moment it reaches the wilting point. Practically, however, this amount of depletion should not be permitted because a small delay in the availability and application of irrigation water would be harmful to the plants, so irrigation should be planned for the time that available moisture has been depleted about 70 percent.

Depletion of available moisture by evapotranspiration does not take place uniformly through the 4-foot root zone, but is in relation to the moisture extraction pattern of the plant roots. This pattern has been determined to be: 40 percent from the first foot, 30 percent from the second foot, 20 percent from the third foot, and 10 percent from the fourth foot for crops that are fairly well matured.

The total amount of readily available moisture in the 4-foot root zone of the Sample Farm can be seen by examination to be governed by the first foot and would be $\frac{1.16 \times 0.70}{0.40} = 2.03$ inches. Under different conditions, other zones of the root-zone profile might limit the amount of total readily available moisture, so all zones must be checked.

Any irrigation application to the land in excess of 2.03 inches would become deep percolation.

.4 Annual Irrigation Schedule. The irrigation schedule for the Sample Farm, as for any farm, will vary from year to year because of variations in crops and acreages. Rainfall and time of planting will also vary from year to year, but for a specific climate, irrigation and cropping practices usually follow a pattern. By taking a long-time average for the factors used in computing consumptive use, the irrigation schedule is found to be about the same each year except for dates. Thus, an average irrigation schedule can be selected. The following tabulations show the calculations for the average irrigation requirement and daily consumptive use of crops best suited to the climate and location of the Sample Farm.

Calculations for Average Consumptive Use and Irrigation Requirement for Sample Farm

Crops	Average percent grown per year	Growing season	Consumptive use coefficient, K	Percent moisture extracted per foot 1/			
				1st	2nd	3rd	4th
Alfalfa	40	15 May to 21 Sept	0.85)	40	30	20	10
Corn	20	15 May to 15 Sept	0.75)				
Beans	20	15 May to 15 Aug	0.75)				
Small grains	20	15 May to 15 Aug	0.75)				

1/Taken from Shockley (1955); this is a general moisture extraction pattern that can be used for most irrigated crops.

Consumptive Use and Irrigation Requirement 2/
for Alfalfa, 15 May to 21 September

Month	Growing days	t	p	f	K	CU	r	Irrigation requirement	Daily CU from irrig
May	16	53.8	10.20	2.84	0.85	2.41	0.93	1.48	0.09
June	30	62.5	10.30	6.44	0.85	5.47	1.17	4.30	0.14
July	31	70.0	10.42	7.29	0.85	6.20	0.97	5.23	0.17
August	31	67.1	9.66	6.48	0.85	5.51	0.64	4.87	0.16
Sept	21	57.1	8.40	3.36	0.85	2.86	0.68	2.18	0.10

Consumptive Use and Irrigation Requirement 2/
for Corn, 15 May to 15 September

Month	Growing days	t	p	f	K	CU	r	Irrigation requirement	Daily CU from irrig
May	16	53.8	10.20	2.84	0.75	2.13	0.93	1.20	0.07
June	30	62.5	10.30	6.44	0.75	4.83	1.17	3.66	0.12
July	31	70.0	10.42	7.29	.75	5.46	0.97	4.49	0.15
August	31	67.1	9.66	6.48	0.75	4.86	0.64	4.22	0.14
Sept	15	57.1	8.40	2.40	0.75	1.80	0.49	1.31	0.09

Consumptive Use and Irrigation Requirement 2/
for Beans and Small Grains, 15 May to 15 August

Month	Growing days	t	p	f	K	CU	r	Irrigation requirement	Daily CU from irrig
May	16	53.8	10.20	2.84	0.75	2.13	0.93	1.20	0.07
June	30	62.5	10.30	6.44	0.75	4.83	1.17	3.66	0.12
July	31	70.0	10.42	7.29	0.75	5.46	0.97	4.49	0.15
August	15	67.1	9.66	3.14	0.75	2.36	0.31	2.05	0.14

2/From Tomlinson (1951).

In the above tabulations:

CU = consumptive use = Kf, inches,

t = mean monthly temperature, ° F,

p = percent of daytime hours of year for each month,

f = consumptive-use factor = $\frac{tp}{100}$ x percent growing days in month,

K = consumptive-use coefficient, and

r = mean monthly precipitation x percent growing days in month.

Weighted Average Daily Consumptive Use
From Irrigation--All Crops

Month	Total inches	Remarks
May 15 to June 1	0.078	
June 1 to July 1	.128	
July 1 to Aug 1	.158	
Aug 1 to Aug 15	.148	
Aug 15 to Sept 1	.154	For alfalfa & corn only
Sept 1 to Sept 15	.097	For alfalfa & corn only
Sept 15 to Sept 21	.100	For alfalfa only

The irrigation schedule in Figure 6 was derived from the computed daily consumptive use of all crops and the readily available moisture (2.03 acre-inches per acre) held in the soil, as follows:

If the first irrigation is assumed to be started on May 15, and the root zone filled to field capacity on that date, then depletions from the available moisture for the remaining 16 days of the month will be 16 x 0.078 = 1.25 inches. Then 2.03 - 1.25 = 0.78 inch will remain in the soil at the end of May. It will take 6 days to deplete this amount at the rate of 0.128 inch per day ($\frac{0.78}{0.128}$ = 6 days). There will be 16 + 6 = 22 days between irrigations, or the second irrigation should be started on June 6. Similarly, it will take $\frac{2.03}{0.128}$ = 16 days for this moisture to be depleted so the third irrigation should be started on June 22. Subsequent irrigation dates through the growing season would be computed in like manner.

.5 Irrigation Deliveries. The records show that irrigation deliveries to the Sample Farm are made at the rate of 5 cfs, or 4.96 acre-inches per hour, and that it requires 88 hours to irrigate the 125 acres in the farm. The depth of water delivered per irrigation is then:

$$\frac{88 \times 4.96}{125} = 3.49 \text{ inches}$$

Since 2.03 inches is the amount readily available to plants within the root zone, the irrigation efficiency is $\frac{2.03}{3.49} = 58$ percent.

In this particular problem, the Sample Farm had been irrigated for a few years and records of these irrigations were available. Similar records may not be available in other areas under study, so a value for irrigation deliveries will have to be obtained by another method. One method would be to assume an irrigation efficiency for the area based on whatever information is available. In general, farm irrigation efficiencies average in the range of 50 to 60 percent, so a judgment value in this range can be assumed if no better information can be obtained.

