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16. Abstract

A survey of existing and developing field permeability test methods was conducted in order to identify and specify capable methods to be used in feasibility investigations or performance monitoring for in situ leaching of ore deposits. Geologic settings of leachable deposits are discussed, as well as representative techniques used for the in situ leaching operation. Subsurface investigation techniques including drilling, core recovery, and down-hole inspection methods relevant to field permeability investigations are discussed. Field experience is examined and capable field permeability test methods are identified. Specifications for test borehole preparation, test performance, and methods of analysis are given. The test methods are comparatively rated in a matrix with respect to costs, ease of use, and data effectiveness for a broad range of conditions. Well Pump and Packer test methods receive high ratings. Recommendations are made for needed developments in field permeability test methods and a comprehensive bibliography is given.

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FOREWORD

This report was prepared by Woodward-Clyde Consultants, Western Region, San Francisco, California under USBM Contract Number J0265045. The Contract was initiated under the Advancing Metal and Non-Metal Mining Technology Research Program. It was administered under the technical direction of the Twin Cities Mining Research Center with Mr. Peter G. Chamberlain acting as the Technical Project Officer. Mr. David L. Vila was the contract administrator for the Bureau of Mines.

This report is a summary of the work recently completed as part of this contract during the period October 1976 to August 1977. This report was submitted by the authors on 31 JUL 77

The authors wish to acknowledge the program direction and constructive review of this report by Dr. Ulrich Luscher, Woodward-Clyde Consultant's Principal-in-Charge for this project, and contributions to the report by Ms. Karen Hee, Staff Geologist and Simon Kisch, staff engineer for equipment data.

1.0 TABLE OF CONTENTS

LIST OF TABLES AND FIGURES

2.0 INTRODUCTION

Mining technology in the United States is seeking to develop new methods by which to mine economically low-grade ore deposits. One such method is the in-situ application of a leaching fluid which dissolves the desired mineral in-situ; the pregnant solution is then recovered, the mineral is removed from solution, and the leach fluid is cycled back into the ore body. Prime targets of current in-situ leaching research are uranium and copper ore bodies at depths ranging from the surface to 2,000 feet.

An important parameter used to estimate the ability of a formation to transmit leaching fluids is the in-situ permeability. The in-situ permeability influences the economic feasibility of a particular leaching project, and whether or not development of the formation by blasting or other means is needed. Where blasting methods are employed, post-blasting permeability measurements can help evaluate the effectiveness of the blasting program.

A need for effective field permeability measurement techniques is evident. With this need in mind, the purposes of this report are

1) to review briefly the geologic environments in which potentially leachable uranium and copper deposits commonly occur;
2) to review the various geologic environments in which field permeability tests have been made;
3) to present details, including test preparation, equipment, procedure and analysis, for field permeability test methods; and
4) to present a basis on which the available field permeability test methods may be compared and the most suitable test method(s) selected.

The report is comprehensive with regard to borehole preparation, test method specifications, and test method analysis procedures. Although careful review of these sections is recommended for a comprehensive understanding of the acceptable test methods, it is not strictly required for the selection of an appropriate test method. Rather, a flow scheme is presented in this section which allows the investigator to select one or more suitable methods based on three figures given in the report. A diagram of the flow scheme is presented in Figure 2-1.

Initially, the investigator should determine an anticipated range in permeability based on the geologic environment of the proposed leaching site. Figure 3-1, which illustrates approximate permeability coefficients for typical rocks and soils, can be used for this purpose. Then Figure 5-1, which presents documented field test permeability ranges, can be used to determine which of the acceptable test methods have been used successfully in the anticipated permeability range. One or more test methods can then be tentatively selected based on the reported field experience. Finally

Figure 10-1, the Field Test Method Capability Matrix, provides a number of criteria by which the tentatively selected field test methods may be compared. After the field test method most favorable for the investigator's specific leach site has been selected, the relevant sections of the report dealing with the specific test method(s) can be reviewed to familiarize the investigator with necessary aspects of borehole preparation, test method procedure, equipment, and analysis.

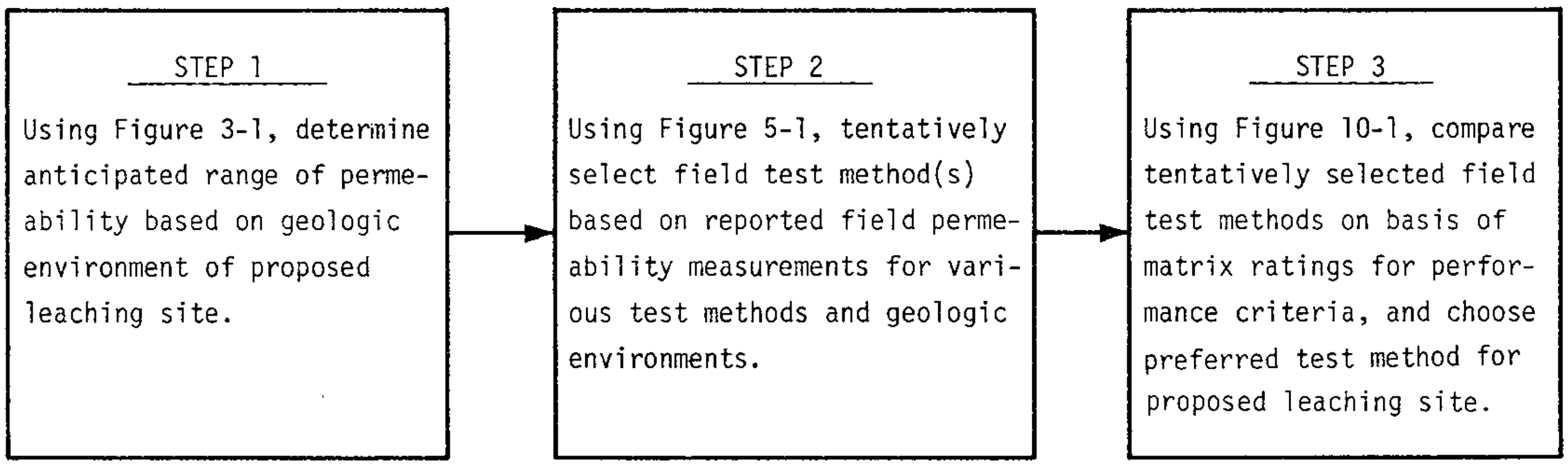


Figure 2-1 **FLOW SCHEME FOR FIELD TEST METHOD SELECTION**

3.0 GEOLOGY OF LEACHABLE DEPOSITS

3.1 INTRODUCTION

Leachable ore deposits of economic importance may occur in many rock types. The physical and chemical characteristics of the host rocks may determine whether solution mining can be carried out economically with present technology. Deposits of salt and other evaporites, sulfur and potash have been solution mined in the past whereas the solution mining of metals is a more recent development. The metals for which solution mining is presently considered economically viable include copper, uranium, lead, zinc, and nickel. This report concentrates on two of these, copper and uranium. A review of the geology of copper and uranium deposits illustrates the rock types and conditions most likely to be encountered in leaching operations.

The hydrology of each ore deposit is unique. A thorough understanding is crucial to successful leaching operations, particularly uranium leaching which is usually conducted below the groundwater table. Geologic factors that affect the natural flow of groundwater as well as the leaching solution include faults, fractures, joints, bedding plane attitudes, and rock types (Whiting, 1976).

3.2 URANIUM DEPOSITS

3.2.1 Background

Uranium is a silver gray metal which oxidizes rapidly in contact with air. The average concentration of uranium in the crust is about 2ppm. Granitic rocks may contain up to 4 ppm, however, uranium is found in varying amounts in virtually all rock types.

Near-surface deposits are usually mined by open pit methods. Where the overburden exceeds a few hundred feet, underground mining is usually preferred for economic reasons. However, several drawbacks exist for these conventional mining methods. In-situ leaching overcomes the environmental problems of open pit mining such as disruption of scenic landscapes and increased sedimentation downstream of mining operations, and the safety hazards of underground mining, which include exposure to radon, a radioactive gas. In-situ leaching of uranium, which has proven successful at several locations, is discussed in detail in Section 4.2.

Uranium enters the geologic cycle from magmas. As a magma crystallizes, uranium, in the tetravalent state, cannot enter the crystal lattices of the common rock-forming minerals because of its large size. Instead, the uranium forms an intergranular film on the rock forming minerals, enters accessory minerals, such as zircon, apatite and monzonite, or forms its own minerals such as uraninite. Uranium also becomes concentrated in late magma differentiates, forming uranium minerals in pegmatites and veins. The uranium bearing accessory minerals generally resist chemical weathering and are deposited unaltered in detrital sediments, resulting in highly concentrated placer deposits. Uranium formed as inter-granular film is readily oxidized to the hexavalent state, which is readily soluble in water and easily transported by the circulating groundwaters. Deposition occurs when the circulating waters encounter a reducing environment. The dissolved uranium may be absorbed by clay minerals and carbonaceous material, and that which reaches the oceans may be precipitated in phosphatic sediments, taken up by organisms, or absorbed by carbonaceous muds (Finch et al., 1973).

In the United States the principal source of minable uranium is continental and marginal marine sandstones and associated rocks in the western states and in the Texas Coastal Plain (Finch et al., 1973). Less than 2% of U.S. uranium production is from vein deposits. Other sources of uranium include some granites, shales, lignites and phosphate deposits.

3.2.2 Peneconcordant Deposits

Most ore bodies in sandstone are tabular masses that lie nearly parallel to the bedding of the sandstone units, and are termed "peneconcordant deposits." Most uranium-bearing sandstones were formed by fluvial or deltaic processes. The sandstone units occur as lenses interbedded with mudstone in deltaic deposits and as lenticular beds laid down in stream channels. A few sandstone deposits were formed by wind action, forming clean massive layers.

Although uranium deposits may occur in sandstone units of all ages except Silurian (Finch, 1967), a majority (60%) of the U.S. uranium resources are deposited in sandstones of late Jurassic age.

Ore bodies forming tabular masses usually have the long axis of the ore body parallel to the long axis of the sandstone lens. Roll-front deposits form curved surfaces that cut across the bedding. The deposits contain concentric bands of rich and lean ore and are thought to have formed at the interface between an oxidizing and reducing environment. As the interface moved with the hydrologic flow, concentric bands of uranium were deposited. Ore bodies range in size from a few feet across and inches thick containing a few tons of ore, to hundreds of feet across and tens of feet thick and containing millions of tons of ore.

Peneconcordant deposits occur most frequently in stratigraphic units of mixed lithology. Uranium deposits occur in sandstone units that are interbedded with mudstone units. Most uranium-bearing sandstones are fine to medium grained, are moderately to well sorted (minimum variation of grain size) and are of quartzose, arkosic or tuffaceous composition. The sandstones are cemented mainly with clay and carbonate minerals. The uranium minerals may fill the pores of the host rocks, replace plant fossils partially or completely, or replace parts of sand grains and the cementing minerals. Uranium also occurs in asphaltite, a noncellular, organic material that is present in some sandstones.

Uranium can occur in a wide range of sandstone types with considerable permeability variations. Well sorted, clean, fractured sandstones will generally have the highest permeabilities; poorly sorted, clay rich sandstones the least. The deposition of the uranium minerals in the pore spaces of the host rock tends to decrease its permeability (Shock and Conley, 1974).

3.2.3 Other Uranium Deposits

Uranium may also occur in vein deposits, fissure fillings in faults, joints and fractures. These deposits are thought to have formed by hydrothermal solutions and in the United States are mainly late Cretaceous to early Tertiary in age. Uraniferous igneous formations such as the Conway granite in New Hampshire are another source of uranium, although concentrations are much less than in sandstone deposits. Uranium is also associated with the marine sediments of phosphate rocks and black shales. Extensive phosphorite beds in the Phosphoria Formation occur in Idaho, Montana, Utah and Wyoming. Most of the uranium-bearing black shale in the United States occurs in the Chattanooga Shale in Tennessee, Kentucky and Alabama (Finch et al., 1973).

3.3 COPPER DEPOSITS

3.3.1 Background

The United States has been the world's largest producer of copper since 1883. In 1968 20% of all the copper was mined in the United States. Most of the copper produced comes from five western states, Arizona, New Mexico, Nevada, Utah and Montana. Lesser amounts are mined in Michigan and Tennessee (Ageton and Greenspoon, 1970).

Most of the copper mining has been by open pit and conventional underground methods. Although the practice of leaching copper deposits has been known since medieval times it has not been used extensively. Leaching operations have been limited to old workings or caved areas of mines where it was uneconomical to recover the copper by other methods. An early example of in-situ leaching of copper ore was that carried out in caved stopes by the Ohio Copper Co. of Utah at Bingham Canyon, Utah during the years 1922-1925 (Anderson and Cameron, 1926).

The copper-bearing minerals of importance in ore-grade deposits are the oxidized copper minerals azurite, malachite, chrysocolla, tenorite and cuprite and the sulfide minerals chalcocite, covellite, bornite and chalcopyrite (Fletcher, 1974).

Cox et al. (1973) have described five major types of copper ore deposits. In order of decreasing abundance, they are (1) porphyry, vein and replacement deposits associated with felsic intrusive rocks, (2) sedimentary deposits, (3) massive sulfide deposits in volcanic rocks, (4) nickel-copper deposits in mafic intrusions, and (5) native copper ores.

3.3.2 Porphyry Deposits

Porphyry deposits account for two thirds of the world's known copper resources. In the United States 90% of the copper mined is from porphyry deposits (Cox et al., 1973). The U.S. porphyry copper deposits are located in the Basin and Range physiographic province of the western states. Deposits with economic concentrations of copper-bearing minerals are associated with the intersections of major orogenic belts and fault zones (Schmitt, 1966) while the richest copper districts occur at "triple-junctions" or more complex intersections. The large porphyry deposits in Arizona, for example, occur at the intersection of the Wasatch-Jerome orogen, the Texas lineament zone, and two strong northeast lineations. Tectonic activity may have faulted and fractured the rock mass in areas resulting in relatively highly permeable zones. Schmitt (1966) concludes that zones of crustal weakness occur at these intersections and have existed for long periods of time, closing and reopening from time to time as the tectonics of the region changed. These zones of weakness become the pathways by which igneous intrusions and associated porphyry intrusions work their way upward in the crust. Mineralizing fluids derived from the mantle and/or lower crust also rise upward through these same zones.

The composition of the copper-bearing porphyry rocks ranges from acidic (alaskite, granite, rhyolite) to a more basic diorite-andesite composition (Stringham, 1966). The ages of the porphyries and their associated igneous rocks range from Mesozoic to early Tertiary (Schwartz, 1966). On the average 70% of the ore is in the porphyry body and 30% is intruded into the surrounding rocks as vein and replacement deposits (Cox et al., 1973). Wide zones of closely and irregularly fractured country rock called crackle zones surround almost all porphyry deposits. The fractures provide avenues for leach solutions to the peripheral porphyry deposits, and are an important consideration when evaluating leach feasibility. Hydrothermal alteration, which is present in all porphyry copper deposits (Creasey, 1966), is usually so pervasive that it is difficult to find specimens of fresh rock for comparison.

The history of a typical porphyry copper deposit has been summarized by Schwartz (1966) as follows:

1) Intrusion of acidic or intermediate stock.
2) Cooling of the upper part of the stock.
3) Fracturing and shattering of part of the stock and to a lesser extent the surrounding rock.
4) Fluids rise into the fractured rock, hydrothermally altering it and depositing sulfides. The sulfides fill the cracks and replace parts of the wall rocks.

5) Erosion and oxidation of sulfide minerals occur. Copper is carried downward by groundwater and deposited near and below the water table (supergene enrichment).

6) The outcrop, leached, and supergene ore zones move downward as erosion proceeds.

3.3.3 Sedimentary Deposits

Sedimentary or stratabound deposits of copper may account for some of the world's largest single reserves but have not been exploited in the United States. Future U.S. discoveries of rich copper deposits may likely be of the sedimentary type, as the major porphyry bodies have probably been detected. Stratabound deposits have been difficult to recognize in the field and their origins are just beginning to be understood (Cox, et al., 1973).

Cox et al. (1973) divide sedimentary copper deposits into three types based on a genetic classification.

Type 1. Rocks of Precambrian age, formed when atmospheric oxygen began to become abundant and to mobilize copper that was previously stable.

Type 2. Syngenetic marine deposits, formed on the sea floor by precipitation or absorption on organic matter.

Type 3. Deposits formed by reaction of connate brines or meteoric groundwater with strata such as black shale, carbonaceous sandstone or carbonate rock.

In the United States known sedimentary copper deposits occur in the Precambrian Nonesuch Shale of Michigan and northern Wisconsin. About 5% of U.S. copper production comes from a single shale bed which is 1 to 8 meters thick. Unless highly fractured, the shale will have an extremely low permeability. Anomalously high concentrations of copper occur in sedimentary rocks of Precambrian age in the Belt Supergroup of Idaho, Washington and Montana. The copper is most common in strata of green color and occurs as disseminations, discrete blebs and veinlets (Harrison, 1972). In the Southwest, copper deposits are associated with late Paleozoic or early Mesozoic red sandstones. The copper most commonly occurs in the light to dark gray sandstones interbedded with red sandstones. The Flowerpot Shale of Permian age in southern Kansas and western Oklahoma contains disseminated copper sulfide minerals in thin beds of gray shale. Interbedded sequences of sandstones, shales and siltstones present a definite problem with respect to in-situ leaching. Large variations in permeability with depth may exist, causing incomplete penetration of acid fluids to the ore. Some means of induced fracturing would most likely be necessary for a leach operation.

3.3.4 Massive Sulfides, Nickel-Copper Ores, and Native Copper Ores

Massive sulfide copper deposits in lavas and pyroclastic rocks of submarine origin form a small but locally important part of the world's copper resources. Such deposits, however, are not found in the United States. Other important resources include byproduct copper, which is obtained from commercial nickel deposits in mafic rocks. Major deposits are located in the U.S.S.R. and Canada, although the U.S. does have nickel-copper ore bodies in Maine and Alaska. Native copper deposits have been mined since 1845 from the Keweenaw Peninsula of Michigan. The copper occurs in amygdaloidal flow tops and conglomerate beds in the Portage Lake Volcanics. The metal was introduced after deformation of the host rocks, filling vesicles, fractures and interstitial pore spaces. Flow patterns can be very complicated in such regions and actual permeabilities might be determined only after a careful test program including geologic and hydrologic investigations.

3.4 PERMEABILITY RANGES FOR VARIOUS SOIL/ROCK TYPES

Figure 3-1 presents a rough estimation of permeability as a function of general geologic conditions. The values and ranges established are not exclusive, and should not be viewed as such. The purpose is to illustrate a likely relative variation in permeabilities through a broad range of potential geologic environments. Actually, the in-situ permeability of a region is likely to be highly variable, and may be influenced by many factors, including depth of deposit, in-situ stresses, geologic history of the deposit, fracture intensity, fracture aperture, filling material of discontinuities, cementing agent and extent of weathering and alteration.

Consequently, Figure 3-1 should only be used as a guide for the selection of an appropriate field test method.

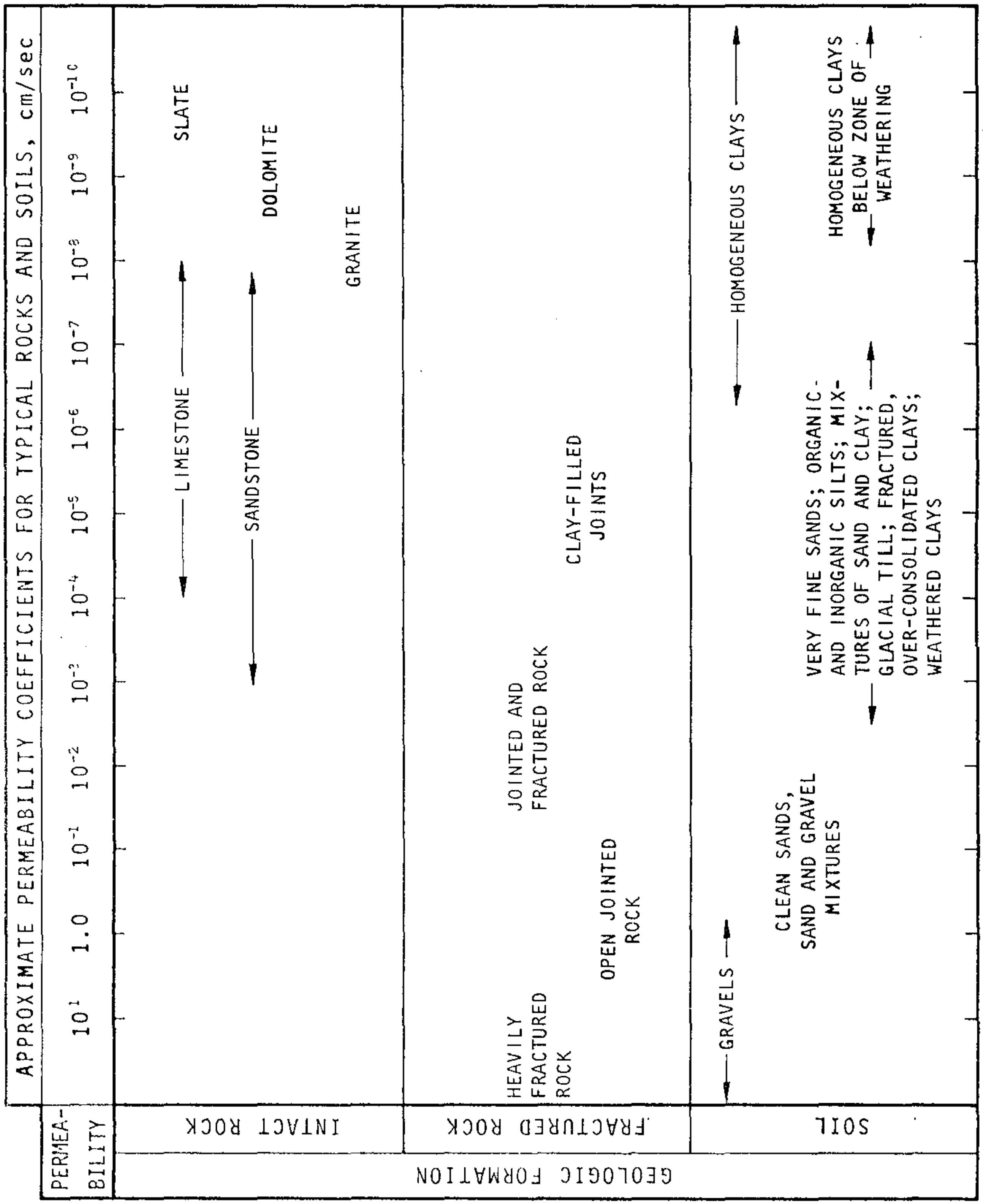


Figure 3-1 RANGES OF PERMEABILITY FOR GEOLOGIC FORMATIONS
(Milligan, 1975; Hoek & Bray, 1974)

Figure 3-1

4.0 IN-SITU LEACHING OPERATIONS

4.1 BACKGROUND

In recent years there has been renewed interest in the solution mining of such metals as copper, uranium, lead, zinc and nickel. Solution mining offers some important advantages over open pit or underground mining methods such as reduced environmental damage and economic recovery from low grade and deeply buried ores. Solution mining also eliminates safety hazards to which open pit and underground mine workers are exposed. Potential disadvantages for solution mining include contamination of groundwater, incomplete recovery of the leached mineral and the nonrecovery of associated minerals. Also, the need for large volumes of water presents problems in arid regions with growing populations.

4.2 LEACHING TECHNIQUES

Leaching operations usually include the general tasks of applying a leaching fluid to the ore, dissolving one or more minerals, collection of the pregnant solution, precipitation of the mineral and recycling of the fluid. The design and production of each leaching operation and the achievement of maximum recovery possible depend on many factors, such as the type of mineralization, the shape, size and orientation of the ore body, the physical and chemical properties of the ore and host rocks and the position of the water table. The permeability of the rocks to be leached is a key parameter, and it has been demonstrated in the field that permeability can be increased by blasting before leaching operations begin. The careful design and installation of injection and production wells with respect to the in-situ permeability is an important consideration toward maintaining an economic level of recovery.

Since the techniques used in the leaching of copper and uranium are somewhat different they are discussed separately. New techniques are being developed as in-situ leaching operations are tested in different environments. With increased use of in-situ solution mining the present techniques will likely be improved and wider application of the techniques may be possible in the future.

4.2.1 <u>Uranium Leaching</u>

Uranium can be leached from ore deposits in sandstones by application of acid or alkaline solutions to the mineralized zone. Since most uranium deposits are located below the water table, the hydrology of the area surrounding the ore body must be studied carefully. Containment of the leaching solutions by the natural groundwater is necessary for successful below-water table leaching (Anderson and Ritchie, 1968).

Leaching takes place in two steps; the tetravalent minerals are first oxidized to the hexavalent state which is readily soluble, and are then complexed to form the relatively stable tricarbonate. The tricarbonate stays in solution during transportation to the production well (Hunkin, 1975). The magnitude and direction of the flow are controlled by the pressure and volume of injected fluid at the input well, the natural groundwater flow regime, and the drawdown at the production well. At Shirley Basin, Wyoming, three injection wells were placed upstream of the natural groundwater flow and one production well downstream. The leach solution moved through the ore zone and was recovered at the production well about 25 feet away (Anderson and Ritchie, 1968). Small scale leaching operations such as at Shirley Basin continue for about a month whereupon new wells are installed in another part of the field. In France, low-grade uranium ore from the Escapiere district is leached by supplementing the natural rainfall with pond water and collecting the solution at the bottom of slopes. The action of anaerobic bacteria on pyrite in the granite forms a dilute sulfuric acid. As this acid seeps through the broken ore, the uranium in the granite is leached out (Anonymous, 1967).

4.2.2 Copper Leaching

Copper was first successfully recovered by leaching at Bingham Canyon, Utah, by the Ohio Copper Company of Utah between 1922 and 1925. The uniformly broken porphyry ore in block caved areas was leached with water (Anderson and Cameron, 1926). Copper leaching of the unbroken ore was also attempted but copper recovery was too low, and deep blasting was used to prepare the leach area. Usually, a solution of sulfuric acid is applied to the top of the ore zone by ponding, sprinkling by rainbirds, perforated pipes or injection wells. As the solution slowly percolates through the ore, the copper oxide minerals are dissolved. The pregnant solution is then collected below the deposit, sometimes in old adits left from previous mining operations. The copper is then precipitated on shredded scrap iron and the sulfuric acid solution recycled. With this procedure the water table must be kept below the collection level.

4.3 PERMEABILITY ENHANCEMENT BY BLASTING

An important factor in a leaching program is the permeability of the zone to be leached. In some applications, the host rock may be sufficiently permeable to allow the leach program to be conducted with no further subsurface work. Alternately, the in-situ mass may be relatively impervious to the leach solution. In this case, it may be possible to induce the desired permeability artificially through a variety of blasting techniques. Various references in the literature consider the application of blasting prior to in-situ leaching (Rosenbaum and McKinney, 1970; Longwell, 1974; Ward, 1974; McLamore, 1974; Lewis et al., 1974; D'Andrea and Runke, 1975; Steckley et al., 1975).

The literature is not specific as to permeability requirements necessary for successful leaching programs. Parameters such as Rock Quality Designation (RQD), percent core recovery by length and by weight, rock block size (Steckley et al., 1975) and fragment size distribution (D'Andrea et al., 1976) have been used to assess the effects of blasting on the ore rock. Although all of the above mechanical parameters may be related to the penetrability of the leach fluid, measuring the permeability of the rock mass before and after induced fracturing is recommended as a more direct approach.

The ease of making permeability measurements before and after blasting can be optimized by attention to the design of test and blast hole locations. Boreholes may be able to serve a dual purpose, particularly in conjunction with well-pumping permeability tests (see Sections 7.1, 7.6). A drilled hole may serve for introducing a blasting agent and as an observation well for pump tests performed before and after blasting. Varying degrees of post-shot hole redevelopment may be required depending on such factors as the depth and size of the blast, the quality of the rock mass, the water table level, and the distance of the drilled hole from the proposed central (pumped) well. The blasting drill hole may also serve for other single-borehole permeability test methods. In either case, new post-shot boreholes for permeability tests might be required due to excessive blast damage in the original holes. Regardless of the degree to which the drilling program for blasting and permeability testing can be overlapped, field testing of permeability before and after blasting is recommended as an effective means of assessing the impact of blasting on the ore body and the enhanced potential of the ore body for in-situ leaching.

4.4 RELATIONSHIPS BETWEEN LEACHING PERFORMANCE AND IN-SITU PERMEABILITY

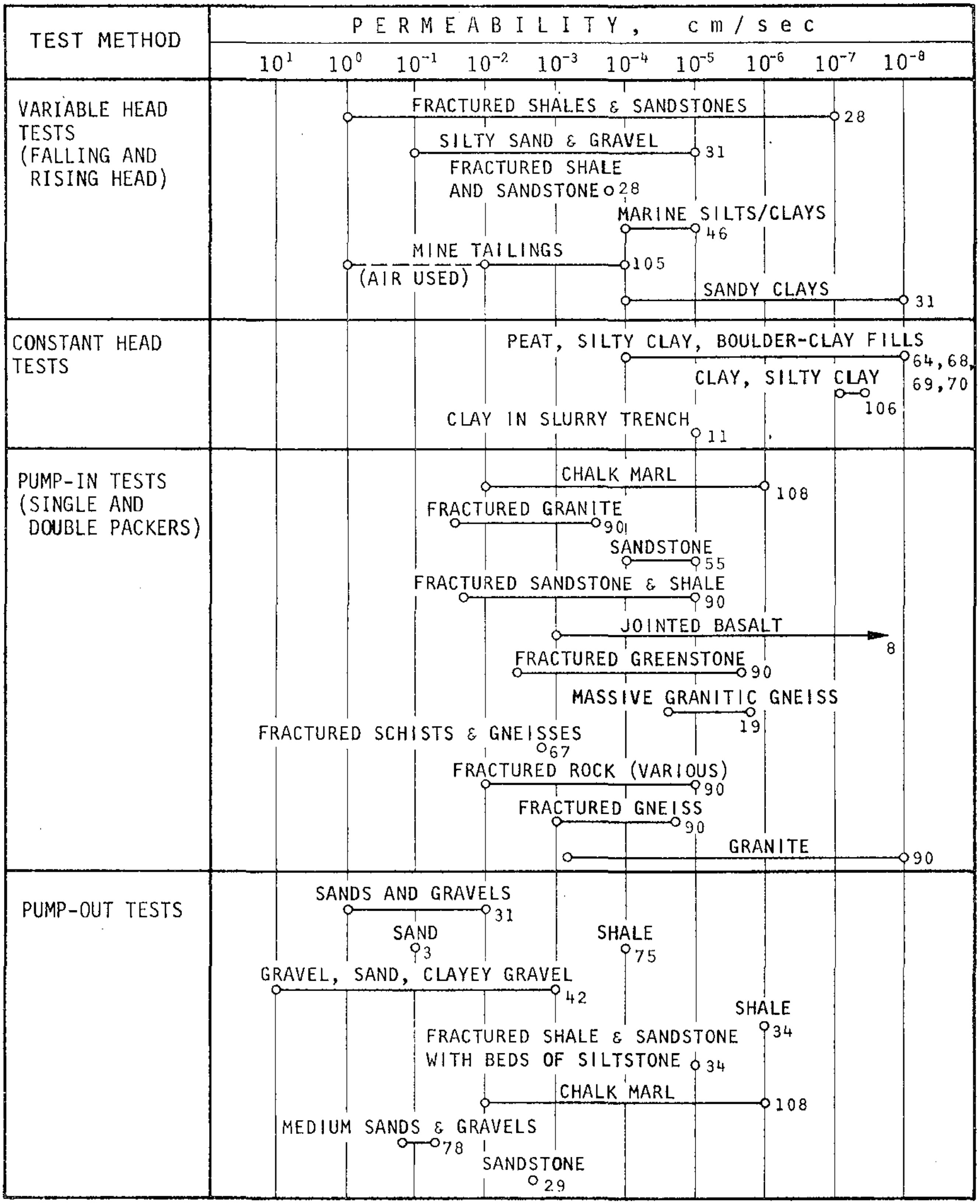
The most important physical characteristic of an ore deposit to determine its leachability is its permeability. The leaching solution must come in contact with as much of the mineral being extracted as possible. Many ore deposits are repeatedly blasted to increase permeability. Factors which affect the permeability of the ore deposit include: rock type, cementation, alteration, pore space, faults, fractures, joints, large or small scale folding, the presence of layers or lenses of impervious material, and the permeability of the host rocks. Leaching operations may cause the permeability to either decrease or increase as leaching progresses. Pore spaces near the wells may be clogged with fines, and the formation of clay along fractures may reduce leachability. However, disintegration of the intact rock surfaces along tight joints and fractures may increase the permeability. Such phenomena require future study.

5.0 FIELD PRACTICE IN IN-SITU PERMEABILITY MEASUREMENT

This section of the report considers field practice in in-situ permeability measurement techniques. Case histories presented in the literature have been reviewed, and where numerical results of permeability are available, such information has been recorded. Specific values or ranges of permeability obtained in the respective test programs are presented in Figure 5-1. The data has been grouped according to the general test method, and the geologic environment of each test has been entered on the graph.

The case histories illustrate an approximate distribution of geologic environments and permeabilities as well as general applicability of a given test in various geologic media. Although the review does not define exact upper and lower bounds for permeability test method application, nor provides a complete estimation of test suitability in terms of rock type, the plot does provide an experience base to facilitate selection of the appropriate test method(s).

Once the investigator determines the general geologic regime of the ore deposit, an approximate range of permeability can be obtained from Figure 3-1. Given this range of permeability, the investigator can proceed to Figure 5-1, wherein one or more tests may be identified as a possible choice. Then Figure 10-1, which is discussed in detail in Section 10, can be used to further identify a suitable test method.



<u>NOTE:</u> Numbers denote respective reference

Figure 5-1 REPORTED RANGES OF FIELD TEST METHODS FOR PERMEABILITY MEASUREMENTS

6.0 GENERAL SPECIFICATIONS FOR FIELD TEST PREPARATION WORK

6.1 INTRODUCTION

This section of the report contains discussions, recommendations, and general specifications for preparation work for permeability field tests. Preparation work for a field permeability measurement program should include drilling and sampling of the formations down to the level of the ore body to be leached. Where the exploratory drill holes are to be used for subsequent field permeability measurements, various constraints relevant to the drilling techniques and drilling fluids must be observed and are described in Section 6.2.

6.2 DRILLING TECHNIQUES

6.2.1 Drilling Tools and Drilling Methods

The formation in which a hole is to be driven will affect the proper choice of drilling method. The following sections describe a few appropriate drilling tools and drilling methods for various types of soil and rock formations to be penetrated and sampled.

6.2.1.1 Unconsolidated Formations - An easy method of penetrating unconsolidated, unsaturated formations to depths of up to about 100 feet is with a continuous-flight auger drill. Such drills may be truck or crawler-track mounted. The continuous-flight auger may be fabricated with a hollow stem to permit insertion of drive-spoon samplers through the auger to the base of the hole in order to retrieve short samples ahead of the auger. While advancing the auger a drilling-log is commonly kept to record the type and visual properties of the soil cuttings with depth.

A much more rapid method of drilling unconsolidated formations is by use of Becker percussion-hammer drills, used extensively at Tarbela Dam (O'Rourke, 1972). A typical tool comprises a double-walled steel drive string with a 6-5/8-inch outside diameter and a 4-7/16-inch inside diameter. Other sizes are available. The drive string has a specially designed cutting-head, which comprises several breaker-teeth with upward-directed air-jet ports. Compressed air is fed down to the cutting head through the double-walled drive string, which is driven down by an air-hammer. The compressed air lifts the cuttings up through the inner opening of the drill string. This drilling method is attractive in that drilling and casing the unconsolidated formation is carried out simultaneously, and well casings, piezometer pipes or tubes, and backfill material may be installed through the drive string before or during withdrawal of the drive string. Up to 18-inch cased holes may be driven using a special hammer which simultaneously drives the casing and an inner drive string. This system was used on the trans-Alaska pipeline in 1975 and 1976. The casing may be withdrawn partially or totally for permeability test purposes by special spider-frames and hydraulic jacks that can be furnished with the rigs.

The wash boring method is a simple and frequently used procedure for advancing a borehole in unconsolidated formations. Initially, casing with a diameter of 2 to 4 inches is driven to a depth of 5 or 10 feet. The casing is then cleaned out by a chopping bit which is connected to the lower end of a wash pipe. Water is pumped down through the wash pipe and out through small bit openings at a high velocity. The water then carries soil fragments up through the annulus between the casing and the water pipe, and into a settling tank at the top of the borehole. The cuttings settle out, and the water is recycled into the wash pipe via a suction hose, pump, and water hose. A swivel connection between the water hose and the wash pipe permits the chopping bit to be twisted as it is raised and dropped on the bottom of the hole. Additional casing is driven and water pipe sections are added as the hole progresses. If the material will stand without caving, however, the casing need only extend 10 to 15 feet below the ground surface (Peck et al., 1974).

Rotary drilling is similar to wash boring except that the drill rods and cutting bit are rotated mechanically while the hole is being advanced. The cutting bit contains ports from which the circulating water emerges. The circulating water carries cuttings up the annular space outside the drill rods. The rods may be pressed downward mechanically or hydraulically while rotating. Various bit types such as drag bits, tri-cone and two cone roller bits can be used, depending on the quality of the medium penetrated.

For both wash boring and rotary boring a drilling fluid other than water may be used. The selection of a drilling fluid compatible with the requirements for subsequent permeability measurements in the borehole is discussed in Section 6.2.2.

6.2.1.2 Rock Formations - There are many drilling methods for rock,each pertaining to a particular set of conditions. If a rock sample is desired, coring methods are available. The use of core barrels is discussed in Section 6.3.

Less expensive, non-coring techniques can satisfy other needs, such as providing inexpensive access to depths where cores are desirable, and for placing piezometers. Small holes 1 to 8 inches in diameter can be drilled by percussive, rotary, or rotary-percussive machines.

The percussive drills remove rock by making a series of indentations, the rotary drills by a planing or cutting action, and the rotary-percussive drills by a combination of indenting and cutting. Various drilling fluids are used to remove the cuttings from beneath the bit with conventional or reverse circulation methods. Drills may be powered by internal combustion engines, diesel engines, compressed air motors, hydraulic fluid motors, or electric motors.

Percussive drilling machines contain a piston that is driven by air or hydraulic fluid. The piston impacts on the steel drill rod which in turn drives the bit to produce rock breakage. The hammer assembly may be located on the drill rig, at the top of the drill rods, or down-hole, just behind the bit. These machines are used only for hard-rock drilling.

Rotary drills, which combine high thrust with bit rotation, are used both in soft, non-abrasive materials and hard rock. The cutting potential is governed by the rig size and bit type (NRC, 1976). In rocks of soft to medium hardness, such as coal, shale, sandstone, evaporites, marls and limestone, carbide drag bits, tricone bits and diamond bits may be used. In hard rocks diamond bits must be used.

If a non-cored hole is desired in hard rocks, non-coring diamond bits can be used. Several advantages are that a very straight hole is produced, the rock mass undergoes a minimum of disturbance, and the rate of advance is high. Table 6-1 presents a summary of various drilling methods used in soil/rock masses.

6.2.2 Drilling Fluids

Drilling fluids used in the various drilling methods can range from clear or muddy water, to conventional soil-bentonite slurries, to a viscous mixture of special-purpose materials. The use of compressed air as a drilling fluid will be discussed later as a special case. In general, a drilling fluid may perform one or more of the following functions:

1) supports the wall of the borehole to prevent caving
2) removes the cuttings from the bottom of the hole
3) cools and cleans the drill bit
4) lubricates bit bearings, mud pump, and drill pipe

In many applications, an additional purpose of the fluid is to seal the wall of the borehole to reduce fluid loss. However, for purposes of permeability testing, this is an undesirable effect. Emphasis must be placed on keeping the borehole wall as clean and free of mud and drill cuttings as possible during drilling operations. If permeable strata are encountered or anticipated, casing of the borehole should be considered in preference to a high-viscosity drilling fluid. The only exception to this rule is in the use of a high-viscosity drilling fluid similar to Johnson Co. "Revert", an organic polymer drilling fluid additive which takes the place of native clay or bentonite. This additive has the advantages of a clay mud drilling fluid in minimizing water loss in permeable strata and stabilizing borehole walls in collapsible formations, and yet will break down to a water-like fluid in 3 or 4 days. Faster or longer "break" times can be achieved by addition of another additive. Consequently, alteration of in-situ permeability around the borehole typical of the clay-based drilling fluids is minimized.

TABLE 6-1 DRILLING METHODS IN SOIL/ROCK MASSES
(after Goodman, 1976)

Augering - Disturbed samples of soil and soil-like rocks; 2 to 24" diameter holes, maximum depth 60 to 100 feet; holes usually vertical.

Wash Boring - Used for making a hole, without return of a sample, in soils and in soft rocks; only in vertical holes.

Center Sample Rotary Drilling (Becker CSR) - Chips or cores of all rocks; holes 3½" and larger, up to 750 feet deep; holes may be inclined up to 45 degrees.

Churn Drilling - Used for making a hole, without return of a sample, in soft and medium hard rocks. Holes commonly 2" to 15" in diameter, up to 4000 feet deep; usually vertical.

Track or Wagon-Mounted Percussive Drilling - Used for making a hole, without a sample, in hard rocks. Holes commonly 1-3/4" to 4-1/2" diameter, up to 100 feet deep, in any orientation.

Down the Hole Percussive Drilling - For making holes 4" to 6" in diameter without a sample, in hard rocks. Usual depth range 120-200 feet. Can be drilled in any orientation.

Rotary Drag Bit Drilling - Very fast drilling in soft to medium hard rocks, but without a sample. 2-3/8" to 7-1/8" diameter holes in any orientation. Maximum depth in the range 100 to 250 feet.

Rotary Tricone Drilling - Holes up to 3" to 12" in diameter to any depth; rigs for drilling up to 200 feet in any orientation are common.

Shot Drilling - A continuous core sample in all but the hardest rocks with holes up to 6 feet in diameter; rather slow. Small diameter shot drilled holes have been drilled up to 1000 feet deep vertically.

Diamond Drilling - Continuous core sample of all rocks except highly fractured rocks of great hardness. Common hole diameters 15/16" up to 7-3/4". Maximum depths of rigs are 200-1500 feet. Wireline equipment can be used in steep holes.

Another important aspect of the drilling procedure is the direction of drill-fluid circulation. The conventional method is to pump drilling fluid or water down through the drill pipe and out through the nozzles in the bit. The drill fluid then flows upward in the annular space around the drill pipe to the surface. In a water-filled borehole, the fluid pressure in the borehole at any depth will be greater than the pressure of any adjacent groundwater (assuming non-artesian conditions). This causes a continuous flow of water from the drill hole into the adjacent strata. The fluid carries with it the return cuttings which are in the form of finely ground rock particles. Sherard (1968) describes a filtering mechanism which may effect a seal of rock fissures intersecting the borehole. Similar effects are obvious in the case of soil pores. Such sealing could dramatically reduce the observed permeability in a water test.

An alternative drilling method has been developed which minimizes the sealing of the fractures or pores around the drill hole. With this technique, called reverse-circulation rotary drilling, the flow of fluid is reversed. The fluid with suspended cuttings is drawn up through the middle of the drill stem and fresh fluid is supplied through the space between the drill stem and the wall of the hole. Consequently, fractures or porosity in the borehole walls are not exposed to the drill cuttings. The technique has the added advantage that the velocity of the drill fluid in contact with the wall of the hole is much lower than with conventional circulation and thus erosion is minimized. It has been recommended that this velocity be kept below 1 foot/sec. To prevent caving in unstable strata the fluid level should be maintained at or above the ground surface. The water level in the borehole should be continuously monitored. If permeable layers are penetrated the water level will suddenly drop. The driller should note the depth at which this occurs, as this will aid in the overall understanding of subsurface conditions. A considerable quantity of water must be immediately available at all times as a supplement to the drill water cycle when drilling in sand formations. Supply requirements could vary from 20 to 500 gpm (Johnson, 1972). A prolonged reduction of water level in the borehole in weak sedimentary formations may result in caving. The use of casing through such weak strata should be strongly considered.

A relatively new development in rotary drilling includes the use of compressed air as the drilling "fluid". Air is circulated in the same manner as is water in the methods described above. Rotary drilling machines designed for this use are usually equipped with a water pump in addition to the high capacity air compressor, since air rotary methods can only be used in consolidated materials. Water can be used when drilling through overburden materials above bedrock, whereupon drilling is continued in the rock using air. Casing may be required through the overburden to prevent caving after changing to air circulation.