.6 Water Sources Causing High Water-Table Conditions on the Sample Farm. Precipitation in the area is low and erratic, so is not considered to be a source contributing to the ground-water body. The remaining sources of high ground water during the irrigation season are: (1) ground water from the adjacent farm to the south moving into the area as subsurface flow; (2) deep percolation in the area resulting from frequent irrigations (the irrigation requirement was high because of low moisture retention in the root zone); and (3) seepage from unlined farm laterals. From A and B, below, these amounts are:

Adjacent areas	= 0.04
Irrigation	= 0.97
Farm ditch loss	= 0.14
Total	= 1.15 acre-inches per acre each irrigation

A. Deep Percolation from Adjacent Areas. The ground-water contours in Figure 2 show that subsurface water is moving under the Sample Farm from the south. An estimate of the volume of water moving into the area can be made using the Darcy principle:

$$Q = kiA$$

where:

Q = flow per unit width,

k = permeability of the water-bearing stratum,

i = slope of the water surface, and

A = cross-sectional area of the water-bearing stratum.

A permeability of 5 inches per hour was indicated by the auger-hole test at Site E-0. Values of i (0.004 foot per foot) and A (8 square feet per foot) were determined from information taken from the north-south profile on the E-line shown in Figure 7. The discharge Q was determined to be 0.0000037 cfs per linear foot. This was a base flow which could increase to about 0.000005 cfs per linear foot after a heavy irrigation, since both the slope and area would increase somewhat. As the south boundary of the sample area was about 2,600 feet wide, the total water moving into the farm could be 0.000005 x 2,600 = 0.013 cfs. This is equivalent to 0.31 acre-inch per day. Assuming an average irrigation cycle of 14 days and that this flow would occur under the entire farm area of 125 acres, the drainage requirement would be about $\frac{0.31 \times 14}{125} = 0.04$ inch per acre. This is a small amount of water that can be easily

removed through spaced drains that will also control the general areal water table. If the amount were larger, an analysis should be made to determine whether an interceptor drain should be constructed at the upper boundary of the Sample Farm. In this particular case, the soil profile and results of piezometer tests showed a clay loam barrier at a depth of about 16 feet along the boundary. It would not be economical to construct a drain at this depth to intercept all of the water moving into the area from the south, regardless of the amount, but for a larger amount it might be justifiable to construct a 9- or 10-foot drain to intercept a portion of the flow.

B. Deep Percolation from Farm Ditches. With irrigation deliveries at the rate of 5 cfs through farm ditches built in firm sandy loam wherein velocities are about 2.5 feet per second, the farm ditch losses can be calculated from the Moritz formula (Paragraph 525.0.10). The value of C for sandy loam is 0.66.

$$S = 0.2C \frac{Q}{V}$$

$$= 0.2 \times 0.66 \times \frac{5.0}{2.5}$$

$$= 0.264 \text{ cfs per mile}$$

The time required for irrigation of the Sample Farm is 88 hours, and during this time about 0.75 mile of farm ditch is carrying water. The seepage loss during each irrigation expressed as inches over the 125 acres is:

$$\frac{0.264 \times 0.75 \times 88 \times 3{,}600 \times 12}{43{,}560 \times 125} = 0.14 \text{ inch}$$

C. Deep Percolation from Irrigation. A portion of the total amount of 3.49 inches delivered to the farm will percolate down to join the ground-water body while the remainder will be disposed of in other ways. An unmeasured amount of the water leaves the farm as surface waste, and this is estimated to be 10 percent, or 0.35 inch. Farm ditch losses are 0.14 inch, and plant use is 2.03 inches, so the amount of deep percolation from irrigation is:

$$3.49 - 0.14 - 0.35 - 2.03 = 0.97 \text{ inch}$$

Total deep percolation on the farm is 0.97 + 0.14 = 1.11 inches

.7 Determination of Barrier Zone. Accurate appraisal of barrier zones is important in the drain-spacing solution, but their identification is not always easy or clear cut. Following the definition given in Paragraph 525.0.6 wherein the barrier zone was defined as a layer which has a permeability value one-fifth or less than the weighted permeability of the layers above it, the following tabulation shows the barrier layer computations for the six areas as shown in Figure 8. The weighted permeability is calculated as in Paragraph 525.0.6.

Computations Showing Selection of Barrier Layer

Sub-area	Depth, feet	Texture	k_1 permeability, in/hr	k_2, weighted permeability, in./hr	$k_1 \times 5$ compared with k_2 of layers above	Remarks
	4-9	SL	1.2	1.2		
	9-14	SCL	0.5	0.85	$0.5 \times 5 = 2.5 > 1.2$	
C-1	14-15	CL	0.2	0.79	$0.2 \times 5 = 1.0 > 0.85$	
	15-20	C	0.1	0.56	$0.1 \times 5 = 0.5 < 0.79$	Barrier
	4-9	SL	1.6	1.6		
	9-14	SCL	0.5	1.05	$0.5 \times 5 = 2.5 > 1.6$	
C-2	14-15	CL	0.2	0.98	$0.2 \times 5 = 1.0 < 1.05$	Barrier
	15-20	C	0.1	0.70		
	4-7	SCL	0.6	0.60		
	7-12	SL	1.2	0.97	$1.2 \times 5 = 6.0 > 0.60$	
A-1	12-16	L	0.8	0.92	$0.8 \times 5 = 4.0 > 0.97$	
	16-18	CL	0.3	0.83	$0.3 \times 5 = 1.5 > 0.92$	
	18-20	C	0.1	0.74	$0.1 \times 5 = 0.5 < 0.83$	Barrier
	4-7	SCL	1.0	1.0		
	7-12	SL	2.2	1.75	$2.2 \times 5 = 11.0 > 1.0$	
A-2	12-16	L	0.8	1.43	$0.8 \times 5 = 4.0 > 1.75$	
	16-18	CL	0.3	1.27	$0.3 \times 5 = 1.5 > 1.43$	
	18-20	C	0.1	1.13	$0.1 \times 5 = 0.5 < 1.27$	Barrier
	4-6	C	0.1	0.10		Barrier
	6-13	SL	1.5	1.20	$1.5 \times 5 = 7.5 > 0.1$	
B	13-17	CL	0.2	0.89	$0.2 \times 5 = 1.0 < 1.2$	Barrier
	17-20	C	0.1	0.74		
	4-9	SL	1.9	1.90		
	9-14	SCL	0.5	1.20	$0.5 \times 5 = 2.5 > 1.90$	
C-3	14-15	CL	0.2	1.10	$0.2 \times 5 = 1.0 < 1.2$	Barrier
	15-20	C	0.1	0.80		

.8 Depth of Drains. Figure 8 shows areas with similar drainage conditions and in-place permeability data for each. Study of these data shows that drains about 9 feet deep would be in the most permeable material. Also,

the benefits as compared with the construction cost for drain depths over 9 feet start decreasing, which gives an economic reason for not going deeper.