Field tests demonstrate that with certain sizes of bits penetration rate is often faster and bit life longer when using air as compared with water. This is achieved through better bottom-hole cleaning (Johnson, 1972).

6.3 SAMPLING TECHNIQUES

Sampling of a proposed in-situ leaching formation is an important part of any permeability test program. Factors which may be important to the understanding of leaching solution flow can be identified by such a program. For example, borings in sandstone uranium deposits may detect significant interbedding of low permeability strata such as siltstones and shales. Core samples can also identify the relative degree of fracturing in a rock mass, a common characteristic of porphyry copper deposits. Discontinuities such as low permeability interbeds and highly permeable fractures can influence the flow regime of an ore deposit significantly. In addition, some sampling is essential for the selection of the borehole preparation method and permeability field test(s).

6.3.1 Soil/Unconsolidated Material

Various augers can be used to obtain soil samples at shallow depth. Although they work well in cohesive materials such as silts and clays, they are ineffective in cohesionless soils below the water table because either the sample will tend to wash off or the hole will not stay open. Consequently, it becomes necessary to employ rotary drilling techniques and drilling fluids. Drive sampling techniques are often used to obtain representative soil samples. Such sampling usually proceeds at 5 foot intervals or at strata interfaces. The sample is taken by driving a sampler or "sample spoon" into the soil at the base of the borehole. Samplers are commonly driven by drop-hammers and sometimes by hydraulic methods. Common sample spoons range in size from 2-to 4-1/2-inch outside diameter and 18-to 24-inch lengths.

Relatively undisturbed samples can be obtained using a thin wall sampler such as a Shelby tube. The sampler tip is rolled inward, providing a 1 percent inside diameter clearance, thereby reducing drag and disturbance of the sample. Tubes made of steel, brass or stainless steel come in 2-to 3-inch outside diameter sizes.

A variation of the Shelby tube is the stationary piston sampler. The sampler is initially sealed at the bottom with a sliding piston so that it can be safely lowered through fluid and soft cuttings without fear of sample contamination. The piston is held stationary as the sampler tube is forced into the formation, and subsequently assists in holding the core in the sampler by a vacuum as the sampler is lifted. This sampler can achieve a better vacuum seal than does the thin wall sampler's ball check valve.

In the Denison Sampler, or Denison Core Barrel, the inner tube and cutter always precede the rotating outer tube into the formation, helping to keep the sample relatively undisturbed and uncontaminated by the drilling fluid. The sample is "floated" by differential pressure of the circulating drill fluid until removed, thereby allowing retrieval in more difficult loose or saturated soils and sediments. The sampler is also effective in sampling mixtures of gravel and clay, in soft shale, and weathered rock interlaced with clay seams. Various bottom assemblies allow a broad range of application (Acker, 1974).

The reader is referred to Hvorslev (1949) for other detailed descriptions of sampling techniques for soils/unconsolidated material.

6.3.2 Consolidated Sediment/Rock

The usual direct method of obtaining a rock sample is by using rotary drills incorporating various types of core barrels and core bits. Continuous core runs can be taken, or a hole can be advanced to a specified depth before a core is taken.

The most important parts of any core drill operation are the cutting bit and recovery vehicle, the core barrel. Most core drilling is carried out using diamond bits which are available in two main forms: (a) "surface set" bits with individual diamonds set in a metal matrix, and (b) "impregnated bits" with fine diamond dust incorporated in the matrix. The latter type of bit is considered to be self-sharpening because as the surface wears away, the new diamonds are exposed. Diamonds used in the surface set bits vary in both quality and size, and are governed by the rock type. In general, the harder the rock, the smaller the size and the higher the quality of the diamonds (Bell, 1975). Bit performance depends on several factors, including the angular velocity of the bit, the applied pressure, the matrix (bit) hardness, and the shape and quality of the diamonds.

Tungsten bits are less costly than diamond bits, but can only be used in softer formations such as shale, till, or a residual soil with boulders. Standard tungsten-carbide bits are manufactured by using carbide inserts instead of diamonds.

The main types of core barrels are as follows:

Single Tube - This is the simplest, least expensive, and most rugged of core barrels. This is often the best core barrel to use when in solid rock formations, where good core recovery is relatively simple. It is also useful in penetrating rock layers above the strata of interest or where a high percentage of core recovery is not necessary.

Double-Tube Rigid - This type of core barrel has an outer and inner tube, both of which are fixed to the core barrel head. A reamer shell and bit are screwed on to the outer tube. During operation, the drilling fluid flows through the head and down the space between the inner and outer barrels. Drilling fluid contacts the core in the bit area. This barrel is useful in materials that tend to wash or dissolve readily. One drawback of this unit is that the inner tube rotates around the core. If the cored material is friable or badly fractured it may be severely disturbed in the coring process.

Double-Tube Swivel - This barrel has the advantage that the inner tube is mounted on anti-friction bearings, which permits the outer tube to rotate while the inner tube remains stationary. This barrel is therefore more useful than the double-tube rigid barrel for coring fractured and friable formations.

The above-mentioned barrel types are commonly designated the "G" Design barrels, as standardized by the DCDMA (Diamond Core Drill Manufacturers Association). DCDMA standards prevail in North and South America, Australia, and South Africa.

The "M" Design Core Barrel (or Face-Ejection Barrel) is basically a double tube swivel type core barrel with improved water flow. An extension, known as the "core lifter case", is fitted to the bottom of the inner tube, extending almost to the bit. This further protects the core from water washing. The "M" Design barrel is widely used for recovering rock cores in fractured and friable formations (Bell, 1975).

Relatively new developments are the Triple Tube and split double tube barrels. These types employ an inner tube which is split into halves longitudinally. When withdrawn from the outer casings of the core barrel, the core can be observed and described without the risk of sample disturbance caused by core extraction. The procedure permits a better evaluation of highly cleaved and jointed rocks (Bell, 1975).

6.4 BOREHOLE INSPECTION

Various techniques have been developed which allow observation of the borehole walls. The advantage of such methods is that structural integrity and lithologic variations are not subject to misinterpretation due to damaged core samples. Of particular importance is an accurate assessment of fracture distribution and spacing in a jointed rock mass. Channelization potential and general anisotropic flow conditions may be more easily estimated using direct down-the-hole techniques. As this problem mostly pertains to fractured rock conditions, a detailed description of the various methods is deferred to the section, "Supplementary Discussions For Fractured Rock." It is emphasized, however, that such techniques may also be effective in identifying impermeable boundaries in non-fractured media, such as impervious siltstone and claystone horizons in an interbedded sandstone sequence.

6.5 BOREHOLE PREPARATION FOR PERMEABILITY TESTS

Seven different methods of borehole preparation are described. Each preparation method is discussed in terms of the geologic conditions and permeability test methods for which it is most suitable, the required equipment and the procedural development. The preparation methods have been given letter designations, and are referred to as such in future sections of the report. The preparation methods are as follows:

A. Unlined Borehole
A-1. Unlined Borehole with Seal Above Test Length
B. Unlined Test Section, Cased Above
C. Gravel-Filled Test Section, Cased Above
D. Perforated Casing Over Test Section, Cased Above
E. Fully Cased Borehole Open at Bottom Only
F. Piezometer Installation

6.5.1 Method A: Unlined Borehole

Geologic Conditions

This method may be used in stable materials such as stiff clays and intact to lightly-jointed rocks. In semi-stable materials such as sandy clays and moderately jointed rocks, test duration will be a factor influencing this method's suitability. It is not suitable in strata that underlie a perched water table, or where a long length of unlined borehole in highly permeable materials will lead to the need for large volumes of test water. For these conditions, Method A-1 is preferable.

Field Test Conditions

This preparation method is suitable for constant head tests above the water table, for packer tests, and for well pumping tests in unconfined aquifers.

Equipment

A drill or auger is required to prepare a 3-inch to 8-inch diameter borehole. The borehole size typically depends upon the formation and the test to be performed, and is usually 6 inches for a constant head test in soils and 3 inches for packer tests in rock. The size of borehole is largely a matter of ease of drilling and installing the necessary test method equipment. For well pumping tests, 3-inch diameters and 6 to 8-inch diameters are typical sizes for observation wells and production (pumped) wells, respectively. A compressor and air hose or a bailing tool and means for surging the borehole are also needed.

For constant and variable head tests, gravel is used to form a 6-inch cushion at the bottom of the borehole (USBR, G-97). Constant head tests also require rigid PVC (Polyvinyl chloride) plastic tubing, from 3/4 to 1-1/4 inches in diameter, to reach from the test zone to the ground surface (the PVC pipes are used to introduce water into the test section and to monitor the water level - see Section 7.3).

Procedure

Drill to the bottom of the proposed test section. The borehole must be cleaned before the test to remove the fines which may clog the side of the borehole. This can be achieved by flushing it with clean water until it runs clear, making sure that the end of the hose is above the base of the borehole to prevent scour.

In stiff clays and fine-grained rocks, the drilling action may smear the sides of the borehole and clog the fissures (Dixon, 1975). To clean the fissures and borehole sides, fill the hole with water well above the test section. Then surge the hole gently by raising and lowering a bailer (cylinder with hydraulic flap valve at bottom). Finally, bail the hole to create water flow from the rock/soil mass into the borehole. Bailing may be conducted with a bailing tool or by lifting the water out of the borehole with compressed air. For the latter method, an air hose from a small (150 cfm) compressor is lowered to the bottom of the hole, and the air is circulated at a rate sufficient to cause water flow out of the top of the hole. This is continued until the running water is clean. The filling, surging and bailing sequence should be repeated until the borehole water is clear. If rock cores show that fissures are normally clay-filled, the bailing procedure might alter in-situ conditions and should not be performed.

In friable materials, constant and variable head tests require a 6-inch gravel cushion at the bottom of the borehole. Constant head tests also require PVC pipe (one or two lengths) which is seated on the top of the gravel cushion, or suspended several inches above the base of the borehole if the gravel is omitted. Figure 6-1 illustrates Borehole Preparation Method A.

6.5.2 Method A-1: Unlined Borehole with Seal Above Test Section

Geologic Conditions

The geologic conditions in which this method is applicable are the same as for Method A, except that it may be used if a perched water table exists. Procedure A-1 may also be used to reduce the volume of water needed for tests in highly permeable materials.

Field Test Conditions

This preparation method is suitable for constant head tests above the water table, and for well pumping tests in unconfined aquifers.

Equipment

In addition to the equipment specified for Method A, the following are required:

- borehole sealing material:
  - i) one bag cement and two bags bentonite, or
  - ii) 1/2-inch diameter commercial compressed bentonite pellets
- pea gravel
- sand
- filter cloth or burlap (optional)
- tamping rod
- tremie pipe (deep holes only)

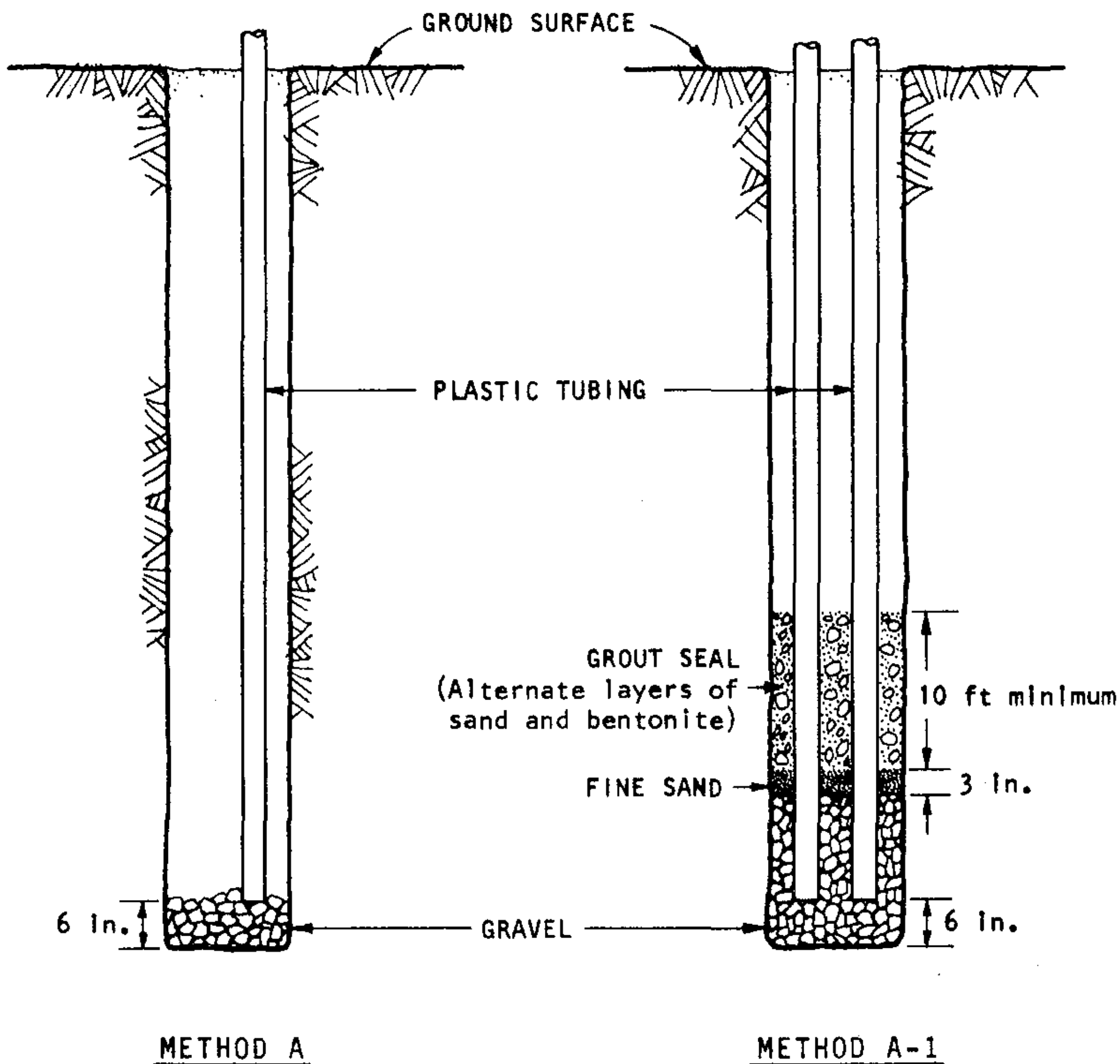


Figure 6-1 BOREHOLE PREPARATION: UNLINED BOREHOLE

Procedure

Borehole preparation procedure follows directly from that given for Method A, except that for constant head tests two, rather than the optional one or two PVC pipes are required. After placing the plastic piping on the 6-inch gravel cushion, finish filling the test section with gravel. In deep holes the gravel should be placed through a tremie pipe to prevent blocking and consequent voids in the hole. Place a 3-inch layer of fine sand over the gravel. To form the seal, use commercially available 1/2-inch diameter compressed bentonite pellets, or prepare mudballs consisting of 2 parts bentonite and one part cement (by volume) with sufficient water to render the mixture sticky. For deep holes or holes below the water table, wrap the prepared balls in a thin layer of netting or cheesecloth. Place a 6-inch layer of commercial pellets or equivalent amount of prepared mudballs into the hole and tamp into place. Add a several-inch layer of sand. Repeat this procedure until the seal either reaches the top of the water table or is 10 feet long. After placing the seal, allow a setting time of 24 hours before testing in the borehole. Figure 6-1 illustrates Borehole Preparation Method A-1.

6.5.3 Method B: Unlined Test Section, Cased Above

Geologic Conditions

This method is suitable for stable materials overlain by unconsolidated deposits, and for marginally stable conditions such as soft or sandy clays and moderately jointed to very blocky and seamy rock. It may be used in medium to stiff clays, but in such low permeability materials, Method F should be considered as an alternate in order to reduce test duration.

Field Test Conditions

This method is suitable for variable and constant head tests, packer tests and well-pumping tests. For constant head tests, where the height of water in the borehole is above the top of the test section, Method D may reduce upward seepage around the casing and give better test results.

Equipment

A drill or auger is required to prepare a 3-inch to 8-inch diameter borehole. The borehole size typically depends upon the formation and the test to be performed, and is typically 6 inches for a constant head test in soils and 3 inches for packer tests in rock. The size of the borehole is largely a matter of ease of drilling and installing the necessary test equipment. For well pumping tests, 3-inch diameters and 6 to 8-inch diameters are typical sizes for observation wells and production (pumped) wells, respectively. A compressor and air hose or a bailing tool and means for surging the borehole are needed for cleaning procedures. Casing is required above the test section.

For constant and variable head tests, gravel is used to form a 6-inch cushion at the bottom of the borehole (USBR, G-97). Constant head tests also require rigid PVC tubing, from 3/4 to 1-1/4 inches in diameter, to reach from the test zone to the ground surface (the PVC pipes are used to introduce water into the test section and to monitor the water level - see Section 7.3).

Procedure

Drill and case the borehole to the top of the test section. Drill 5 to 20 feet further, with preferred lengths of 5 feet for constant and variable head tests, and 10 feet for packer tests. For well pumping tests, the test length should preferably penetrate the entire stratum under investigation. The borehole should then be cleaned as described for Method A. In friable materials, constant and variable head tests require a 6-inch gravel cushion at the bottom of the borehole. Constant head tests also require PVC pipe (one or two lengths) which is seated on the top of the gravel cushion, or suspended several inches above the base of the borehole if the gravel is omitted. Figure 6-2 illustrates Borehole Preparation Method B.

6.5.4 Method C: Gravel-Filled Test Section, Cased Above

Geologic Conditions

This method is suitable for unstable materials such as soft clays, sands and gravels, and crushed rock. In unstable materials of high permeability, Method E should be considered as an alternate because it may reduce the volume of water needed for the test. For clays, Method F should be considered as an alternate because it may shorten the test duration. Due to difficulties with installing the gravel and plastic tubing, the performance of constant head tests may be limited to depths of approximately 100 feet.

Test Conditions

This method is suitable for constant and variable head tests. For constant head tests where the height of water in the borehole is above the test section, Method D is preferable because upward seepage around the casing is reduced (USBR, G-97). Although Method C can be used for well-pumping tests, Method D is preferable for such tests.

Equipment

A drill or auger is required to prepare a 6-to 8-inch diameter borehole. A compressor and air hose or a bailing tool and means for surging the borehole are needed for cleaning procedures. Sufficient gravel is required to fill a 5 foot (minimum) test section length. Casing is required above the test section. A tremie pipe is required to place the gravel in deep holes.

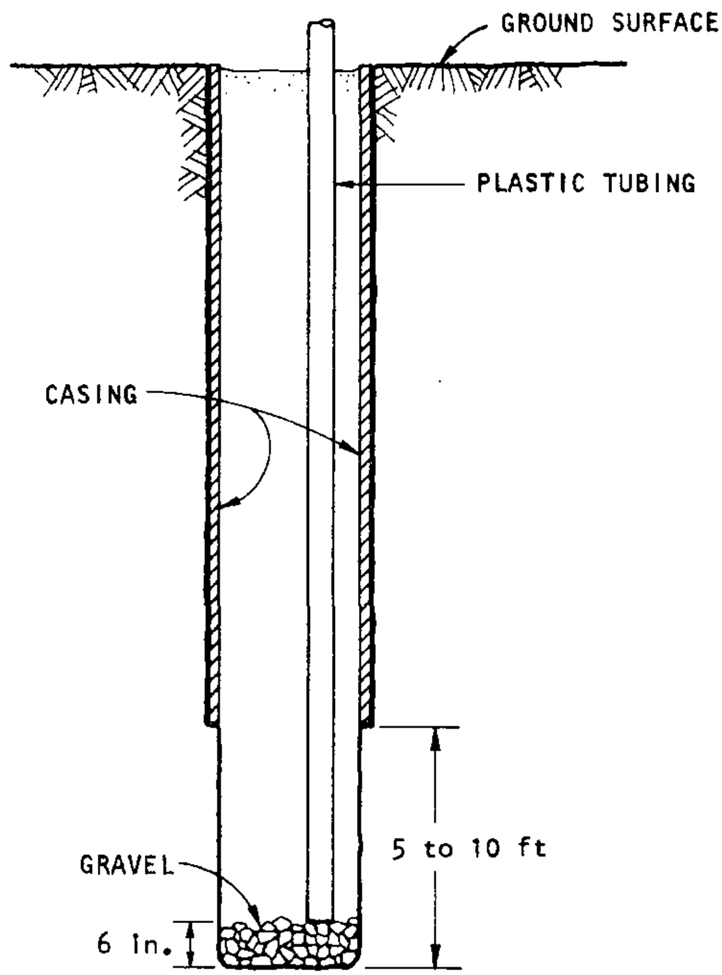


Figure 6-2 BOREHOLE PREPARATION METHOD B: UNLINED TEST SECTION, CASED ABOVE

For constant head tests, rigid PVC tubing, from 3/4 to 1-1/4 inches in diameter is required to reach from the test zone to the ground surface (the PVC pipes are used to introduce water into the test section and to monitor the water level - see Section 7.3).

Procedure

The preparation procedure for this method varies slightly for relatively stable and unstable materials. If the borehole will stand unsupported for a sufficiently long time, drill to the top of the test section, case over the entire length, and then drill a test section of approximately 5 feet. Clean the test section as described for Method A. For variable head tests, place sufficient gravel (using a tremie pipe at large depths) so that it extends several inches into the casing. For constant head tests, create a 6-inch gravel cushion, place the required PVC pipe on the cushion, and then backfill with gravel so that it extends several inches into the casing.

If the borehole will not stand unsupported, drill and case the borehole to the bottom of the test section. Withdraw the casing a short distance, flush the hole, and place gravel to support the unlined section. For variable head tests, continue the casing withdrawal-flushing-gravel backfill cycle until the desired test section length has been achieved. Extend the gravel section several inches into the casing. The procedure is the same for constant head tests, except that the PVC pipe is placed after a 6-inch gravel cushion has been created. Continue the cycle, being careful not to alter the position of the PVC pipe. When the desired test length has been achieved, extend the gravel section several inches into the casing. Figure 6-3 illustrates Borehole Preparation Method C.

6.5.5 Method D: Perforated Casing Over Test Length, Cased Above

Geologic Conditions

This method is suitable for unconsolidated materials which will cave without support. Using this method, the results of constant head tests are progressively more accurate in strata of greater grain size and higher degrees of uniformity and compaction (USBR, G-97).

Field Test Conditions

This method is suitable for variable and constant head tests above and below the water table in all ground conditions. This is the preferred preparation method for gravity tests where the height of water in the borehole is to be maintained above the test section, because upward seepage around the casing is reduced. Method C is preferable for constant head tests where the water level is to be maintained within the test length because the gravel section will minimize heave of the base of the borehole.

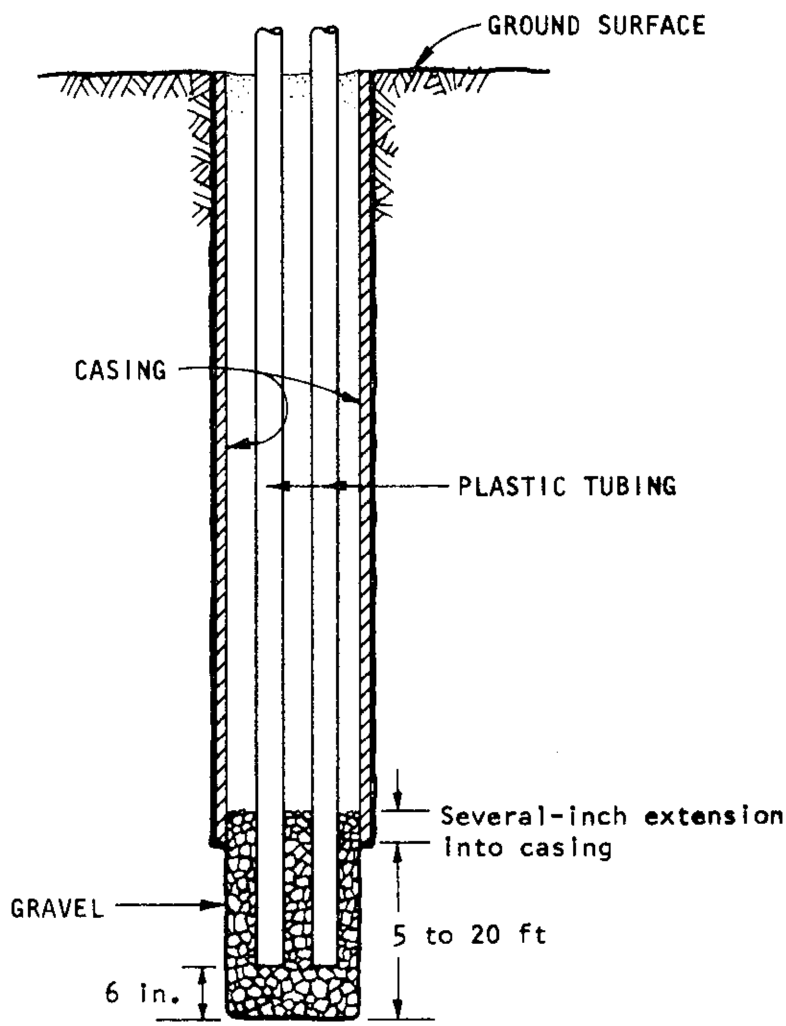


Figure 6-3 BOREHOLE PREPARATION METHOD C: GRAVEL-FILLED TEST SECTION, CASED ABOVE

Equipment

A drill or auger is required to prepare a 3-to 6-inch diameter borehole. The borehole size typically depends upon the formation and the test to be performed, as is discussed for Method A. Casing is required over the entire borehole, with the test section perforated with 1/16 to 1/2-inch diameter holes, depending on the grain size of the material to be tested. For well-pumping tests, a well screen may be preferred to the perforated casing. A minimum test length of 5 feet is required for constant and variable head tests, while for well-pumping tests, the entire saturated stratum should be penetrated if possible.

The bottom of the perforated section of casing may be bevelled on the inside and case-hardened to provide a cutting edge, or a drive shoe may be attached. Drive shoes are recommended only for gravelly soils or fractured rock where the casing cannot otherwise be driven (USBR, G-97). The drive shoe will cause compaction of the immediate ground mass and may influence the test results.

Gravel requirements will vary with the preparation method and length of the test section. If a well screen is used for a well pumping test, a gravel filter is required to fill an annulus created between the screen and the borehole wall. This is particularly important in sandy and clayey strata. The appropriate filter gradation is a function of the grain size distribution of the test zone material and is determined as follows. Sieve a representative sample from the test zone. Then, define D_{15}, D_{50}, and D_{85} as the grain sizes at which 15%, 50%, and 85% by weight of the sample has a smaller diameter, respectively. Select a filter material that has no clay particles, sieve a representative sample to obtain the D_{15}, D_{50} and D_{85} grain sizes as was done for the test zone material, and apply the following limit criteria to the ratios of the two materials:

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{In-Situ Sample}} < 5$$

$$4 < \frac{D_{15}\ \text{Filter}}{D_{15}\ \text{In-Situ Sample}} < 20$$

$$\frac{D_{50}\ \text{Filter}}{D_{50}\ \text{In-Situ Sample}} < 25$$

(Lambe and Whitman, 1969)

Additional information regarding well screens is available in Johnson (1972).

For variable and constant head tests, and well-pumping tests where perforated casing is used, a 6-inch gravel cushion is required at the bottom of the borehole. Constant head tests also require one or two lengths of PVC pipe, from 3/4 to 1-1/4 inches in diameter, to reach from the test zone to the ground surface. The PVC pipe is used to inject water and/or monitor water levels in the borehole as recommended in Section 7.3.

Procedure

If perforated casing is used, insert the casing to the required test depth by drilling and driving. Clean the borehole as described for Method A. In addition to removing drill cuttings from the well, this removes some of the fines immediately outside the test section and forms an annulus of coarse material around the perforated casing. This zone will reduce the possibility of clogging of the well during actual testing. For constant and variable head tests, a 6-inch gravel cushion is then placed at the bottom of the borehole. For the constant head test, place the PVC pipe so that it rests on top of the 6-inch cushion.

If a well screen is used in a well pumping test, several installation procedures are available. One procedure is to drill and case to the bottom of the test section. The well screen is then placed at the bottom of the hole, and the filter gravel pack is backfilled around the well screen as the casing is gradually withdrawn. The gravel should be added slowly so that the well screen is not pulled by the casing. After the casing has been withdrawn to the top of the test section, a packer provides a seal between the larger diameter casing and the smaller diameter well screen. Other installation procedures are available in Johnson (1972). Clean the well as described for Method A. Figure 6-4 illustrates Borehole Preparation Method D.

In situations where a weak, caving material is overlain by competent, intact rock, the borehole will require extending casing through the competent mass to the weaker material below. When performing a constant head test under such conditions, it may be desirable to provide a seal at the base of the competent stratum to prevent upward seepage of water through the annulus between the casing and the intact borehole. It should be emphasized that an installation of this type is intended to be temporary in nature, as are most permeability testing installations.

Two possible methods of preparation follow. Drill the hole to the top of the weak stratum. Using a tremie pipe, place sufficient grout so as to create a three-foot plug at the base of the hole. Wait overnight to allow the grout to set. Drill concentrically through the soft grout plug and soft stratum. Then advance casing which is 1) slightly undersized in diameter, 2) perforated over the desired test length, and 3) fitted with a cutter shoe. The remaining grout plug ring provides a base upon which bentonite slurry is placed in the annulus between the borehole wall and the undersized casing in the competent stratum. Place about 10 ft of slurry above the grout plug and allow 24 hours before testing is begun.

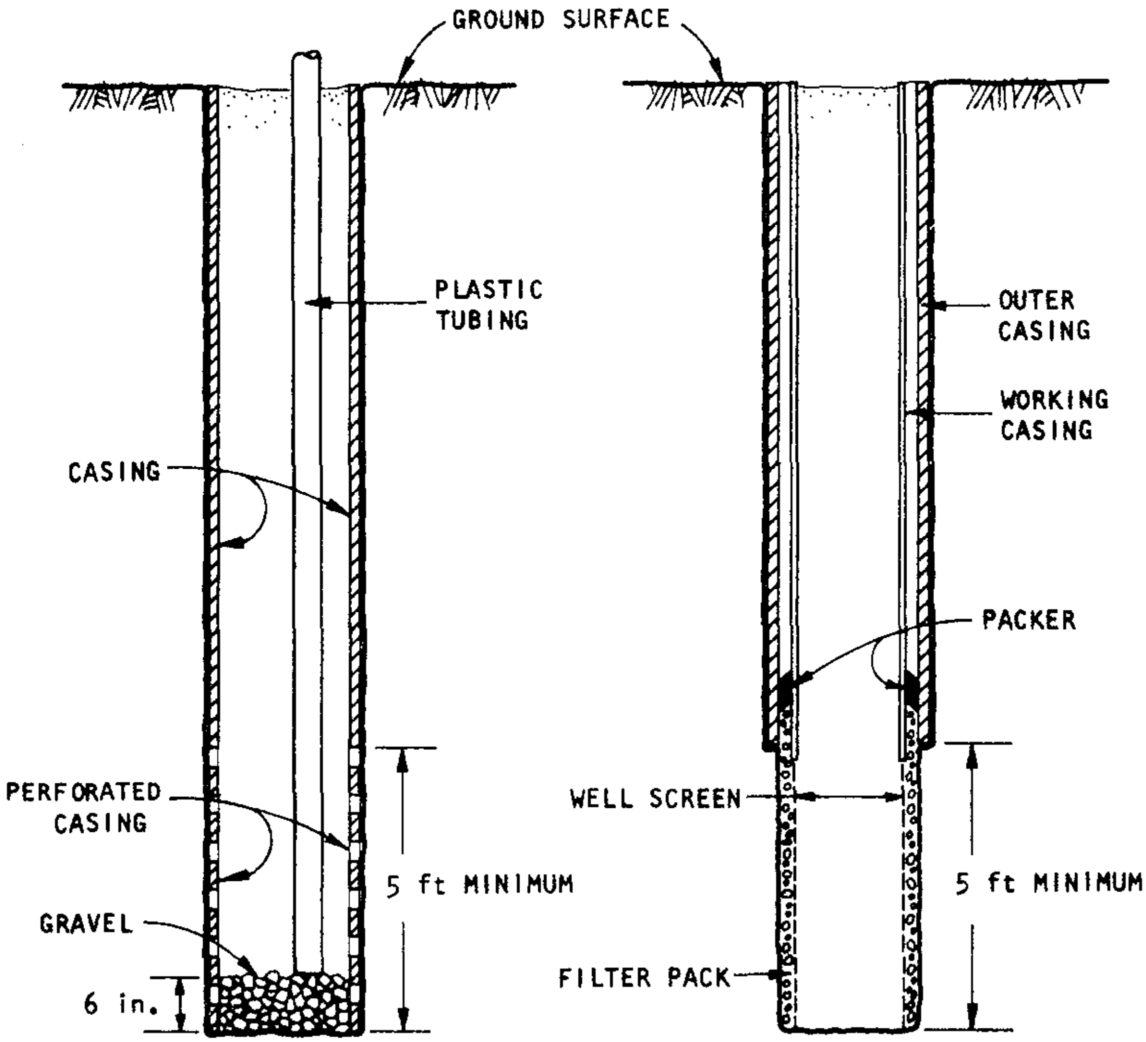


Figure 6-4 BOREHOLE PREPARATION METHOD D: PERFORATED CASING OVER TEST LENGTH, CASED ABOVE

The second procedure allows for the use of a well screen rather than perforated casing. The procedure deviates slightly from the well screen discussion above. In addition to backfilling the entire screened length with the filter gravel pack as the casing is withdrawn, place approximately one foot of clean sand on top of the filter material. Fill the annulus to the top of the borehole with a bentonite slurry. Clean the well as described for Method A.

6.5.6 Method E: Fully Cased Borehole, Open at Bottom Only

Geologic Conditions

This method is suitable for sands, gravels and fractured, highly permeable rocks. The very small test area requires the rock/soil to be fairly uniform. This method should not be used in medium or low permeability strata.

Field Test Conditions

This method is suitable for constant and variable head tests.

Equipment

A drill or auger is required to prepare a 3-to 6-inch diameter borehole. Casing is required over the full length of the borehole. Gravel requirements will vary with the field test method and geologic conditions. Constant head tests require one or two lengths of rigid PVC pipe, from 3/4 to 1-1/4 inches in diameter,to reach from the test zone to the ground surface.

Procedure

Advance the borehole and place the casing. During the drilling operation keep the hole full of water if below the water table to prevent piping (Dixon, 1975). Seal the casing joints well to prevent water leakage in or out of the casing. Flush the borehole with clean water, being sure to keep the water hose off the bottom of the borehole to prevent erosion. If the borehole extends below the water table, the borehole should not be bailed more than a few feet for cleaning purposes. This will minimize the possibility of piping. However, it is important to clean the hole carefully. If sediment is deposited on the bottom of the hole, the permeability may be underestimated in a constant or falling head test.

For constant and falling head tests in friable materials, place a 6-inch gravel cushion at the bottom of the borehole. Constant head tests also require PVC pipe (one or two lengths) which is seated on top of the gravel cushion or suspended several inches above the base of the borehole if the gravel is omitted (the PVC pipes are used to introduce water into the test section and to monitor water level - see Section 7.3). Figure 6-5 illustrates Borehole Preparation Method E.

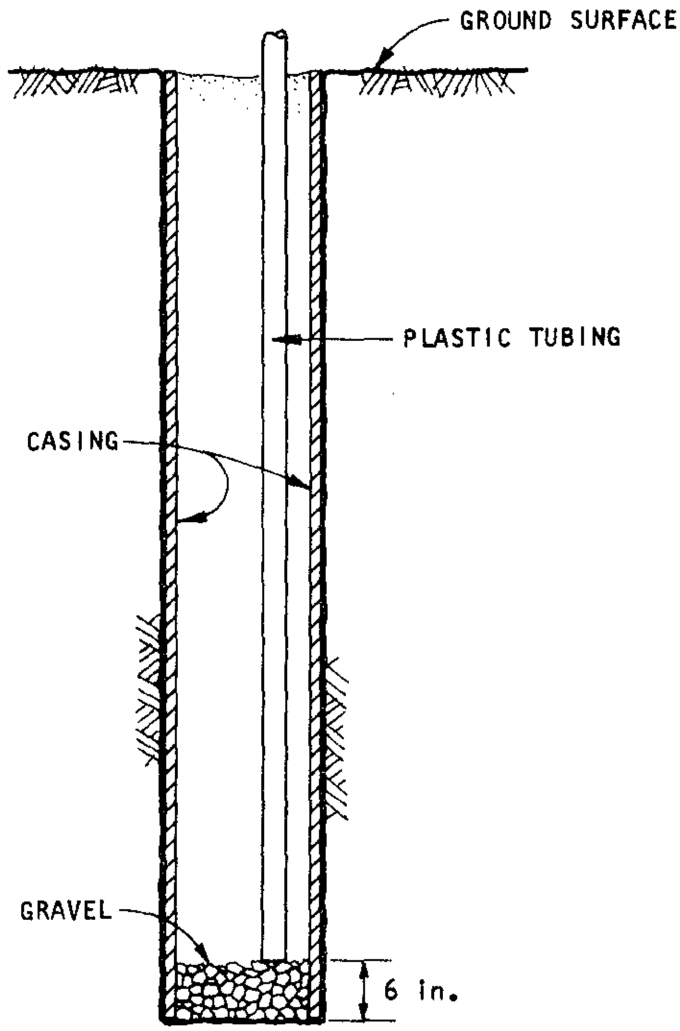


Figure 6-5 BOREHOLE PREPARATION METHOD E: FULLY-CASED BOREHOLE, OPEN AT BOTTOM ONLY

6.5.7 Method F: Piezometer Installations

Background

Piezometer installations are included in the borehole preparation section because for these field tests which require piezometers as a fixed monitoring system, the piezometer installation is an integral part of the borehole preparation sequence. Piezometers, or down-hole instruments to measure pore water pressure, cover a wide spectrum. Some are very simple and easy to install, while others are more sophisticated, requiring experienced and qualified personnel to install them.

The literature is often inconsistent with regard to precise piezometer classification nomenclature. However, the many piezometer types may be divided into five general classifications: open piezometers or observation wells, standpipe piezometers, closed hydraulic piezometers, electrically indicating piezometers and air-actuated piezometers.

Open piezometers or observation wells are simply open-ended cased or uncased boreholes wherein the water level is measured using a down-hole probe; such probes are easily constructed of an ohmmeter and a length of 2 conductor electric wire or are commercially available. This is the simplest variation of a piezometer, and is only useful in permeable strata (greater than 10^{-4} cm/sec, Hoek and Bray, 1974; Terzaghi and Peck, 1967). A disadvantage of this system is that different layers of soil which may be acted upon by independent water levels are now connected by the borehole, and subsequent water level measurements may have little relation to true, in-situ values.

Where such conditions exist, it is necessary to use standpipe piezometers to sense the true, in-situ water levels for each soil layer. Standpipe piezometers have a porous element embedded within a pervious borehole backfill material, usually sand, within the stratum to be monitored. A riser pipe of equal or smaller diameter is used to connect the porous element to the surface for monitoring. The porous element is hydraulically sealed from communication with any other soil layer contacting the borehole by backfilling the borehole with an impervious seal just above the intake element. Consequently, the rate at which water will flow out of, or into the porous element is a function only of the permeability and existing pressure head of the stratum communicating to the porous element.

In low permeable materials ($K<10^{-4}$ cm/sec) the "time lag" can become excessive if large diameter open piezometers or large diameter riser pipes for standpipe piezometers are used. The time lag is the time taken for the observed water level in the uncased or cased hole or standpipe, or the indicated pressure of a closed system, to reach equilibrium after a groundwater pressure change. Hoek and Bray (1974) stress that time lag is one of the most important factors in choosing an appropriate piezometer system. The time lag is reduced in low permeable materials by using a standpipe piezometer with a riser pipe of small diameter connected to a larger diameter porous intake element. The relatively small volume of water within the standpipe reduces the response time. Several standpipe piezometers can be installed in a single borehole, but the practical difficulties of placing effective seals between successive porous elements in the borehole are considerable. Unless extensive experience has been obtained, single rather than multiple installations are recommended.

The third type, closed hydraulic piezometers, can be used when the permeability falls below 10^{-6} cm/sec. In such low permeable strata, the time lag of standpipe piezometers becomes excessive. A closed hydraulic piezometer generally comprises a small, down-hole porous element connected by twin water-filled plastic tubes to the surface. The tubing and porous element are initially filled with water from a surface station, or terminal. Groundwater contacts the water in the sensor unit through a porous stone, and communicates through the water-filled tubes to pressure gages at the surface. Consequently, they only monitor changes in groundwater pressure heads that are consistently higher than the elevation of the terminal.

Electrically indicating piezometers, also referred to as pore-pressure cells, generally consist of a small, borehole-sized hydraulic chamber which houses a thin steel diaphragm. Groundwater enters the chamber through a porous stone on one side of the diaphragm and causes it to deflect. The deflection, which is proportional to groundwater pressure, is sensed by a strain gage mounted to the back side of the diaphragm. The electrical signal from the strain gage is monitored on a readout set at the surface. Such piezometers can have an almost instantaneous response time.

Air-actuated piezometers are similar to the electrical piezometers except that when making a measurement, the diaphragm is deflected back to its datum position by gas pressure supplied from a bottle at the surface. The instrument is calibrated such that when a certain constant volume of gas is escaping past an inlet and outlet port in the diaphragm seat, the gas pressure acting on the diaphragm will equal the ambient water pressure.

Geologic Conditions

As mentioned above, an important parameter to consider in an installation is the time lag. Open piezometers or observation wells can be used in sands, gravels, and moderately to heavily fractured rock masses where permeabilities are generally greater than 10^{-4} cm/sec. Standpipe piezometers may be used in weathered clays, silts, and weathered or lightly fractured rock where permeabilities are not less than 10^{-6} cm/sec. In low permeability formations such as silty clays, homogenous clays and intact rock, time lag considerations may require closed hydraulic, electrical, or air-actuated piezometers. The latter piezometer types can be used in any moderate to low permeability formations and are practical at large depths.

Field Test Conditions

All piezometers described here may be used to monitor groundwater pressure changes during performance of constand head and falling head tests, and can be used in observation wells for constant head and well-pumping tests. However, closed-hydraulic piezometers are limited to those cases where the terminal can be located below the effective head to be measured. Open piezometers, standpipe piezometers, and closed hydraulic piezometers may themselves serve as the test apparatus through which the water is either removed or introduced during the performance of constant head or variable head tests.

Equipment

The following equipment is required for a standpipe piezometer. A drill or auger is required to advance a 3- to 4-inch diameter borehole. Casing is required in unstable materials. Rigid PVC pipe, 3/4 to 1-1/2 inches in diameter depending on the permissible time lag, is required in sufficient length to reach from the position of the intake element in the test section to the ground surface.

The porous intake element consists of either a ceramic piezometer tip (Casagrande type) or a slotted plastic pipe wrapped in burlap or 1/32-inch nylon mesh.

Sand or gravel filter material, typically 1 to 2 cubic feet, should be selected based on the grain size of the test zone material. The selection criteria are discussed in Section 6.5.5. The volume of filter material depends on the use of the piezometer and the permeability of the stratum.

Also needed are several cubic feet of fine sand, a tremie pipe (for deep installations only), commercial compressed bentonite pellets or bentonite/cement mixtures, a grout pump and hose, and cheesecloth or nylon mesh.

The closed hydraulic electrical, and air-actuated piezometers require essentially the same borehole backfill materials as the standpipe piezometer, and the preparation procedure which follows is similar.

Procedure

Advance the borehole, using casing only if necessary. Clean the borehole by flushing, being careful to keep the flushing hose above the base of the borehole to prevent erosion.

In stiff clays and fissured rock clean the borehole by filling, surging, and bailing as discussed in Method A. If the base of the hole is in sands, gravels, or crushed rock below the water table, do not lower the water level in the hole more than 10 feet, as this could cause piping or heave of the bottom of the hole.

If casing is used, the cleaning procedure should be carried out as the casing is withdrawn.

Place the gravel filter material in the borehole up to the desired location of the piezometer intake element or piezometer sensor unit. This location should be a minimum of 6 inches above the base of the borehole. Install the intake or sensor unit, connected by the appropriate plastic pipe or sensor leads to the surface. Add additional gravel filter material until the top of the proposed filter section is reached. At least 6 inches of gravel should cover the intake or sensor. Add 3 inches of sand.

For the seal, use commercially available 1/2-inch diameter compressed bentonite pellets, or prepare mudballs consisting of 2 parts bentonite and one part cement (by volume) with sufficient water to render the mixture sticky. For deep holes or holes below the water table, wrap the prepared balls in a thin layer of netting or cheesecloth. Place a 6-inch layer of commercial pellets or equivalent amount of prepared mudballs into the hole and tamp into place. Add a several-inch layer of sand. Repeat this procedure until the seal is at least 3 feet long. Grout the remainder of the borehole with a mixture of 2 parts bentonite, 1 part cement and 7 parts water (by volume). Figure 6-5 illustrates Borehole Preparation Method F.

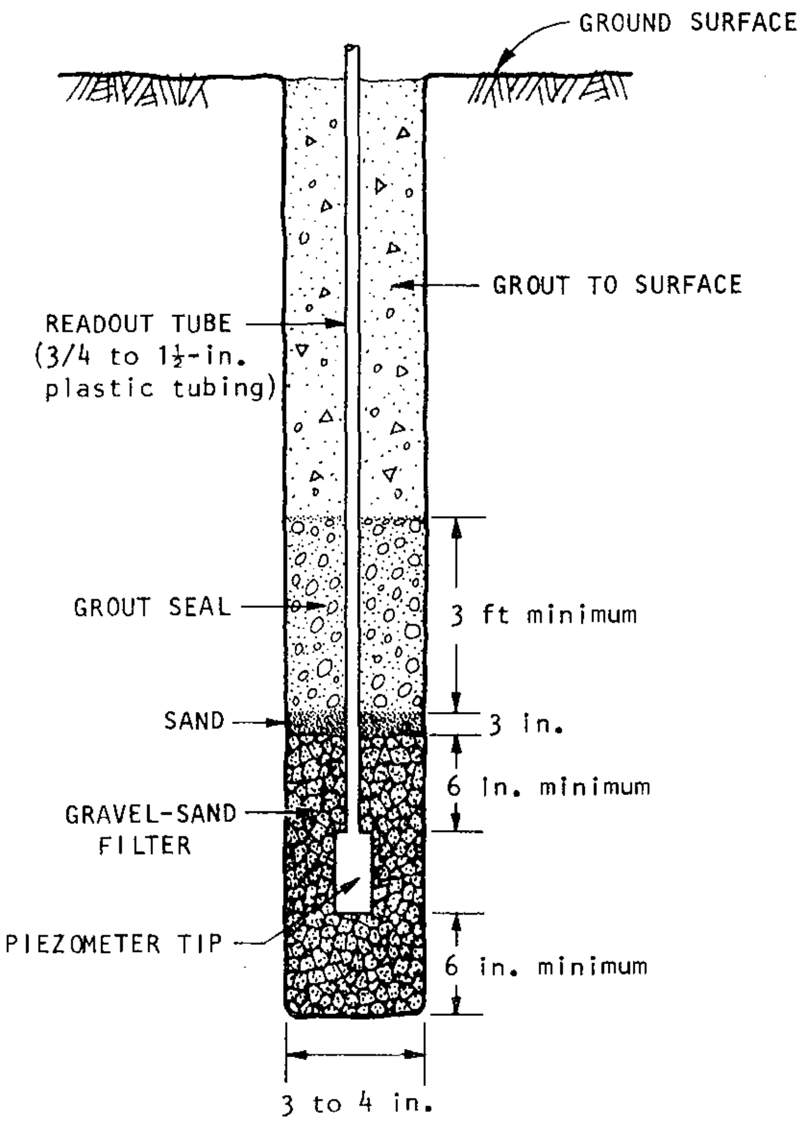


Figure 6-6 BOREHOLE PREPARATION METHOD F: PIEZOMETER INSTALLATION

7.0 TEST METHOD SPECIFICATIONS

7.1 BACKGROUND

The most frequently used direct measurements of in-situ permeability can be divided into two major groups: those which pump water into the ground and those which pump water out. Descriptions of the general features of these two groups of tests follow, some limitations and areas needing improvement are commented on, and supplementary investigative techniques are identified for later discussion.

7.1.1 Pump-In Tests

Pump-in tests may be used above or below the water table. The only pump-in test that is commonly analyzed for non-equilibrium conditions is the falling head test. In this test, the borehole is filled with water above the normal water level and the rate at which the water level drops is recorded.

The remainder of the pump-in tests are similar; water is pumped in until the flow into the well is constant, and the height of the water level in the well is constant. The various pump-in tests are differentiated by the manner in which the water pressure is applied to the test section. In a gravity test, the column of water pumped into the test section does not rise above ground level. In the literature a gravity test with a constant head applied is usually called a "constant head test." In practice, a packer test is also a constant head test; however the column of water extends through drill rods to the surface and additional pressure is applied with a pump at the surface. In a gravity test steady state conditions may be reached by maintaining a constant water head and allowing the flow into the borehole to stabilize, or by supplying a constant flow and allowing the water level to stabilize. In a packer test, a constant pressure is usually applied and the flow is allowed to come to equilibrium.

The analysis procedures for the pump-in tests in the steady state condition are independent of whether constant head or constant flow is used to reach the steady state and are independent of the magnitude of the applied head. However, if one or more observation wells are available, more sophisticated and accurate analysis procedures can be utilized.

7.1.2 Pump-Out Tests

Pump-out tests can only be conducted below the water table. The rising head test is a simple non-equilibrium pump-out test. In this test, the borehole is bailed or pumped and the rate of rise of the water in the borehole to pre-test conditions is observed.

Well pumping tests are generally considered the most accurate tests available below the water table. There are basically two kinds of well pumping tests: constant discharge and constant drawdown. Water may be pumped out of the well at a constant discharge rate and the water-level drawdown is measured, or a constant drawdown is maintained and the pumping rate is recorded. The analysis of a well pumping test is simplified if equilibrium conditions are reached. Often, however, it is not possible to reach equilibrium because of time limitations or because of the nature of the recharge to the pumped zone. Accordingly, a number of analysis methods are presented for non-equilibrium conditions.

7.1.3 Supplementary Techniques

Various monitoring techniques are available to enhance the information provided by the primary permeability tests. For example, measurement of the rate of flow with depth over the test zone can help detect channelization or identify interbedded layers of contrasting permeabilities such as sandstones and siltstones. In a jointed rock mass, the permeability is a function of joint size and spacing. Calipers or borehole cameras can be used to estimate the frequency and dimensions of joint fractures. The use of the cameras is particularly recommended. Electric logging of a borehole in conjunction with exploration drilling can often furnish useful information for planning permeability field test programs. These techniques are described in Section 9.0, "Supplementary Discussions for Fractured Rock."

7.1.4 Available and Developing Capabilities

In-situ permeability studies for fractured media have received generally inadequate attention in the field but are important in applications to in-situ leaching operations. For testing in rock, the packer test is believed to be the best method available for assessing variability of permeability.

Review of past practice has revealed several inadequacies with regard to constant head packer tests. The standard packer test method has primarily evolved from techniques for permeability testing of dam foundations to 100 or 200 ft. depths. While careful use of the standard packer test method will enable measurements to be carried out to depths of about 150 feet, the equipment and procedures are considered inadequate at greater depths.

Suitable modifications to the standard packer test are recommended, based on in-house experience and recent research adaptations noted in the literature. Overall efficiency should increase as a result of these recommendations, including reduced manpower costs, operating costs and operating time, as well as increased operating ease and accuracy of test results.

Most of the specified permeability test methods are in general accordance with past practice and consequently available test methods will be found generally capable. Because available packer test methods were found to be lacking in some respects, the specifications for packer tests incorporate the research and development advances mentioned.

7.2 VARIABLE HEAD TESTS

Two variable head tests are in common use: the falling head test and the rising head test. Falling head tests may be used above and below the ground water table. Rising head tests are possible only below the ground water table.

7.2.1 Equipment

The following equipment with associated approximate costs is needed to perform variable head tests, in addition to equipment used to prepare the borehole.

Falling Head Test

Equipment	Approximate Cost, $
Water supply, somewhat warmer than the groundwater, if possible (to prevent formation of air bubbles in the test fluid)	Indeterminate
Pump	Up to 1,000
Thermometer	8
Water level indicator (electrical probe or linen tape with sinker)	100

Rising Head Test

Equipment or Facility	Approximate Cost, $
Water Disposal Area (the disposal area must not allow water to percolate into the test zone)	Indeterminate
Bailer: Cylinder with bottom check valve, such as dart valve bailer and drill rig/hoist to operate bailer; or compressor with air hose; or suction pump for lifts of less than 15 ft.	a) Dart valve bailer: 250 b) 125 cfm air compressor and hose: rental per day: 35
Water level indicator (electric probe or linen tape with sinker)	up to 100

7.2.2 Manpower Requirements

Two persons are required to perform rising and falling head tests: an engineer, to determine pump-out/pump-in rates needed to attain desired drawndown/excess head levels and to lend general supervision, and a technician, who should assist the engineer in developing the well and recording the necessary data during the test. If stage testing is performed, that is, if tests are performed incrementally with depth as the borehole is advanced, a drill will be available after each stage has been completed and assistance in test performance may be provided by the driller and/or his helper under the supervision of the engineer (or competent technician).

7.2.3 Hole Preparation

Select a borehole size so that the variable head test can be run in a reasonable length of time. It is desirable to run the test to 90 percent equalization, i.e., until 90 percent of the differential head created by bailing or adding water is dissipated (Hvorslev, 1951). A table of 90 percent equalization time for various borehole dimensions is given in Figure 7-1, as well as a method to compute the time to 90 percent equalization for any borehole diameter.

If the test is run in a piezometer, the filter length of the piezometer should be sufficiently long to give a representative value of permeability within a 24-hour test period (Tabor) or longer. In boreholes, the test section length should be about 5 feet (USBR, G-97) with the borehole cased above it. The analysis becomes less accurate if the entire borehole is uncased.

The borehole should be prepared by one of the following preparation methods, listed in order of decreasing desirability.

Falling Head Test

F: Standpipe Piezometer
(very low permeability materials only)

D: Perforated Casing over Test Section, Cased Above

E: Fully Cased Borehole open only at base
(very high permeability materials only)

C: Gravel-filled Test Section, Cased Above

B: Unlined Test Section, Cased Above

Rising Head Test

D: Perforated Casing over Test Section, Cased Above

C: Gravel-filled Test Section, Cased Above

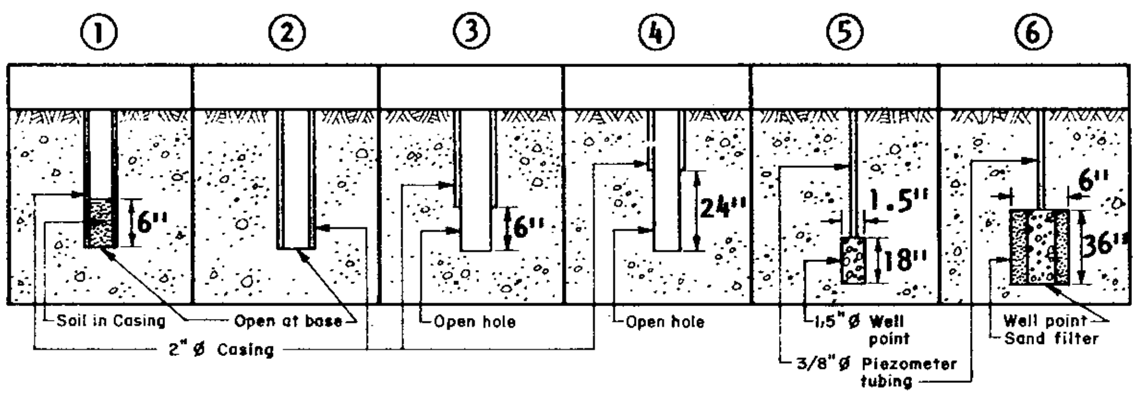
B: Unlined Test Section, Cased Above

7.2.4 Test Procedures

Falling Head Test

1. Prepare borehole as prescribed above.

APPROXIMATE SOIL TYPE	TIME FOR 90 PERCENT EQUALIZATION = T_{90} (m = minutes, h = hours, d = days) SAND			SILT			CLAY			
COEFFICIENT OF PERMEABILITY IN CM/SEC	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}
1 2-IN. CASING - SOIL IN CASING, L = 3D = 6-IN.	6^m	1^h	10^h	4.2^d						
2 2-IN. CASING - SOIL FLUSH BOTTOM CASING	0.6^m	6^m	1^h	10^h	4.2^d					
3 2-IN. CASING - HOLE EXTENDED, L = 3D = 6-IN.		1.5^m	15^m	2.5^h	25^h	10^d				
4 2-IN. CASING - HOLE EXTENDED, L=12D=24-IN.			6^m	1^h	10^h	4.2^d	42^d			
5 3/8-IN. PIEZOMETER WITH WELL POINT DIAMETER 1½-IN., LENGTH 18-IN.				3^m	30^m	5^h	50^h	21^d		
6 3/8-IN. PIEZOMETER WITH WELL POINT AND SAND FILTER, D = 6-IN., L = 36-IN.					12^m	2^h	20^h	8.3^d	83^d	



NOTE:

The time to 90% equalization can be estimated for a soil/rock mass as follows; $T_{90} = 2 \cdot 3 \cdot \frac{A}{F \cdot k}$ where

A is the cross-sectional area of the standpipe/borehole

k is the estimated permeability

F is the shape factor (Hvorslev) shown on Figures 8-2 and 8-3

Figure 7-1 TIME TO 90% EQUALIZATION (after Hvorslev, 1951)

2. If possible, bail a small amount of groundwater and record its temperature.
3. Record the temperature of the water to be added.
4. Fill the borehole/piezometer to top with clean water. In highly permeable material the water level should be raised as high as is practical.
5. Using an electric probe or weighted linen tape, read the depth to water from the top of the casing. Withdraw and dry the tip of the level indicator after each reading.

   Read the depth to water:
   - at 30 sec intervals for 5 min
   - then at 1 min intervals for 10 min
   - then at 5 to 10 min (or longer) intervals depending on the rate of fall for the remainder of the test and the estimated value of T_{90}.

   Record data until 90 percent of the excess head has dissipated. The time required for 90 percent equalization is a function of soil/rock permeability and borehole geometry and may vary from a few minutes to several days. Generally though, an hour should be allowed for each test (Dixon and Clark, 1975). A table of 90 percent equalization times is given in Figure 7.1.
6. Record the data on the data sheet shown in Figure 7-2.

<u>Rising Head Test</u>

1. Prepare the borehole as prescribed above.
2. Lower the water level in the borehole by bailing, by dewatering with a compressor air line, or by pump where total suction lift is not more than about 15 feet.
3. Using an electric probe or weighted linen tape, read the depth to water from the top of the casing. Withdraw and dry the top of the level indicator after each reading.

   Read the depth to water:
   - at 30 sec intervals for 5 min
   - then at 1 min intervals for 10 min
   - then at 5 to 10 min intervals depending on the rate of rise for the remainder of the test and the estimated value of T_{90}.

   Record data until 90 percent of the excess head has dissipated. The time required for 90 percent equalization is a function of soil/rock permeability and borehole geometry and may vary from a few minutes to several days. Generally though, an hour should be allowed for each test (Dixon and Clark, 1975). A table of 90 percent equalization time is given in Figure 7-1.

4. Record the data on the data sheet shown in Figure 7-2.

7.3 CONSTANT HEAD AND CONSTANT FLOW TESTS

7.3.1 Equipment

The following equipment with associated approximate costs is needed to perform constant head and constant flow tests, in addition to equipment used to prepare the borehole.

Equipment	Approximate Cost $
Water supply	Indeterminate
Hose and Suction Pump	100
Storage Tank	150
Test Pump and Suction Line	350 gpm delivery against total head of 275 feet: 1,200
Water Meter, Flow Meter or Volume-Calibrated Container	800
Water level indicator (electrical probe, weighted linen tape or steel tape) for test well and each observation well	up to 100

A water level indicator is required if the water level in the casing is below a convenient depth for visual observation. For water levels near the top of the borehole, a weighted linen tape or steel tape is satisfactory to measure the depth to water.

Additional important equipment criteria relevant to constant head and constant flow tests are included in Appendix B of this report.

7.3.2 Manpower Requirements

Essentially the same personnel are needed to perform a constant head or constant flow test as with the variable head tests. Two persons (engineer and technician) are required to develop the well, set up the equipment, and perform the test. If observation wells are incorporated during the test, an additional person is needed to observe and record the water level in each observation well.

Location: ____________ Name: ____________

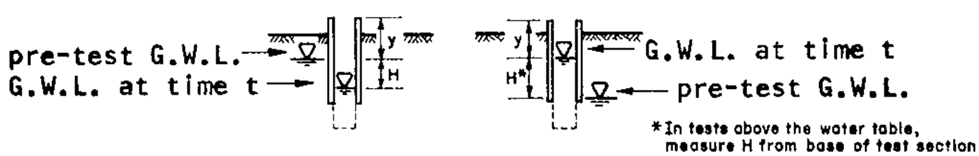
Borehole No: ____________ G.W. Elevation before Test: ________

Date of Test: ____________ Type of Test (Falling/Rising): ________

Ground Elevation: ____________ G.W. Temperature ___°F Added Water ___°F

DIMENSIONS OF

Borehole	or Piezometer
Depth of BH before test ________ (Below G.L.)	Depth of Top of Filter ________ (Below G.L.)
Depth of BH after test ________ (Below G.L.)	Depth to Bottom of Filter ________ (Below G.L.)
Depth of Unperforated Casing ________ (Below G.L.)	Length of Filter Section ________
Length of Test Section below Casing (May be "0") ________	Diam. of Filter Section ________
Diam. of Test Section ________	Piezometer Type ________
Height of Casing above G.L. ________	Depth to Top of Piez. ________ (Below G.L.)
Test Section: ▱ Unlined ▱ Well Screen ▱ Open at Bottom Only	Depth to Base of Piez. ________ (Below G.L.)
Inside Diam of Unperforated Casing ________	Outside Diam. of Piez. ________
	Inside Diam. of Piez. ________
	Inside Diam. of Tubing above Piez. ____
	Height of Tubing above G. L. ________



RISING HEAD TEST **FALLING HEAD TEST**

ELAPSED TIME	Depth to Water from Top of Casing : y	Excess Head : H
t = 0		H_0 =
t = 30 sec		
t = 1 min		
t = 1.5 min		
t = 2 min		
t = 2.5 min		
t = 3.0 min		

(continued)

Figure 7-2 VARIABLE HEAD TEST DATA SHEET

Location: ____________________ Name: ____________________________

Borehole No: ____________________

Date of Test: ____________________

Elevation of Base of Test Section: ____________________

Elevation of Top of Test Section: ____________________

ELAPSED TIME	Depth to Water from Top of Casing : y	Excess Head : H
t = 3.5 min		
t = 4.0 min		
t = 4.5 min		
t = 5.0 min		
t = 6.0 min		
t = 7.0 min		
t = 8.0 min		
t = 9.0 min		
t = 10.0 min		
t = 11.0 min		
t = 12.0 min		
t = 13.0 min		
t = 14.0 min		
t = 15.0 min		
t = 20.0 min		
t = 25 min		
t = 30 min		
t = 35 min		
t = 40 min		
t = 45 min		
t = 50 min		
t = 55 min		
t = 60 min		

Figure 7-2 VARIABLE HEAD TEST DATA SHEET (CONTINUED)

7.3.3 Borehole Preparation

1. The borehole should be prepared by one of the following borehole preparation methods, listed in order of decreasing accuracy:

   F: Standpipe Piezometer (best for low permeability materials below water table)

   B (or C): Unlined (Gravel-filled) Test Section, Cased Above (best method above water table if filled only to top of test section)

   D: Perforated Casing over Test Section, Cased Above (best method below water table, unless F can be used)

   B (or C): Unlined (Gravel-filled) Test Section, Cased Above (if the height of water during the test is maintained above the test section, this method is acceptable only if the casing fits very tightly to the borehole wall)

2. A plastic pipe, inserted into the test well to 2 to 6 inches above the base of the hole, should be used to introduce water into the well. This will prevent entering water from splashing the water level indicator cable, thus precluding withdrawal and drying of the indicator after each reading.

Observation Well Preparation

If possible, install one or more observation wells. Particularly for tests conducted below the water table, observation wells can improve the estimate of permeability. Above the water table, observation wells are useful only if the intake and observation wells penetrate to an impermeable stratum. For tests below the water table, the observation wells should extend at least 5 feet below the water table.

The observation wells should have as small a diameter as possible to minimize the time needed for water to percolate into the well and rise to a height representative of the groundwater pressure. This time period has been discussed previously, and is called the time lag. However, the observation wells should have an intake of sufficient area so that clogging is not a problem.

Observation wells should be drilled and cleaned with the same care as used for test well preparation. Specific methods and procedures for well preparation are presented in Section 6.5.

Any of the following borehole preparation methods can be used to prepare the observation wells.

F: Piezometers - for a low permeability medium. These should not be used in high permeability strata because the permeability of the tip may be less than that of the surrounding mass.

A or B: Unlined Test Section, may be cased above

C: Gravel-filled Test Section, Cased Above

7.3.4 Test Procedure

1. For both the constant head and constant flow tests, select the limiting height of water to be used in the test.

   In unconsolidated deposits above the water table, the problem of upward seepage around the casing during the test can be eliminated by filling the hole with water only to the top of the test section.

   In weakly cemented rock and cohesive soils, hydraulic fracture may occur if the applied excess head is too great. The applied excess head is defined here as the difference between the height H of water in the test well during the test and the height H_w of water in the well before the test ($H > H_w$), and should not exceed 0.5 D/γ_w, where D is the depth to the test section and γ_w is the unit weight of water.

2. Record the temperature of the water to be added. If possible, bail a small quantity of groundwater and measure its temperature. The added water should be slightly warmer than the groundwater to prevent bubbles from forming as the added water infiltrates the soil or rock (USBR G-97).

At this point the procedures diverge. For the constant head test, proceed with steps 3, 4, 6 and 7. For the constant flow test, proceed with steps 5, 6 and 7.

3. (Constant head test only) If measurements are going to be at depth, lower the electrical probe into the hole until the electrodes are at the desired depth, i.e., the height at which the water level will be maintained during the test. If two probes are available, lower them so that one is several inches above the other. Fix them securely in place.

   When a constant head is maintained, the signal at the single probe readout will go on and off repeatedly. With two probes, a constant head can be maintained so that one probe is continuously on and one continuously off.

4. (Constant Head Test only) Begin flow into the hole. Vary the flow to maintain a constant height of water in the well. If the flow is from a constant head tank, no flow adjustment is needed. Record the flow rate directly from a flow meter, or by measuring the volume passing the water meter over 1 minute intervals. In a low permeability medium it may be possible to shut off the main flow and measure the flow needed to maintain a constant head by pouring water into the hole with a calibrated container over a 1 minute interval.

5. (Constant Flow Test only) Begin the flow. Maintain the flow at a constant rate, measured with a flowmeter, water meter or calibrated container (depending on the rate of flow). Record the depth to the top of the water with an electrical probe, weighted linen tape, or steel tape.

6. (Both Tests) Record the flow and depth to groundwater at the following intervals: 5 minute intervals for 20 minutes
   15 minute intervals for 4 hours
   1 hour intervals for 24 hours
   If the test is conducted above the water table, it may be terminated at any time after the first 20 minutes if the last two readings differ by less than ± .2 foot (USBR, Earth Manual).

7. Record the data on the data sheet, Figure 7-3.

7.4. CONSTANT HEAD TEST WITH PACKERS - TEST METHOD A

A packer test is a type of constant head test and is analyzed similarly. However, the equipment and procedures required to operate at high pressures are more complex than for a constant head test with gravity-induced water pressure. As a result, the methods for conducting packer tests are presented separately.

Two methods for conducting packer tests are presented in Sections 7.4 and 7.5. Both are acceptable permeability measurement methods, however one may prove more "capable" than the other under certain circumstances. Both methods are evaluated in the Test Method Capability Matrix, Section 10.0.

The first method, Method A, has been generally accepted in past practice. Test accuracy, however, is dependent upon proper calibration of required equipment. Conditions in the field may restrict such calibration operations, thereby reducing the reliability of the results. Other physical aspects such as packer leakage may affect the results as well. In view of such potential inaccuracies in the accepted test method, an alternate packer test method, Method B, has been specified. The primary difference between the two methods is that Method A requires calibration of friction head losses in the system to estimate the effective fluid pressure in the test cavity, while Method B uses a sensing device in the test section to monitor the cavity pressure directly. Method A is presented in this section, and Method B follows in Section 7.5.

Location______________________

Injection Well No.______________________

Complete if data recorded for Injection Well:

Depth to top of Test Section in Injection Well__________

Depth to base of Test Section in Injection Well__________

I.D. of Injection Well______________________

Perforated Casing (if any) in Injection

Well: % area of slots______________________

Height of Gravel in Injection Well, if any__________

If Piezometer: Type______________________

Depth to Top/Base of Tip______________________

Depth to Top/Base of Filter______________________

Elevation of Injection Well______________________

Depth to Groundwater______________________

Elevation of Groundwater______________________

Temperature of added water______________________

Temperature of ground water______________________

Name of Operator______________________

Observation Well No.______________________

Complete if data recorded for Observation Well:

Depth to top of Observation Section in well__________

Depth to base of Observation Section in well__________

I.D. of Observation Well______________________

Perforated Casing (if any) in Observation

Well: % area of slots______________________

Height of Gravel in Injection Well, if any__________

If Piezometer: Type______________________

Depth to Top/Base of Tip______________________

Depth to Top/Base of Filter______________________

Elevation of Observation Well______________________

Depth to Groundwater______________________

Elevation of Groundwater______________________

t_i	Δt	$t_{ave.}$	V_t	ΔV	Q	D_w	H_t
Time	Time Between Readings	Time at Middle of Time Interval $t_{ave} = \frac{t_{i+1}+t_i}{2}$	Water Meter Reading at Time t	Change in Water Meter Reading over Δt	Flow Over Time Δt $Q = \Delta V/\Delta t$ Q = flow at time $t_{ave.}$	Depth to Water at Time t	Height of Water in Well at Time t

Figure 7-3 CONSTANT HEAD TEST DATA SHEET

Figure 7-3

7.4.1 Equipment

The following equipment, with associated approximate costs, is needed to perform Packer Test Method A.

Equipment	Approximate Cost, $
Drill rig or equivalent to lower and raise the water pipe and packers	Indeterminate approximate rental per hr: 60
Water meter, range 1 to 8 inches, depending on the permeability of the test zone and the surface area of the test section. Generally 1-inch and 4-inch meters will be appropriate.	4-inch meter: 800 3-inch meter: 400 1-inch meter: 100
Pressure gage, range (in psi) to at least 1.5 times test depth (in feet).	30
Pump, centrifugal-type, capacity to 350 gpm is adequate for most tests.	1,200
One or two packers.	
Drill rods or 1-1/4-inch diameter water pipe.	Rubber hose per ft: 10 Water pipe per ft: 1
Water supply with suction pump, hose and settling tank if water is obtained from a local pond, river, etc.	Indeterminate
Swivel and hose to pump; swivel connects hose to drill rods or water pipe.	550
Compressed air and air hose if air-inflatable packers used.	Nitrogen bottle (230 cubic ft): 10

The USBR (G-97) recommends arranging the equipment as follows, beginning from the water source:

> "Source of water, suction line, pump, water line to settling and storage tank or basin, suction line, centrifugal test pump, line to water meter adapter if required, or to water meter, short length of pipe, plug valve, water line to swivel, sub for gage, pipe or rod to packer. All connections should be kept as short and straight as possible with a minimum number of changes in diameter of hose, pipe, etc."

This will minimize head losses.

A possible equipment set-up for packer testing is shown in Figure 7-4.

Further comments on selection of equipment are made below.

a) Packer Selection - A packer is an expandable plug which is used at the top, or the top and bottom of a permeability test section to hydraulically isolate the test section from the remainder of the borehole. Packer selection is the key to a successful pressure test. The packer must have sufficient dimensions so that leakage past it is not a problem. The packer should be selected so that its length is at least 5 times the borehole diameter (USBR G-97). In erodable formations a longer packer or several packers in series may be required to get a good seal. The packer must be flexible enough to deform to the irregular shape of the borehole.

   Two principal types of packers are acceptable for packer testing: Mechanical packers and pneumatic packers. There are several types of mechanical packers including wedge type, bottom set and screw set. Wedge type packers are simple but useful only to 25 psi (Acker, 1974). Both bottom set and screw set packers have a rubber cylinder which is mechanically expanded against the sides of the borehole by compressing the cylinder. With the bottom set type, the rubber cylinder is located between drill rods attached to the drill. The rubber cylinder is located between drill rods extending to the bottom of the borehole and drill rods attached to the drill. The rubber cylinder is compressed and expanded laterally when the drill rods are loaded by jacking them against the drilling machine. With the screw type solid rubber packer, an adjusting nut is used to compress the packer. This type of packer is suitable for hard rock and moderately jointed, non-caving, non-erodable formations. Although it can be used to higher pressures than pneumatic packers, the difficulty of applying torque at greater depth limits its flexibility.

   Pneumatic packers are by far the most popular type of packer in current practice (Maini, 1971). They are recommended for sedimentary formations, irregular borehole profiles, and caving and erodable material. However, operational pressure criteria may pose some limitations to currently practiced pneumatic packer construction. A basic criterion to in-situ permeability measurement by the packer test method is that the excess test cavity pressure may not exceed 0.5 psi/ft depth to the test section. Otherwise, hydraulic fractures may occur, significantly altering subsequent test results. Generally, this is not a problem for present test methods and equipment down to depths of about 400 ft. All that has been found necessary is to expand the packer at a small differential pressure above the test cavity pressure. A minimum differential of 5 psi is needed (Herndon and Lenahan, 1976), and Gale (1975) has found that a differential pressure of 30-40 psi is sufficient to seal most leaks.

   Considering that about 200 psi is the maximum differential pressure reported for the previously constructed pneumatic packers (Sherard et al 1963) about 165 psi will be found to be the maximum desirable differential injector pressure over any in situ groundwater pressure in the test zone when making in situ permeability test measurements. Consequently, for

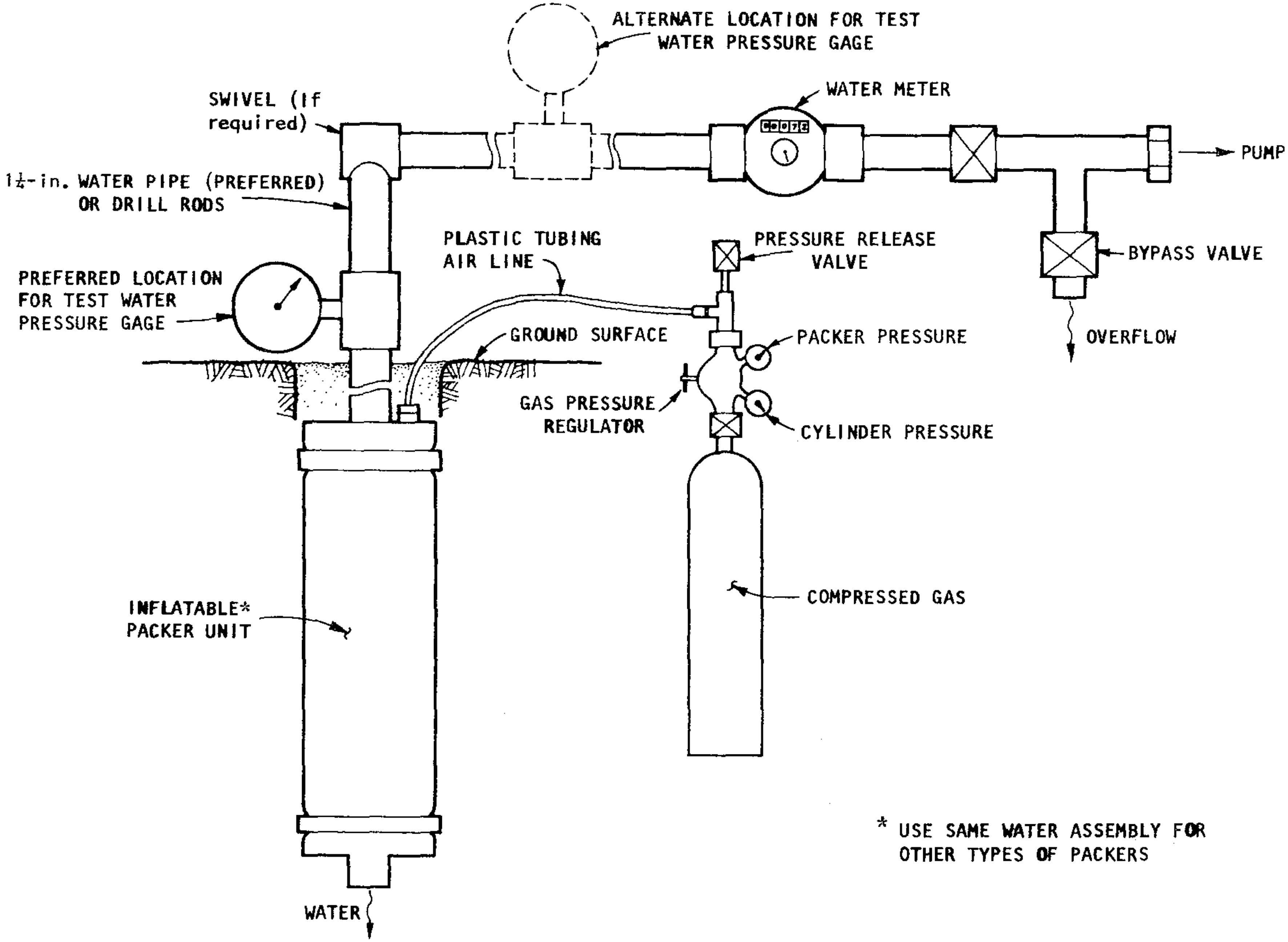


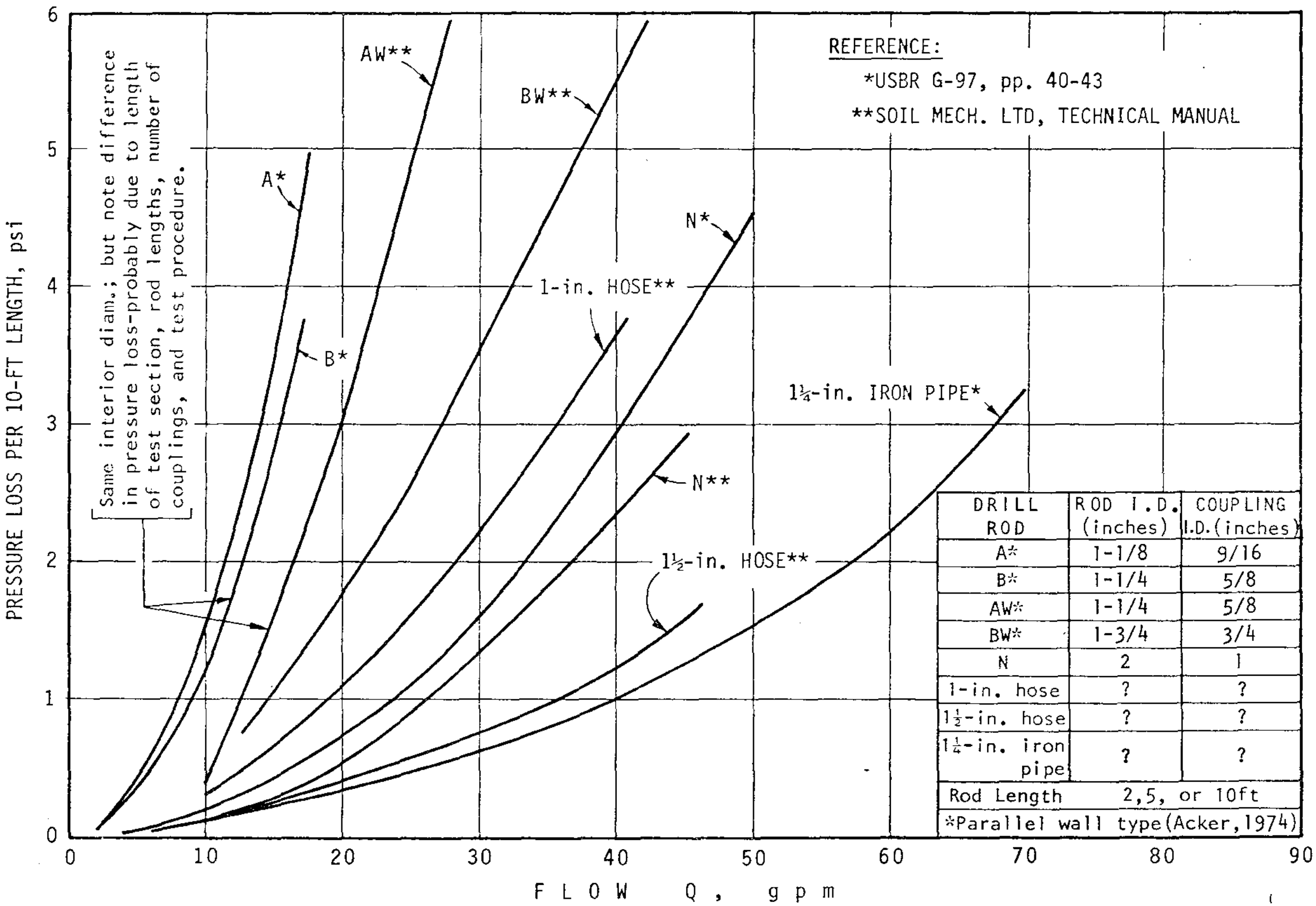
Figure 7-4 PACKER TEST SET-UP

test zones greater than about 380 ft in depth and where there is no offsetting groundwater pressure acting on the test zone, a downhole pressure regulating valve could be installed in the injection pipe above the packer to limit the injection water pressure to 165 psi and thus avoid leakage past a current packer inflated at maximum pressure.

Hydraulic packers have also been reported in field permeability testing. Depending on the suitability of hydraulic pressure control furnished for the packer operation, such packers can be equally as effective as pneumatically actuated packers. Automatic, continuous hydraulic pressure regulation during testing, equivalent to the gas regulation system shown in Figure 7-4, is desirable. Hydraulic packers that utilize shear pins for control of the seating pressure and permit no pressure adjustment thereafter during testing are considered to be too insensitive for permeability testing requirements.

Packer tests can be run either in a stage test format, where tests are performed incrementally as the borehole is advanced, or in a continuous series of tests after the entire length of borehole has been advanced. The former technique uses only one packer, because the bottom of the borehole represents the bottom of the test section. A minimum of two packers are required where the borehole is drilled to its final depth before testing begins. In the latter case, the packers are connected by a perforated pipe. This pipe spans the test cavity and can be 5 to 20 feet long. It should have a perforated area at least twice the cross-sectional area of the pipe (USBR G-97). In erodable formations where it is difficult to seal the packer, it may be desirable to use several packers in series to obtain a good seal. In caving formations where casing is required, the top few inches of the pneumatic packer may be left in the casing to facilitate easy packer withdrawal. However, this shortens the length of packer seal below the casing, and may allow leakage past the packer (USBR G-97).

b) Size of Rod or Pipe - Drill rods are not recommended for use in packer testing. Friction losses become excessive when flow through the rods exceeds 15 gpm or when the length of the rods is more than 50 feet. 1-1/4-inch pipe is more satisfactory for moderate depths. A graph showing pressure losses per 10-ft section at various delivery rates of water, for several drill rod sizes and 1-1/4-inch pipe is given in Figure 7-5. Those curves plotted by the USBR (G-97) were compiled from tests in which the pressure gage was set between the swivel and the pump and, therefore, the swivel friction losses are included. 1 and 1-1/2-inch swivels were used in the tests, with nominal diameters of 1/2 and 3/4 inch, respectively. It can be observed that at high flow rates (>100 gpm) even the 1-1/4-inch diameter pipe will effect significant head losses. When testing at large depths in highly permeable strata, large flow rates may be required to produce desired pressures. In such cases, a larger diameter pipe is required to reduce friction losses.



DRILL ROD	ROD I.D. (inches)	COUPLING I.D. (inches)
A*	1-1/8	9/16
B*	1-1/4	5/8
AW*	1-1/4	5/8
BW*	1-3/4	3/4
N	2	1
1-in. hose	?	?
1½-in. hose	?	?
1¼-in. iron pipe	?	?
Rod Length	2,5, or 10ft	
*Parallel wall type (Acker, 1974)		

Figure 7-5 CALIBRATING PRESSURE LOSSES IN DRILL RODS/WATER PIPE

c) Pumping Equipment - In past practice, many packer tests have been run using the circulation pump on the drill for pumping the water. Such pumps are often the multiple cylinder type, which delivers a fluctuating pressure. These pumps are not recommended, because the fluctuating pressures are often difficult to read accurately, and averaging is required to determine the true pressure.

   Instead, a centrifugal pump should be used. The required capacity of the pump will vary with the depth of the test section and the permeability of the rock/soil mass. However, a 350 gpm pump capacity against a dynamic head of 300 feet (excess head in test cavity) should be adequate for most tests. When testing highly permeable strata at depths greater than about 150 feet, higher capacity pumps may be required to overcome friction losses in the pipe. Drill rigs used for making pressure tests should be equipped with auxiliary pumps of this type (USBR G-97).

d) Swivels - Swivels used on most drill rigs have a narrow constriction which carries a considerable loss of pressure as the water passes through. Swivels with a uniform inside diameter are recommended for use in packer testing.

e) Location of Pressure Gage - In most tests the pressure gage is located between the pump and the water meter or between the water meter and the swivel. Although the latter location is less objectionable, the necessity for estimating pressure loss in the water swivel can be avoided if the pressure gage is located near the top of the pipe or rod used for testing, that is, between the packer and the swivel (see Figure 7-4). The fitting for the gage in this case should be located below the bottom of the swivel a distance at least 10 times the diameter of the pipe or rod (USBR G-97).

f) Recommended Types of Water Meters - Required water deliveries in packer tests may range from less than 1 gpm to as much as 400 gpm. No one meter is sufficiently accurate to be used for all ranges. Therefore, two meters for each rig are recommended: a 4-inch impeller-type meter to measure flows greater than 50 gpm and a 1-inch disk-type meter for flows between 1 and 50 gpm. When possible, water meters should be tested at least once a month.

   Adapters should be available for each meter. The adapters should be at least 10 times as long as the diameter of the rated size of the meter. This length of adapter permits the water flow to become steady and eliminates the turbulence due to a change in pipe diameter. The accuracy of most meters is influenced adversely by turbulent flow. An adapter should be used on the upstream side of each meter where the water line from the pump to the meter has a different diameter than the nominal size of the meter (USBR, G-97).

7.4.2 Manpower Requirements

A crew of 4 persons is recommended to perform Packer Test Method A. Calibration of the equipment prior to testing will require two technicians to observe and record the necessary information at either end of the system. During actual testing, the hoist or drill operator and his helper, a knowledgeable technician, and one engineer (supervisor) will be required. The four-person crew is recommended for both the stage test (single packer) and continuous test (double packer) procedures.

7.4.3 Hole Preparation

There are two ways to drill a borehole for packer testing. One drilling procedure is to drill to the desired test depth, case (if required) to the top of the test section, insert the packer and perform the test. The packer is then removed and the borehole is drilled to the next test depth. A simpler procedure is possible in sound rock where casing is not required. The hole may be drilled completely before testing. In this case, the testing begins at the bottom of the borehole and proceeds upwards.

The borehole may be prepared by one of the following borehole preparation procedures:

A: Unlined Borehole (consolidated deposits)
C: Unlined Test Section, Cased Above (unconsolidated deposits)

If the borehole is unlined (Method A) and is drilled to its final depth before testing begins, it is only necessary to clean the hole once over its full length. If a stage test is conducted in an unlined borehole, the hole must be cleaned prior to each test.

Select a hole diameter between AX and NX (up to 3-inch diameter) (Milligan, 1975; Sherard, 1963). Small hole diameters are desirable because the packers perform better. (The smaller the hole diameter, the less the total uplifting force on the packer. For a packer of fixed length, the total force displacing the packer increases with r^2 but the shear resistance along the sides of the packer increases only as "r".) However, if down-the-hole-inspection methods are used in conjunction with the permeability test program, a borehole of sufficient diameter to accept the instrument is required.

The test section should be roughly 10 feet long (USBR G-97) but may vary from 5 to 20 feet. The hole length can be varied in order to get good packer seating or to isolate a specific zone. In very permeable formations, the shorter test length may be needed to build up a back pressure (USBR G-97).

7.4.4 Test Procedure

1. Prepare the borehole as discussed above.

2. Determine the head loss in the drill rods or water pipe as follows. Assemble the drill rods (water pipe), swivel, pressure gage, water meter and pump at ground level. Fit the last drill rod with a pressure gage, a short length of pipe, and a valve and hose as shown in Figure 7-6.