.9 Required Depth of Root Zone. Under irrigation, the depth of the root zone above which the water table should not be allowed to fluctuate is usually considered to be the distance from the ground surface to the water table at the point midway between drains 48 hours after irrigation. The required root zone varies with climate and crops. Where salts are not a problem, the climate is relatively cool, and irrigation water has low total salts, the root zone can be only 2 to 3 feet deep and give satisfactory crop yields. For soils in arid regions under irrigation where salts are usually a problem, this depth should be at least 4 feet. A 4-foot root depth was used in all drain-spacing computations for the sample farm.

.10 Drain-Spacing Determinations and Drain Location. Drain spacing is determined by the methods described in Chapter 526.4. The following tabulation shows the calculated drain spacings for each of the subar[illegible]. These spacings are rounded to the nearest 10 feet.

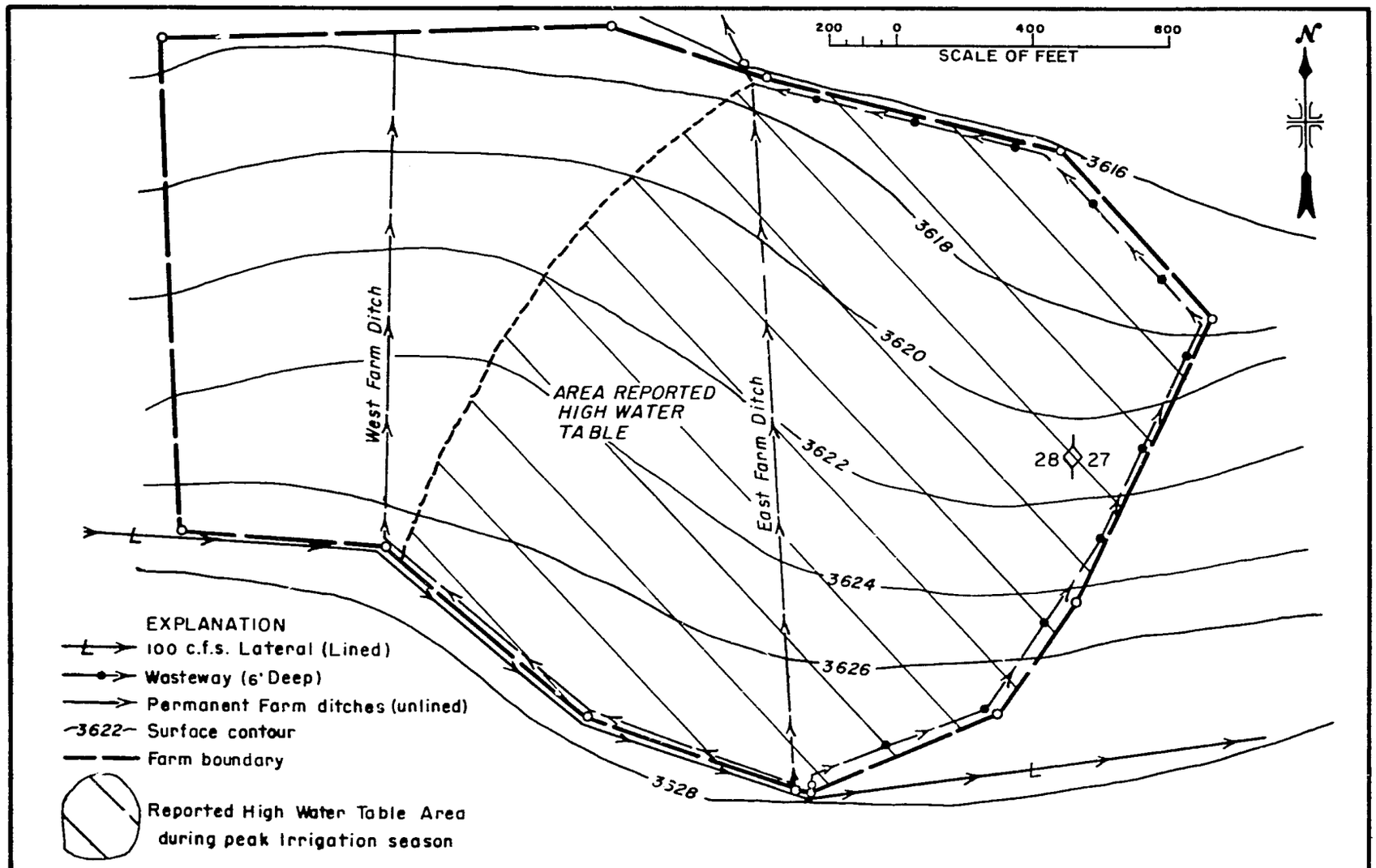
Drain Spacings on Sample Farm

Subarea	Drain spacing, feet
A-1	240
A-2	350
B	170
C-1	250
C-2	300
C-3	350

To be most effective, drains should be located in the more permeable layers. The in-place permeability data obtained at each hole were used in conjunction with the drain-spacing determination in selecting 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 Holes D-3 and E-3. In this case, drains could be located on either side of the less permeable area and still meet spacing requirements.

Figure 9 shows the location of the drains for the sample farm. These drains were spaced to give adequate drainage under present seepage conditions with irrigation and cropping practices remaining about the same. It has been suggested that a probability study of past climatological and cropping conditions could be used in determining the most economical spacing. Although this could be done, seldom, if ever, would the cost of such accuracy be warranted for only a portion of the many indefinite factors affecting drain spacing.