   Pump at a range of expected flow rates and compute the head loss between the two pressure gages for each flow rate. Use the valve at the end of the drill rods to control the flow. The maximum flow rate used should deliver a maximum pressure of 0.5 psi/ft depth to the test section. For example, for the length of drill pipe (water pipe) corresponding to a test depth of 40 feet, the maximum flow rate should produce a pressure of 20 psi in the gage at the far end of the system. Other test flow rates can be percentages of the maximum value determined. Test pressures above 0.5 psi/ft depth may fracture the formation, resulting in an overestimation of the true permeability.

   Repeat this process over the range of drill rod lengths required for anticipated test depths. Curves are available to estimate head loss per 10 foot length of drill rod but they do not include the effects of the swivel, pipe connections, and individual characteristics of the drill rods (water pipes) used. The longer the drill rods, the greater the head loss and the more important it becomes to calibrate the equipment (Little et al., 1963). Dick (1975) stresses the importance of careful, routine calibration of equipment which conducts the water from the point of pressure measurement to the section of borehole being tested. The accuracy of packer test results depends significantly on the correct estimation of the head loss in the system.

3. Record the data in Step 2 on a log similar to that shown in Figure 7-7. The data may be plotted in either of two ways, depending on personal preference. Plot head loss (vertical scale) vs. flow, connecting the data points according to the length between the pressure gages, or plot head loss (vertical scale) vs. the length of the system, connecting the data points according to flow rate. Since it may prove difficult to obtain consistent flow values throughout the calibration procedure, the former plot is the easier of the two to develop. The plot will be used in the analysis of the packer tests.

4. Seat the packer or packers so that a test section of approximately 10 feet is obtained. If two packers are used to isolate the test section, begin testing at the bottom of the borehole. The seating of packers can be determined in conjunction with geologic information, or according to a prespecified interval. For example, if a 4-foot thick fractured section is sandwiched in massive rock, it should be isolated

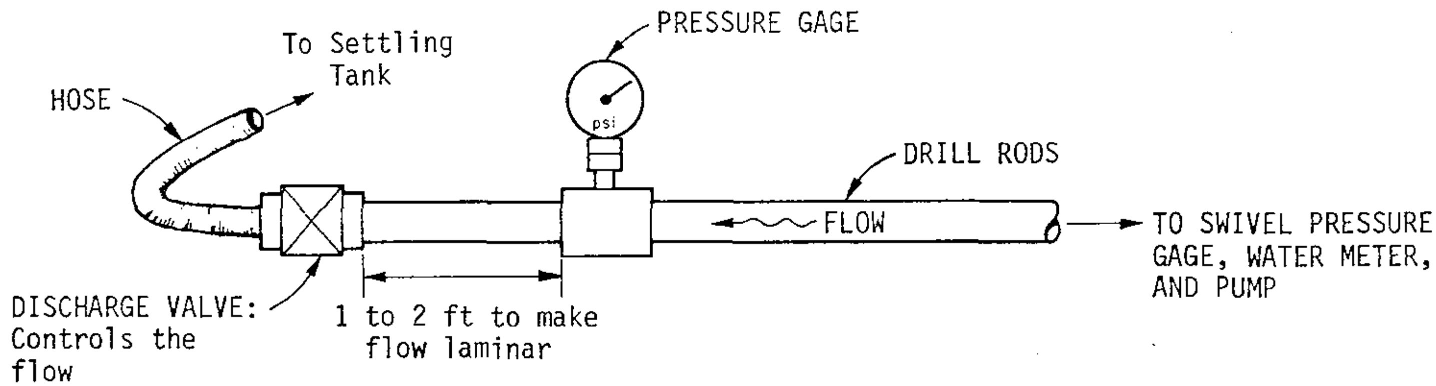


Figure 7-6 DRILL ROD ARRANGEMENT TO CALIBRATE HEAD LOSS IN DRILL RODS

Drill Rod/Water Pipe Type__________

Nominal I.D.__________

Length of Hose between Upper Pressure Gage and Drill Rod_____

Swivel Type (if below pressure gage)__________

I.D. Swivel__________

Length of Drill Rod Between Lower Pressure Gage and Valve____

Name__________

Date__________

Location__________

Borehole No.__________

Nominal I.D. of Hose__________

Sketch drill rod/swivel/pressure gage arrangement

L	t	V_w	Δt	ΔV_w	Q	G_u	G_L	ΔG_t	$\Delta G_{\Delta t}$
Length of Drill Rod Between Gages	Time of Reading	Water Meter Reading at Time t	Time Between Readings	Change in Water Meter Reading Over Δt	Flow = $\Delta V_w/\Delta t$	Pressure at Upper Gage at Time t	Pressure at Lower Gage at Time t	Pressure Loss at Time t = $G_u - G_L$	Average Pressure Loss Over Interval Δt = $\Delta G_t - \Delta G_{t-1}$

Figure 7-7 DRILL ROD CALIBRATION DATA SHEET

Figure 7-7

Location__________
Borehole No.__________
Depth of Bottom of Borehole__________
Depth of Top of Test Section__________
Depth of Base of Test Section__________
Depth of Casing__________
Diameter of Borehole__________
Type of Drill Rod/Water Pipe__________
I.D. of Drill Rod/Water Pipe__________
Length of Drill Rod__________
Packer Type__________
Packer Length (Upper)__________

Name__________
Date__________
Elevation__________
Depth to Ground Water__________
Elevation of Ground Water__________
Height of Pressure Gage above Ground Level__________
Length of Hose between pressure gage and drill rod__________
Nominal I.D. of hose__________
Swivel type__________
Nominal I.D. of swivel__________

If two Packers Used:
Packer Length (Lower)__________
Length of Test Section__________
Water pipe I.D.__________
Area of pipe perforations__________

SYMBOL:	P_P	P_L	H_E	P_E	P_T	t	V_w	Δt	ΔV_w	Q	H_T
ITEM:	Water Gage Pressure	Pressure Loss in Drill Rods and Hose*	Static Head Gain: Elev. of Press. Gage - Elev. of Middle of Test Section	Pressure Gain due to H_E = $H_E \cdot \gamma_w$ Where γ_w = unit Weight of Water	Test Section Pressure $= P_E + P_P - P_L$	Time (min) From Start of Test	Water Meter Reading at Time t	Time (min) Between Readings	Change in Water Meter Reading Between Readings	Flow = $\Delta V_w/\Delta t$	Head at Test Section $= P_T/\gamma_w$ γ_w = unit Weight of Water
UNITS:						min		min			

*If calibration test run, see Figure 7-7. Otherwise, see Figure 7-5.

Figure 7-8 PACKER TEST DATA SHEET

with packers and pressure tested. A complete profile can be obtained by repositioning the packers at intervals equivalent to the test zone length.

5. Pump water into the test section at a specified constant pressure for 15 to 20 minutes, taking readings of total water flow and pressure at 5 minute intervals (USBR G-97). The test at this pressure is completed when the flow over two successive 5 minute intervals differs by less than 10 percent. In very permeable materials, a test duration of 5 or 10 minutes may be sufficient. In this case, record flow and pressure over 1 minute intervals until stable conditions are reached. Perform this procedure at varying applied water pressures. A recommended test pressure sequence is 1/2 P_{max}, 3/4 P_{max}, P_{max}, 3/4 P_{max}, and 1/2 P_{max}, where P_{max} is defined as follows:

   $$P_{max} = P_t + P_l$$

   where P_t is the maximum allowable excess pressure at the test section, typically 0.5 psi/ft depth to the test section, but subject to any limitations on permissible packer pressures as discussed in Section 7.4.1, and

   P_l is the pressure loss due to friction in the testing apparatus, determined using the procedure presented in Step 2, or estimated using Figure 7-5.

6. Record the data on a form similar to that in Figure 7-8. Analysis of the data to compute permeability is discussed in Section 8.2.

7. Plot the data, pressure (vertical scale) vs. flow, and compare with Figure 7-9 for evidence of problems such as leakage around packer, erosion of test zone, and clogging of fissures. A high quality test should provide a linear plot similar to that corresponding to laminar flow in Figure 7-9.

8. For the stage test procedure, remove the packer and prepare the borehole for the next test. In a predrilled hole, where two packers are used to isolate the test section, the packers are raised to the next test depth.

7.5 CONSTANT HEAD TEST WITH PACKERS - TEST METHOD B

It can be observed that an essential step in the Method A test procedure is the careful calibration of test equipment prior to testing. The desired data in a packer test are the flow rate and excess pressure applied to the given test section. Calculation of the true excess pressure at depth, as performed on the data sheet in Figure 7-8 requires the pressure loss in the packer stem (drill rod or water pipe) due to friction. In practice, short cuts are often taken, calibration is not performed regularly, and one set of "standard"

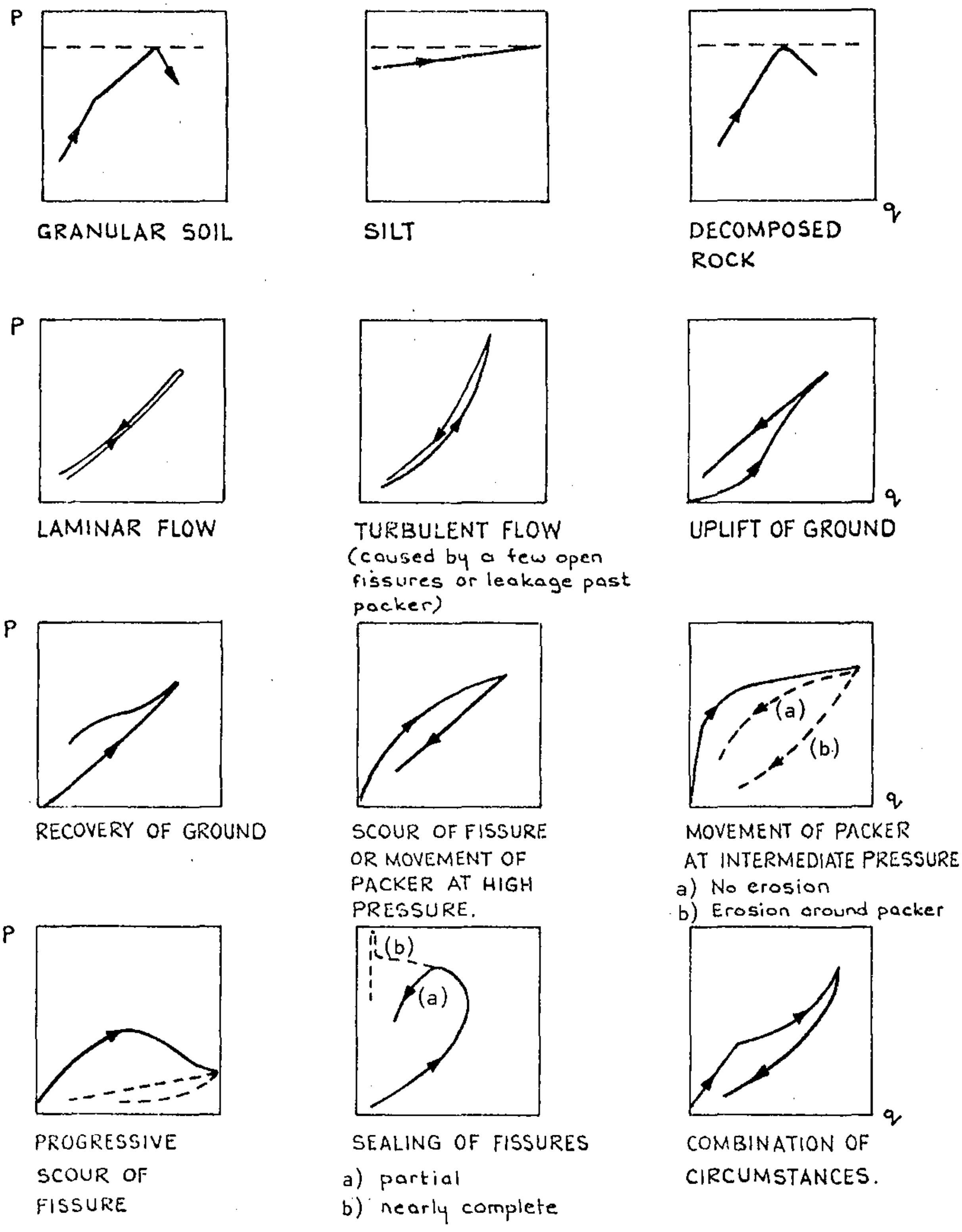


Figure 7-9 TYPICAL PRESSURE VERSUS FLOW CURVES FOR PACKER TESTS
(Dixon, 1975)

calibration curves is used for all tests. Such practice can lead to significant error in permeability tests. Dick (1975) discusses many sources of variable head loss, including flexible hoses, fittings, placement of pressure meters, and variable age of the equipment.

As the depth of testing increases, the frictional head loss in the packer stem increases significantly. It is at large depths where equipment calibrations are most crucial. The amount of time and effort required to calibrate the system increases with the increased stem length, as does the temptation to "standardize" calibration curves. Also, areal constraints imposed by a particular test site may make calibration impractical.

It is with this calibration difficulty in mind that a second method is proposed, a method which replaces the need for calibrating friction losses entirely. This is achieved by using a sensing device, a pressure transducer, which is placed down the hole in the test section and measures the test cavity pressure directly. This device also provides a means to detect and eliminate possible packer leakage. Using Test Method A, this information is frequently only noticed after the fact, if at all, from interpretation of P-q plots.

7.5.1 Equipment

Equipment	Approximate Cost, $
Drill rig or equivalent to lower and raise the water pipe and packers	Indeterminate approximate rental per hr: 60
Water meter, range 1 to 8 inches, depending on the permeability of the test zone and the surface area of the test section. Generally 1-inch and 4-inch meters will be appropriate.	4-inch meter: 800 3-inch meter: 400 1-inch meter: 100
Pump, centrifugal type, capacity to 350 gpm is adequate for most tests	1,200
One or two pneumatic packers	Indeterminate
1-1/4" diameter water pipe	Rubber hose per ft: 10
Water supply with suction pump, hose and settling tank if water is obtained from a local pond, river, etc.	Indeterminate
Swivel and hose to pump; swivel connects hose to drill rods or water pipe	550

Equipment	Approximate Cost, $
Compressed air and air hose for pneumatic packers	Nitrogen bottle (230 cubic ft): 10
Pressure transducer, cables and read out unit	750-2,000
Optional continuous recorder for transducer	1,000

Similar specifications as to equipment arrangement, set-up, pipe size, pump equipment, swivels and water meters apply to Test Method B as are described in Section 7.4.1 for Test Method A. Pneumatic packers are recommended for use with Test Method B. In-line pressure gages are not required for Test Method B, but are useful as a secondary data source.

Pressure transducers (electrical piezometers) are discussed in Section 6.5.7. Various types are commercially available, see Appendix C. Piezometer cables may be easily attached to the packer stem as the packer assembly is lowered into the borehole, or a spool can control the cable feed as the assembly is lowered. A typical test arrangement is given in Figure 7-10.

7.5.2. Manpower Requirements

Manpower requirements for Packer Test Method B are essentially the same as for Test Method A. A drill operator and his helper are required to advance the hole and hoist the packer assembly. A knowledgeable technician is required to perform the test and observe that equipment is operating properly. An engineer is required to guide the placement of packer sections and generally supervise the operation.

7.5.3. Hole Preparation

Hole preparation follows identically from the discussion for Test Method A, in Section 7.4.3.

7.5.4 Test Procedure

1. Prepare the borehole as per Section 7.4.3.

2. Position the packer or packers so that a test section of approximately 10 feet is obtained. If two packers are used to isolate the test section, begin testing at the bottom of the hole. The seating of packers to define the test section length can be adjusted according to geologic information obtained from core samples and/or down-the-hole surveys, or can be according to a pre-specified interval.

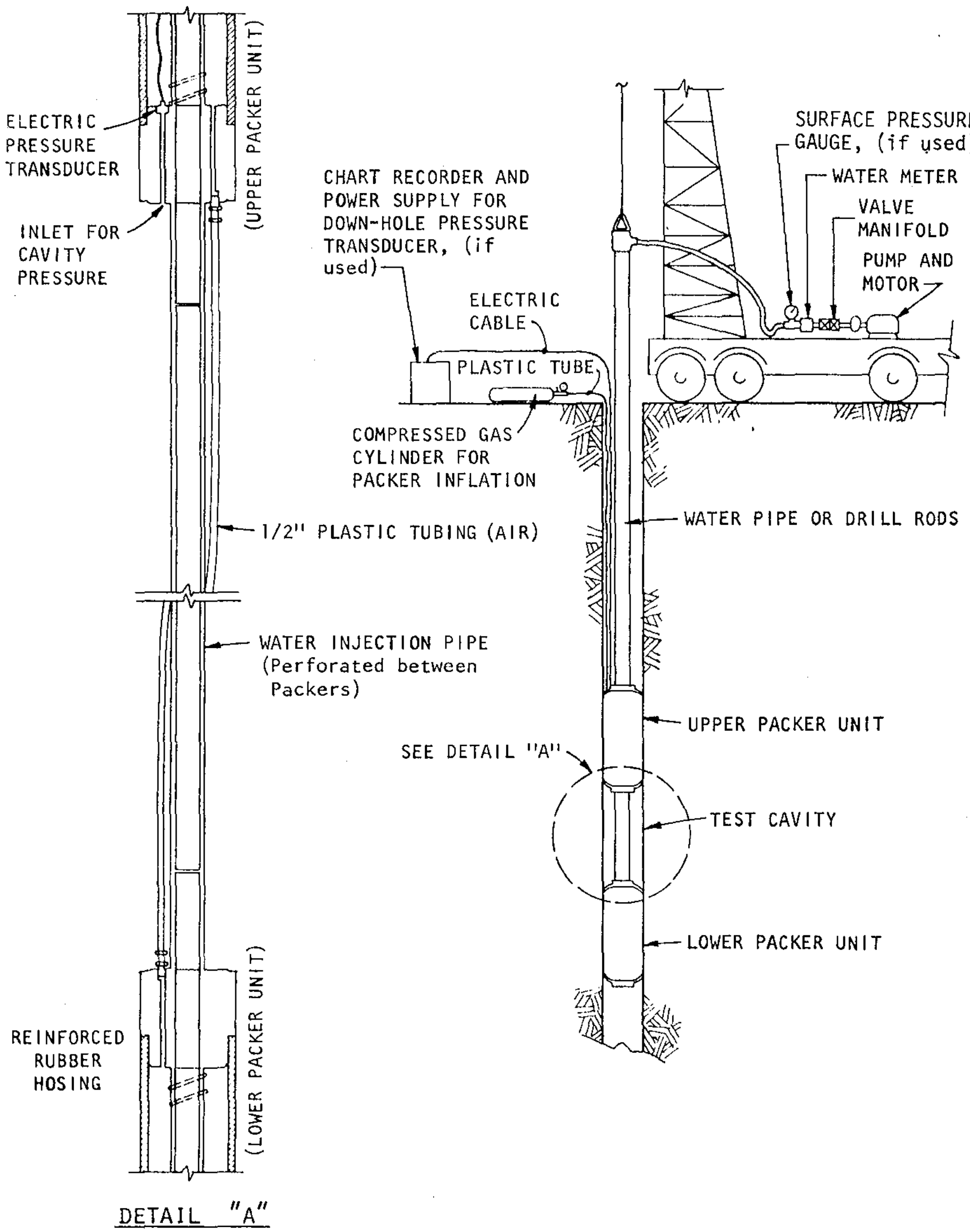


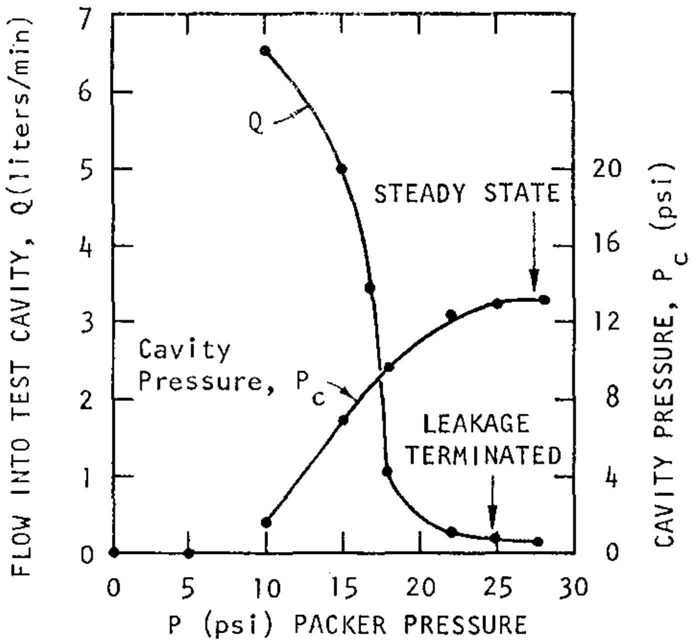
Figure 7-10 PUMP-IN TEST ARRANGEMENT USING PNEUMATIC PACKERS

3. Prior to expanding the packer(s), record the pre-test water pressure (if below the water table) in the test cavity. The difference between this pressure and the cavity pressure measured during testing is the excess pressure applied to the immediate ground mass. This is the pressure which is plotted vs. the observed flow rate into the test section.

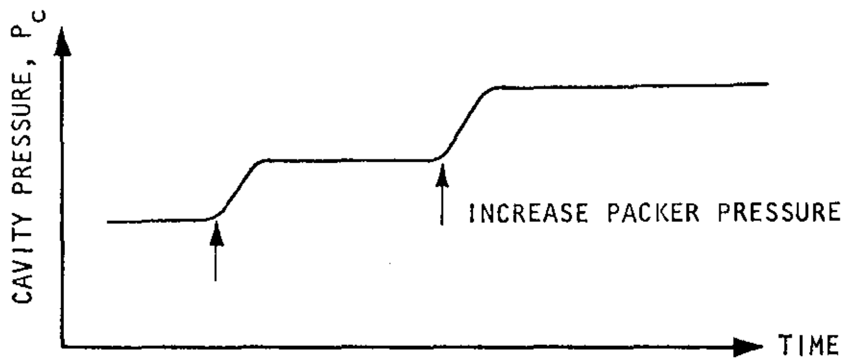
4. Investigate possible packer leakage. This can be done with a continuous reading instrument (chart recorder) monitoring the pressure in the test cavity. This procedure can be applied to either the stage test (single packer) method or the double packer method, as follows. Commence inflow into the system and then incrementally increase packer pressure. This will aid in the removal of any large air pockets within the test cavity. As the packer pressure increases, leakage past the packers will decrease, the pressure in the test cavity will increase, and the flow rate into the test cavity (measured by the water meter at the top of the borehole) will consequently decrease. After a certain point, further increase of packer pressure will only effect a temporary slight increase in cavity pressure at the instant the packer pressure is increased. This is a dynamic effect, and the cavity pressure will quickly return to a steady state value. Figure 7-11 contains three graphs which illustrate typical relationships between packer pressure, test cavity pressure, and test cavity inflow throughout the packer sealing procedure.

   Caution must be used to assure that the excess test cavity pressure does not exceed 0.5 psi/ft depth to the test section during this procedure. Otherwise, hydraulic fractures may occur, significantly altering subsequent test results as discussed earlier.

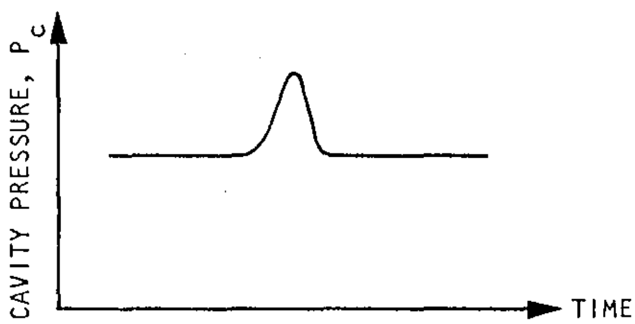
5. Pump water into the test section at a specified constant pressure for 15 to 20 minutes, taking readings of total water flow and pressure at 5-min intervals (USBR G-97). The test at this pressure is completed when the flow over two successive 5-min intervals differs by less than 10 percent. In very permeable materials, a test duration of 5 or 10 minutes may be sufficient. In this case, record flow and pressure over 1-min intervals until stable conditions are reached. Perform this procedure at varying applied water pressures. The maximum excess pressure applied should be 0.5 psi/ft depth to the test section. As an example, consider a test section depth of 100 feet, where the existing water table is 50 feet below ground level. The pre-test pressure in the test section is approximately 22 psi($(62.4 \text{ lb/ft}^3 \times 50 \text{ ft}) \div 144 \text{ in}^2/\text{ft}^2$). The maximum allowable excess pressure is 50 psi (0.5 psi/ft x 100 ft). Therefore, during the test, the maximum observable pressure is 72 psi (22 psi existing hydrostatic pressure plus the maximum allowable excess pressure of 50 psi). The value of 72 psi is called P_{max}, and a recommended pressure sequence is $1/2\ P_{max}$, $3/4\ P_{max}$, P_{max}, $3/4\ P_{max}$, and $1/2\ P_{max}$.



(a) TYPICAL DATA INDICATING SEALING OF PACKER



(b) CHART RECORD OF TEST CAVITY PRESSURE WHILE INCREASING PRESSURE IN A LEAKY PACKER (Increases in Packer Pressure divert additional leakage into test cavity, thereby raising cavity pressure, P_c)



(c) CHART RECORD OF TEST CAVITY PRESSURE WHILE INCREASING PRESSURE IN A NON-LEAKING PACKER (Transient Peak in Cavity Pressure, P_c, due to increase in packer pressure indicates effective packer seal)

Figure 7-11 MONITORING PACKER LEAKAGE IN PACKER TESTS

(Maini, 1971)

6. Record the data on a form similar to that in Figure 7-8, where the Test Section Pressure P_T is directly determined by the transducer. The data can be analyzed to compute permeability as discussed in Section 8.2.

7. Plot the data, pressure (vertical scale) vs. flow. Compare with Figure 7-9 for evidence of problems such as leakage around packer, erosion of test zone, and clogging of fissures. A high quality test should provide a linear plot similar to that corresponding to laminar flow in Figure 7-9.

8. For the stage test procedure, remove the packer and prepare the borehole for the next test. In a predrilled hole, where two packers are used to isolate the test section, the packers are simply raised to the next test depth.

7.6 WELL PUMPING TESTS

7.6.1 Equipment

The following equipment with associated approximate costs is needed to perform well pumping tests, in addition to equipment used to prepare the borehole.

Equipment	Approximate Cost, $
Pump - various types of pumps may be used for deep well operations (below 25 ft): plunger, displacement, airlift, submersible, and jet (Todd, 1959). Required pump capacity range is 10 gpm to 1000 gpm.	Submersible, 1,000 gpm, total head 450 feet: 7,700 Submersible, 500 gpm, total head 450 feet: 5,000
Flow meter	800
Optional borehole flow meter for borehole profiling	2,500
Water Disposal Area - water should not be allowed to infiltrate within the influence area of the well. Water may be discharged into a stilling tank and then conducted away from the test area in a polyethylene-lined ditch.	Indeterminate

Equipment	Approximate Cost, $
Water level indicator (electrical probe or linen tape with sinker)	up to 100
Readout equipment for closed system piezometers if used.	1,000

Required pump capacity may range from 10 gpm to 1000 gpm, depending on the permeability of the material, hole geometry, and extent of the aquifer. The USBR indicates that a pump capacity of 60 to 75 gpm is adequate for moderately permeable material whose saturated thickness is less than 150 feet.

Several criteria have been suggested for the selection of a pumping rate: the pumping rate should cause roughly 0.2D or 0.2H drawdown at the pumped well; the drawdown at a distance D/2 or H/2 should be no more than 0.1D or 0.1H, and the rate of discharge should allow laminar flow through the well screen (Walton, 1970; Tabor, and USBR G-97), where:

D = saturated thickness of the artesian aquifer

H = saturated thickness of the water table aquifer

To maintain laminar flow with a fully penetrating well, the discharge rate Q should be maintained below

$$Q = 7.48\ S_L A_o V_c \quad \text{(Walton, 1970)}$$

where Q = discharge rate (gpm)

S_L = optimum length of screen (feet)

A_o = effective open area (square feet per foot) of screen

V_c = optimum screen entrance velocity given below:

OPTIMUM SCREEN ENTRANCE VELOCITIES, V_c

Permeability gpd/ft^2	Permeability cm/sec	V_c ft/min
greater than 6000	greater than .30	12
6000	0.30	11
3000	0.15	8
2000	0.10	6
1000	5×10^{-2}	4
500	2.5×10^{-2}	3
less than 500	less than 2.5×10^{-2}	2

For compatible units, $Q = S_L A_o V_c$.

The criterion that the drawdown be less than 0.1D or 0.1H at a distance D/2 or H/2 will be met if the drawdown at the pumped well is 0.2D or 0.2H. To estimate the flow that will give 0.2D or 0.2H drawdown the following equations for steady-state flow can be used:

$$Q = \frac{K\,(2\pi D)\quad(S)}{\ln\,(r_w/R)} \qquad \text{(for artesian conditions)}$$

$$Q = \frac{K\,(\pi)\quad(H^2 - S^2)}{\ln\,(R/r_w)} \qquad \text{(for water table conditions)}$$

where R = the radius at which the pumped well causes no drawdown; R is commonly assumed to be 1000 ft to 3000 ft.

r_w = the test well radius

K = the permeability of the rock/soil mass. Estimate from chart given in Figure 3-1 and convert to compatible units using conversion factors in Appendix E.

S = the drawdown in the pumped well. Let S = 0.2D for artesian conditions and 0.2H for water table conditions.

Various flow meters can be used. Mansur and Dietrich (1965) use a 15-foot straight section of pipe fitted with an orifice and meter. The jet from the orifice is directed into a stilling tank.

It may also be desirable to use flow meters at depth in the pumped well to obtain a profile of flow rate over the depth of the test section. Indications of relative permeability variations can be obtained by observing the change in total flow along the borehole. Highly permeable regions will produce large increases in flow rate, while the flow rate increases will be relatively smaller in regions of low permeability. Figure 7-12 shows a schematic of an acceptable device, whereby the flow is measured by an electrical impulse counter connected to a down-hole propeller-type sensor unit. Tate et al. (1970) discuss a similar instrument. A polypropylene impeller 1 cm in diameter employs blades whose top surfaces are plated with a reflecting material. The beam from a light source directed on the impeller is reflected, triggering a photocell operating a counter at the surface. Mansur and Dietrich (1965) describe an 8-inch well velocity meter which was used to record flow rates at 5-ft intervals in a well pumping 1,200 gpm overall.

In order to facilitate a general understanding of the well-pumping test, typical terminology is presented in Figure 7-13.

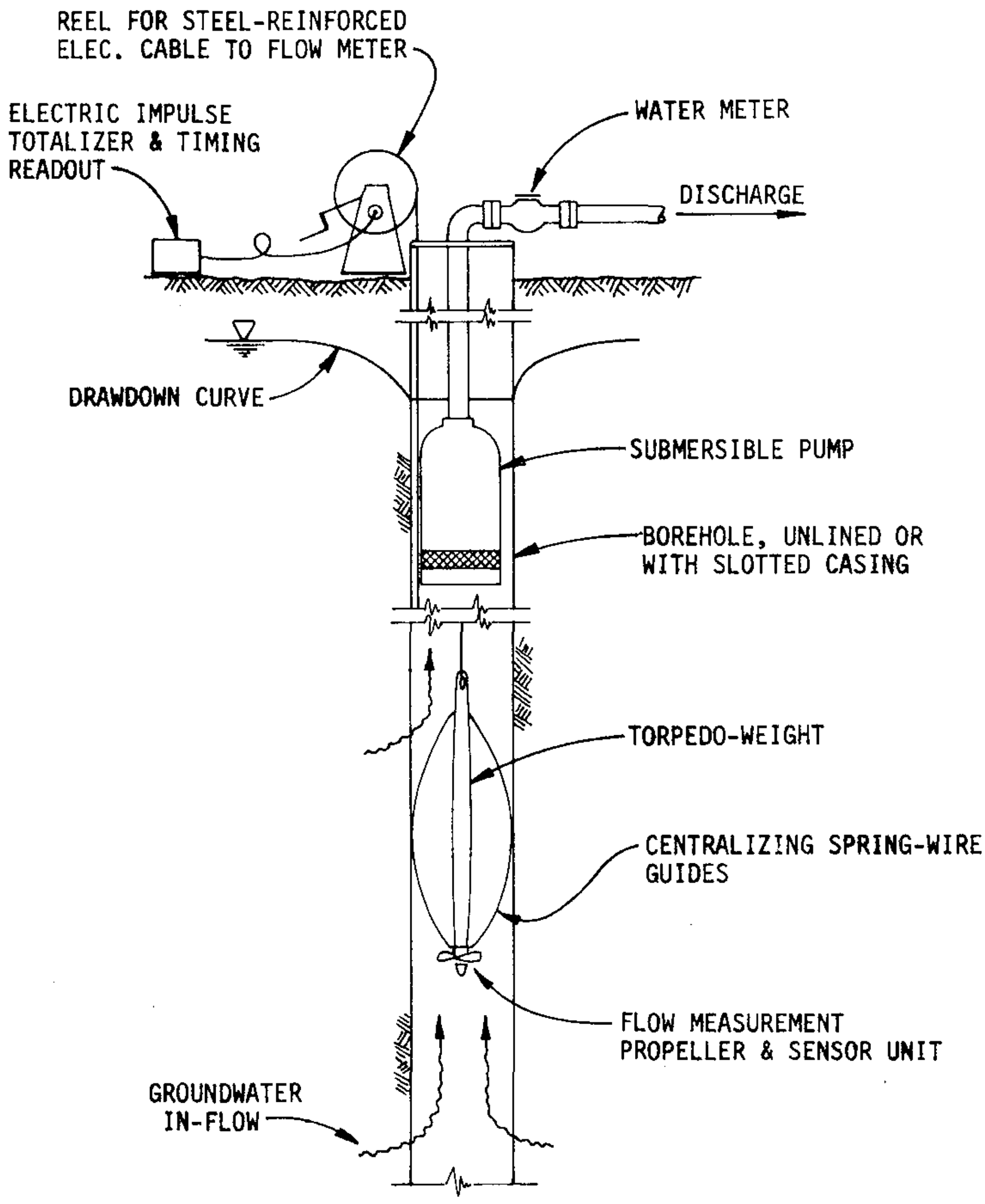


Figure 7-12 PERMEABILITY PROFILING WITH FLOW METER

7.6.2 Manpower Requirements

A minimum of one knowledgeable technician and one supervising engineer are required, in addition to a drill operator and his helper, to perform a well pumping test. This minimum manpower level can operate the pumped well and make observation readings at one observation well.

Total manpower requirements, however, are subject to great variability. In a pump test, the accuracy and reliability of the measured permeability increases with the number of observation wells employed. Well defined contours of equal drawdown can be used to assess important aquifer characteristics such as high flow gradients, lateral anisotropic permeability, etc.

Due to the short intervals at which readings are made at the beginning of a pump test (see recording schedule, Section 7.6.4), one person must be stationed at each observation well for the first hour of the test. After this point, the required recording frequency decreases and one (or more) person(s) can circulate and read each water level, thus reducing the manpower requirement. When the pump is shut off, the recovery observations follow the same time schedule as the pump segment of the test. Once again, for the first hour of recovery, a person is stationed at every observation well, after which time one (or more) person(s) can circulate, and manpower requirements decrease.

7.6.3 Hole Preparation

7.6.3.1 Pumped Well - Drill or auger a 6-inch or larger diameter hole. Record the saturated thickness of the aquifer and preserve core samples from the zone to be tested, if possible. The hole should not deviate more than 6 inches in 100 feet from vertical to facilitate pump installation (Todd, 1959). The casing diameter should be 2 nominal pipe sizes larger than the pump to be used (Walton, 1970). Walton (1970) suggests various casing diameters for a range of pumping rates. Methods to estimate the pumping rate are discussed in the next section. Suggested casing diameters are (see conversion factors in Appendix E):

Pumping rate gpm	Diameter of well, in.
less than 100	6
200	8
400	10
600	12
900	14
1,200	16
1,800	20
3,000	24
greater than 3,000	30

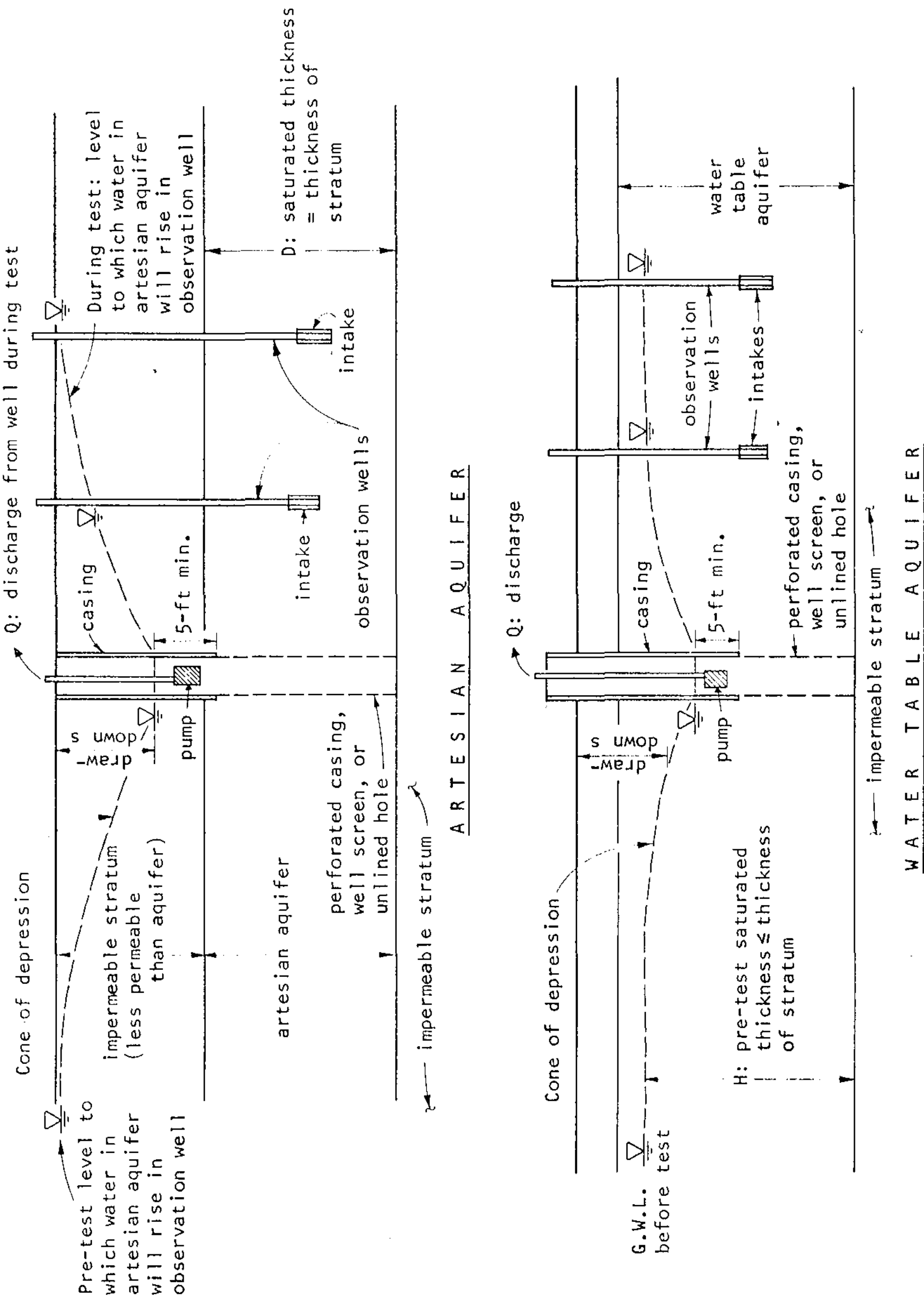


Figure 7-13 BASIC WELL PUMPING TERMINOLOGY

Prepare the borehole by any of the methods listed below:

Water Table Aquifer	Artesian Aquifer or Water Table Aquifer Underlying Other Aquifers
A: Unlined Borehole (consolidated deposits)	B: Unlined Borehole, Grout Seal Above Test Section (consolidated deposits)
D: Perforated Casing Over Test Section, Cased Above (unconsolidated deposits)	D: Perforated Casing Over Test Section, Cased Above (unconsolidated deposits)

Several types of recommended well configurations are given in "AWWA Standard for Deep Wells" (1966).

The screened or unlined test section should penetrate the entire water bearing stratum if possible. The water level in the well must be kept above the top of the screen. Accordingly, in a water table aquifer, the perforations should begin 5 feet below the maximum groundwater drawdown level (AWWA, 1966). A drawdown of about 0.2D or 0.2H is a reasonable target for a well pumping test (Tabor) (see Figure 7-13 for definition of D and H).

7.6.3.2 Observation Wells - Drill or auger a minimum of three observation wells, located on a straight line with the pumped well. The observation and pumped wells should be spaced at a distance D/2 (or H/2) from adjacent wells (USBR, G-97). The observation wells should penetrate a sufficient depth into the stratum so that they do not become dry during the test. A penetration of 0.2D (or 0.2H) should be sufficient.

If the pumped well is not fully penetrating, the flow paths to the well will not be horizontal, equipotential head lines will be curved, and hence the piezometric head at any radius from the well will not be constant with depth. In this case, at locations within a distance D (or H) from the pumped well, two observation wells are recommended with one intake located at depth 0.1D (or 0.1H) and one at depth D (or H) below the top of the saturated zone (USBR G-97). The average of the two observation well heads may then be used in permeability computations.

If the surface of the groundwater table is not horizontal before the test, at least three observation wells should be located up-gradient and three down-gradient from the pumped well (Wenzel, 1942).

Observation wells should have as small a diameter as possible to reduce the response (lag) time. In deep test zones, where the depths make electrical probe measuring impractical, closed system piezometers should be used as observation wells. All observation wells should be cleaned using the procedures recommended for test wells, Section 6.5.

The following borehole preparation procedures, listed in order of preference, may be used to prepare observation wells.

Water Table Aquifer	Artesian Aquifer or Water Table Aquifer Underlying Other Aquifers
F: Piezometer	F: Piezometer
A: Unlined Borehole (consolidated deposits)	B: Unlined Test Section, Cased Above (consolidated deposits)
C: Gravel-Filled Test Section, Cased Above (unconsolidated deposits)	C: Gravel-Filled Test Section, Cased Above (unconsolidated deposits)
D: Perforated Casing Over Test Section, Cased Above (unconsolidated deposits)	A-1: Unlined Borehole, Grout Seal Above Test Section (consolidated deposits)
	D: Perforated Casing Over Test Section, Cased Above (unconsolidated deposits)

7.6.4 Test Procedures

Well pumping tests can be performed as constant discharge or constant drawdown tests. Constant discharge tests are preferable to constant drawdown tests because more analytical solutions are available. Both procedures are presented.

7.6.4.1 Constant Discharge Pump Test

i) Prepare the borehole and observation wells as discussed above. The placement of the observation wells is based on the following considerations:

   a. type of aquifer - The propogation of head loss is extensive in confined aquifers. The loss of head can be measured at large distances from the pumped well, as much as 1000 feet. In water table aquifers, propogation of head loss is relatively slow, and maximum distances between the pumped well and observation wells will be approximately 300 feet (Kruseman and DeRidder, 1970).

   b. permeability - A highly permeable aquifer material will produce a wide, flat cone of depression, while that of a low permeability material will be steep and narrow. Observation wells should be spaced farther away in the former than in the latter case.

c. discharge rate - the size of the cone of depression will increase with the pumping rate.

d. well-screen length - A fully penetrating well will develop horizontal flow, and observation wells may be located close by. In a partially penetrating well, a non-uniform distribution of drawdown will result near the well due to vertical flow. Drawdown readings taken close to the well may lead to incorrect results. Generally, the nearest observation well should be placed at a distance equal to the aquifer thickness. This will ensure horizontal flow in most cases (Kruseman and DeRidder, 1970).

e. stratification - Stratification can result in large vertical variations in permeability. The greater the distance from the pumped well, the less is the effect of stratification upon the drawdown distribution.

No fixed rule is available for placement of observation wells. However, placing them about 30 to 300 feet from the pumped well will give good results in most cases. These distances should be adjusted to 300 to 800 feet for thick, or stratified confined aquivers (Kruseman and DeRidder, 1970). The USBR (G-97) suggests that the nearest observation well be located at least 0.5H from the pumped well, where H is the thickness of the penetrated aquifer. Other holes are located at whole number multiples from the pumped well.

ii) At least 24 hours before the test, install the pump to be used for the test. Pump for about an hour at the estimated discharge rate to see if the drawdown is adequate.