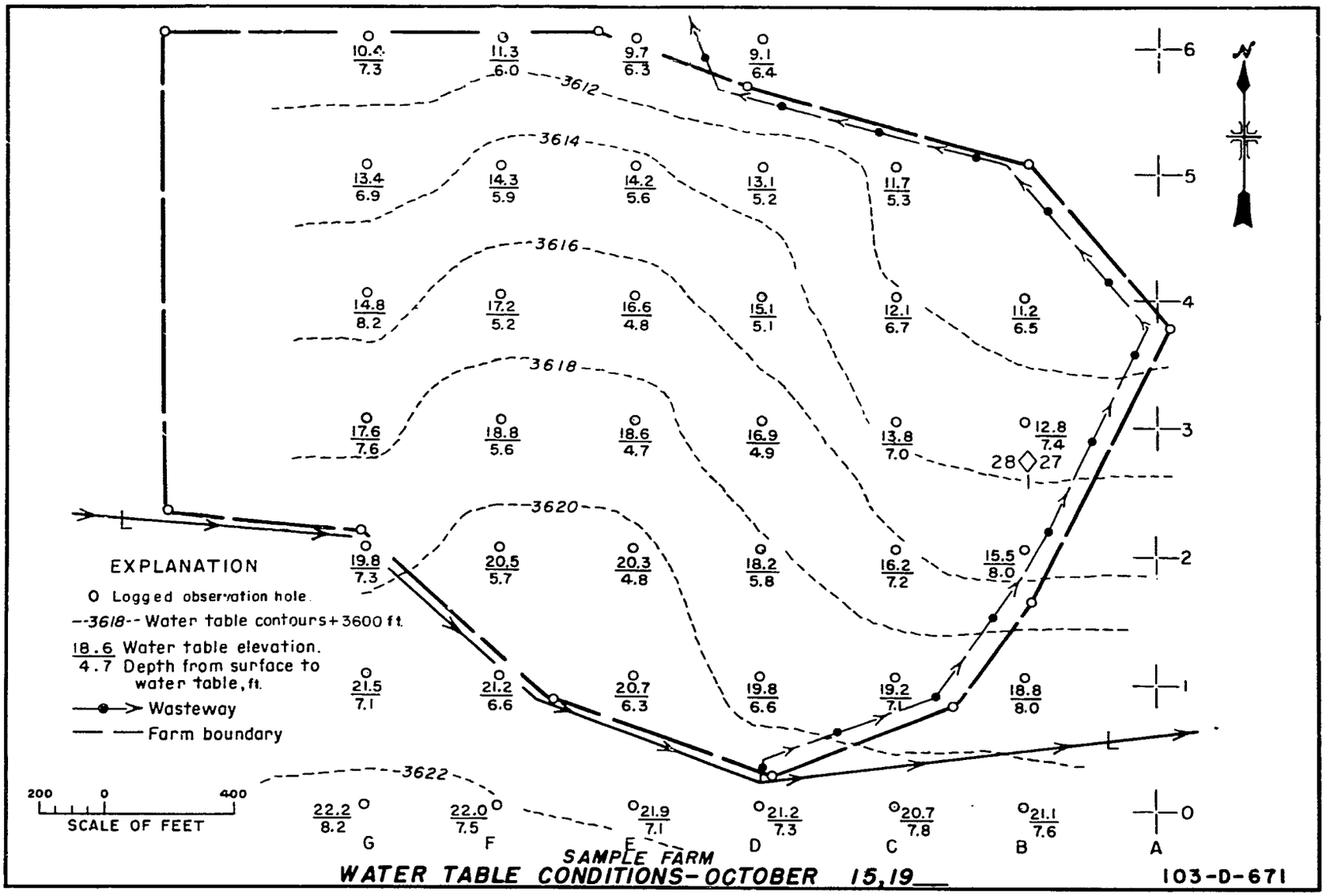
In the final analysis, drainage of waterlogged lands is not an exact science. However, more efficient and economical drainage systems will be constructed if all available data are used during the investigation, location, and design. As permeability is a key factor in almost all phases of drainage work, the more accurate data obtained by in-place tests justify their use.