In very permeable zones, it may be a problem to find a sufficiently high rate of discharge to achieve at least several feet of drawdown in the pumped well. In a very permeable material ($K = 10^{-1}$ cm/sec), rapid drawdown will occur at the well during the first hour of pumping. Depending on the nature of recharge, an hour's worth of pumping may cause 50 percent to 100 percent of the ultimate drawdown right at the well.

In a low-permeability material, ($K \approx 5x10^{-7}$ cm/sec) pumping for an hour will produce only a small fraction of the ultimate steady-state drawdown at the well. The problem here may be to find a sufficiently low discharge so that steady-state drawdown is just achieved or is still being approached by the end of the test, or so that the well is not pumped dry before the end of the test. If a pumping rate of less than 5 gallons per minute is required, it may be more convenient to carry out a constant drawdown test, to a drawdown of about 0.2D or 0.2H at the pumped well. For the latter technique, the pump-out flow rate is controlled to produce the desired result.

iii) Wait at least 24 hours after the test described above. If the well has been pumped previously, a longer wait may be required until the static ground water levels are stabilized.

iv) Start the pump. Pump at a constant rate, recording the drawdown in the pumped well (if possible) and all observation wells at the following intervals:

Pumped Well Readings:

Time Since Pumping Started	Time Interval
0-5 minutes	30 seconds
5-60 minutes	5 minutes
1-2 hours	20 minutes
2-24 hours	60 minutes
24 hours - pump shut down	2-4 hours

Observation Well Readings:

Time Since Pumping Started	Time Interval
0-5 minutes	30 seconds
5-20 minutes	1 minute
20-60 minutes	5 minutes
1-3 hours	20 minutes
3-7 hours	30 minutes
7-24 hours	60 minutes
24 hours - pump shut down	2-4 hours

v) When the pumping period (24 hours to 1 week) is complete, shut off the pump. Continue recording water level data until the groundwater levels in the observation wells stabilize or until the plot of drawdown versus log time is a straight line for each observation well. Record the recovery of the water level in the pumped well and all observation wells according to the same schedule established above for the pumping period.

Recovery data is important. Todd (1959) notes that it "provides an easy check on pumping test results; also, it implies a constant discharge Q, which is often difficult to control accurately in the field."

vi) Record all data on the data sheet given in Figure 7-14.

Location ______________________ Name ______________________

Well No. ______________________ G.W. Elev. (Obs. Well) before Test ________

No. of Pumped Well ______________________ G.W. Elev. (Pump. Well) before Test ________

Date of Test ______________________ Radius of Well from Pumped Well ________

Ground Elevation ______________________ G.W. Temperature ________ °F

DIMENSIONS:

PUMPED WELL

Depth of Base of Well ________
Depth of Top of Screen/Unlined Hole ________
Length of Screen/Unlined Hole ________
Depth of Base of Aquifer ________
Depth of Top of Aquifer ________
Thickness of Aquifer ________
I.D. of Screen/Unlined Hole ________
O.D. of Gravel Pack ________
Type of Well Screen/Perf. Casing:* ________

*For example: ¼-in. horizontal louvered slots 2-in. long

OBSERVATION WELL

Depth of Base of Well ________
Depth of Top of Screen/Unlined Hole ________
Length of Screen/Unlined Hole ________
Depth of Base of Aquifer ________
Depth of Top of Aquifer ________
Thickness of Aquifer ________
I.D. of Perf. Casing/Unlined Hole ________
I.D. of Standpipe ________
Type of Observation Well: ☐ Open Hole, ☐ Cased Hole, ☐ Hole Cased to El., ☐ Standpipe Piezometer (open), ☐ Standpipe Piezometer with grout seal at top of filter length.

Time of Day: ________

Elapsed Time	Depth to Water from Top of Casing	Drawdown	Rate of Discharge at Pumped Well

Figure 7-14 WELL PUMP TEST DATA SHEET

7.6.4.2 Constant Drawdown Pump Test

i) Carry out Steps i)- iii) of Section 7.6.4.1,

ii) Start the pump. Vary the flow so that a constant drawdown is maintained. The drawdown should vary at most by 10 percent or ± 6 inches, whichever is less. If a direct reading flow meter is not available, determine the flow rate by measuring the volume of flow over a 1 minute period. Record the flow at the intervals listed in Step iv) of Section 7.6.4.1.

iii) Carry out Steps v) and vi) of Section 7.6.4.1.

In a very low permeability material, the flow may be so low that the pump cannot operate continuously. This is acceptable as long as the drawdown is maintained within the ranges stated in Step ii) above.

7.7 OTHER METHODS

In addition to the principal methods discussed in the earlier subsections of Section 6, other methods have existing or potential usefulness in estimating in-situ permeability. This section discusses several of these methods.

7.7.1 Permeability Tests Using Air

Air has been used for permeability testing of jointed rock above the water table. The evaluation of air permeability tests relies on the same assumptions of fluid flow as those for water permeability tests. One problem in this application is that air is a compressible fluid. Also, high temperatures may develop during air pumping tests (Sherman and Banks, 1970). Advantages for air as a field permeability test fluid lie in the relatively compact, portable and economic storage and delivery systems for even very large fluid delivery requirements, at any test site.

Two methods can be used for air permeability measurements in a borehole:

a. Unsteady flow and spherical flow pattern - air is pumped into a packed-off section, and the pressure is raised rapidly. The flow is then cut off, and the decrease of the cavity pressure is determined as a function of time. The solution for this case considers the decrease in air pressure with time of a spherical cavity in a semi-infinite porous medium.

b. Constant flow and radial flow pattern - this method is essentially a constant flow test using air rather than water, where the test cavity pressure is monitored. Radial flow from the cavity is assumed in the analysis (Einstein, 1967 in Sherman and Banks, 1970). Pressure tests in the vicinity of the water table using air as a permeating fluid have been attempted, although severe limitations exist. Air will not penetrate a saturated medium until the air pressure overcomes the capillary forces retaining water in the mass. Therefore, the observed air "permeability" is variable, depending upon the applied pressure. As the air pressure is increased, a larger portion of void space will drain and the air permeability will progressively increase.

   In layered strata, the threshold pressures required for air to penetrate the medium will usually be lower parallel to the laminations than perpendicular to them (Blight, 1971). Unless the air pressure exceeds the threshold value for flow across such laminations, the mass will be permeable to air flow in one direction only. Blight (1971) has also found that in the field, it is extremely difficult if not impossible to achieve steady-state conditions in a saturated medium. The overall situation complicates considerably when partly saturated media are considered.

   Although air is used widely as a fluid for laboratory testing of rock core samples in the petroleum engineering field, Baptist (1967) points out several drawbacks of its use, which can lead to erroneous results.

7.7.2 Tracer Methods

Below the water table, in-situ values of permeability can also be obtained using tracer tests. Two test methods are tracer travel time and tracer dilution. The tracer travel time method may not produce consistent results due to several geologic and hydrologic factors (Lewis et al., 1966). The tracer dilution method has been more successful and warrants discussion. This method involves injection of radioactive isotopes into a well, and monitoring of the isotope activity as a function of time with a "scintillation" probe in either the same or a second well. The apparent groundwater velocity and permeability of the medium can be calculated from measurements of tracer concentrations at various times after injection.

A modification of the tracer dilution method employs fluorescent dye tracers and physical sampling and analyses to determine dilution (Lewis et al., 1966). Lewis et al. (1966) summarizes several other modifications and applications of the tracer method, as well as several means of analysis.

7.7.3 Temperature Profiling

An indirect method of assessing the in-situ permeability below the watertable is by measuring the distribution of temperatures underground, which is an indirect manifestation of the groundwater velocity. Thermometric measurements are made with a thermistor probe and a wheatstone bridge. An adaptor which balances the bridge and a recorder can provide continuous temperature readings on a strip chart at various depth scales. The equipment can be designed to read either the temperature of a single thermistor, or the differential temperature between two matched thermistors (Tate et al., 1970). With the addition of head measurements over a given area, the permeability can be calculated using differential equations which relate this parameter to the temperature and hydraulic gradients in a medium (Stallman, 1960). The technique is useful in detecting regions of fissure flow and water yielding horizons (Robinson, 1974). An investigation of seepage through the abutments of a dam in central Pennsylvania was successfully carried out by profiling temperature gradients of the water in existing standpipe piezometers located within the embankment and abutments of the dam. Isotherm contours plotted with the data clearly showed the localized seepage path (Trautwein, 1969). Temperature profiling in the observation wells of a pump-out test may yield sufficient data to plot anisotropy of the aquifier as demonstrated by the preferred seepage path when not pumping.

7.7.4 Geophysical Logging

Borehole geophysical methods such as self-potential method and resistivity logging, may also be useful in profiling permeable strata in sedimentary sequences below the water table. Briefly, the self-potential, or spontaneous potential, method involves the placement of a single electrode in a fluid-filled borehole. A potentiometer measures the natural electric potential between the submerged electrode and another electrode fixed to the ground surface. Due to the ionic nature of clay and strata with clay content, voltages recorded adjacent to these strata differ significantly from those recorded adjacent to strata with low clay content. The self-potential method is thus able to differentiate between impermeable clay or shale horizons and other sedimentary strata which may be aquifers.

Resistivity measurements involve the use of four down-hole electrodes, where an electric current is passed between two outer electrodes in the conductive fluid of the borehole and the potential difference between the two inner potential electrodes is measured. The potential is related through Ohm's Law to the resistivity of the formations near the measuring electrodes. Electrode spacings and interpretation curves vary with the application (Hamilton and Myung). The resistivity is a function of the amount of water in the rock/soil pore spaces. Observed resistivity jumps may indicate lithologic variations in a vertical sedimentary sequence, and thus aid in planning and analyzing further permeability field test programs and results.

8.0 TEST METHOD ANALYSES

8.1 VARIABLE HEAD TESTS

8.1.1 Background

The classic reference for variable head tests in saturated media was written by Hvorslev in 1951. Schmid and Kirkham have provided alternate solutions (Schmid, 1967). Schmid (1967) has demonstrated that the value of permeability calculated by the three methods (Schmid, Kirkham and Hvorslev) predict consistent permeability values within a range of a factor of 2. Since Schmid's and Kirkham's solutions are similar, only the former will be presented.

Schmid has also provided a solution for falling head tests above the water table (unsaturated media). The solution requires assumptions concerning the degree of saturation and porosity of the soil/rock mass. Consequently, falling head tests can only give a rough estimate of permeability above the water table.

In their analysis of variable head tests in saturated media both Schmid and Hvorslev assume:

1. The soil is saturated; no gas in the system.
2. The undisturbed groundwater level is constant or predictable with time. Only constant groundwater conditions are considered.
3. The soil is incompressible.
4. The shape factor F is constant throughout the test. F is a function of the intake geometry and certain permeability ratios.

8.1.2 Analysis Methods

8.1.2.1 Below the Groundwater Table

i) Find the basic time lag T as follows (see Appendix A for a discussion of the meaning of the term "basic time lag"):

   a) Calculate H/H_0 where H is the excess/deficit head at time t and H_0 is the initial excess/deficit head (see Figure 8-1). Then, plot log H/H_0 vs. t and find the best fit straight line. The time when $H/H_0 = 0.37$ is called the basic time lag T. Refer to Figure 8-1 for a sample calculation of T.

   b) Alternatively, plot H vs. t. Mark off equal intervals of time Δt (it is not necessary to start at t = 0). Find ΔH_1 for the first time interval, ΔH_2 for the second, etc. Calculate M = average ($\Delta H_1/\Delta H_2$, $\Delta H_2/\Delta H_3$,. . .).

Then $T = t/\ln M$

Figure 8-1 shows an example of this method of calculating T.

ii) Calculate the permeability k.

k (or k_h if the soil is anisotropic) $= \frac{A}{FT}$

where A = cross-sectional area of standpipe
F = shape factor shown in Figures 8-2 and 8-3
k = isotropic permeability
k_h = horizontal permeability (in anisotropic medium)
T = basic time lag

Formulas have been derived for computing k directly without computing T, and are available from Hvorslev (1951) and Schmid (1967). In general, the method outlined above is preferred.

Where the soil/rock mass is anisotropic, the engineer must estimate the ratio of horizontal to vertical permeability $k_h/k_v = m^2$ or determine it from laboratory tests. Then, the value of m is introduced into the computation of the shape factor F, as shown in Figure 8-2. The error in the permeability calculations due to an error in selection of m is generally less than the inherent error in variable head tests.

Where a gravel filter is placed in the casing, the engineer must also estimate $n = k'_v/k_v$ where k'_v is the vertical permeability of the filter material and k_v is the vertical permeability of the soil/rock mass. Then "n" is introduced into the computation of the shape factor F, as shown in Figure 8-2.

8.1.2.2 <u>Above the Groundwater Table</u> - Schmid presents a solution only for a fully cased well open at the bottom.

i) Assume the degree of saturation in the zone wetted by the test is $S = 0.85$. It may be desirable to test the sensitivity of the calculated permeability to a range of S from 0.75 to 0.95.

ii) Estimate the porosity $n = V_v/V$ where V_v = volume of voids and V = total volume of a rock/soil sample.

iii) At any time t_i, the height of water in the borehole is H_i measured from the bottom of the well. Select any two data points H_1, t_1 and H_2, t_2.

iv) Calculate $$k = \frac{R}{4} \; \frac{\ln (H_1/H_2)}{t_2\left[\left(\frac{3\,(H_1 - H_2)}{4S\,n\,R}\right) + 1\right]^{1/3} - t_1}$$

Where R = the interior radius of the cased borehole
n = the porosity
H_i = height of water in the borehole at time, t_i
S = final degree of saturation in zone wetted by test

DEFINITION OF SYMBOLS

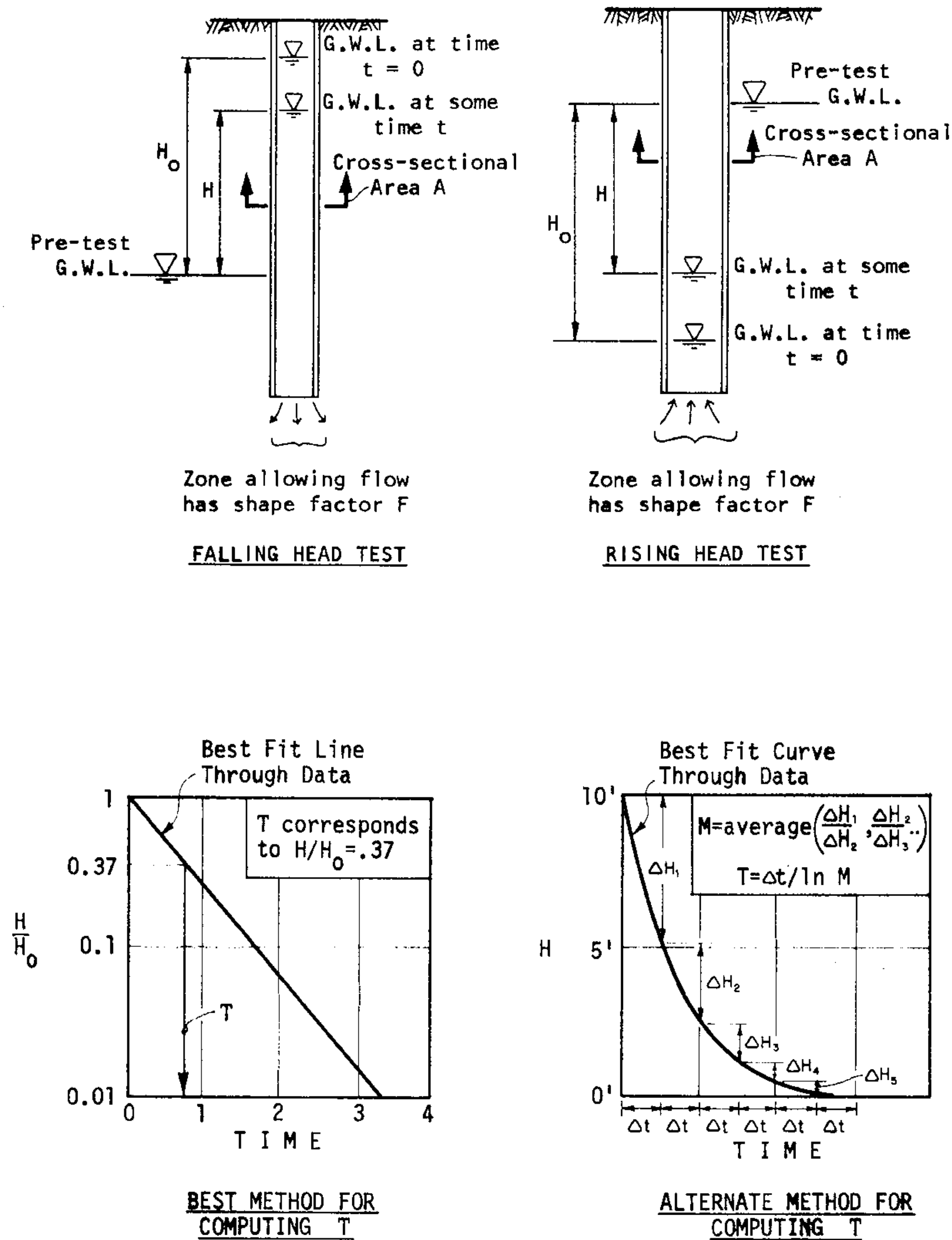


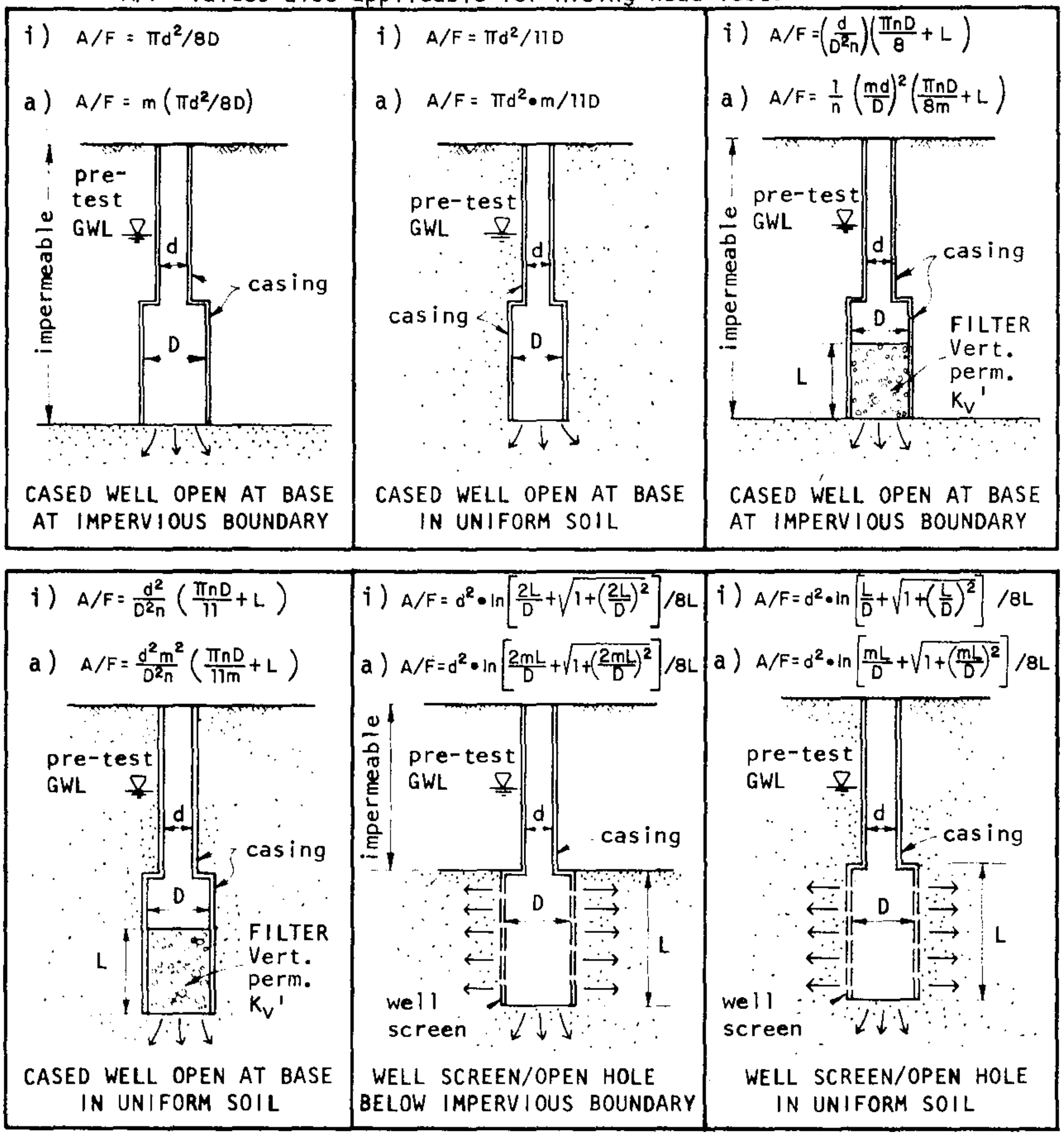
Figure 8-1 CALCULATION OF BASIC TIME LAG, T, FOR VARIABLE HEAD TESTS
(Hvorslev, 1951)

i = isotropic conditions: $K_h = K_v = K$

a = anisotropic conditions: $K_h \neq K_v$

$$K \text{ or } K_h = \frac{A}{(F \bullet T)}$$

NOTE: Flow direction shown for Falling Head Tests for clarity; "A/F" values also applicable for Rising Head Tests



DEFINITIONS: $K_m = \sqrt{K_v K_h}$; $m = \sqrt{K_h/K_v}$; $n = K_v'/K_v$

where K_v = vertical permeability of soil/rock mass

K_h = horizontal permeability of soil/rock mass

K_v' = vertical permeability of filter in casing

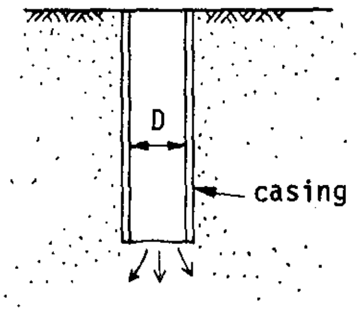
T is termed the basic time lag. See text for best method to determine representative value of T

Figure 8-2 SHAPE FACTORS FOR VARIABLE HEAD TESTS BELOW THE WATER TABLE (Hvorslev, 1951)

Assumed Isotropic Conditions: $k_h = k_v = k$

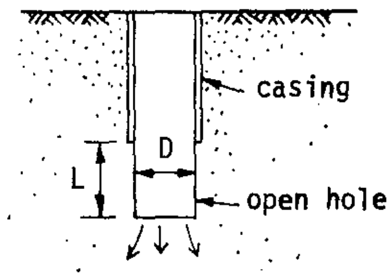
$$K = \frac{A}{(F \cdot T)}$$

$$\frac{A}{F} = \frac{D}{8}$$



Schmid : Falling Head

$$\frac{A}{F} = \frac{D^2}{8(D+L)}$$



Schmid : Falling Head

Figure 8-3 SHAPE FACTORS FOR VARIABLE HEAD TESTS BELOW THE WATER TABLE (after Schmid, 1967)

8.2 CONSTANT HEAD, CONSTANT FLOW AND PACKER TESTS

8.2.1 Background

In this section the borehole, piezometer or pipe into which water is pumped will be called an injection well.

This analysis applies to any pump-in test where a steady state condition is achieved; i.e., both Q, the in-flow, and H, the height of water in the injection well, become constant over time. This analysis, therefore, can be used for constant head tests, packer tests and constant-flow tests.

Several analysis methods are available in the literature; research by Schmid (1967) has shown that there is good agreement among the range of formulas. For example, the variation in value of K computed may vary by a factor of 2 between the methods of analysis developed by Schmid and Hvorslev. Such a variation is within the standard deviation of field test results, even from similar, carefully performed field tests using a single analysis method.

Selected analysis methods are presented in this section. The methods cover both saturated and unsaturated media and a wide variety of well geometries.

In saturated media, Hvorslev (1951) developed the basic analysis for constant head tests. His assumptions are the same as those given for variable head test analyses in Section 8.1.1. The USBR (Earth Manual) analysis method, which is applicable for packer tests, is considered more accurate for tests below than above the groundwater level. Where one or more observation wells are available, Schmid (1967) has developed a solution which gives the permeability over a larger area and is less influenced by local variations in the soil/rock medium. If observation well data is available, it should be used in the permeability calculation. Hvorslev's, the USBR (Earth Manual) and Schmid's analysis methods are presented.

Other analyses available in the literature include those by Cornwell (in Zangar, 1953), which is similar to Hvorslev's, by Gibson, whose solution is modified to account for the compressibility of soils, and by Hvorslev (1951) who provides solutions for linear or sinusoidal fluctuation in base groundwater levels.

Two methods are presented for unsaturated media; for a zone above the water table saturated by capillary action and for an unsaturated material overlying on impermeable bed (USBR,G-97).

8.2.2. Analysis Methods

i) Determine Steady State Conditions

Let H = height of water in well above base of test zone

Q = flow rate of water

t = time

For constant flow tests, plot H versus log t (Note: If transducer is used to measure head in psi, use conversion 1 psi = 2.31 ft of water). Find H as $t \rightarrow \infty$. This is the steady state head in the well.

For constant head tests, plot Q versus log 1/t. Find Q as $1/t \rightarrow 0$. This is the steady state flow. For packer tests, prepare one plot for each applied pressure.

ii) Determine Effective Head at Test Zone (Packer Test Only)

If pressure is measured with a pressure gage above the ground surface, the following procedure is necessary. Estimate the head loss, H_L, in the drill rods and hose between the pressure gage and the test section. This data must be provided by calibrating the equipment before the test, as described in Section 7.4.4. The head loss $H_L = P_L/\gamma_w$, where P_L is the measured pressure loss in the system and γ_w is the unit weight of water. If calibration curves have not been developed for the specific test equipment, the head loss H_L can be estimated by the use of Figure 7-5.

H_p is the pressure head added by the pump and measured at the pressure gage.

H_{elev} is the height of the column of water from the bottom of the test section to the pressure gage.

Then $H = H_p + H_{elev} - H_L$ = the effective head at the test zone.

Note: If a pore pressure transducer is used, the effective head at the test zone is obtained directly by measuring the pressure in the test cavity and converting to the equivalent head of water.

iii) Determine the "Zone" of the Test

The zones defined below are developed in USBR publication G-97.

Zone 1: Above the water table and unsaturated.

Zone 2: Above the water table, but saturated by capillary action or close enough to the water table to create a continuous saturated zone between the test section and the water table during the test.

Zone 3: Below the water table.

Let H = the constant height of water in the well or the effective head, as defined for packer tests.

D_W = the depth from the base of the test section to the groundwater level or to an impermeable stratum. For purposes of creating a partially saturated zone in the vicinity of the test section, an impermeable stratum is equivalent to the water table (USBR, Earth Manual) (Zones 1 and 2).

L = length of the test section.

H_W = the pretest groundwater level in the well.

To determine whether the test is in Zone 1 or Zone 2, calculate:

$$Y = \frac{H + D_W}{L}$$

$$X = \frac{H}{H + D_W}$$

Then enter Figure 8-4 with X and Y to determine the Zone.

iv) Determine the constant head H_C for calculations.

Zone 1: $H_C = H$

Zone 2: $H_C = \frac{(H + D_W) + (H - L)}{2}$

Zone 3: $H_C = H - H_W$

v) To compute the shape factor F for a given test well geometry and test zone in constant head tests using data only from the test well, refer to Figures 8-5 through 8-9 for the USBR G-97 method, to Figure 8-10 for the USBR Earth Manual method (packer test only), or to Figure 8-11 for Hvorslev's method. Figures 8-5 through 8-7 require supplemental reference to Figures 8-8 or 8-9, which contain graphs for the determination of conductivity coefficients under saturated and unsaturated conditions, respectively. For the single curve contained in Figure 8-8, an equivalent equation is presented for the investigator's convenience. However, Figure 8-9 contains a family of curves, thereby precluding a simplified analytic representation. Note that the USBR G-97 method for unsaturated conditions is limited in depth of application as a result of the limited range of conductivity coefficients presented in Figure 8-9. With regard to the USBR Earth Manual method, the formula is considered to be more accurate for tests below the groundwater table than above it. When using Hvorslev's method for anisotropic soils and rock, the engineer must estimate $m = \sqrt{K_h/K_v}$. If the well is packed with gravel, the engineer must also estimate $n = K'_v/K_v$, where:

K_v = the estimated vertical permeability of the rock or soil mass.

K_h = the estimated horizontal permeability of the rock or soil mass.

K'_v = the vertical permeability of the gravel filter in the well (probably at least 10^{-2} cm/sec).

The values of m and n enter into the computation of the shape factor F in Figure 8-11.

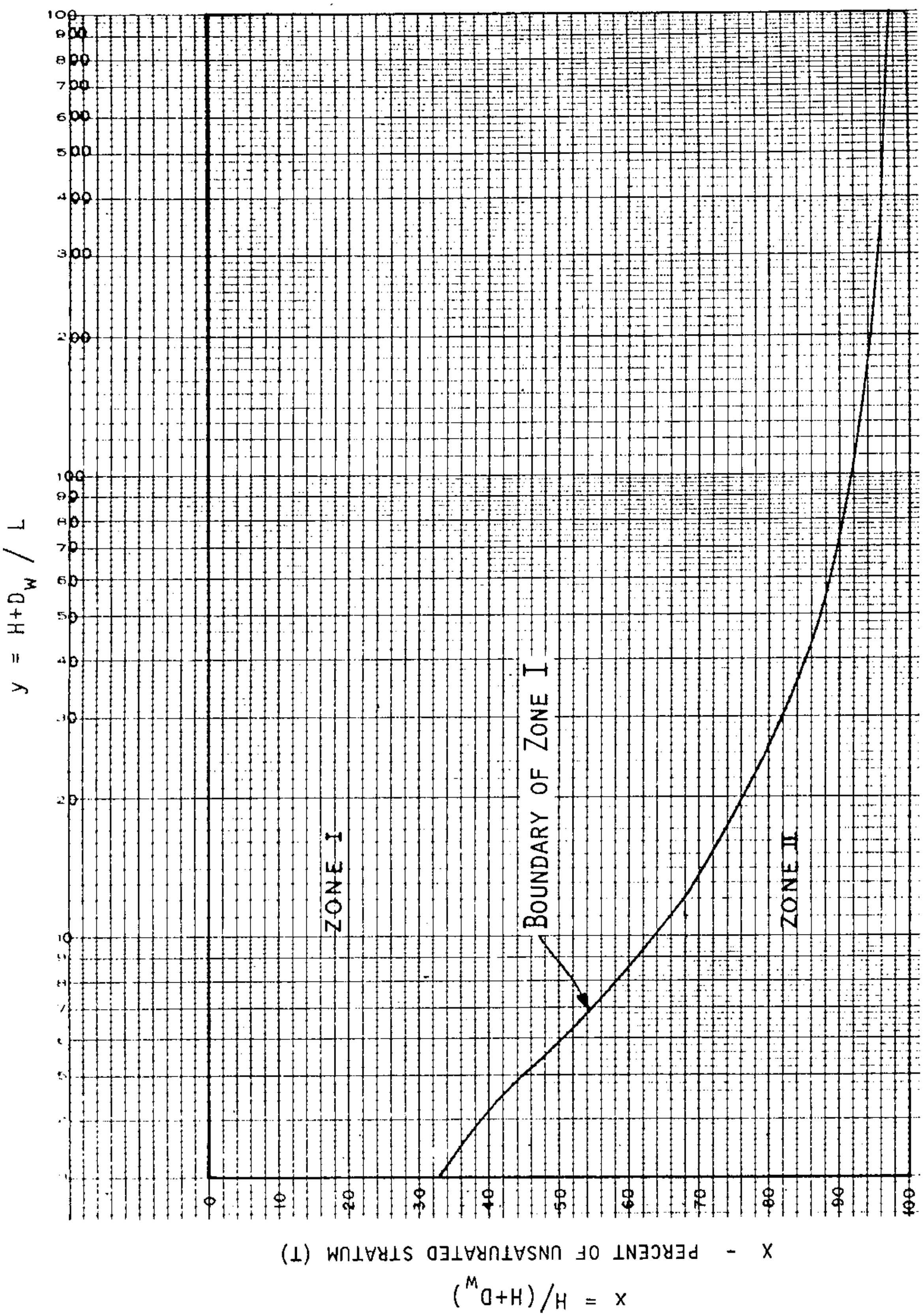


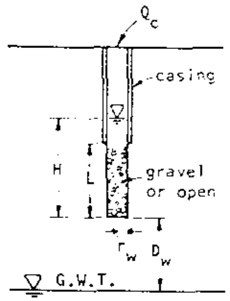
Figure 8-4 LOCATION OF ZONE I / ZONE II BOUNDARY
(USBR, G-97)

Partly Cased Well With Uncased Test Section

Analysis is valid for : Zone 2

$L > 10r_w$

$\frac{Q}{2\pi r_w L + \pi r_w^2} \leq 0.1$ fps



$K = \frac{Q_c}{F \cdot H_c}$

$H_c = \frac{(H + D_w) + (H - L)}{2}$

$F = (C_s + 4) \cdot r_w$

Values of C_s are given in Figure 8-8 , or can be approximated from:

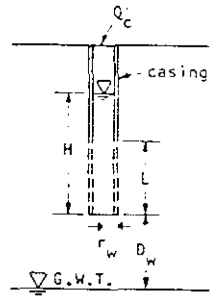
$C_s = 5.30\left(\frac{L}{r_w}\right)^{0.73}$

Partly Cased Well With Slotted Test Section

Analysis is valid for : Zone 2

$L > 10r_e$

$\frac{Q}{2\pi r_e L + \pi r_w^2} \leq 0.1$ fps



$K = \frac{Q_c}{F \cdot H_c}$

$H_c = \frac{(H + D_w) + (H - L)}{2}$

$F = C_s r_e + 4r_w$

r_e = (% open slotted area) $\cdot r_w$

Values of C_s are given in Figure 8-8 , or can be approximated from:

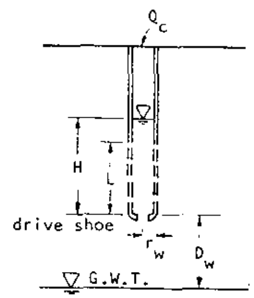
$C_s = 5.30\left(\frac{L}{r_e}\right)^{0.73}$

Partly Cased Well With Slotted Test Section and Drive Shoe

Analysis is valid for : Zone 2

$L \geq 10r_e$

$\frac{Q}{2\pi r_e L + \pi r_w^2} \leq 0.1$ fps



$K = \frac{Q_c}{F \cdot H_c}$

$H_c = \frac{(H + D_w) + (H - L)}{2}$

$F = C_s r_e + 4r_w$

where r_e = (% open slotted area) $\cdot r_w$

Values of C_s are given in Figure 8-8 , or can be approximated from:

$C_s = 5.30\left(\frac{L}{r_e}\right)^{0.73}$

(Omit the term $4r_w$ if a clay seal placed at bottom of casing)

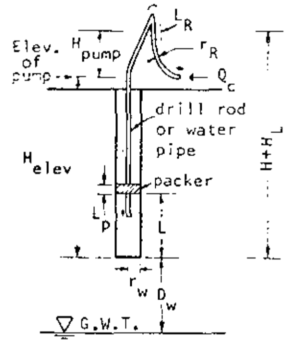
Unlined Test Section With Single Packer

Analysis is valid for : Zone 2

$L > 10r_w$

$L_p > 10r_w$

$\frac{Q}{2\pi r_w L + \pi r_w^2} \leq 0.1$ fps



$K = \frac{Q_c}{F \cdot H_c}$

$H_c = \frac{(H + D_w) + (H - L)}{2}$

$H = H_{pump} + H_{elev} - H_L$

where H_L = friction loss in drill rods = fct (L_R, r_R)

H_L can be computed from calibration curves.

$F = (C_s + 4) \cdot r_w$

Values of C_s are given in Figure 8-8, or can be approximated from:

$C_s = 5.30\left(\frac{L}{r_w}\right)^{0.73}$

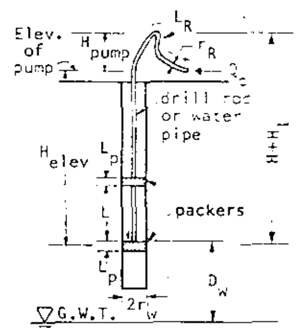
Unlined Test Section With Two Packers

Analysis is valid for : Zone 2

$L > 10r_w$

$L_p > 10r_w$

$\frac{Q}{2\pi r_w L} \leq 0.1$ fps



$K = \frac{Q_c}{F \cdot H_c}$

$H_c = \frac{(H + D_w) + (H - L)}{2}$

$H = H_{pump} + H_{elev} - H_L$

where H_L = friction loss in drill rods = fct (L_R, r_R)

H_L can be computed from calibration curves.

$F = C_s \cdot r_w$

Values of C_s are given in Figure 8-8, or can be approximated from:

$C_s = 5.30\left(\frac{L}{r_w}\right)^{0.73}$

Figure 8-6 USBR ANALYSIS METHOD FOR CONSTANT HEAD TEST - ZONE 2 (ABOVE WATER TABLE, SATURATED BY CAPILLARY ACTION) (after USBR, G-97)

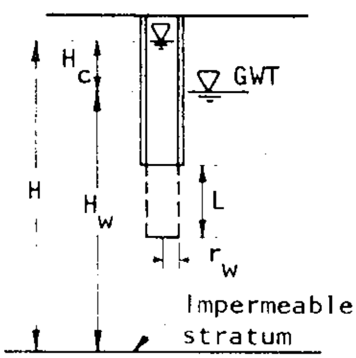
Partly Cased Well With Slotted Test Section

Analysis is valid for : Zone 3

$L > 10r_e$

$$\frac{Q}{2\pi L r_e + \pi r_w^2} \leq 0.1 \text{ fps}$$

$H_w > 5L$



$$K = \frac{Q_c}{F \cdot H_c}$$

$H_c = H - H_w$

$F = C_s \cdot r_e + 4r_w$

where

r_e = (% open slotted area)$\cdot r_w$

Values of C_s are given in Figure 8-8, or can be approximated from:

$$C_s = 5.30\left(\frac{L}{r_e}\right)^{0.73}$$

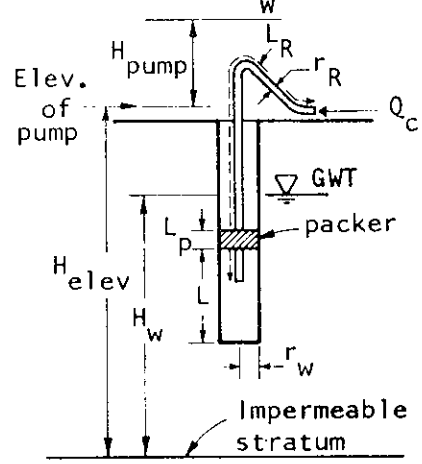
Unlined Test Section With Single Packer

Analysis is valid for : Zone 3

$L \geq 10r_w$

$$\frac{Q}{2\pi L r_w + \pi r_w^2} \leq 0.1 \text{ fps}$$

$H_w \geq 5L$



$$K = \frac{Q_c}{F \cdot H_c}$$

$H_c = H - H_w$

$H = H_{elev} + H_{pump} - H_L$

where H_L = head loss due to friction in drill rods.

H_L = fct (L_R, r_R) and H_L may be computed based on calibration curves

$$F = (C_s + 4) \cdot r_w$$

Values of C_s are given in Figure 8-8, or can be approximated from:

$$C_s = 5.30\left(\frac{L}{r_w}\right)^{0.73}$$

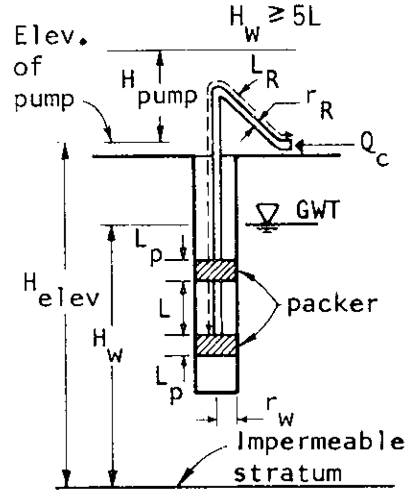
Unlined Test Section With Two Packers

Analysis is valid for : Zone 3

$L \geq 10r_w$

$$\frac{Q}{2\pi L r_w} \leq 0.1 \text{ fps}$$

$H_w \geq 5L$



$$K = \frac{Q_c}{F \cdot H_c}$$

$H_c = H - H_w$

$H = H_{elev} + H_{pump} - H_L$

where H_L = head loss due to friction in drill rods.

H_L = fct (L_R, r_R) and H_L may be computed based on calibration curves

$$F = C_s \cdot r_w$$

Values of C_s are given in Figure 8-8, or can be approximated from:

$$C_s = 5.30\left(\frac{L}{r_w}\right)^{0.73}$$

Figure 8-7 USBR ANALYSIS METHOD FOR CONSTANT HEAD TEST - ZONE 3 (BELOW WATER TABLE)
(after USBR, G-97)

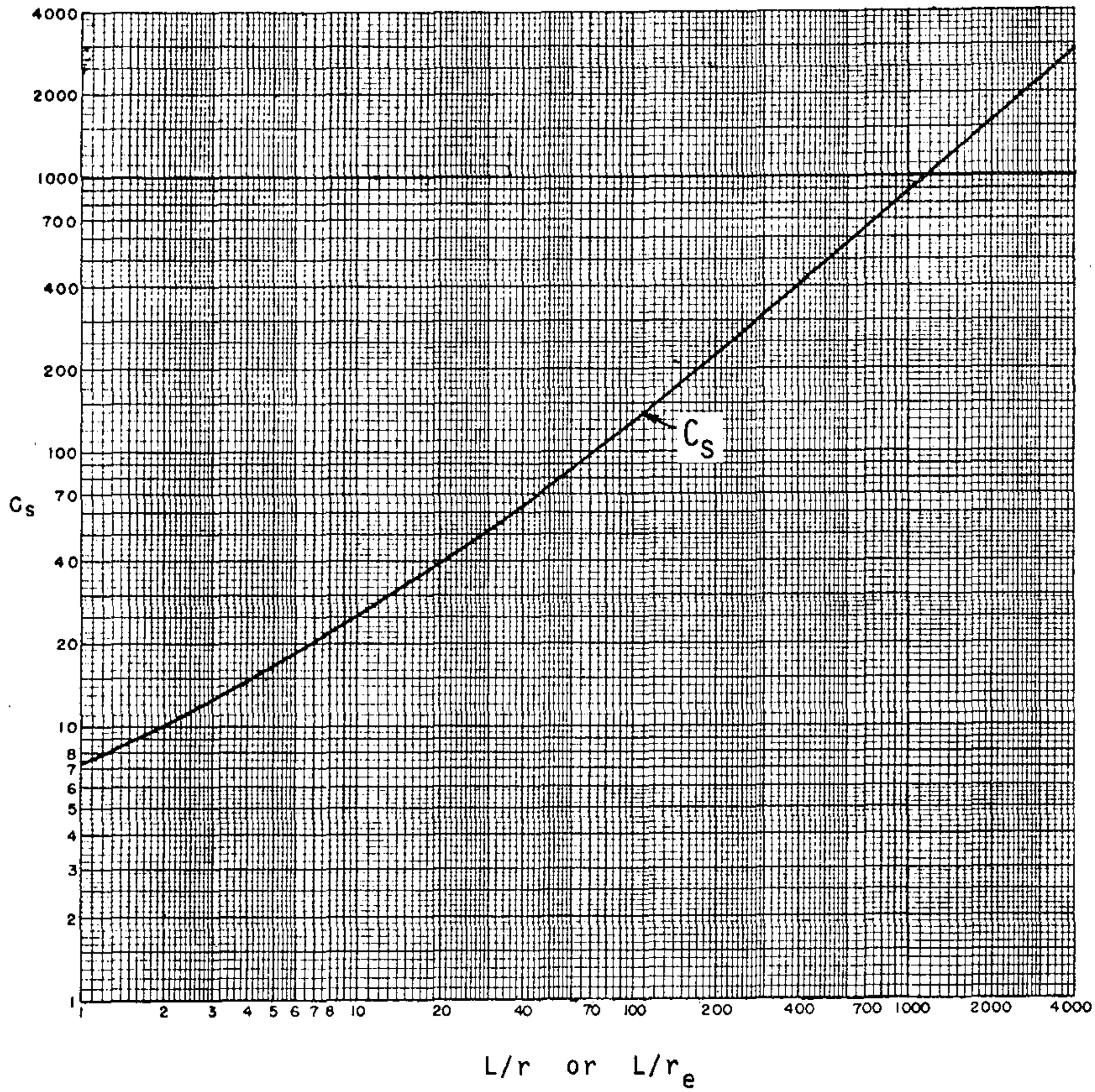
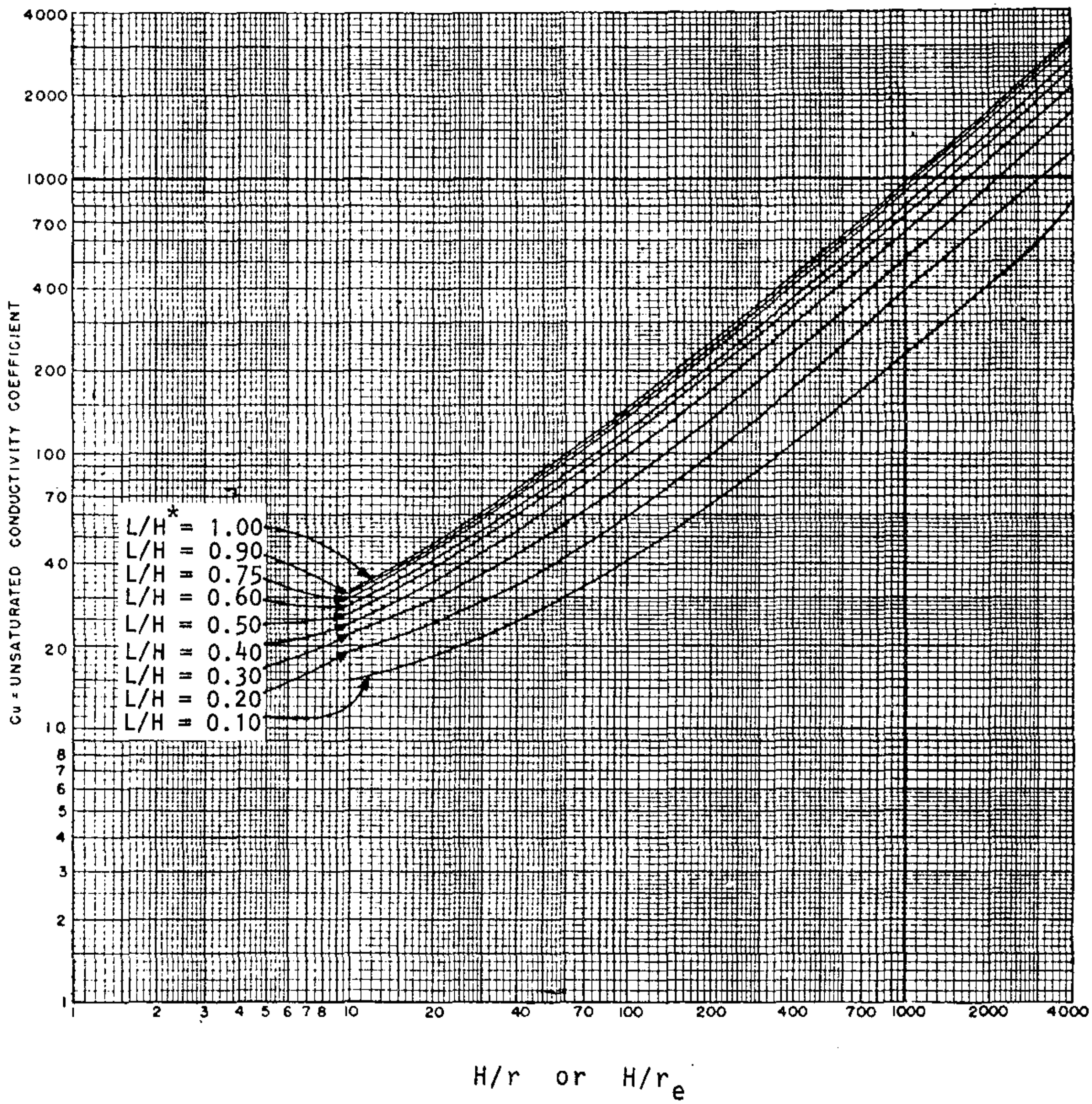


Figure 8-8 C_s VALUES FOR USBR CONSTANT HEAD TEST ANALYSIS
(USBR, G-97)



*L/H_{elev} for packer tests

Figure 8-9 C_u VALUES FOR USBR CONSTANT HEAD TEST ANALYSIS (USBR, G-97)

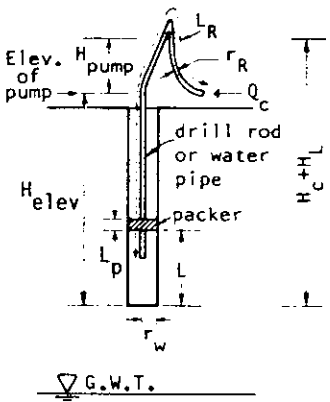
• ABOVE THE WATER TABLE • | **• BELOW THE WATER TABLE •**

Unlined Test Section With Single Packer (above the water table)

Analysis is valid for :

$L > 10r_w$

$L_p > 10r_w$



$$K = \frac{Q_c}{F \cdot H_c}$$

$H_c = H_{pump} + H_{elev} - H_L$;

H_L = head loss due to friction in rods. $H_L = fct\ (L_R, r_R)$

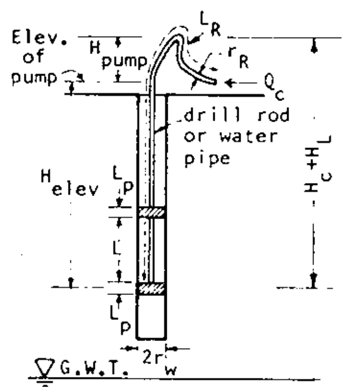
Values of H_L may be computed from calibration curves.

Unlined Test Section With Two Packers (above the water table)

Analysis is valid for :

$L > 10r_w$

$L_p > 10r_w$



$$K = \frac{Q_c}{F \cdot H_c}$$

$H_c = H_{pump} + H_{elev} - H_L$

H_L = head loss due to friction in drill rods. Computation for H_L obtained from calibration curves. $H_L = fct\ (L_R, r_R)$

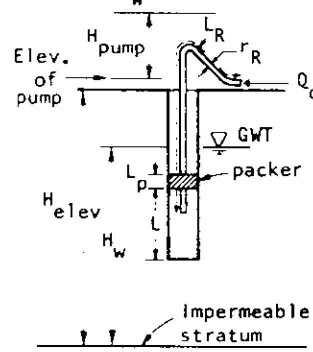
Unlined Test Section With Single Packer (below the water table)

Analysis is valid for :

$L \geq 10r_w$

$L_p > 10r_w$

$H_w \geq 5L$



$$K = \frac{Q_c}{F \cdot H_c}$$

$H_c = H - H_w$

$H = H_{elev} + H_{pump} - H_L$

where H_L = head loss due to friction in drill rods.

$H_L = fct\ (L_R, r_R)$ and H_L may be computed based on calibration curves

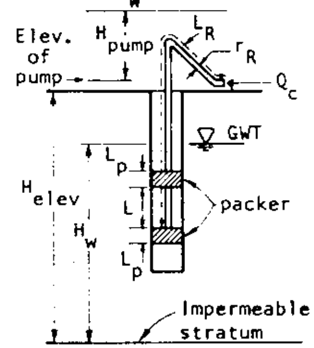
Unlined Test Section With Two Packers (below the water table)

Analysis is valid for :

$L \geq 10r_w$

$L_p > 10r_w$

$H_w \geq 5L$



$$K = \frac{Q_c}{F \cdot H_c}$$

$H_c = H - H_w$

$H = H_{elev} + H_{pump} - H_L$

where H_L = head loss due to friction in drill rods.

$H_L = fct\ (L_R, r_R)$ and H_L may be computed based on calibration curves

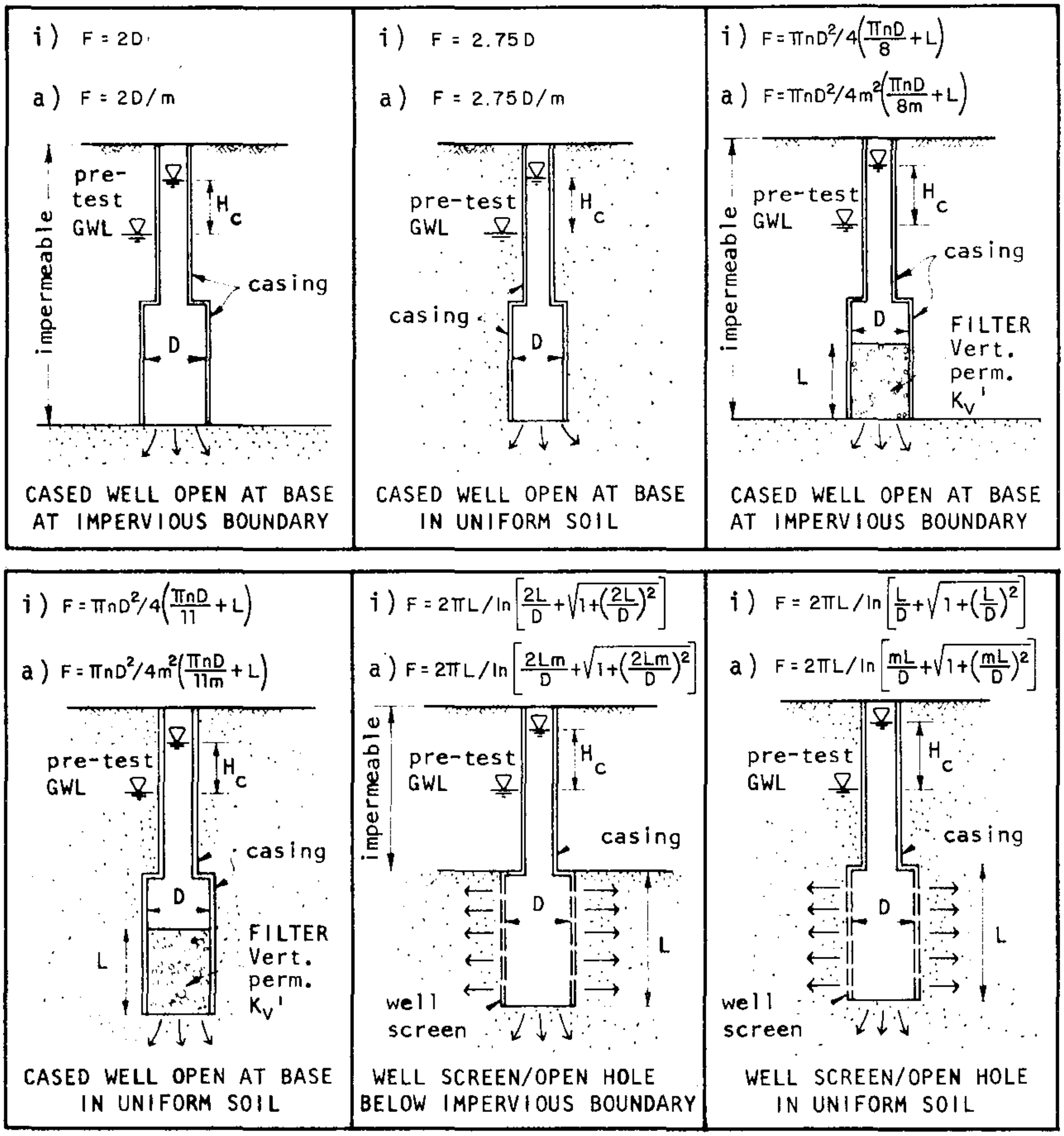
For all cases: $F = \dfrac{2\pi L}{\ln(L/r_w)}$

Figure 8-10 USBR (EARTH MANUAL) ANALYSIS METHOD FOR CONSTANT HEAD TEST
(after USBR Earth Manual, 1974)

i isotropic conditions: $K_h = K_v = K$

a – anisotropic conditions: $K_h \neq K_v$

$$K \text{ or } K_h = \frac{Q_c}{(F \cdot H_c)}$$



DEFINITIONS: $K_m = \sqrt{K_v K_h}$; $m = \sqrt{K_h/K_v}$; $n = K_v'/K_v$

where K_v = vertical permeability of soil/rock mass

K_h = horizontal permeability of soil/rock mass

K_v' = vertical permeability of filter in casing

Figure 8-11 HVORSLEV'S ANALYSIS METHOD FOR CONSTANT HEAD TEST - ZONE 3 (BELOW WATER TABLE) (1951)

vi) Then $K = \frac{Q_c}{F\ H_c}$

where Q_c is the constant flow under steady state conditions.

vii) Additional methods of analysis are available if data can also be obtained from observation wells near the test well. Refer to Figures 8-12 and 8-13 and determine if the conditions are similar to the conditions of the test to be analyzed. Figure 8-12 is for Zone 1, overlying an impermeable bed. Figure 8-13 is for Zone 3. The formulas for computing the permeability are given on each figure.

8.3 WELL PUMPING TESTS

8.3.1 Background

The analysis of well pumping tests has been covered extensively in the technical literature. The background for the solution methods used here can be found in the references cited below.

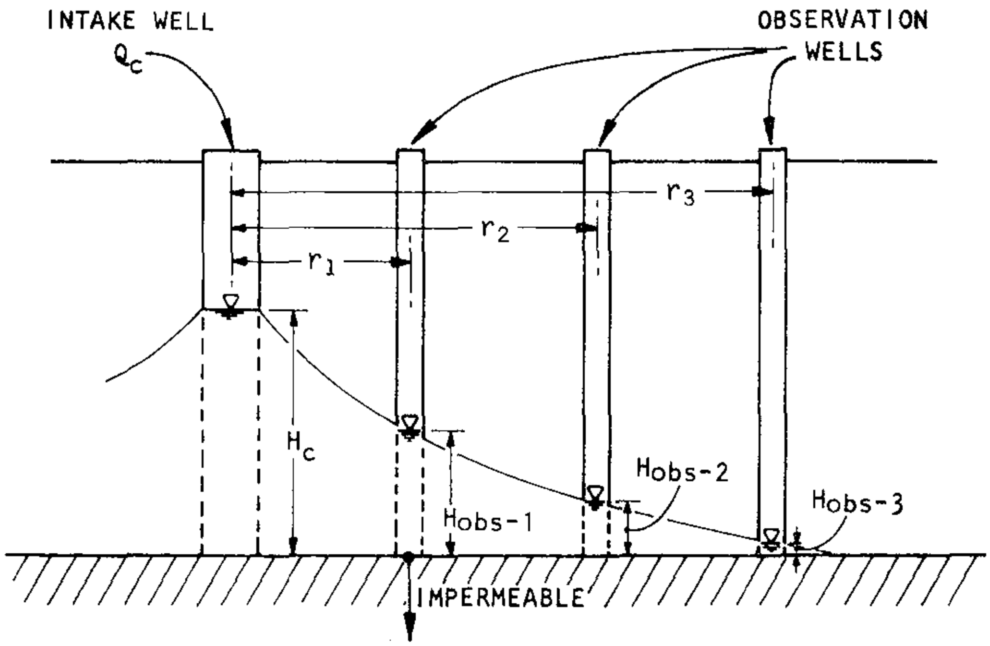
Equations presented in this section may conform to traditional and frequently inconsistent units of measurement. In such cases, two forms of the equation are given - one which requires those traditional units specified, and one which is valid for any consistent system of units. The latter form is referred to in the text and figures as that valid for "compatible units."

In the following discussion, the water-bearing stratum whose permeability is determined by well-pumping tests is called an "aquifer." If the aquifer is overlain/underlain by a less permeable stratum, then that overlying/underlying layer is called an "aquitard."

Well pumping tests are usually run for a maximum of several days. As a result of the short time span of the test and the nature of recharge to the aquifer, equilibrium conditions may not be reached during the pumping test. Consequently, many well-pumping analysis methods are for non-steady state conditions, i.e. before equilibrium is reached. Equilibrium methods are easy to use but are only appropriate when steady-state conditions have been achieved.

Figure 7-13 is a schematic of a well pumping test. It defines some of the basic parameters used in the analyses.

Permeability, K, is a function of the more common hydrologic term, transmissivity, T. K = T/m where m is the saturated thickness of the aquifer. The value "m" is determined by borings or other exploratory methods. As a result of the analysis of the pumping



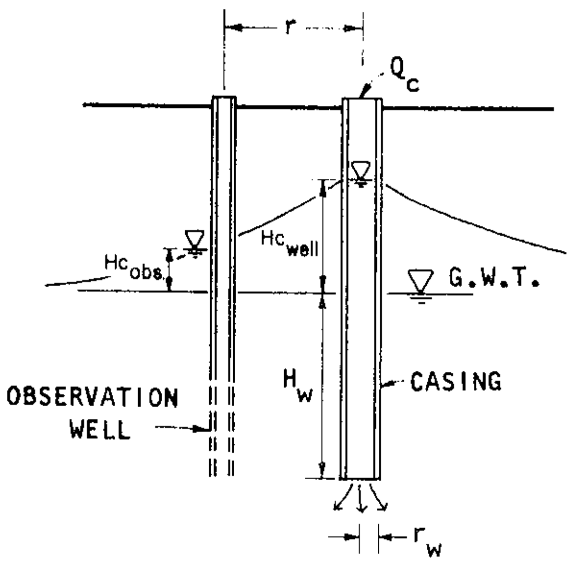
$$k = \frac{2.3\ Q_c \log(r_3/r_2)}{\pi(H^2_{obs-2} - H^2_{obs-3})} = \frac{2.3\ Q_c \log(r_3/r_1)}{\pi(H^2_{obs-1} - H^2_{obs-3})} = \frac{2.3\ Q_c \log(r_2/r_1)}{\pi(H^2_{obs-1} - H^2_{obs-2})}$$

Q_c - in units of flow per unit time

r_i, H_i - in units of length

Figure 8-12 USBR ANALYSIS METHOD FOR CONSTANT HEAD TEST USING OBSERVATION WELLS - ZONE 1 (ABOVE WATER TABLE, UNSATURATED)
(after USBR, G-97)

VALID FOR: ZONE 3
1 Observation Well
required



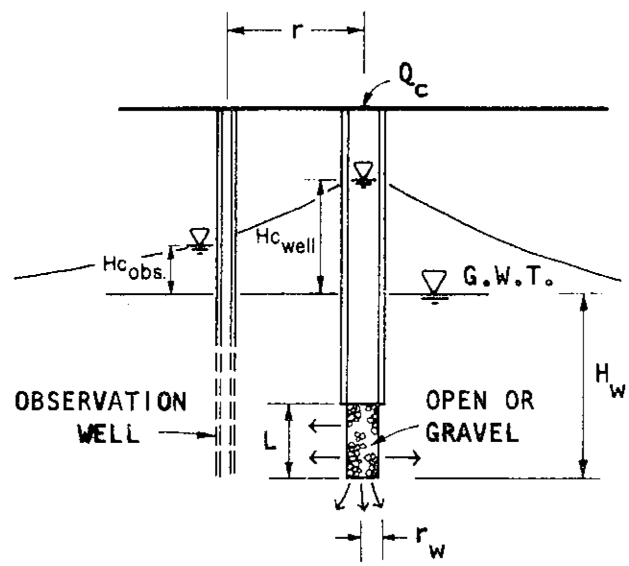
$$k = \frac{Q_c}{F \cdot \Delta H_c}$$

where $\Delta H_c = H_{c_{well}} - H_{c_{obs}}$

$$F = \frac{4\pi r_w \cdot r}{r - r_w}$$

CASED WELL OPEN AT BASE

VALID FOR: ZONE 3
1 Observation Well
required



$$k = \frac{Q_c}{F \cdot \Delta H_c}$$

where $\Delta H_c = H_{c_{well}} - H_{c_{obs}}$

$$F = \frac{4\pi\, L/2}{\ln \left[\frac{r\,(r_w + L/2)}{r_w (r + L/2)} \right]}$$

PARTLY CASED WELL WITH
UNLINED TEST SECTION

Figure 8-13 SCHMID'S ANALYSIS METHOD FOR CONSTANT HEAD TEST USING OBSERVATION WELL - ZONE 3 (BELOW WATER TABLE) (1967) test, it may also be possible to assess the coefficient of storage, S, of the aquifer and the values of K, T and S for an overlying aquitard (if it exists).

These and other terms commonly used to define aquifer and aquitard characteristics and pumping test data are defined below.

T = transmissivity: the rate of horizontal flow through a vertical strip 1 foot wide and m feet high (where m = the saturated thickness) under a hydraulic gradient of 1. T = (K)x(m). Transmissivity is usually expressed in "gallons per day/foot" units, or more simply, gpd/ft.

S = coefficient of storage: the volume of water released from or taken into storage per unit surface area per unit change in head. The coefficient of storage is dimensionless. For water table aquifers, S is called the specific yield.

K = permeability: the rate of flow through a unit area under a hydraulic gradient of 1. Permeability units are usually cm/sec or gpd/ft^2 (2.1 x 10^4 gpd/ft^2 = 1 cm/sec).

m = saturated thickness of aquifer or aquitard. "m" is usually given in feet.

s = drawdown, usually expressed in feet.

Q = discharge from pumped well, commonly expressed in gpm.

r_w= radius of pumped well, usually given in feet.

r = radius from center of pumped well to center of observation well, usually given in feet.

Although the units presented in the above descriptions are those most commonly used in practice, conversion to other units can be obtained by using appropriate factors in Appendix E.

It is helpful to identify some basic aquifer characteristics before discussing methods to interpret well-pumping tests.

Artesian or Water Table Aquifer: In an artesian aquifer the pre-test groundwater level will rise in an observation well to a height above the top of the aquifer, while in a water table aquifer the groundwater level will stabilize below the top of the aquifer.

Fully or Partially Penetrating Wells: The pumped well is fully penetrating if the casing or well screen is slotted or open for the full thickness of the aquifer (and not slotted above or below the aquifer). A partially penetrating well has an intake section over only a portion of the aquifer.

Leaky or Non-Leaky: If the overlying strata allow water to percolate from upper aquifers or from the surface to the test aquifer, the aquifer is leaky.

Water Released/Not Released from Storage in the Aquitard: If the groundwater levels in the overlying stratum (aquitard) are lowered during the test, then water is probably "released from storage" in the aquitard.

8.3.2 Analysis of Constant Discharge Tests

In a constant discharge test, the pump is turned on and allowed to pump at a constant flow rate, Q. Three methods are discussed below: Jacob's straight line method, the type curve fitting method, and the equilibrium formula method. Both Jacob's method and the type curve fitting method are solutions for non-equilibrium conditions. Jacob's method is a quick solution, but less accurate than the type curve fitting method. If equilibrium is reached, then the equilibrium formula method, accurate and easy to use, should be used.

8.3.2.1 Jacob's Straight Line Method - This method is based on Theis' curves for non-leaky artesian aquifers with fully penetrating wells. However, it can be used as a first approximation for water table aquifers (Walton, 1970). It is most appropriate for sands and gravels or materials of high permeability because the drawdown vs. log time curve becomes linear more rapidly in materials with high permeability. The value of the coefficient of storage computed by this method may not be very accurate, but the calculation of transmissivity and permeability should be adequate if enough data is available (Walton, 1970).

A general methodology holds for both the Drawdown-Time and Drawdown-Distance analyses. An iterative scheme is used to obtain successive approximations of S and T. For the initial line-fitting procedure, data from the first hour of the test should not be used, and the later part of the test should be given more weight. After initially approximating S and T, as determined from the formulas in Figure 8-14, determine the time

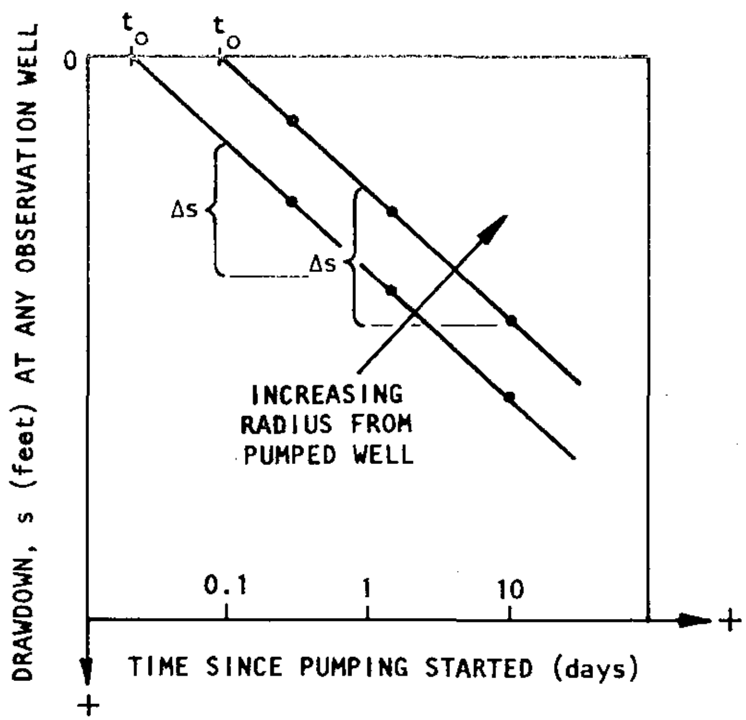
$$t_{s1} = \frac{1.35 \times 10^5 r^2 S}{T}$$

where t_{s1} = time after pumping starts before a semilog time-drawdown or distance-drawdown plot will yield a straight-line graph, in minutes

r = distance from pumped well to observation well, in feet

T = transmissivity, in gpd/ft

S = coefficient of storage, fraction (Walton, 1970)



$$T = \frac{264Q}{\Delta s}$$

$$S = \frac{0.3Tt_o}{r^2}$$

$$k = \frac{T}{m}$$

Where:

Q = flow (gpm)

Δs = drawdown over 1 log cycle (feet)

T = transmissivity (gpd/ft)

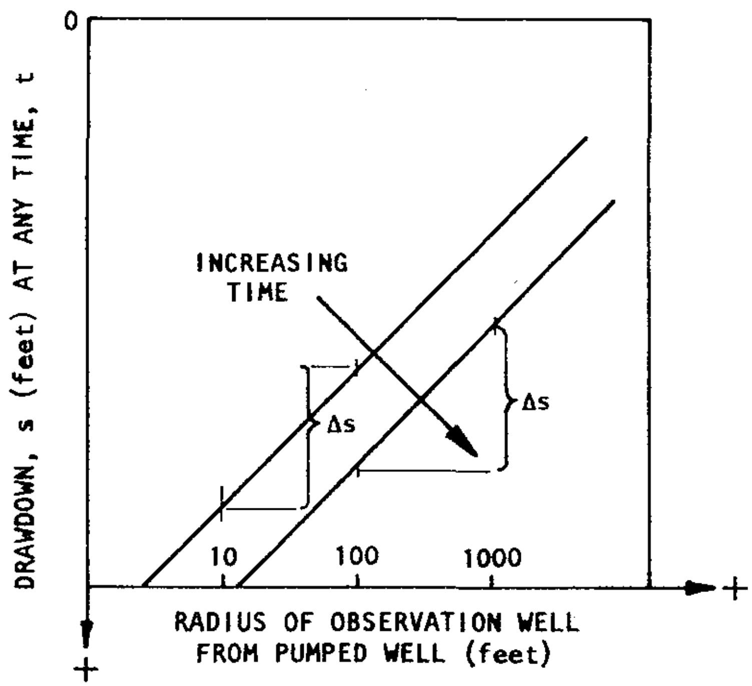
t_o = time intercept when s = 0 (days)

r = radius of observation well from pumped well (feet)

S = coefficient of storage

k = permeability (gpd/ft^2)

NOTE: For compatible units, $T = \frac{0.183Q}{\Delta s}$, $S = \frac{2.25Tt_o}{r^2}$



$$T = \frac{527.7Q}{\Delta s}$$

$$S = \frac{0.3Tt}{r_o^2}$$

$$k = \frac{T}{m}$$

Units as Above Except:

Δs = drawdown over 1 log cycle of r(feet)

r_o = radius intercept when s = 0 (feet)

NOTE: For compatible units, $T = \frac{0.366Q}{\Delta s}$, $S = \frac{2.25Tt}{r_o^2}$

Figure 8-14 STRAIGHT LINE METHOD (CONSTANT DISCHARGE)

For compatible units, $t_{s1} = \frac{701.3\ r^2\ S}{T}$

Then recalculate S and T using only data recorded after time t_{s1}. This iterative process will give a more accurate estimate of S and T.

The specific analysis procedures are now presented.

i) Drawdown-Time Method (Refer to Figure 8-14)

a) Plot drawdown, s, vs. log time, t, for any observation well. Time, t, may be in days or minutes for the logarithmic plot.

b) Fit a straight line through the data and extend it to s = 0. Find t_o = time at s = 0, and convert time to days.

c) Find Δs, change in drawdown, over any log cycle of time.

d) Solve for T, S and K as follows:

$$T = \frac{264Q}{\Delta s}$$

Q = flow (gpm)

Δs = change in drawdown (feet) over 1 log cycle

T = transmissivity (gpd/ft)

$$S = \frac{0.3Tt_o}{r^2}$$

r = distance to observation well from pumped well (feet)

t = time (days)

m = sat. thickness of aquifer (feet)

$$K = T/m$$

K = permeability (gpd/ft^2)

For compatible units, $T = \frac{0.183Q}{\Delta s}$, $S = \frac{2.25\ Tt_o}{r^2}$

The analysis of the test results at all observation wells should give the same values of S, T and K. See comments in Section 8.3.2.4 if results differ between wells.

ii) Drawdown-Distance Method (Refer to Figure 8-14)

a) Plot drawdown, s, vs. log radius, r, at a given time, t, for a series of observation wells.

b) Fit a straight line through the data and extend it to s = 0. Find r_o = radius at s = 0.

c) Find Δs over any log cycle of radius.

d) Solve for T, S and K as follows:

$T = \frac{527.7Q}{\Delta s}$ — Q = flow (gpm); Δs = drawdown (feet) over 1 log cycle

$S = \frac{0.3\ Tt}{r_o^2}$ — T = tranmissivity (gpd/ft); S = coefficient of storage

$K = T/m$ — m = aquifer thickness (feet); t = time (days); K = permeability (gpd/ft^2)

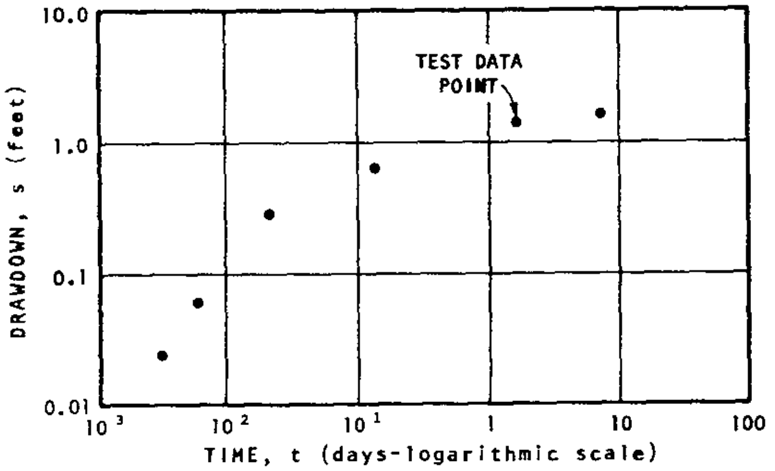
For compatible units, $T = \frac{0.366Q}{\Delta s}$, $S = \frac{2.25\ Tt}{r_o^2}$

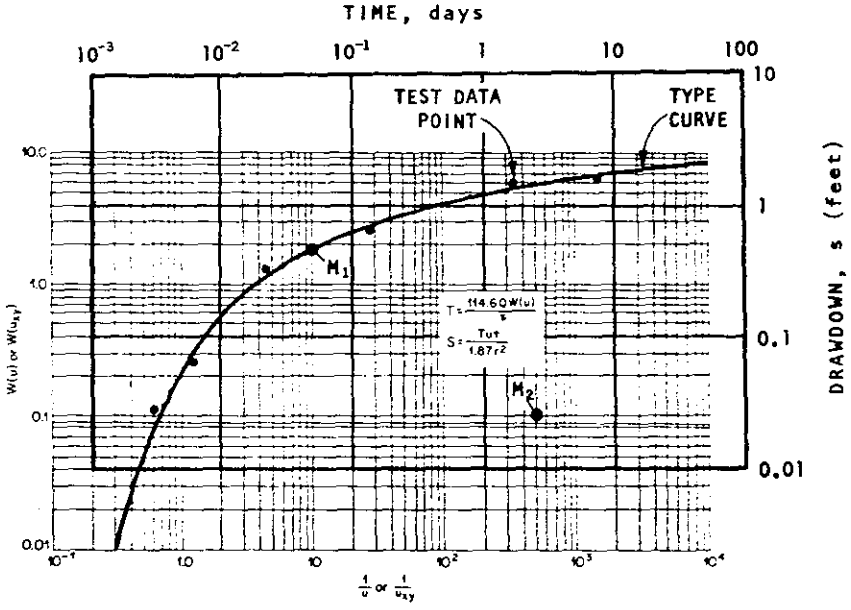
This method should give the same values of S, T and K for any time t. The values of S, T and K should be the same as computed by the drawdown-time procedure. See comments in Section 8.3.2.4 if the values of S, T and K vary with time.

The above methods can be used for analysis of both pumping and recovery data. For pumping data, the following criteria (Todd, 1959) apply. Use the drawdown $s = h_o - h$, where h_o = the initial height of water in the well before the test and h = the height of water in the well at time t. Measure time t from the point that pumping commences. For recovery data, use the residual drawdown $s' = h_o - h$ where h_o is defined as above and h is the height of water in the well at time t'. Measure time t' after pumping is stopped. The discharge Q used in the calculations is the constant discharge used during the test.

8.3.2.2 Type Curve Fitting Method - The straight line method presented in the previous section is limited because it is an approximate solution most appropriate to large values of time and permeability, and non-leaky artesian aquifer conditions. More precise solutions for a variety of well-pumping situations are available. Mathematical solutions have been found for various combinations of those aquifer characteristics itemized in Section 8.3.1. These solutions are plotted in the form of a well function W(u) and are called "type" curves. Figure 8.15 illustrates the type curve fitting method, and Figures 8-16 through 8-21 present the type curves for different well-pumping situations.

TEST DATA GRAPH





TYPE CURVE FITTED TO DATA
(Example shown in Non-Leaky Artesian Aquifer Curve)

SYMBOLS:

T = transmissivity (gpd/ft)
S = coefficient of storage
s = drawdown at observation well (feet)
t = time after pumping began (days)
r = radius from pumped well to observation well
Q = discharge (gpm)

M_1, M_2 = Arbitrarily selected match points. Either point will give same result.

For example:

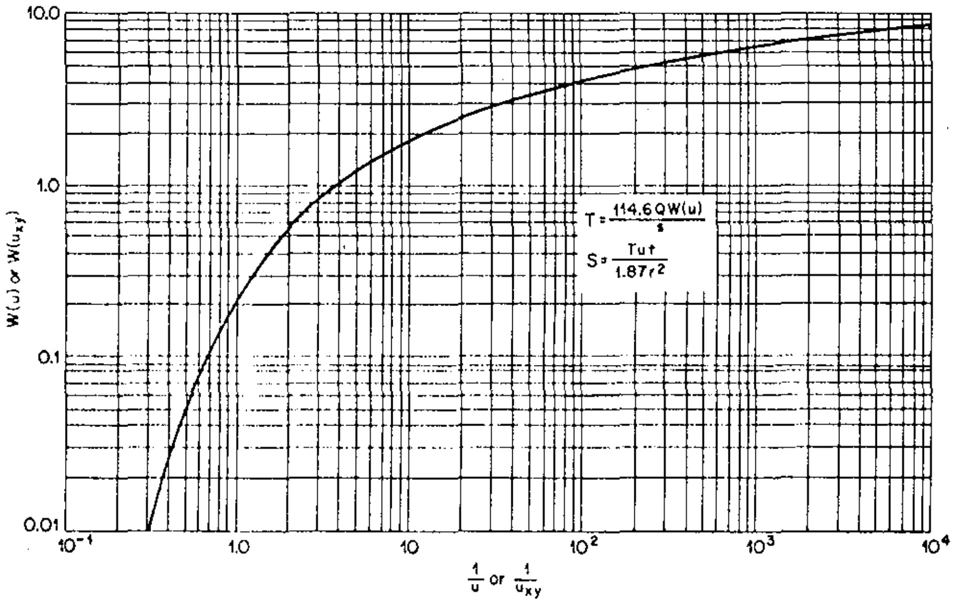
- at $M_1: \frac{1}{u} = 10$; $W(u) = 2$; $t = 5.2 \times 10^{-2}$; $s = 5.0 \times 10^{-1}$
- at $M_2: \frac{1}{u} = 500$; $W(u) = 0.1$; $t = 2.6$; $s = 2.5 \times 10^{-2}$

and either set of four coordinates will yield equal values for S and T.

NOTE: For compatible units, $T = \frac{Q\,W(u)}{4\pi s}$, $S = \frac{4Tut}{r^2}$

Figure 8-15 PROCEDURE FOR MATCHING "TYPE" CURVE TO DATA

Figure 8-15



<u>Where:</u>

T = transmissivity (gpd/ft)

S = coefficient of storage

s = drawdown at observation well (feet)

t = time after pumping began (days)

r = radius from pumped well to observation well

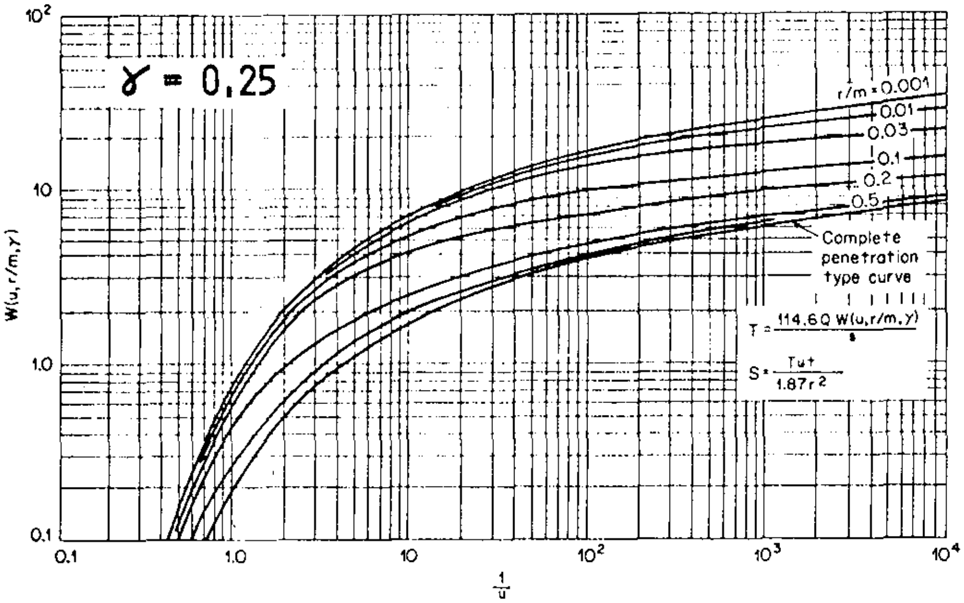
Q = discharge (gpm)

For compatible units:

$$T = \frac{Q}{4\pi s} W(u), \quad S = \frac{4Tut}{r^2}$$

Figure 8-16 TYPE CURVE FOR NON-LEAKY ARTESIAN AQUIFER WITH:

- FULLY PENETRATING PUMPED WELL
- NO WATER RELEASED FROM STORAGE IN AQUITARD
- CONSTANT DISCHARGE (Walton, 1970)



<u>Where:</u>

T = transmissivity (gpd/ft)

S = coefficient of storage

r = radius from pumped well to observation well(feet)

m = thickness of saturated layer of aquifer (feet)

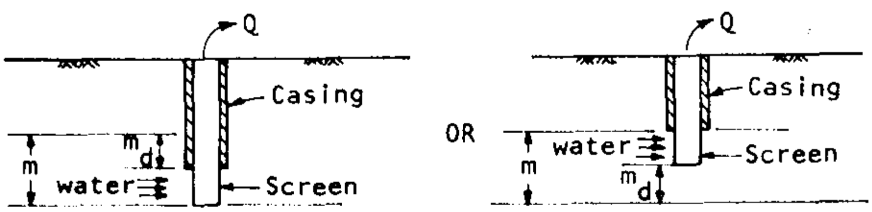
m_d = thickness of saturated layer which well penetrates without open screen (feet)

γ = $(m - m_d)/m$

s = drawdown at observation well (feet)

t = time after pumping began (days)

Q = discharge (gpm)

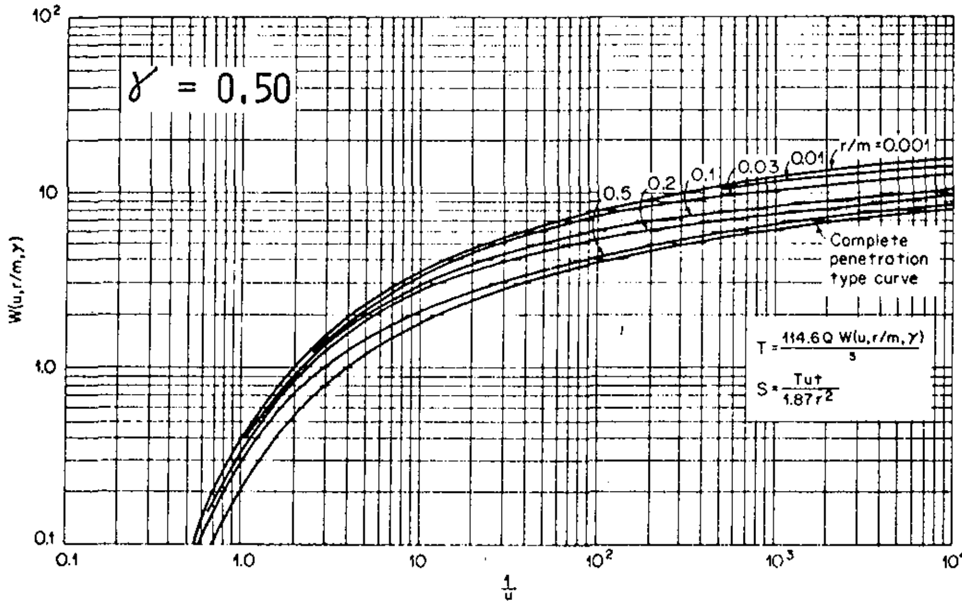


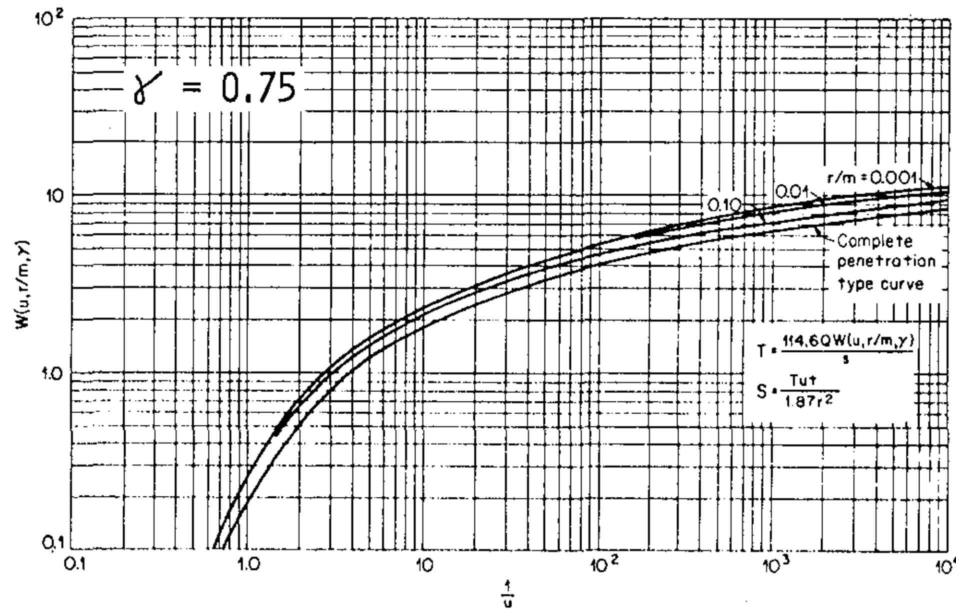
For compatible units:

$$T = \frac{Q\,W(u, {}^{r}/_{m}, \gamma)}{4\pi s}, \quad S = \frac{4Tut}{r^2}$$

Figure 8-17 TYPE CURVES FOR NON-LEAKY ARTESIAN AQUIFER WITH:

- PARTLY PENETRATING PUMPED WELL
- NO WATER RELEASED FROM STORAGE IN AQUITARD
- CONSTANT DISCHARGE (Walton, 1970)



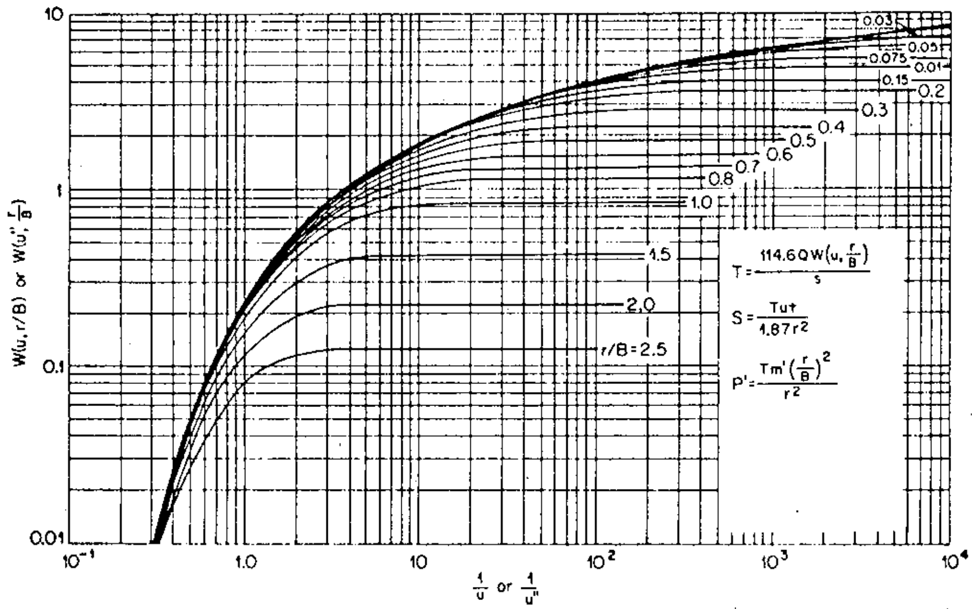


For compatible units:

$$T = \frac{Q\ W(u,\ {}^{r}/_{m},\ \gamma)}{4\pi s},\quad S = \frac{4Tut}{r^2}$$

Figure 8-18 TYPE CURVES FOR NON-LEAKY ARTESIAN AQUIFER (CONTINUED)

(Walton, 1970)



<u>Where:</u>

T = transmissivity (gpd/ft)
S = coefficient of storage
Q = discharge (gpm)
s = drawdown at observation well (feet)
r = radius from pumped well to observation well (feet)
B = 'leakage factor'; determined by curve selection:

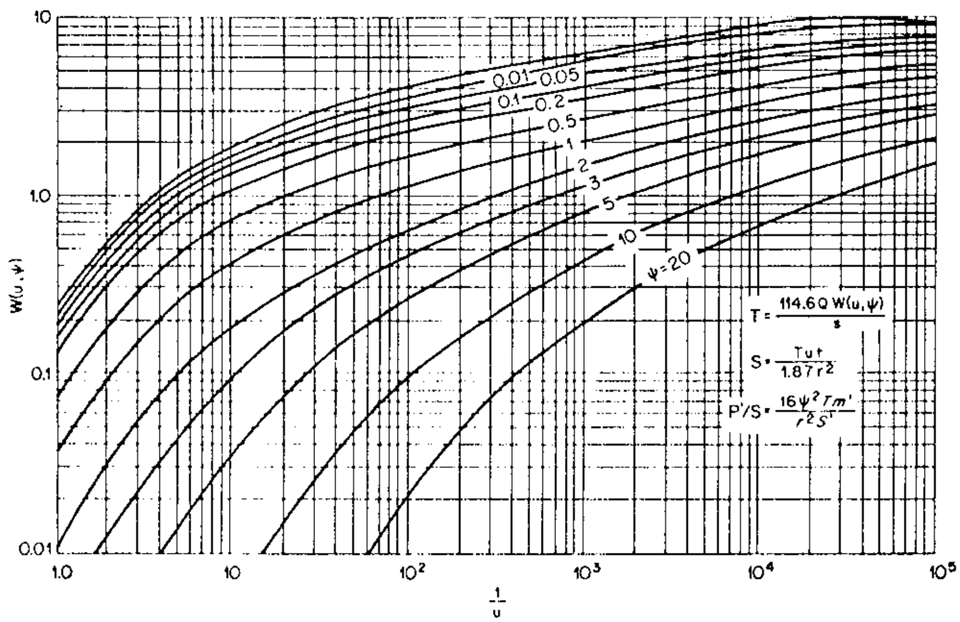
$$\frac{r}{B} = \frac{r}{\sqrt{T/(P'/m')}}$$

t = time after pumping began (days)
P' = permeability of aquitard (gpd/ft^2)
m' = saturated thickness of aquitard (feet)

For compatible units: $T = \frac{Q\ W(u,\ {}^{r}/B)}{4\pi s}$, $S = \frac{4Tut}{r^2}$

Figure 8-19 TYPE CURVE FOR LEAKY ARTESIAN AQUIFER WITH:

- FULLY PENETRATING PUMPED WELL
- NO WATER RELEASED FROM STORAGE IN AQUITARD
- CONSTANT DISCHARGE (Walton, 1962)



Where:

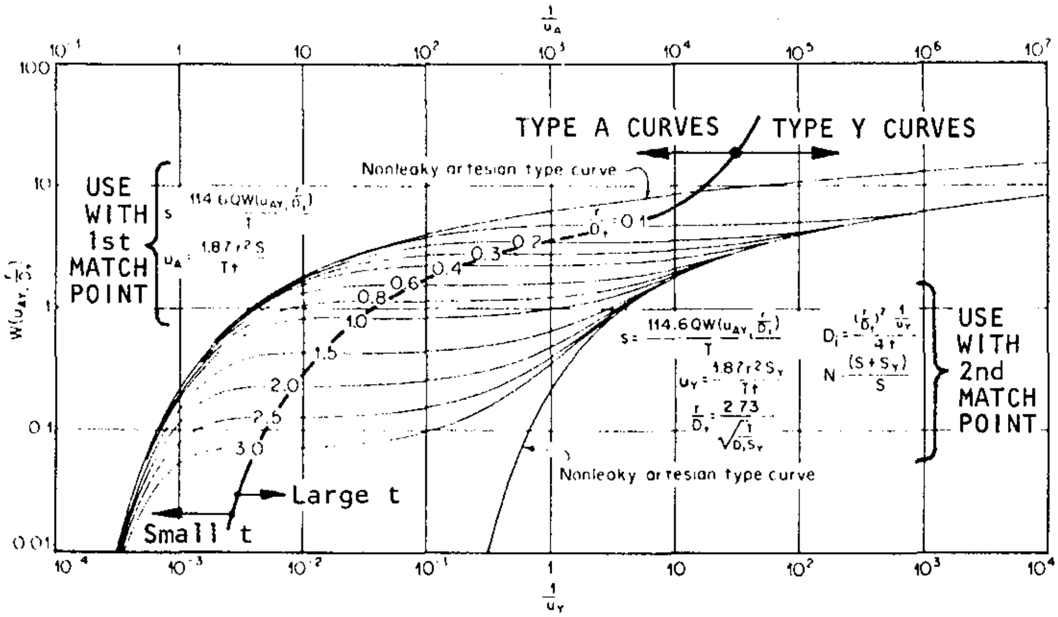
T	=	transmissivity (gpd/ft)
S	=	coefficient of storage
t	=	time after pumping began (days)
r	=	radius from pumped well to observation well (feet)
P'	=	permeability of aquitard (gpd/ft^2)
S'	=	storage coefficient of aquitard
m'	=	saturated thickness of aquitard (feet)
ψ	=	'storage factor'; determined by curve fitting procedure

$$\psi = \left(\frac{r}{4}\right)\left(\sqrt{\frac{S'P'}{TSm'}}\right)$$

For compatible units: $T = \dfrac{Q\ W(u,\ \psi)}{4\pi s}$, $S = \dfrac{4Tut}{r^2}$

Figure 8-20 TYPE CURVE FOR LEAKY ARTESIAN AQUIFER WITH:

- FULLY PENETRATING PUMPED WELL
- WATER RELEASED FROM STORAGE IN AQUITARD
- CONSTANT DISCHARGE (Walton, 1970)



Where:

- T = transmissivity (gpd/ft)
- S = coefficient of storage (artesian conditions)
- Q = discharge (gpm)
- t = time after pumping began (days)
- r = radius from pumped well to observation well (feet)
- S_y = coefficient of storage associated with gravity drainage of pore spaces = specific yield
- D_i = reciprocal of delay index (days^{-1})
- D_t = 'gravity factor': $D_t = \sqrt{\frac{T}{D_i S_y}}$

NOTES:
1. Match early data to type A curve; find match point s_1, t_1, $W(u_a, \frac{r}{D_t})$, $\frac{1}{u_a}$; solve for S, T.
2. Slide type curve (same r/D_t) horizontally and match later data to type Y curve (same r/D_t). Find match point s_2, t_2, $W(u_y, r/D_t)$, $\frac{1}{u_y}$; Solve for T, S_y. T should be approximately the same as in step 1.

For compatible units:

$$s = \frac{Q\, W(u_{ay}, {}^{r}/D_t)}{4\pi T}, \quad u_a = \frac{r^2 S}{4Tt}, \quad u_y = \frac{r^2 S_y}{4Tt}, \quad \frac{r}{D_t} = \frac{r}{\sqrt{T/D_i S_y}}$$

Figure 8-21 TYPE CURVE FOR WATER TABLE AQUIFER WITH:
- FULLY PENETRATING PUMPED WELL
- CONSTANT DISCHARGE (Prickett, 1965)

The procedure for the type curve fitting method follows (Walton, 1970):

i) Characterize the aquifer as accurately as possible as to artesian conditions, degree of well penetration, leaky/non-leaky conditions, water release from storage in the aquitard, etc. This will aid in the selection of the "type" curves. However, it is not necessary to know in advance of the curve fitting procedure if the aquifer is leaky or if water is released from storage in the aquitard during testing.

ii) Plot log drawdown, s (vertical scale) vs. log time, t, as shown in Figure 8-15. The scale of one log cycle of the "test" data should be the same as that of one log cycle on the "type" curves. The units used to plot drawdown and time should be feet and days, respectively, to facilitate the use of the type curves.

iii) Select the type curve best representing your initial characterization of the aquifer. Superimpose the "test" curve over the "type" curve and move until a good fit is obtained, keeping the axes parallel (see Figure 8-15). Try to fit the data over other "type" curves until the best fit is obtained.

iv) Pick any point on the graph where the type curve and data curve are matched and read off W and 1/u from the "type" curve, and s and t from the test data curve. This point is called the "match point." See Figure 8-15.

v) Calculate S and T from the equations printed on the type-curve graphs and the values of W, 1/u, s and t at the match point. Then $K = T/m$.

Additional parameters needed for the selection of "type" curves and calculations are given on Figures 8-16 through 8-21.

8.3.2.3 Equilibrium Method - If equilibrium is reached, the data analysis is simplified greatly. At equilibrium, both flow Q and drawdown are constants in the pumped well, and drawdown is constant in the observation wells. Only leaky aquifers or aquifers with a source of recharge can reach equilibrium. At least two observation wells are necessary to solve for the permeability using this equilibrium method.

i) Equilibrium Formula for Leaky Artesian Aquifer (Kruseman and DeRidder, 1970) with fully penetrating pumped well; water may or may not be released from storage in aquitard.

Procedure:

a) Prepare a type cruve by plotting values of the Bessel function $K_0(r/B)$ versus the variable (r/B) on log-log paper, using the values in Table 8-1.

TABLE 8-1

VALUES OF THE FUNCTION $K_o(r/B)$ FOR GIVEN VALUES OF THE VARIABLE, r/B
(after Walton, 1970)

N \ r/B	$N \times 10^{-3}$	$N \times 10^{-2}$	$N \times 10^{-1}$	N
1.0	7.0237	4.7212	2.4271	0.4210
1.5	6.6182	4.3159	2.0300	0.2138
2.0	6.3305	4.0285	1.7527	0.1139
2.5	6.1074	3.8056	1.5415	0.0623
3.0	5.9251	3.6235	1.3725	0.0347
3.5	5.7709	3.4697	1.2327	0.0196
4.0	5.6374	3.3365	1.1145	0.0112
4.5	5.5196	3.2192	1.0129	0.0064
5.0	5.4143	3.1142	0.9244	0.0037
5.5	5.3190	3.0195	0.8466	
6.0	5.2320	2.9329	0.7775	0.0012
6.5	5.1520	2.8534	0.7159	
7.0	5.0779	2.7798	0.6605	0.0004
7.5	5.0089	2.7114	0.6106	
8.0	4.9443	2.6475	0.5653	
8.5	4.8837	2.5875	0.5242	
9.0	4.8266	2.5310	0.4867	
9.5	4.7725	2.4776	0.4524	

Examples

for r/B = .001 = 1×10^{-3}, K_o = 7.0237

for r/B = .003 = 3×10^{-3}, K_o = 5.9251

b) Plot on another sheet of log-log paper with the same scale the steady-state drawdown, s, of each observation well versus its corresponding distance from the pumped well, r.

c) Superimpose this data plot on the Bessel function type curve and while keeping the axes parallel, adjust so that the plotted points achieve an optimal fit to a portion of the type curve (see Figure 8-22 for illustration).

d) Select an arbitrary match point A on the overlapping portion of both sheets and record the values of s and $K_0(r/B)$ corresponding to that point.

e) Substitute the values of s and $K_0(r/B)$ into the following equation to obtain T:

$$T = \frac{229Q}{s} K_0(r/B)$$

The permeability of the aquifer may then be computed from the expression: $K = T/m$.

where:

$r/B = r/\sqrt{T/(K'/m')}$

s = drawdown at observation well (feet)

r = distance from pumped well to observation well (feet)

Q = discharge (gpm)

$K_0(r/B)$ = Bessel function given in Table 8-1

T = transmissivity of aquifer (gpd/ft)

K = coefficient of permeability of aquifer (gpd/ft^2)

K' = coefficient of permeability of aquitard (gpd/ft^2)

m = thickness of aquifer (feet)

m' = thickness of aquitard (feet)

Note: For compatible units, use:

$$T = \frac{Q}{2\pi s} K_0(r/B)$$

Note that a coefficient of storage, S, cannot be determined from steady-state data.

ii) Equilibrium formulas for an artesian or a water table aquifer with a source of recharge at the edge of the cone of depression with a fully penetrating pumped well.

This method does not specify the nature of the source of recharge. However, it implies that the cone of depression does not extend past a distance R from the pumped well; that is, recharge exists at the periphery of the cone of depression. Generally, R is not known, and data from at least two observation wells is needed. Only K or T = Km can be calculated in the steady state. The storage coefficient S cannot be obtained.

Water table aquifer: $$K = \frac{1055Q \log(r_2/r_1)}{h_2^2 - h_1^2}$$ (Johnson, 1972)

(For compatible units, $$K = \frac{2.3Q \log(r_2/r_1)}{\pi(h_2^2 - h_1^2)}$$)

Artesian aquifer: $$K = \frac{528Q \log(r_2/r_1)}{m\ (h_2 - h_1)}$$ (Johnson, 1972)

(For compatible units, $$K = \frac{1.15Q \log(r_2/r_1)}{m\ (h_2 - h_1)}$$)

Combined artesian and gravity flow (flow becomes artesian at distance r_t from pumped well):

Gravity flow: $$K = \frac{1055Q \log(r_t/r_1)}{m^2 - h_1^2}$$ for $r<r_t$

(For compatible units, $$K = \frac{2.3Q \log(r_t/r_1)}{m^2 - h_1^2}$$)

Artesian flow: $$K = \frac{528Q \log(r_t/r_2)}{m - h_2}$$ for $r>r_t$

(For compatible units, $$K = \frac{1.15Q \log(r_t/r_2)}{m - h_2}$$)

where h_i = the height of water in observation well "i" above the base of the aquifer (feet)

r_i = the distance from observation well "i" to the pumped well (feet)

K = permeability (gpd/ft^2)

Q = pumping rate (gpm)

m = saturated thickness of artesian aquifer (feet)

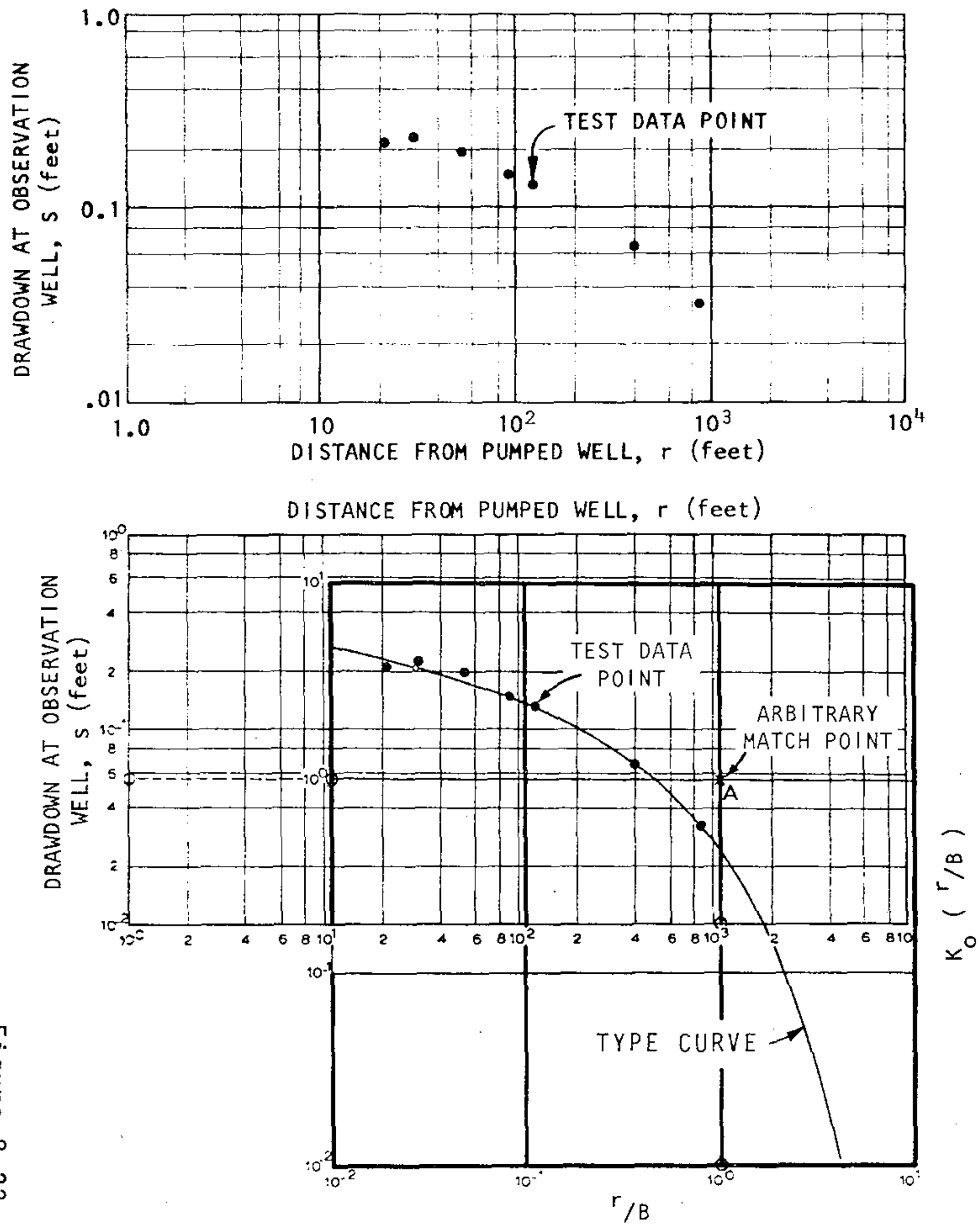


Figure 8-22

SYMBOLS:

T = transmissivity (gpd/ft)

s = drawdown at observation well (feet)

r = distance from pumped well to observation well (feet)

Q = discharge (gpm)

B = leakage factor (feet) = $\sqrt{T/(K'/m')}$

$K_o(^r/B)$ = modified Bessel function of the second kind and of zero order (See Table 8-1)

K' = permeability of leaky aquitard (gpd/ft^2)

K = permeability of aquifer (gpd/ft^2)

m' = thickness of leaky aquitard (feet)

m = thickness of aquifer

A = arbitrarily selected match point

$$T = \frac{229\ Q}{S} K_o\ (^r/B)$$

For compatible units:

$$T = \frac{Q}{2\pi s} K_o\ (^r/B)$$

Then $K = {}^T/_m$

Figure 8-22 PROCEDURE FOR MATCHING "TYPE" CURVE TO DATA

(after Kruseman and DeRidder, 1970)

8.3.2.4 Analysis Modifications -

i) Transmissivity of Water Table Aquifers:
Where water table aquifers are involved, the data used for non-equilibrium analysis should be adjusted to allow for decrease in transmissivity of the aquifer during pumping. The transmissivity, T = K•m, varies because drawdown reduces the saturated thickness, m, of the aquifer. As a result, m varies with time and distance during the test. To compensate for this effect in the calculations, the drawdown is transformed as follows:

$$s = s_{obs} - \frac{(s_{obs})^2}{2m}$$

where m = the initial saturated thickness of the aquifer
s_{obs} = the observed drawdown under water table conditions (Walton, 1970)

Then carry out the non-equilibrium analysis.

ii) Time Effect of Gravity Drainage on Water Table Aquifers:
In a water table aquifer, when the water level is lowered due to pumping, gravity drainage of the interstices occurs. With time, the drainage occurs farther and farther away from the pumped well; the movement of water near the well is due only to water flowing from the edge of the cone of depression to the well. Eventually, within the area covered by the observation wells, no more water is released from storage. That is, water is released from storage outside the observation area and flows through the observation area to the well. A water table aquifer may be analyzed as an artesian aquifer once gravity drainage in the observation area is complete (Walton, 1970). However, the drawdown must still be adjusted as noted in (i) above.

iii) Partially Penetrating Wells:
When the pumped well partially penetrates the aquifer, several non-equilibrium solutions are available, as shown in Figures 8-17 and 8-18. Note that for analysis purposes a partially penetrating well can be treated as fully penetrating when the observation well is further than 2m from the pumped well (Todd, 1959) (m = saturated thickness of aquifer). At this distance, flow becomes essentially radial.

For partially penetrating wells, flow lines are longer and head loss greater than for fully penetrating wells (Todd, 1959). As a result, for the same drawdown, the flow out of a partially penetrating well is less than that out of a fully penetrating well. If the well is only partially penetrating, then the value of the transmissivity will be substantially underestimated if the solutions for fully penetrating wells are used to analyze the data.

A correction for the drawdown can be applied to the equilibrium analysis to account for the effect of partial penetration (Todd, 1959). This procedure is valid if the perforated section of the well penetrates from the top of the aquifer or if the well is cased near the top of the aquifer and open only at the base.

Let m = saturated thickness of the saturated zone of the aquifer

h_{2m} = height of water in an observation well above the base of the aquifer at radius 2m from the pumped well.

h_w = height of water in the pumped well above the base of the aquifer.

h_s = length of penetration of pumped well below the top of the saturated zone of the aquifer.

h_o = pretest height of groundwater above base of aquifer.

r = distance from pumped well to observation well.

r_o = distance from pumped well at which there is no drawdown.

r_w = radius of pumped well.

Then, the drawdown can be obtained as follows:

Artesian Aquifer: $$h_o - h_w = \frac{Q}{2\pi k}\left[\frac{1}{h_s}\ln\left(\frac{\pi h_s}{2r_w}\right) + \frac{0.1}{m} + \frac{1}{m}\ln\left(\frac{r_o}{2m}\right)\right]$$

or $$h_{2m} - h_w = \frac{Q}{4\pi k}\left[\frac{2}{h_s}\ln\left(\frac{\pi h_s}{2r_w}\right) + \frac{0.2}{m}\right]$$

Valid for $(1.3\ h_s) \leq m$ and $(h_s/2r_w) \geq 5$

Water Table Aquifer:
$$h_{2m} - h_w = \frac{Q}{4\pi k}\left[\frac{2}{h_s}\ln\left(\frac{\pi h_s}{2r_w}\right) + \frac{0.2}{m}\right]$$

iv) Boundary Effects of Analysis of Constant Discharge Tests: Hydrogeologic boundaries can distort the drawdown curves. A fault may act as a barrier to flow, or a stream bed may act as a source of recharge.

A barrier to flow alters the drawdown curve as if there were another well pumping on the opposite side of the barrier. A recharge source such as a stream alters the drawdown curve as though there were a well injecting water on the opposite side of the boundary. The imaginary well on the other side of the boundary is called an image well. The image well is located on a line perpendicular to the boundary at the same distance from the boundary as the actual well. Data handling procedures should be modified to account for boundaries. Three methods to do this are presented in the following pages.

a) Straight Line Method Adjustment for Boundaries:
If a barrier is present, the slope of the drawdown-log time curve plotted for the straight line method will steepen as the cone of depression reaches the barrier. Similarly, a recharge source will flatten the plot. The effects of a boundary on the straight line graph and solution are summarized in Table 8-2 (Johnson, 1972).

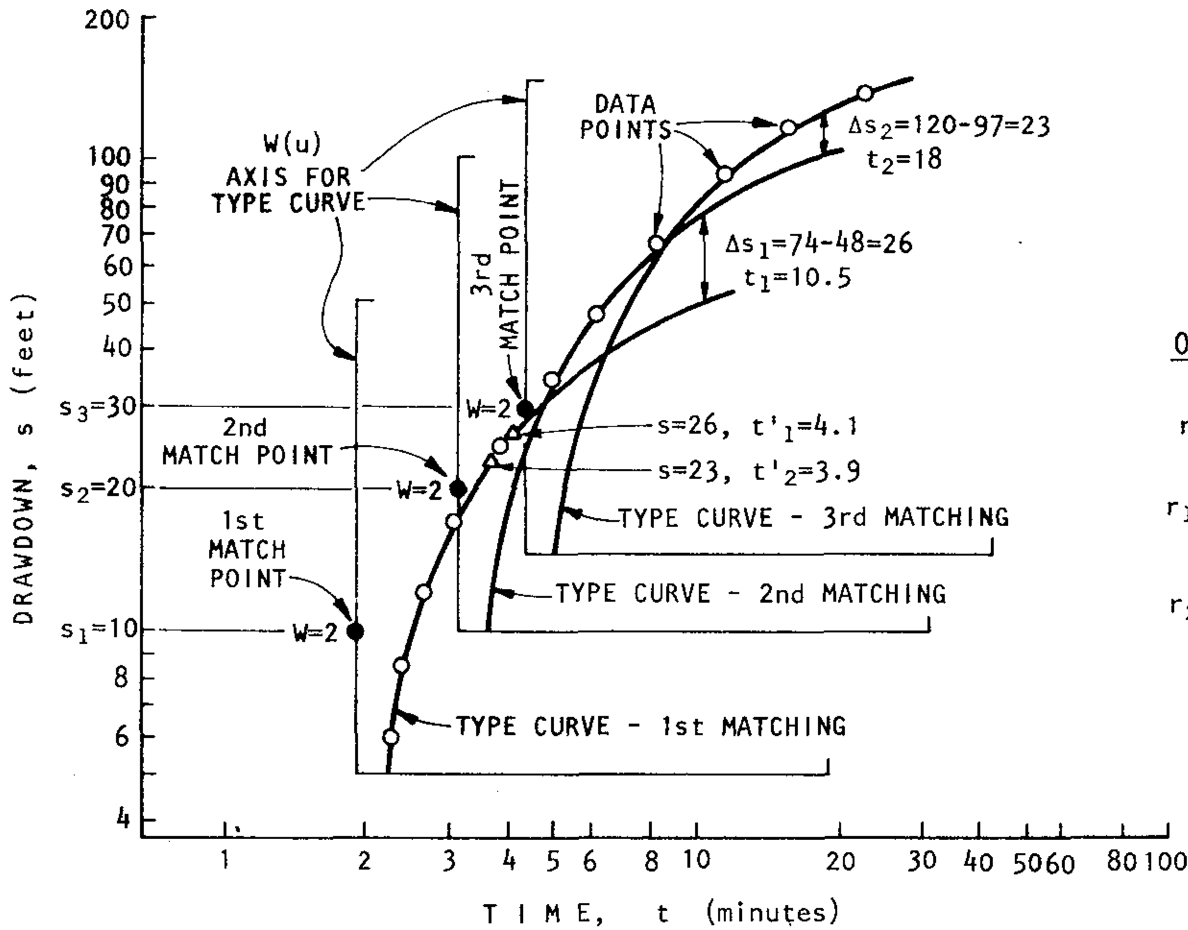
Note that a bend in the graph can also indicate a decrease in aquifer transmissivity with distance rather than a hydrogeologic barrier.

If the s-log t data from some or all of the observation wells do not plot as a straight line, a boundary may be present. If the change in the slope in the s-log t graph is readily detectable, use the data from the earlier portion of the test to calculate T, K and S, the transmissivity, permeability, and coefficient of storage, respectively. Note here, however, that the straight line method is often substantially in error for early test data (Walton, 1970).

Where a clear bend in the test graphs is not evident, plot s-log t for the observation wells on one sheet of graph paper and compute S (Johnson, 1972). Similarly, plot s-log r for a series of times and compute S. If the average value of S calculated from the s-log t graphs is substantially larger/smaller than the average value of S from the s-log r graphs, a barrier/recharge source is probably present and the true value of S lies somewhere in between. A reasonable assessment of T can be made from the s-log t test graphs.

b) "Type" Curve Method Adjustment for Boundaries:
Where barriers or recharge sources are known to exist, the distance to the boundary as well as the aquifer properties can be calculated.

To determine aquifer properties, match a type curve to the early log s-log t data and compute the aquifer properties as previously described. Use observation well data for the time period before the cone of depression encounters the boundary. A detailed example of the calculation of boundary distance is presented in Figures 8-23 and 8-24 (after Walton, 1970).



OBSERVATION WELL:

r = 100 feet

$$r_1 = 100\sqrt{\frac{10.5}{4.1}} = 160 \text{ feet}$$

$$r_2 = 100\sqrt{\frac{18}{3.9}} = 215 \text{ feet}$$

EXAMPLE SHOWN: 2 Boundaries Encountered

NOTE: See Figure 8-24 for Procedure

Figure 8-23 EXAMPLE OF BOUNDARY DISTANCE CALCULATION

Select the data from any observation well:

1. Find best type curve fitting early data.
2. Fit same type curve to later data as follows:
   a) pick <u>any</u> match point W_1, s_1 from the first curve matching;
   b) line up axes for second matching so that $s_2 = 2s_1$ is lined up with W_1;
   c) slide type curve horizontally until some portion of it fits the data. Trace it over the data.
3. Repeat Step 2 but line up W_1 with $s_3 = 3s_1$.
4. For the n^{th} boundary line up W_1 with $s_n = n \cdot s$.
5. At any arbitrary time t_1, find the deviation Δs_1 between type curves 1 and 2.
6. At any arbitrary time t_n, find the deviation Δs_n between type curves n and n + 1.
7. Find t_1' at which $s = \Delta s_1$ on the first type curve.
8. Find t_n' at which $s = \Delta s_n$ on the first type curve.
9. Find the radius from the observation well to the <u>first</u> image well $r_1 = r\sqrt{\frac{t_1}{t_1'}}$; for the n^{th} image well $r_n = r\sqrt{\frac{t_n}{t_n'}}$.

Repeat steps 1 - 9 for the data from each observation well

10. To find the exact location of the n^{th} image well, find r_n for each observation well. For each observation well, draw a circle around it using the radius r_n calculated for that well. The n^{th} image well is located at the intersection of the circles. Note that if the boundary is not perfectly vertical, the circle arcs will not intersect at one point, however the image well location can be approximately defined.
11. The boundary is located halfway between the image well and the pumped well.

Figure 8-24 PROCEDURE FOR BOUNDARY DISTANCE CALCULATION

TABLE 8-2

Comparisons of Recharge and Boundary Effects on Semilog Diagrams (straight-line solution method)

Recharge Effect During Pumping Test

Time-drawdown Graph	Distance-drawdown Graph
1. Slope of graph becomes flatter. If transmissivity is calculated on the basis of the flatter slope it will be higher than the true value.	1. Slope of straight line remains almost unchanged. Aquifer transmissivity calculated from the graph is usually close to its true value.
2. Extending straight line of flatter slope results in an erroneous value of t_0 making it too low. A calculation using this figure gives a value for the storage coefficient that is smaller than the correct one.	2. Straight line is displaced upward. Extension to zero drawdown gives a value of r_0 which makes calculated storage coefficient higher than the correct one.

Barrier Effect During Pumping Test

Time-drawdown Graph	Distance-drawdown Graph
1. Slope of graph becomes steeper. If transmissivity is calculated on the basis of the steeper slope it will be lower than the true value.	1. Slope of straight line remains almost unchanged. Aquifer transmissivity calculated from the graph is usually close to its true value.
2. Extending line of steeper slope results in erroneous value of t_0 which is too high. A calculation using this figure gives a storage coefficient that is larger than its correct value.	2. Straight line is displaced downward. Extension to zero drawdown gives erroneous value of r_0 which makes calculated value of storage coefficient smaller than the correct one.

c) Equilibrium Formula Adjustment for Boundaries:
Under equilibrium conditions the drawdown at any point is due to the sum of the drawdowns caused by all real or image wells in the vicinity. If the boundary location is known, the image well can be located on the opposite side of the boundary on a line with the pumped well. For a non-leaky artesian aquifer with a recharge boundary, Walton (1970) provides the following equilibrium formula:

$$T = \frac{528Q}{s_{obs}} \log \left({}^{r}i/_{r} \right)$$

where r_i = distance from observation well to image well (feet)

r = distance from observation well to pumped well (feet)

Q = discharge (gpm)

T = transmissivity (gpd/ft)

s_{obs} = observed drawdown at observation well (feet)

For compatible units, $T = \frac{1.15\ Q}{\pi s_{obs}} \log \left({}^{r}i/_{r} \right)$

8.3.3 Analysis of Constant Drawdown Tests

In some cases it is easier to maintain a constant drawdown rather than a constant discharge. This is true for tight (low permeability) formations. For example, if the flow is less than several gallons/minute, it is possible to maintain a constant drawdown ($\pm$ 1 foot) by operating the pump intermittently. It may be difficult to conduct an accurate constant flow test at such low discharge rates. In addition, if a constant flow test is attempted in a low permeability material, the danger exists that the drawdown will reach pump level before the desired end of the test.

Constant drawdown tests are also an appropriate method where observation wells may not be available. Constant drawdown tests, unlike constant discharge tests, can be analyzed for non-equilibrium conditions without observation well data.

The available analysis methods for constant drawdown tests are fewer and less versatile than for constant discharge tests.

8.3.3.1 Straight Line Method - The straight line method is not appropriate since a simple equation relating T, s, t and r over a log cycle of Q is not available.

8.3.3.2 Type Curve Method - Some type curves are available for idealized cases simulating a variety of aquifer conditions. They are expressed in terms of the well function $W(\lambda)$. Solutions are only available for artesian aquifers with fully penetrating wells (Walton, 1970).

The analysis procedure is similar to that for constant discharge conditions. Refer to Figure 8-25.

i) Characterize the aquifer if possible.

ii) Plot log Q (vertical axis) vs. log t so that the scale of one log cycle of data is the same as for one log cycle on the type curves.

iii) Select the most appropriate type curve and attempt to match the curve keeping axes parallel. Repeat with various type curves until a good match is obtained.

iv) Select any point on the data curve. Call this the match point. Read Q, t, $W(\lambda)$ and λ corresponding to that point.

v) Solve for S and T using the equations printed on the graphs in Figure 8-25.

8.3.3.3 Equilibrium Formula Method - Because few non-equilibrium solutions are available for the constant drawdown test, the best method of analysis is to use equilibrium data, if equilibrium conditions can be obtained. The equilibrium formulas for constant drawdown data are the same as for constant discharge data.

8.3.3.4. Analysis Modifications

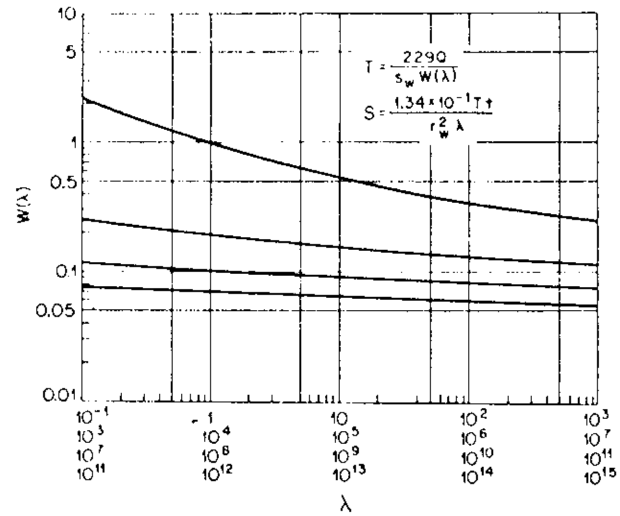
i) Water Table Aquifers: The artesian solutions can be applied to water table aquifers if the restrictions discussed in Section 8.3.2.4 (ii) are followed.

ii) Partial Penetration: The flow to a partially penetrating well is less than that to a fully penetrating well for reasons discussed in Section 8.3.2.4 (iii). The effect of partial penetration on equilibrium analysis is also presented in that section.

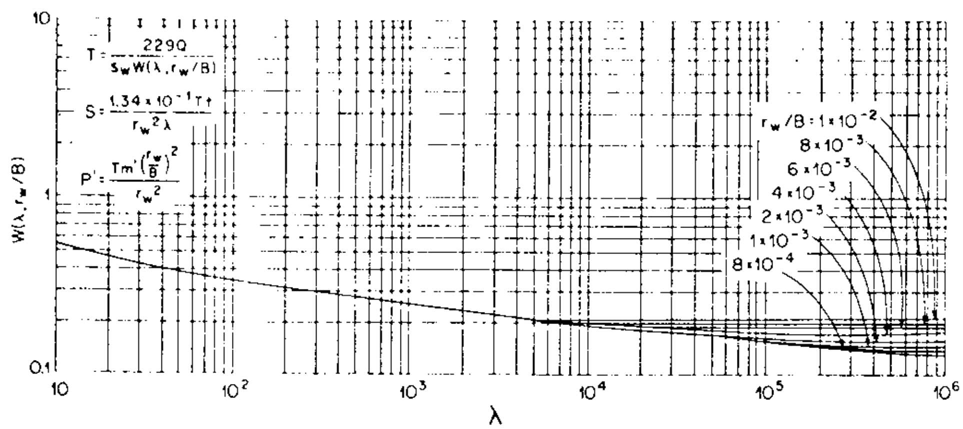
iii) Boundaries: A simple procedure for assessing the effects of boundaries is not generally available for constant-drawdown tests, except under equilibrium conditions where the comments for equilibrium constant discharge tests apply.

8.3.4 Anisotropic Aquifers

8.3.4.1 Background - In many instances, aquifers may be considered homogeneous, but not isotropic. Aquifers whose permeability varies in different directions are called anisotropic. Significant anisotropy may occur in wind-blown sands and beach deposits, and is often found



(a) NON-LEAKY



(b) LEAKY

Where:

T = transmissivity (gpd/ft)
S = coefficient of storage
r_w = radius of pumped well
Q = discharge (gpm)
r_w/B = 'leakage factor'; determine from curve matching procedure
t = time after pumping began (days)
s_w = drawdown at pumped well (feet)
P' = permeability of aquitard (gpd/ft^2)

For compatible units: $T = \frac{Q}{2\pi S_w W(\lambda)}$, $S = \frac{Tt}{r_w^2 \lambda}$

Figure 8-25 TYPE CURVES FOR NON-LEAKY AND LEAKY ARTESIAN AQUIFERS WITH:

- FULLY PENETRATING PUMPED WELLS
- NO WATER RELEASED FROM STORAGE IN AQUITARD
- CONSTANT DRAWDOWN

(Jacob & Lohman, 1952; Walton, 1970)

in stream deposits. The transmissivity (permeability x the aquifer thickness) in the major direction of anisotropy may be from two to ten times greater than that in the minor direction (Kruseman and DeRidder, 1970). This could have a significant effect on the flow pattern of injected leach fluids.

Two methods have been developed by Hantush (1966) and Papadopulos (1965) for the pumping test analysis of anisotropic aquifers. The Hantush method is preferable, although a sufficient number of observation wells are required to develop accurate drawdown contours. Several maps of drawdown contours at different times are desirable for the Hantush analysis.

8.3.4.2 Hantush Method for Anisotropic Aquifers - A non-leaky aquifer is assumed (confined or unconfined). Three radial lines of three observation wells each are required (nine wells total), and time-drawdown and time-recovery data should be collected at each of them.

Definitions:

t = time since pumping started (days)

t' = time since pumping stopped (days)

s = drawdown (feet)

T_x, T_y, T_r = transmissivity in directions x, y and r (gpd/ft)

r = distance from observation well to pumped well (feet)

Q = discharge (gpm)

S = coefficient of storage

i) Plot s (vertical axis) vs. log t for each observation well for the pumping data including the recovery period.

ii) Use any isotropic method to solve for T and S at each well.

iii) Find T_i, the average transmissivity along each ray i. Similarly, find S_i, the average coefficient of storage.

iv) Find $T_{ave} = \sum_{i=1}^{n} T_i/n$ Find $S_{ave} = \sum_{i=1}^{n} S_i/n$

v) Select a time t_f near the end of the pumping period. Plot contours of equal drawdown, using the s-log t curves to fill in the missing information. Draw a contour through each observation well. If the transmissivity is indeed anisotropic, the contours should be concentric ellipses, centered about the pumped well (see Figure 8-26).

vi) For each observation well, find the length of the major and minor axes of their respective ellipses, measured from the pumped well. Call these distances a and b.

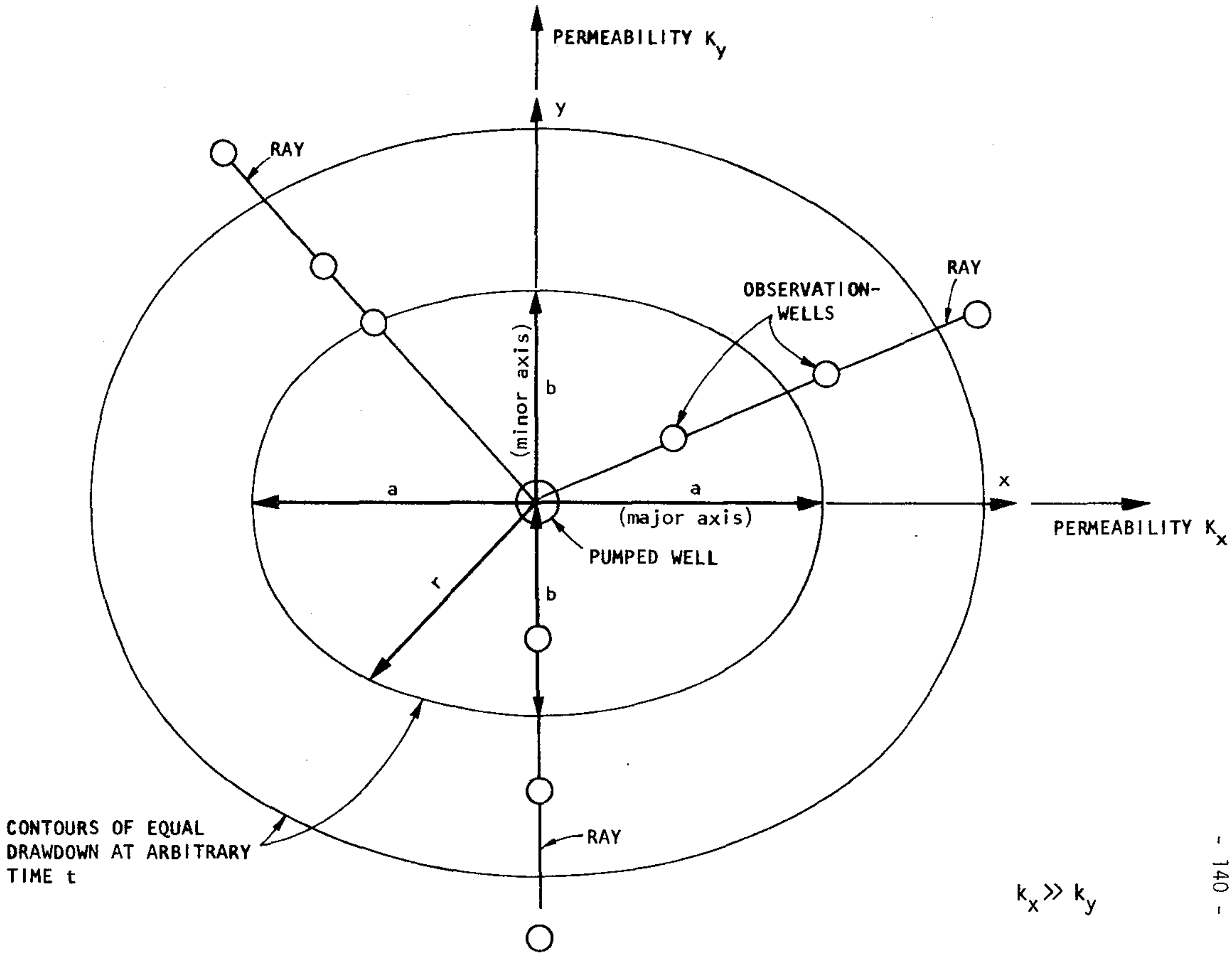


Figure 8-26 ELLIPTICAL DRAWDOWN CONTOURS FOR ANISOTROPIC AQUIFERS
(Hantush, 1966)

vii) Then solve for $T_r = (r^2/ab)\ T_{ave}$

$T_x = (a/b)\ T_{ave}$

$T_y = (b/a)\ T_{ave}$

$S = (4T_{ave}t_f/ab)\cdot u$

where $u = W^{-1}(4\pi T_{ave}s/Q)$

and W = well function for non-leaky isotropic aquifers shown in Figure 8-16.