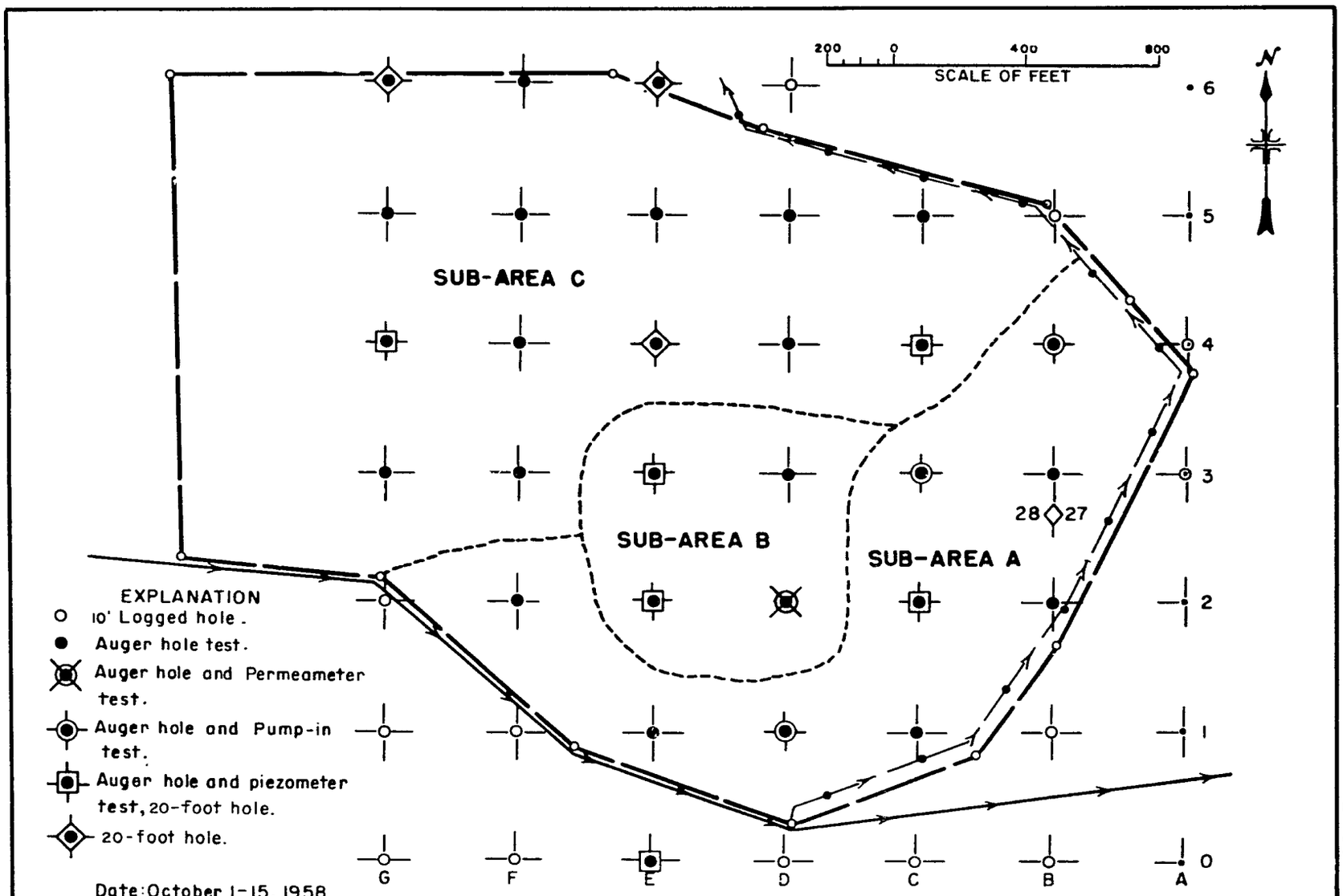


103-D-670 **SURFACE TOPOGRAPHY AND IRRIGATION FACILITIES--SAMPLE FARM**

Figure 1
Par. 526.6.1

Figure 2
Par. 526.6.2

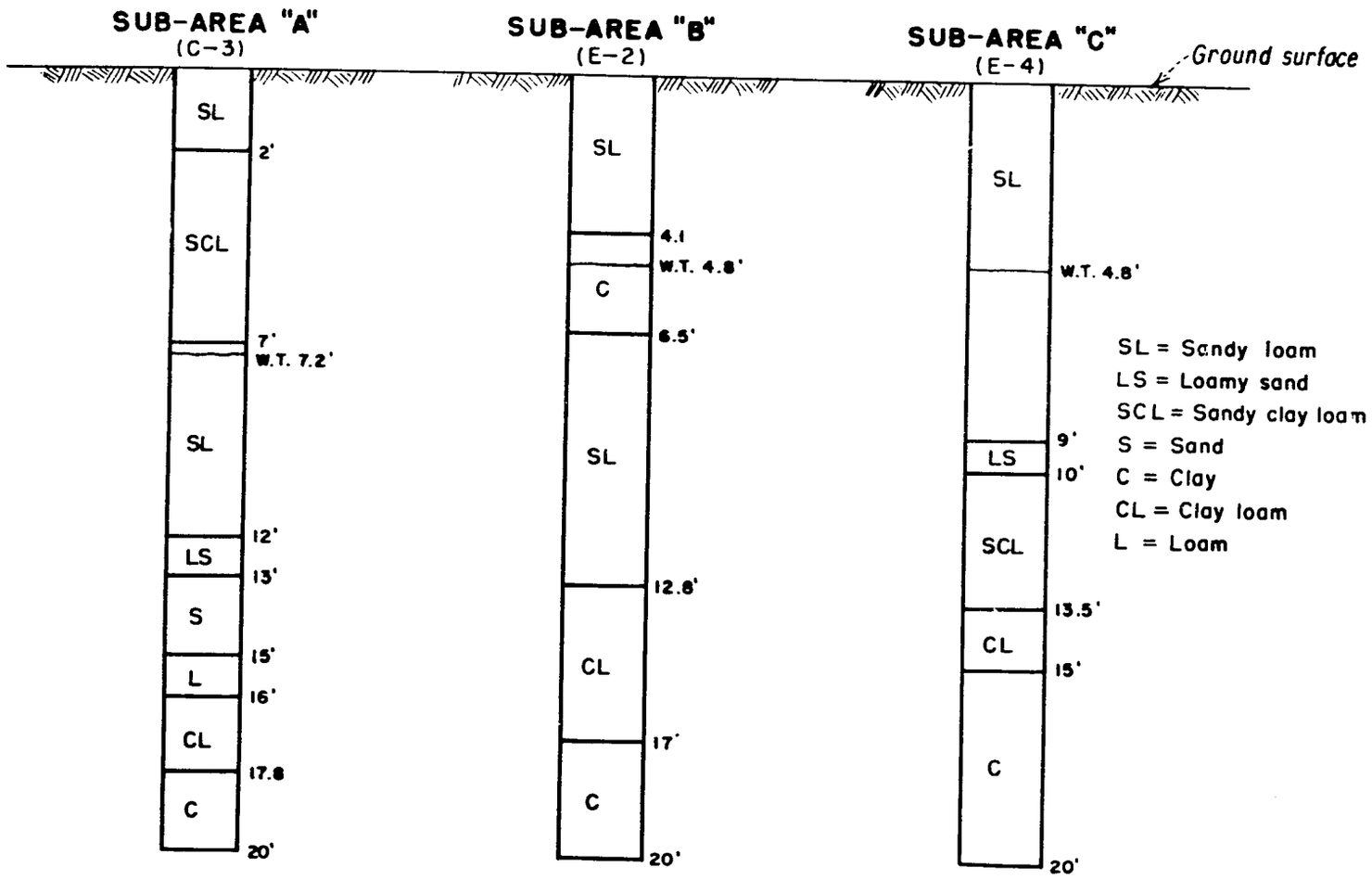




SAMPLE FARM
GRID SYSTEM AND TYPE OF IN-PLACE PERMEABILITY TESTS

103-D-672

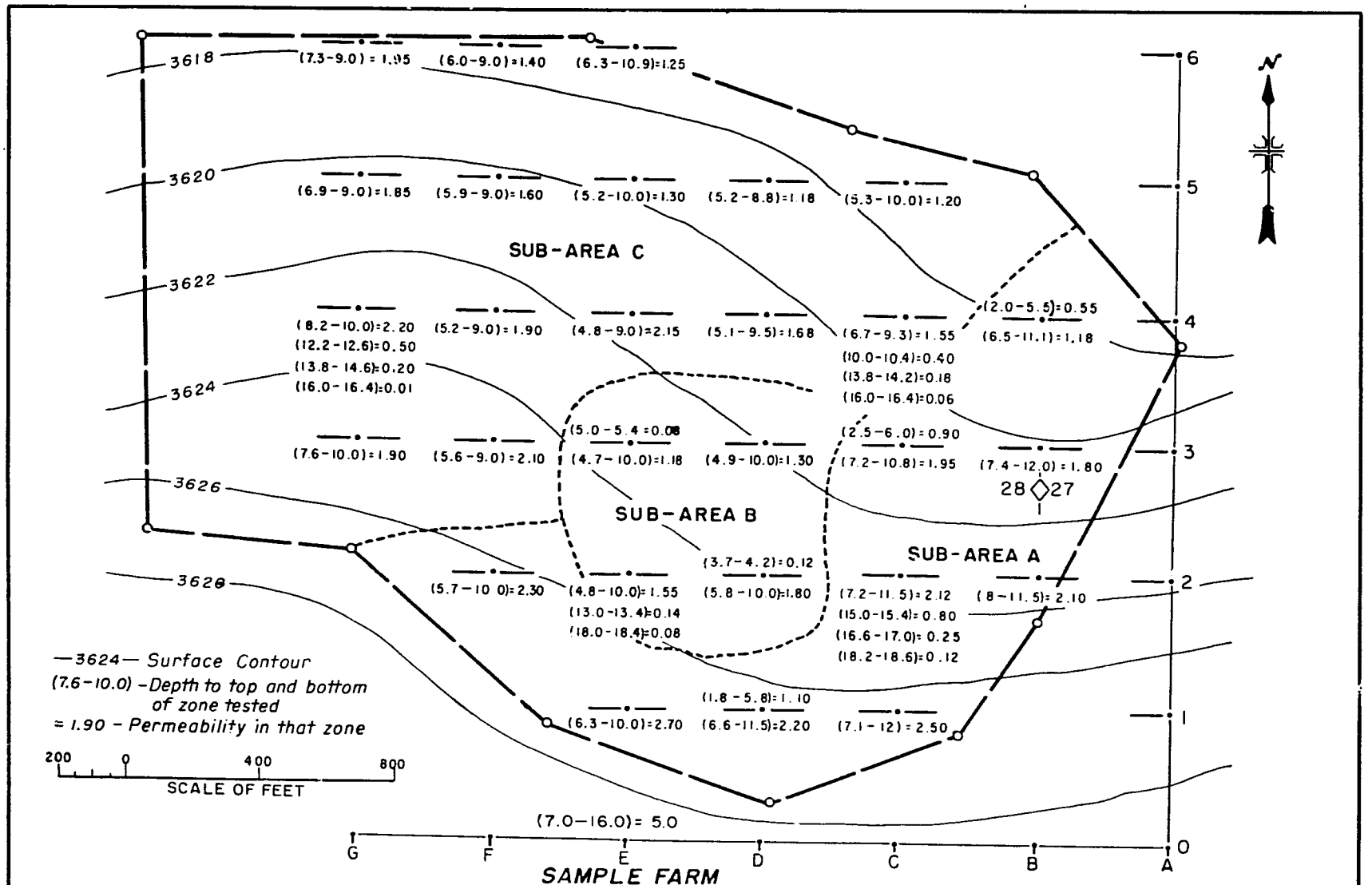
Figure 3
Par. 526.6.2



TYPICAL SOIL PROFILES--SAMPLE FARM

103-D-673

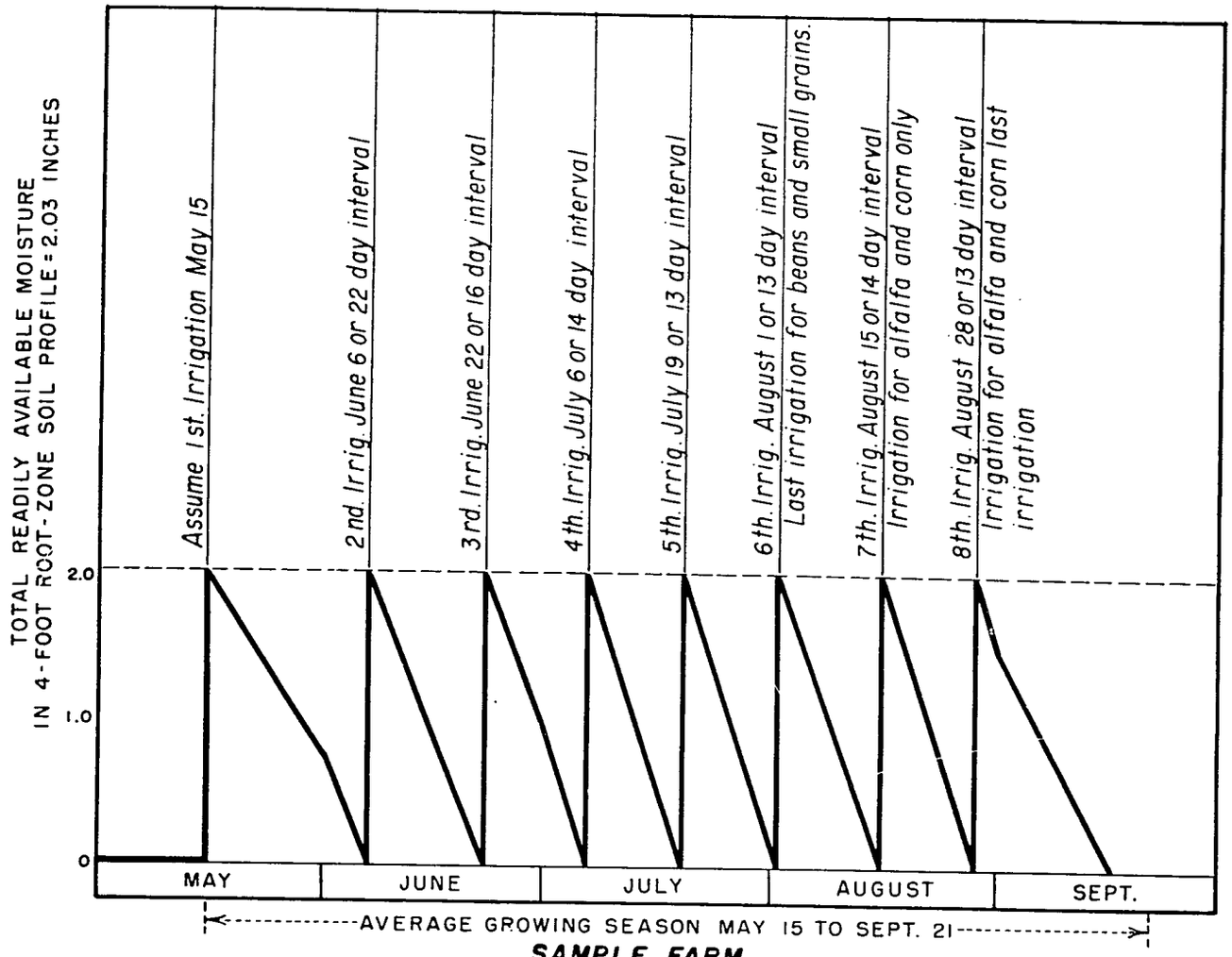
Figure 4
Par. 526.6.2



SAMPLE FARM
IN-PLACE PERMEABILITY MEASUREMENTS--INCHES PER HOUR

103-D-674

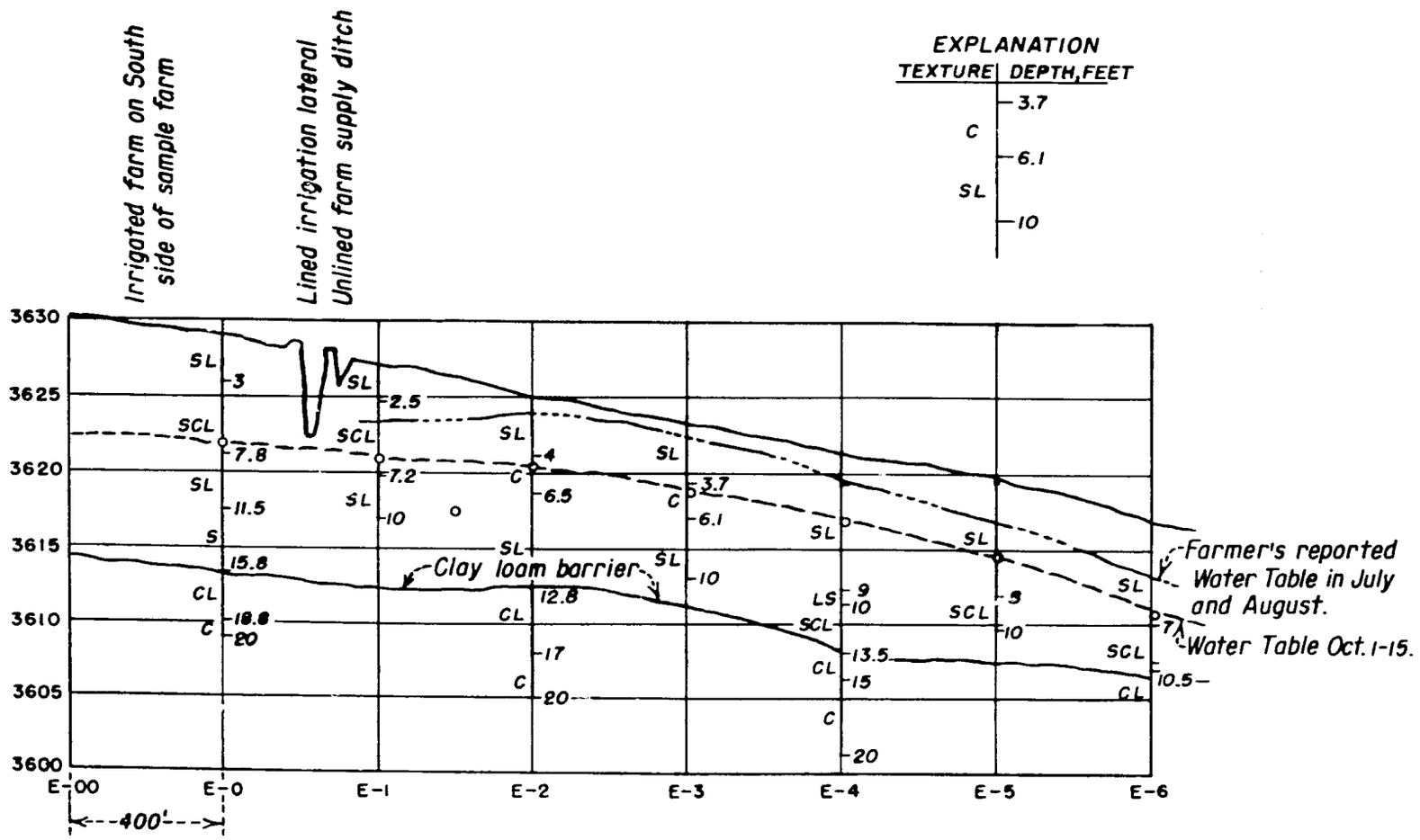
Figure 5
Par. 526.6.2



SAMPLE FARM
AVERAGE YEARLY IRRIGATION SCHEDULE

103-D-678

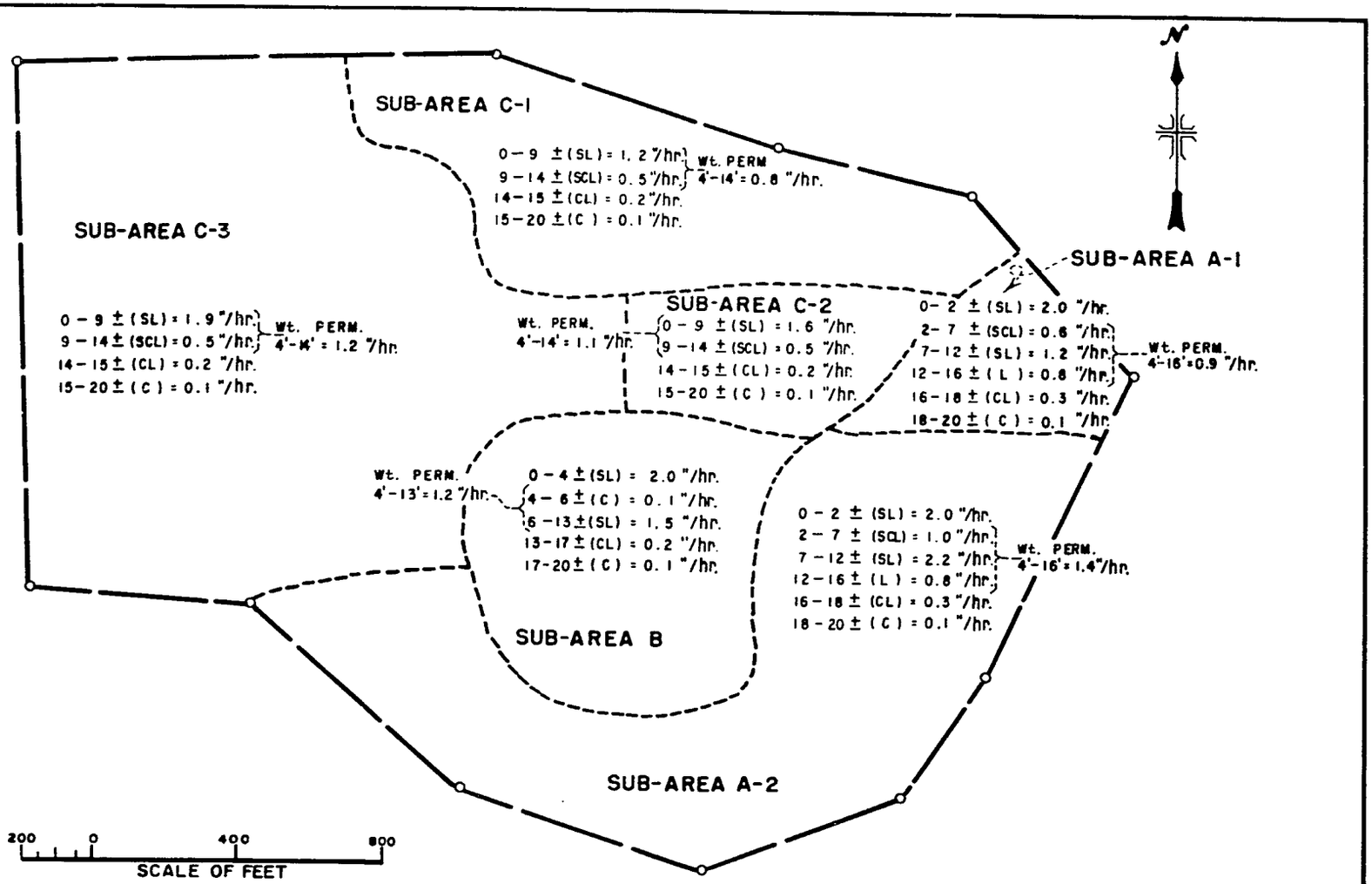
Figure 6
Par. 526.6.4



NORTH-SOUTH PROFILE ON E-LINE--SAMPLE FARM

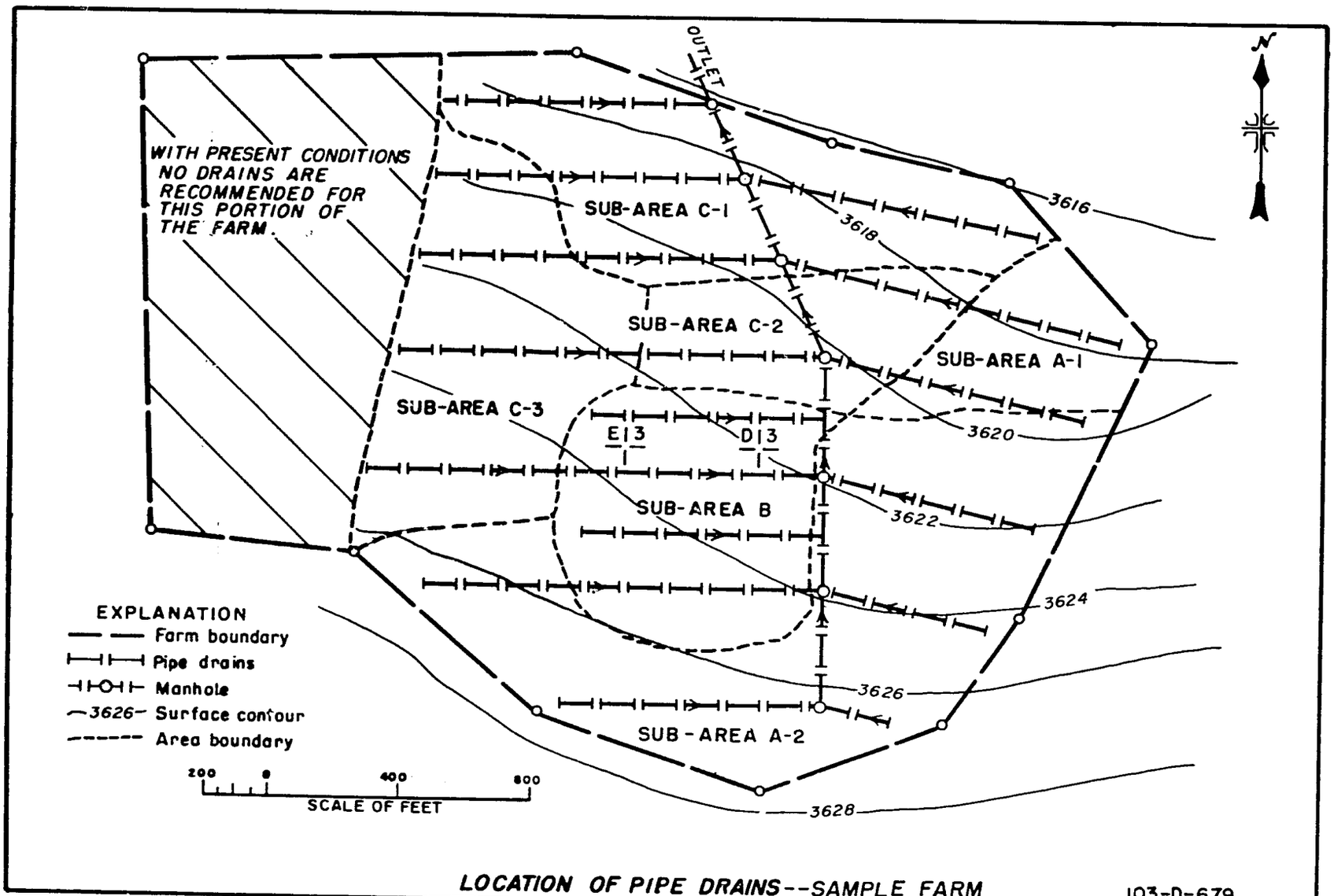
103-D-675

Figure 7
Par. 526.6.6



SAMPLE FARM
AREAS WITH SIMILAR DRAINAGE CONDITIONS AND AVERAGE IN-PLACE PERMEABILITY DATA FOR EACH

103-D-676



LOCATION OF PIPE DRAINS--SAMPLE FARM

103-D-679

Figure 9
Par. 526.6.10

CHAPTER 7 TYPICAL OUTLINE FOR DRAINAGE STUDY

.1 Introduction. It is not possible to prepare a standard comprehensive outline that will be pertinent for all types of drainage studies on any project area, because of the divergence of problems encountered. Each project will have problems peculiar to it which will require that the study be slanted toward solution of those problems. Thus, there will be a difference in the emphasis in the report for each different project. However, certain basic considerations will be required from all projects so that parts of all reports will be similar. In general, the subjects discussed herein will almost all be pertinent to any complete analysis and report although with varying degrees of stress. Following is a typical outline for studies of a specific project which points out the type of study made and which may be helpful in developing an outline for a similar type project.

.2 Outline for Weber Basin Project, Utah.

WEBER BASIN PROJECT
UTAH

CONTENTS

Chapter I Introduction

Purpose of supplement

Material included in supplement

Revisions to the drainage plan

Changes in drainage service area

Changes in project drainage requirement

Changes in farm drainage requirement

Summary of drainage plan

Geology of the East Shore Area

Historical geology

Physical geology

Physiography

Salinity and alkalinity (East Shore Area)

Chapter II Investigations

Investigation program

Water table investigations

Subsurface exploration

Permeability measurements

Test drains

Hooper Pilot Drain

Bountiful A-1 Drain

Syracuse B-5 Drain

Artesian leakage studies

Formulation of drainage plan

Drainage concepts

Project drainage

Farm drainage

Determination of the drainage requirement

Maximum water table

Estimate of deep percolation

Irrigation intervals

Average permeability and depth to barrier

Specific yield

Drain layout

Chapter III Drainage Area "A"--characteristics, problems, and plans

Introduction

Bountiful Lake Plains Subarea

Location and area boundaries

Topography and physiography

Present agricultural development

Water table characteristics

Characteristics of soils and subsurface materials

Origin and occurrence

Structure and permeability

- Existing drainage facilities
- Conditions contributing to present drainage deficiencies
- Drainage deficiencies under project operation
- Drainage requirements

Farmington Lake Plains Subarea

- Location and description
- Topography and physiography
- Water table characteristics
  - Irrigated area
  - Nonirrigated area
- Artesian leakage
- Characteristics of soils and subsurface materials
  - Origin and occurrence
  - Texture
  - Permeability
- Existing drainage facilities
- Summary of conditions contributing to drainage deficiencies
- Drainage deficiency under project operation
- Drainage requirements

Kaysville-Farmington Repayment Block

Location and topography

Agricultural development

Subsurface and water table conditions

Drainage conditions under project operation

(Additional chapters contain similar information on other drainage areas)

FIGURES

Sample auger hole test sheet

Isopiestic profile--pressure before relief well was opened

Isopiestic profile--pressure after 90 days

Isopachous profile--pressure changes after 90 days

Hydrograph of piezometric pressure in various zones

Water table hydrograph and climatic records

Sample drain layout

Hydrographs (Of water table observation wells in all areas)

Multiple Profile (At various locations in different areas)

Hydrographs (Of recorders on various operating drains)

TABLES

Summary of drain spacing data and calculations

526.7.2-4

MAPS

Water table studies

Land classification

Subsurface exploration

General drainage map

GPO 853-937