Note: u can be obtained by entering the quantity $(4T_{ave}t_f/ab)$ on the vertical scale of the plot in Figure 8-16, and inverting the corresponding value on the horizontal scale.

A second calculation of the transmissivities can be obtained using the recovery data.

viii) Extrapolate the s vs. log t pumping data to t = infinity. Set the drawdown at $t_{infinity} = s_o$.

ix) For the recovery data, plot $s_o - s$ vs. log (t'), where s is now the residual drawdown, and t' is the time since shutdown. Solve for T and S by Jacob's method:

$$T = \frac{264Q}{\Delta(s_o - s)}$$

where $\Delta(s_o - s)$ is determined over one log cycle of (t'). Note that data obtained soon after pumping is ceased should be used to calculate $\Delta(s_o - s)$.

x) Perform Steps (iv) through (vi) above; however, in Step (v), instead of t_f, select a time t' during the recovery period so that

$$\frac{Sr^2}{4Tt'} > 0.01$$ for all observation wells.

xi) Solve for $T_r = (r^2/ab)\ T_{ave}$

$T_x = (a/b)\ T_{ave}$

$T_y = (b/a)\ T_{ave}$

xii) The major and minor principal permeabilities can be obtained by dividing T_x and T_y by the aquifer thickness, respectively.

9.0 SUPPLEMENTARY DISCUSSIONS FOR FRACTURED ROCK

9.1 INTRODUCTION

When attempting to assess the in-situ permeability of a rock mass, the existence of joints and fractures is an important aspect and must be considered carefully. It has been demonstrated frequently in the literature that in a fractured medium the overall permeability is dominated by the conducting joints and fractures. The matrix permeability (the permeability of the intact rock blocks) will be orders of magnitude less than that of the fissures, and can be considered effectively zero (Maini, 1971; Maini et al., 1972; Banks, 1972; Milligan, 1975). Because of the dominating effect discontinuities have on the in-situ permeability of a rock mass, and the frequency with which they occur in the field, this section of the report considers the problem of fractures exclusively.

9.2 SURFACE MAPPING OF JOINTS

As with all in-situ permeability investigations, a detailed geological investigation should precede actual permeability testing. If the rock mass in question is exposed in outcrops, a surface joint survey should be conducted to assess general orientation and spacing of any joint systems present. Surface evidence of joints and fractures will be more abundant in areas of high relief. When considering leaching operations in an abandoned mine, additional surfaces will be exposed, yielding additional information. Generally prevailing joint orientations can be determined using techniques of pole density contours on stereographic nets as discussed by Goodman (1975) and Hoek and Bray (1974). If the rock mass is located beneath soil cover or other rock strata, a surface investigation may not be possible. Should this be the case, a detailed borehole investigation should be conducted.

9.3 BOREHOLE INVESTIGATIONS

Whether or not a surface mapping investigation has been conducted, a detailed study of joint orientation at depth is recommended. Conditions at the surface may be quite different from those at depth, due to weathering of exposed surfaces, shallow stress relief, and mining operations such as blasting and excavation. Joint distribution in the immediate vicinity of the leachable deposit is a controlling factor in the effectiveness of the overall leaching operation. The fractures will serve as the effective paths of solution ingress into and egress from the rock mass. Also, their spacing and orientation may affect the degree of channelization which occurs. Knowledge of existing fractures and their locations may also prove valuable in the design of a successful injection-recovery well pattern.

9.3.1 Core Samples

Core samples of the proposed leach zone provide a method of assessing general rock quality and degree of fracturing in the rock. Methods for obtaining core samples are discussed in Section 6.3. Simple logging of the core can provide a rough approximation of fracture spacing, an important parameter in determining the permeability of a fractured mass. One method of representing the degree of fracturing is the RQD (Rock Quality Designation) value. RQD refers to the percent modified core recovery; i.e., the cumulative length of core pieces equal to or greater than 4 inches in length divided by the length of the core run (Deere et al., 1967).

However, even with the most carefully conducted coring procedures, a certain degree of induced fracture of the rock will occur. Drilling-induced fracturing will cause RQD values to be correspondingly lower, and initial approximations of the permeability based on core analysis will be too high. An additional problem is expressed by Maini (1971): "When a disintegrated shaley mass is withdrawn from the core barrel, it is important to decide whether this represents a highly porous void or an impermeable plug which has been shattered by drilling."

A more accurate means of identifying in-situ fracture conditions is provided by down-the-hole logging techniques, a discussion of which follows.

9.3.2 Down-The-Hole Logging

Down-the-hole logging techniques are methods by which an oriented, accurate representation of the borehole wall can be obtained. Recent in-situ leaching research substantiates the need for such methods (D'Andrea et al., 1974). Several techniques have been described in the literature. A summary of these methods is presented.

9.3.2.1 Acoustical Methods - King and McConnell (1973) describe an acoustic-velocity measuring sonde which can be used in water-filled or dry AX-size (1-7/8-inch diameter) boreholes to a maximum depth of 200 feet. The reflection of acoustic waves against the borehole wall detects inhomogeneities such as joints and fractures, and changes in lithology. Hamilton and Myung (no date) discuss another acoustical logging device called the Seisviewer System, which is manufactured by the Birdwell Division of the Seismograph Service Corporation. This device logs a continuous acoustic picture of a borehole wall, with a north-designated orientation. Physical variations in the wall are displayed as changes in the picture intensity. Another variation of an acoustic device is the "3-D" velocity log, also developed by the Birdwell Division of SSC. This method utilizes transmitting and receiving transducers placed at known distances down the borehole. A pulse is generated and arrival times of compressional and shear waves are monitored by the receiver. Output can be evaluated for fracture locations (Myung et al., 1972).

9.3.2.2 Optical Methods - Optical techniques can be used for direct observation of borehole walls. Krebs (1967) discusses the application of a borehole periscope. With a diameter of 2-1/4 inches, the unit can be used to a depth of 110 feet. It allows immediate investigation, produces an image in color, and permits color photographs to be taken from the surface. It can be used beneath the water table, however, muddy water will reduce image clarity. Simple techniques can orient fracture surfaces as to their strike and dip.

Krebs (1969) describes a borehole television camera, which can make direct observations of geologic structure in boreholes to depths of 1600 feet. The unit, 2-1/2 inches in diameter, consists of a miniature television camera in a probe. The picture is transmitted through a cable to a television screen at the surface. A compass pendulum within the probe enables simultaneous readings of the inclination of the borehole and the azimuth of the inclination. Various video recorder-playback interfaces can be accommodated to obtain permanent records of the visual logs.

The third type of direct borehole viewer is the film camera. The film-type borehole camera obtains an image on one of many types of photographic film, by direct exposure through a lens, by indirect exposure through a system of prisms, or by photographing an image reflected on a flat or conical mirror. A borehole film camera can fit in an NX-sized borehole. At large depths, a strobe-light attachment provides the required light source. Muddy water renders the camera essentially useless (Lundgren et al., 1968).

All the borehole techniques can provide important information regarding fracture spacing, orientation, and distribution. An analysis of the core samples can augment the investigation with regard to degree of fracture weathering, gouge and joint filling material, etc. Information from a borehole investigation can be combined with available surface observations to further identify joint spacing and orientation. Biasing may result from surface and/or borehole observations, as discussed by Terzaghi (1965).

9.3.2.3 Borehole Disturbance Due to Drilling - If borehole logging is incorporated in the preliminary investigation, a drilling method should be used which minimizes the disturbance of the borehole walls. The borehole should not be advanced using the percussion drill method. Gale (1975) used a borehole periscope in both percussion drilled wells and those advanced with a diamond drill. The former method left a relatively rough borehole wall and thus only the major fractures could be detected, while the smooth walls of the diamond drill borehole provided an excellent background on which the faintest fracture trace could be detected. Where core samples are taken, the diamond drill provides a good quality wall for down-the-hole inspection. If the borehole is advanced without core recovery, the full faced diamond bit rotary method is recommended.

9.4 FACTORS WHICH AFFECT TEST RESULTS

It has been expressed in the preceding subsections that the permeability of a fractured rock mass is governed by the fractures themselves. In general, two fundamental aspects of fractures will affect the in-situ permeability: fracture aperture (degree of opening) and spacing between fractures. Average values of these parameters can be effectively measured from down-the-hole investigations described previously. Given these averaged quantities, a rough approximation of the permeability can be determined. Should selected depths or sections be individually analyzed (i.e., intensely fractured horizons within a relatively less-fractured mass), variations in permeability can be approximated. Figure 9.1 shows the general influence that fracture aperture and spacing have on the permeability. It is emphasized that the plot is based on several simplifying assumptions. Values of permeability obtained are rough approximations and should be treated accordingly. The graph should never be used as a substitute for a carefully conducted packer test program.

Recent studies have shown that the observed permeability of a fractured rock mass is influenced by the method and procedure of the permeability test. Some aspects have been mentioned in previous sections of the report and are repeated here.

The analysis procedure for the packer test, presented in Section 8.2, incorporates a plot of flow rate vs. excess pressure in the test cavity. If the relationship is linear, the determination of the permeability is straight forward. Nonlinear plots make the analysis more difficult. Principal causes of nonlinearity are (Maini et al., 1972):

i) Deformation of the fractures due to excess injection pressure. The fluid-conducting fissures are not rigid; they deform under hydrodynamic pressure. The fissure opening reflects an equilibrium state influenced by gravity and the fluid pressure (Noorishad et al., 1971). Studies show that the opening of a fissure is minimized if only one fracture system is operant. More favorable kinematic conditions for movement exist if another fracture set nonparallel to the first one is present. Such deformation is often referred to as hydraulic fracturing of the rock mass.

ii) Turbulence due to high injection pressure resulting in excessive velocities. One assumption in the test analysis is that all fluid flow is laminar. The general permeability equation used in the test analysis states that the permeability is proportional to the ratio of Q (flow rate) over H_c (constant excess head) (see Section 8.2). Turbulence effectively reduces this ratio, thereby reducing the observed permeability. The potential for both turbulent flow and hydraulic deformation of the fractures can be minimized by establishing a safe upper bound for the allowable excess (above hydrostatic) cavity pressure. This bound has been specified (Section 7.4) as 0.5 psi/ft depth to the test section.

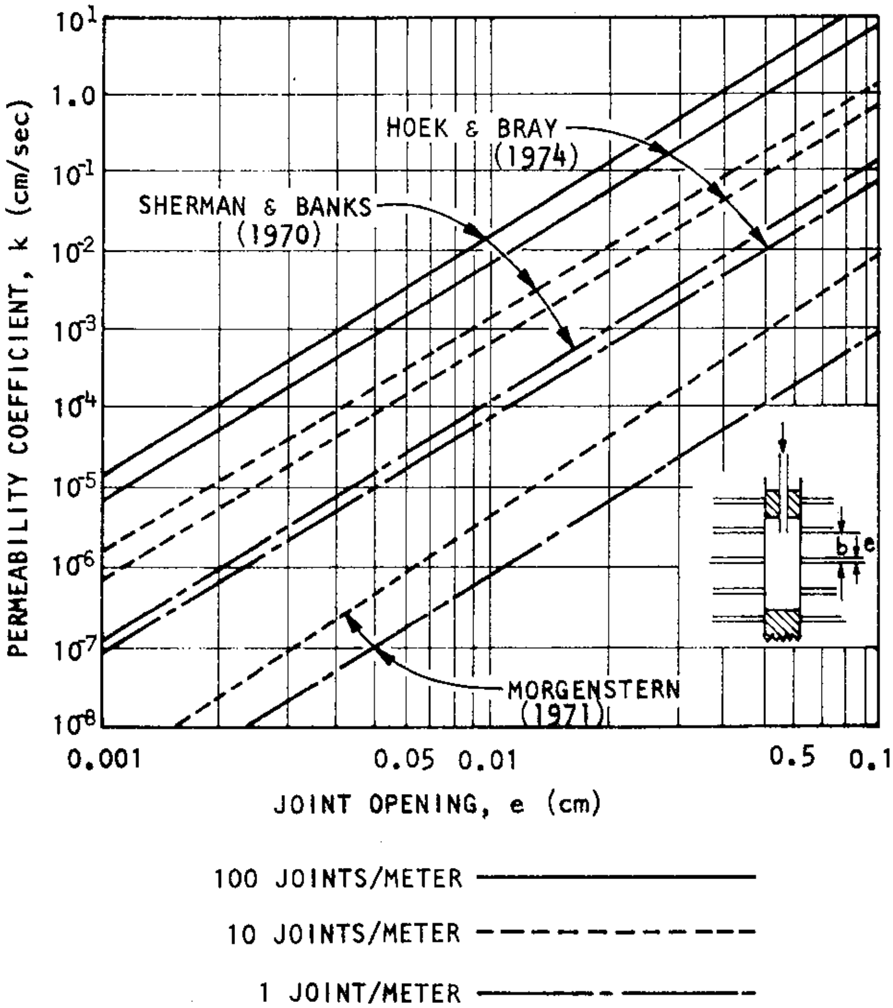


Figure 9-1 INFLUENCE OF JOINT OPENING e AND JOINT SPACING b ON THE PERMEABILITY COEFFICIENT k IN THE DIRECTION OF A SET OF SMOOTH PARALLEL JOINTS IN A ROCK MASS

iii) Leakage past packers. Packer leakage will result in erroneously high flow rates and correspondingly high permeability estimates. A procedure has been developed and presented in Section 7.5, whereby such leakage can be detected and minimized.

The general concept of the mass permeability of a jointed rock mass has been studied. General consensus is that scale has an important effect on applicability of a mass permeability. Fractured rock beneath a dam may be justifiably referenced in terms of its "mass permeability," however, the term may be unacceptable in describing a 50-foot thick ore body with a fracture spacing of 10 feet. In the latter case it may be more appropriate to discuss the characteristics of the individual fractures rather than the "mass permeability."

An important requirement for identifying fracture "permeability" is to test those fractures exclusively. The most effective means of testing a system of parallel fractures is by intersecting them perpendicularly with the borehole (Maini, 1971; Maini et al, 1972; D'Biago et al., 1972; Milligan, 1975; Louis et al., 1970). Unfortunately, the difficulty and expense of preparing the hole and performing the test increase substantially as the borehole varies from the vertical position. However, the consequences of testing with vertically oriented boreholes regardless of the fracture system orientation should be recognized:

i) One or several fracture systems may be intersected obliquely. This improves the kinematic conditions for crack deformation as discussed above.

ii) Rather than representing a single fracture system, the observed permeability may reflect the average of two or more obliquely intersected systems. This effect is substantiated under field test conditions (Maini, 1971).

The permeability of a homogeneous isotropic medium is considered to be a constant value, assuming no physical changes (e.g., in void ratio, porisity, fabric, etc.) occur. The same is true for a fracture. However, it has been established using laboratory analogies (Wilson and Witherspoon, 1970; Louis, 1969; Snow, 1965), and demonstrated under field test conditions (Iwai, 1976) that the permeability of a fracture varies as the aperture squared, and the flow rate varies as the aperture cubed. Thus, a relatively small change in aperture can have a significant effect on the fracture's ability to transmit a fluid.

Another important effect of fluid flow through fractures is the possibility for permanent deformation of the fractures after they have been opened or closed. This effect bears on both pump-out and pump-in tests. Iwai (1975) and Gale (1975) have shown that a degree of permanent, irrecoverable deformation occurs whether a fracture is subjected to a cyclic increase or decrease in fluid pressure. This is particularly evident in pump-out tests below the water table, where a decrease in fluid pressure increases the effective stress across the fracture contacts. Finite closure results due to inelastic deformation of the asperities and filling materials in the fracture. This effect may be most important when the fractures have been weathered and asperities are relatively weak.

10.0 PERMEABILITY FIELD TEST METHOD CAPABILITY MATRIX (TMCM)

This section of the report presents and discusses the matrix of field test methods and evaluation criteria (see Figure 10-1). The matrix is designed so that the user may select an appropriate test method on the basis of his individual needs and restrictions. A discussion of this design "philosophy" is presented.

10.1 TEST METHODS

The matrix evaluates the major acceptable test methods which have been discussed in Sections 7 and 8. They are as follows:

- Falling Head Test
- Rising Head Test
- Constant Head Test
- Packer Test with Calibration
- Packer Test with Pore Pressure Transducer
- Well Pump Test, Equilibrium Analysis
- Well Pump Test, Non-equilibrium Analysis

The matrix has been divided into two sections, corresponding to tests performed above and below the water table. The test methods are evaluated for each criterion on a relative numerical scale between 1 and 4. The meaning of these values for each criterion is described in the following section.

10.2 EVALUATION CRITERIA

Each criterion used as a basis for test method evaluation is discussed below, including the significance of the rating scale.

Hole Preparation Cost - This criterion pertains to the general cost of preparing the borehole prior to the test. Ratings are based on the preparation of a single borehole except for the well pump tests, where several boreholes are required.

Rating: 4 = least expensive
1 = most expensive

Equipment Cost (Procurement Cost) - This criterion reflects the purchase or rental costs for equipment required to perform the test. It is assumed that relative cost ratings will remain the same whether the equipment is purchased or rented, i.e., rental rates are proportional to the purchase price. The criterion does not include costs of hole preparation as this comprises a separate evaluation criterion.

Rating: 4 = least expensive
1 = most expensive

Performance Cost - This criterion reflects the cost of performing the field test, including required equipment and manpower considerations.

Rating: 4 = least costly
1 = most costly

Operation Time - This criterion considers the amount of time needed to perform a test. For the packer test, it is assumed that the "continuous test" procedure is used (see Sections 7.4 and 7.5). Two ratings are given for the variable and constant head tests, reflecting the time needed to complete the single test and stage test procedures.

Rating: 4 = least time requirement
1 = greatest time requirement

Operation Ease - This criterion reflects the relative ease with which a test may be conducted. It considers the complexity of the equipment and the required competence level of the operator(s). It does not include the analysis or interpretation of the data obtained from the test, as this comprises a separate evaluation criterion.

Rating: 4 = most easy to perform
1 = most difficult to perform

Ease of Analysis - This criterion considers the relative ease with which test data can be analyzed to determine a permeability value. Generally the analyses are not complicated. Although well pumping test interpretations are the most difficult to perform, an experienced interpreter can obtain additional valuable information about the aquifer from a properly conducted test.

Rating: 4 = relatively easy to analyze
1 = relatively difficult to analyze

Test Accuracy with Depth - This criterion demonstrates the relative accuracy of each test method as a function of the depth of the test section. The rank can be interpreted as the "reliability" of a permeability value obtained with the test method. The category is divided into two classifications: shallow test sections (depths of 150 feet or less) and deep test sections (depths greater than 150 feet). The maximum accuracy of any test method is about 1/2 of an order of magnitude. The significance of a local, 'actual' permeability may be limited, since the permeability of many geologic formations can be expected to vary naturally throughout their mass and such variation may exceed the estimated test accuracy.

Rating: 4 = most accurate
1 = least accurate

Test Accuracy with Permeability Range - This criterion evaluates the relative accuracy of the test method in high and low permeability deposits. For this matrix, high permeabilities are defined as greater than or equal to 10^{-4} cm/sec, low permeabilities as less than 10^{-4} cm/sec.

Rating: 4 = most accurate
1 = least accurate

Geologic Sensitivity - This criterion reflects the ability of the test method to accurately assess the permeability, based on the geologic environment. The category is divided into three classifications as follows:

a. Homogeneous - The mass is free from unconformities and discontinuities, either lithologic or structural, which is conducive to a consistent permeability value throughout. An example would be a clean, uniform sandstone deposit free of interbeds, clay seams, etc.

b. Horizontally Stratified - The mass contains various horizontal discontinuities, mostly lithologic, which result in a vertical variation in permeability. An example would be an alluvial deposit, with frequently occurring interbeds of silt seams, siltstones, shales, etc.

c. Complex - The mass contains a complex system of discontinuities, including joints, fractures, fault zones, weathered zones, lithologic variations, etc. Examples would be fractured or folded sedimentary sequences and tectonic regions with associated joints, faults, and intrusives.

Rating: 4 = most accurate
1 = least accurate

Permeability Discrimination - This criterion evaluates the regional extent over which the test method assesses permeability, in either the vertical or lateral (areal) directions. Several methods contain two rankings for their ability to discriminate vertically. The first number refers to a single test, while the second number refers to the multiple stage test procedure. Note that the single test/stage test rankings have corresponding values in the categories "Operation Time," "Operation Ease," and "Geologic Sensitivity: Stratified." Assumptions are made that well pump tests are conducted with a minimum of three observation wells, and that variable head, constant head, and packer tests are performed in one borehole. The areal representation of data with the latter group of tests is limited but can be improved with a series of tests in a number of boreholes throughout the site.

Rating: 4 = Maximum ability to assess permeability in a vertical/lateral direction
1 = Minimum ability to assess permeability in a vertical/lateral direction

10.3 OVERALL RATING

An overall rating of the different test methods has not been performed. Such an evaluation would be misleading to the investigator, for it would weight each of the criteria in the matrix equally. Rather, the intent is for the investigator to enter the matrix, select which criteria are relevant on an individual priority basis, and select an appropriate method based on those criteria. For example, one investigator may desire a method which will evaluate a shallow deposit above the water table in a short amount of time, and where the relative test accuracy is of less importance. A second investigator may have a large budget, a deep zone of interest below the water table, and a high accuracy requirement. The priority lists for the two individuals are different. They may concern themselves with all or only a few of the criteria presented in the matrix. Based on their respective criteria priorities, their test method selections will probably be different.

After determining which criteria are most important for his application, the investigator may require a means of combining the individual criterion ratings into one value for overall comparison purposes. The method recommended is to multiply the rating for any selected matrix criteria by a weighting factor, and then sum the adjusted ratings to obtain a composite rating for a particular test method.

KEY:

4/1 — 4: Refers to single test; 1: Refers to stage test

NOTE:

4 is most favorable

1 is least favorable

TEST METHOD		EVALUATION CRITERIA: HOLE PREPARATION COST	EQUIPMENT COST	PERFORMANCE COST	OPERATION TIME	OPERATION EASE	EASE OF ANALYSIS	ACCURACY WITH DEPTH: SHALLOW (<150 ft)	ACCURACY WITH DEPTH: DEEP (>150 ft)	ACCURACY WITH PERMEABILITY RANGE: HIGH ($k \geq 10^{4} \frac{cm}{sec}$)	ACCURACY WITH PERMEABILITY RANGE: LOW ($k < 10^{4} \frac{cm}{sec}$)	GEOLOGIC SENSITIVITY: HOMOGENEOUS	GEOLOGIC SENSITIVITY: STRATIFIED	GEOLOGIC SENSITIVITY: COMPLEX	PERMEABILITY DISCRIMINATION: VERTICAL	PERMEABILITY DISCRIMINATION: LATERAL (AREAL)
ABOVE THE WATER TABLE	FALLING HEAD TEST	3	4	4	4/2	4/2	4	1	1	3	1	2	1/2	1	1/4	1
	CONSTANT HEAD TEST	4	3	4	3/2	4/2	4	2	2	3	2	3	1/2	1	1/4	1
	PACKER TEST WITH CALIBRATION	4	2	2	2	1	4	3	1	4	4	4	4	4	4	1
	PACKER TEST WITH PRESSURE TRANSDUCER	4	2	2	2	1	4	4	4	4	4	4	4	4	4	1
BELOW THE WATER TABLE	FALLING HEAD TEST	3	4	4	4/2	4/2	4	1	1	3	1	2	1/2	1	1/4	1
	RISING HEAD TEST	3	4	4	4/2	4/2	4	1	1	3	1	2	1/2	1	1/4	1
	CONSTANT HEAD TEST	4	3	4	3/2	3/2	4	2	2	3	2	3	1/2	1	1/4	1
	PACKER TEST WITH CALIBRATION	4	2	2	2	1	4	3	1	4	4	4	4	4	4	1
	PACKER TEST WITH PRESSURE TRANSDUCER	4	2	2	2	1	4	4	4	4	4	4	4	4	4	1
	WELL PUMP TEST, EQUILIBRIUM ANALYSIS	1	1	1	1	2	3	4	4	4	2	4	3	3	1	4
	WELL PUMP TEST, NON-EQUILIBRIUM ANALYSIS	1	1	1	1	2	1	3	3	4	3	4	3	3	1	4

Figure 10-1 FIELD TEST METHOD CAPABILITY MATRIX

11.0 SUMMARY AND RECOMMENDATIONS

The requirements for field permeability testing systems to meet the needs for evaluating geologic formations for in-situ leaching were investigated. The investigation included geologic studies of the currently important leaching applications comprising copper and uranium deposits, and a general review of the techniques and problems of in-situ leaching of these minerals as presented in the literature and available job files. These studies comprised the background data for evaluating current field permeability measurement techniques.

The literature was searched for all field permeability measurement techniques that might have any capabilities for determining the potential leachability of a formation. The methods of borehole preparation, test performance, and data analysis were studied for each method. Specifications for field permeability testing using the most promising of the available techniques were developed, giving careful attention to the need for any provisions that would overcome or avoid inaccuracies or defects noted in the reported use of the field test methods. The currently practiced field test methods referenced in this study were summarized in a chart showing geologic formations and the measured permeability range obtained in the reported use of the test method. The capabilities of available field test methods performed in accordance with specifications of this study, as well as the capability of one developing method, were comparatively rated against key performance criteria in a Field Test Method Capability Matrix.

A problem may exist when attempting to define the permeability of a fractured rock mass. Fractures over the rock matrix dominate fluid flow through the medium. Also, Darcy's Law, a well documented and accepted relationship between permeability and flow through a homogeneous porous medium, may not adequately define flow through fractures. Recent attempts to quantify fracture flow relationships were reviewed and significant results were presented. The importance of identifying existing joint systems in a fractured rock mass was emphasized in the report, and down-the-hole investigative techniques for identifying fracture orientations, spacings and aperture openings were discussed.

Four test methods are considered suitable for field permeability tests above the water table, as follows.

Falling Head Test
Constant Head Test
Packer Test with Calibration
Packer Test with Pressure Transducer

In addition to the test methods listed above, the following are considered to be suitable test methods for investigations conducted below the water table.

Rising Head Test
Well Pump Test using Equilibrium Analysis
Well Pump Test using Non-Equilibrium Analysis

Above the water table, the packer test is generally considered the most favorable test available. It evaluates the permeability of discrete, isolated lengths along the borehole and can provide a permeability profile along the borehole relatively easily. The falling head test, although less accurate, is generally considered the most easily and quickly performed test available above or below the water table.

Below the water table, the well pump test is generally considered the most accurate test method available. The method can assess variations in permeability over a large area. Time and financial considerations are important, as the well pump test requires multiple boreholes, increased manpower, and pumping durations as long as a week or more.

Packer tests are generally considered the most versatile method available in that vertical permeability variations may be established in individual boreholes, and tests in multiple boreholes may detect lateral variations. Thus, a three dimensional picture of the permeability can be obtained. The use of a pressure transducer in the packer test section is recommended to improve the accuracy of the packer test by giving accurate pressure measurements at great depths below the ground surface.

It is concluded from the Field Test Capability Matrix that no single permeability test will be most suitable in all instances. The proper selection is based on the geologic environment of the ore deposit, the depth to the test zone, financial considerations, desired accuracy, time required to conduct the test, ease of analysis, and other criteria presented in the capability matrix. The matrix is designed and presented so that the investigator can select the most suitable test based on a personal priority scale.

Recommendations for future study are:

(1) A suitable analysis method for constant drawdown well pump tests should be developed. Such tests are suitable for low-permeable regions, where the performance of constant discharge pump tests is difficult.

(2) The "sink-source" test should be further developed. In this test three packers define two test sections, one of which provides a sink by having water pumped out. This technique could be used to assess the vertical permeability in the immediate vicinity of the borehole, and to detect the presence of horizontal impermeable strata through a profiling

procedure (see Appendix D). The capability might also be significant in evaluating localized permeability of a rubbilized zone at depth, independent of any effects from the in-situ permeability of the surrounding undisturbed formation.

(3) Evaluation techniques for a fractured zone should be further developed. Applied test methods and analyses need to be improved for the evaluation of fractured media, where the fractures rather than the rock matrix govern fluid flow through the mass. Practical difficulties remain in estimating the three-dimensional flow pattern through a mass laced with multiply-intersecting joint systems.

(4) Further development of the 'Packer Test with Pressure Transducer' method is needed to optimize the design and selection of packer materials and components in order to facilitate testing under greater pressures than is presently possible.

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APPENDIX A

BASIC TIME LAG
(Hvorslev, 1951)

The basic time lag, T, is a characteristic property of the test well and its environment, and is independent of the test performed or the excess/deficit head applied. T is defined as the time it would take the excess/deficit head H_o to dissipate if the initial rate of flow $q_o = K \cdot H_o \cdot F$ (Darcy's Law) were maintained.

q_o = initial rate of flow under head H_o (length^3/unit time)

K = soil/rock permeability (length/unit time)

H_o = initial excess/deficit head (length)

F = shape factor dependent on geometry of test zone and standpipe (length)

From the definition given,

$T = V_o/q_o$ where V_o = volume of standpipe to be filled or emptied in order to reach equilibrium or $V_o = A_s \cdot H_o$ where A_s = cross-sectional area of standpipe

$$T = V_o/q_o = (A_s \cdot H_o)/(K \cdot H_o \cdot F) = A_s/KF$$

Therefore T depends only on the geometry of the system and the soil/rock permeability. T is independent of the value of H_o used in the test.

During time dt the volume of flow under the excess/deficit head is $q \cdot dt = -A_s \cdot dH$. From Darcy's Law, we know that q = K H F if H is the head at time t. So $-\frac{dH}{H} = \frac{FK}{A_s} \cdot dt = \frac{dt}{T}$

$$-\ln H = t/T + \text{CONSTANT}$$

boundary cond.: $t = 0$ at $H = H_o$

then $\ln(H_o/H) = t/T$

So for $\ln(H_o/H) = 1$ or $H/H_o = 0.37$, $t = T$

This is the basis for the lower left graph on Figure 8-1.

Once T is known, then

K = A/FT where A and F are functions of the geometry of the system.

APPENDIX B

CONSTANT HEAD AND CONSTANT FLOW TEST EQUIPMENT

The following comments on selection of equipment for constant head and constant flow tests are edited segments from the USBR publication G-97.

B.1 RECOMMENDED PIPE USAGE

In making constant head and constant flow tests, insertion of a 1-1/4-inch pipe (PVC) in the hole is helpful. In an uncased test section in friable materials liable to wash, the end of the pipe should rest on a 4- to 6-inch cushion of coarse gravel at the bottom of the hole. In more stable material the pipe may be suspended above the bottom of the hole, but the bottom of the pipe should be at least 2 feet below the top of the water surface to be maintained in the hole.

Water may be introduced through a 1-1/4-inch pipe and measurements of water level made in the annular space between the pipe and the casing; where tests are made in uncased sections in materials liable to wash, this is the preferable method. The water may also be poured into the annular space between the pipe and the casing and measurements of water level made through the pipe. The latter method, using a 45° "Y" branch fitting at the top of the casing, is generally the most convenient when test sections are supported by perforated casing.

B.2 RECOMMENDED TYPES OF WATER METERS

Required water deliveries in constant head and constant flow tests may vary, necessitating more than one water meter, as no one meter is sufficiently accurate at all flow rates. Therefore, two meters are recommended for each rig: a 3-inch turbo-type meter to measure flows above 50 gpm and a 1-inch disk-type meter to measure flows between 1 and 50 gpm. It is recommended that, when possible, water meters be calibrated at least once a month.

Adapters should be available for each meter. The adapters should be at least 10 times as long as the diameter of the rated size of the meter. This length of adapter permits the water flow to become steady and eliminates the turbulence due to a change in pipe diameter. The accuracy of most meters is influenced adversely by turbulent flow. An adapter should be used on the upstream side of each meter where the water line from the pump to the meter has a different diameter than the nominal size of the meter.

In the interest of pumping efficiency, etc., sharp bends in hoses, 90 degree fittings on pipes, and unnecessary changes in pipe and hose diameters should be avoided.

All joints, connections and hose between the water meter and the casing should be tight in order that no water loss occurs between the meter and the test section.

On some constant head and constant flow tests in which the test sections take a very small amount of water a control arrangement has proved effective. A plug valve is used in the water line from the meter to the casing; a bypass line around the plug valve contains a 1/2-inch needle valve. The entire set-up is connected directly to the outlet valve of a constant head tank. A 1-inch water meter will not measure accurately the low flows used under such conditions and, after stabilization is obtained, actual flow is determined by the time required to fill a container of known volume.

B.3 PUMPING EQUIPMENT

Many tests are run using the circulation pump on the drill for pumping the water. Such pumps are generally the multiple-cylinder type with a uniform fluctuation in pressure. They have a maximum capacity of about 25 gpm and if not in good condition may have capacities as small as 17 to 18 gpm. Many tests are failures because such pumps do not have sufficient capacity to develop back pressure in the length of hole being tested. When this happens, the tests are generally reported "took capacity of pump, no pressure developed." This result does not permit determination of permeability of the material tested other than it is probably high. In addition, the fluctuating pressures of multiple-cylinder pumps, even when an air chamber is used, are often difficult to read accurately as the high and low readings must be averaged to determine the approximate true effective pressure. This difficulty may be a source of error in some cases, and should be avoided. Rather, centrifugal pumps should be used. Centrifugal pumps can be obtained to meet a variety of testing conditions. A typical pump should have a 350 gpm capacity against a total dynamic head of approximately 275 feet. An example of such a unit is the Jacuzzi Centrifugal Pump Model ATEM4, whose performance curves are given in Figure B-1.

B.4 WATER SUPPLY

The quality of water used in all permeability tests is of primary importance. Water should be clear and silt free. A recommended practice is to pump from the source of supply into a settling and storage tank. A 3,000-gallon plastic tank is ideal for this purpose; but if one is not available, a pit can be dug. The end of the suction line from the centrifugal test pump should be supported several inches above the bottom of the tank or pit to avoid pumping the settled silt and clay. Water for testing should never be stored in or pumped from the slush or mud pit.

The use of water a few degrees warmer than the ground temperature of the test section is a desirable practice in order to avoid plugging of the rock pores by air bubbles. Such water is not obtainable in many places, particularly in the winter, but if procurable, it should be used.

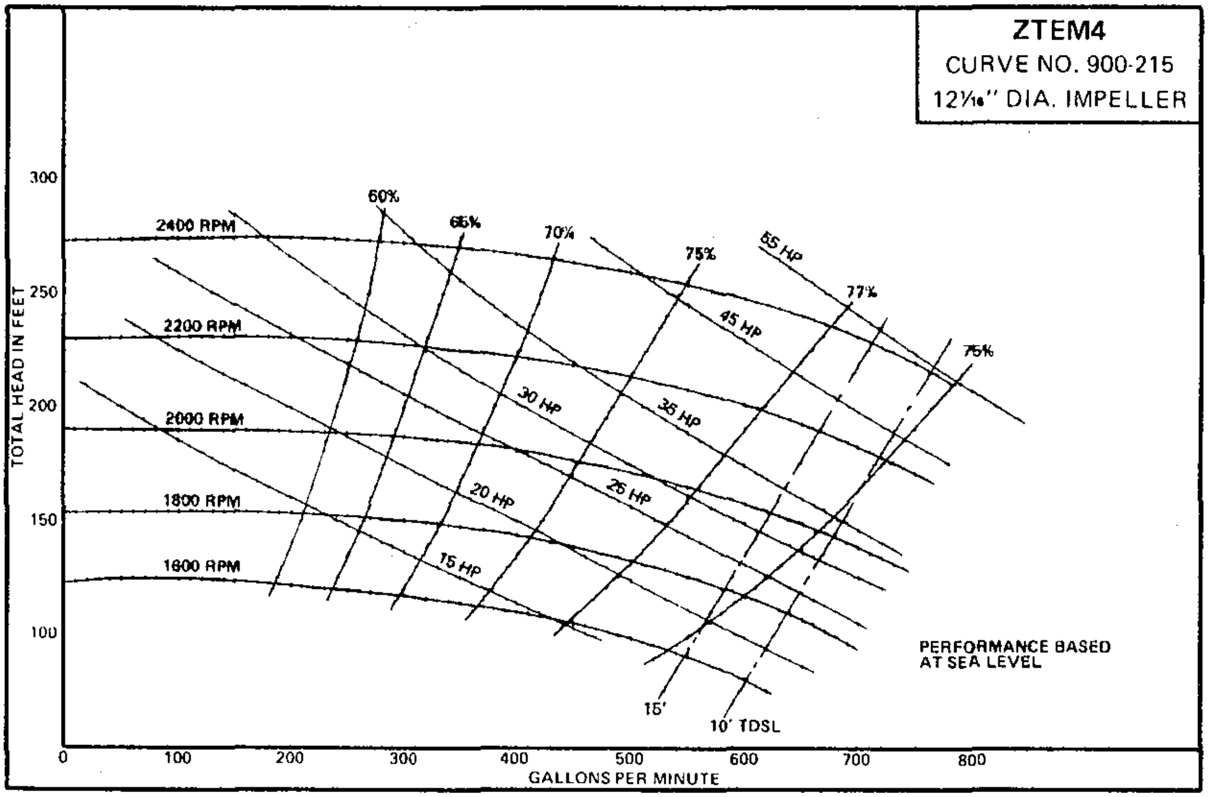


Figure B-1 PUMP PERFORMANCE CURVES - JACUZZI MODEL ZTEM4

B.5 EQUIPMENT ARRANGEMENT

In constant head and constant flow tests the recommended arrangement of equipment beginning at the source of water is as follows.

When using tandem connected pump and meter, the arrangement is water source, suction line, pump, water line to sump of storage tank, suction line, centrifugal test pump, line to water meter adapter if required, water meter, gate or plug valve, and water line to casing. A gate valve on the angular arm of the "Y" branch connection is convenient when this arrangement is used. When an electrical probe is used, stabilization of water level is facilitated with a valve at this location.

When using a constant head tank, the arrangement is the source of water, suction line, pump, water line to storage tank or sump, pump to constant head tank, constant head tank with overflow line back to sump or tank, and water line from tank to the casing.

APPENDIX C

FIELD TEST EQUIPMENT MANUFACTURERS

C.1 DRILLS AND DRILLING EQUIPMENT

C.1.1 Droptool Cable Drill

English Drilling Equipment Company, Ltd.
Lindley Moor Road
Huddersfield
Yorkshire
HD3 3RW
England

C.1.2 Drills - Auger Type

Acker Drill Company, Inc.
Shady Lane Road
Scranton, Pennsylvania 18501

Atlas Copco
70 Demarest Drive
Wayne, New Jersey 07470

Mobile Drill Company, Inc.
Indianapolis
Indiana 46227

Penndrill
1201 Chartiers Avenue
Pittsburgh, Pennsylvania

C.1.3 Drills - Rotary Type

Acker Drill Company, Inc.
(see C.1.2)

Atlas Copco
(see C.1.2)

English Drilling Equipment Company, Ltd.
(see C.1.1)

Longyear Drilling Company
925 Delaware Street, S.E.
Minneapolis, Minnesota 55414

Mobile Drill Company, Inc.
(see C.1.2)

Penndrill
(see C.1.2)

C.1.4 Becker Drills

Becker Drills, Inc.
5502 Pearl Street
Denver, Colorado 80216

C.1.5 Drills - Rotary-Percussive

Acker Drill Company, Inc.
(see C.1.2)

Atlas Copco
(see C.1.2)

C.2 CORE BARRELS AND DRILL BITS

B - Bits (Diamond + Tungsten Carbide)

C - Core barrels without split inner tubes

CS - Core barrels with and without split inner tubes

Acker Drill Company, Inc.
(see C.1.2)

Atlas Copco *(B, C)*
(see C.1.2)

Christensen Mining Products
P. O. Box 387
Salt Lake City, Utah 84110

English Drilling Equipment Company, Ltd. *(C)*
(see C.1.1)

Longyear Drilling Company *(B, CS)*
(see C.1.3)

Mobile Drill Company, Inc. *(B, C)*
(see C.1.2)

Penndrill *(B, C)*
(see C.1.2)

C.3 PUMPS

C - Centrifugal - used for packer tests or general pumping.

S - Submersible - used for well pumping tests.

P - Piston - may be used for moving water at ground level where constant pressure or flow rate are not important.

Allis-Chalmers *(C)*
1150 Tennessee Avenue
Cincinnati, Ohio 45229

Byron Jackson *(C, S, P)*
P. O. Box 2017
Terminal Annex
Los Angeles, California 90051

Crane Company *(C, S)*
Deming Division
P. O. Box 450
Salem, Ohio 44406

Jacuzzi Brothers, Inc. *(C, S)*
11511 New Benton Highway
Little Rock, Arkansas 72203

Peabody Barnes *(S)*
Mansfield
Ohio

C.4 SWIVELS

If a down the hole pressure transducer is being used then the use of standard swivels manufactured by one of the listed drill manufacturers will suffice. If excessive pressure loss through a swivel is of concern, either because of limited pump capacity or because "Packer System A" is being used, then a large diameter swivel should be used.

C.4.1 Large Diameter Swivels

King Oil Tools, Inc.
2401 Wilson Road
Humble, Texas 77338

C.5 HOSES AND COUPLINGS

The Gates Rubber Company
P. O. Box 5887
Denver, Colorado 80217

Goodyear Industrial Hose
Goodyear Tire and Rubber Company
Industrial Products Division
Akron, Ohio 44316

C.6 FLOW METERS

Badger Meter, Inc.
Flow Products Division
4545 West Brown Deer Road
Milwaukee, Wisconsin 53223

Hersey Products, Inc.
Water Meter and Control Division
250 Elm Street
Dedham, Massachusetts 02026

Rockwell Water Meters
400 North Lexington Avenue
Pittsburgh, Pennsylvania 15208

C.7 WATER LEVEL INDICATORS

Soil Instruments, Ltd.
Townsend Land
London
NW9 8TR
England

Soil Test, Inc.
2205 Lee Street
Evanston, Illinois 60202

C.8 PIEZOMETERS

C.8.1 <u>Casagrande</u>

Soil Instruments, Ltd.
Townsend Lane
London
NW9 8TR
England

Soil Test, Inc.
2205 Lee Street
Evanston, Illinois 60202

C.8.2 Electric

Irad Gage
14 Parkhorst Street
Lebanon, New Hampshire 03766

Slope Indicator Company
3668 Albian Place North
Seattle, Washington 98103

Terra Technology
3018 Western Avenue
Seattle, Washington 98121

C.8.3 Air Activated

Earl B. Hall, Inc.
P. O. Box 4306
San Rafael, California 94903

Glötzl
Franz Glötzl, Baumesstechnik
D-7512 Rheinstetten 4-Fo
W.-Germany

Slope Indicator Company
(see C.8.2)

C.9 PRESSURE GAUGES

C.9.1 Mechanical

ACCO
Helicoid Gage Division
929 Connecticut Avenue
Bridgeport, Connecticut 06602

Enerpac
Butler
Wisconsin 53007

H T L Industries, Inc.
373 South Fair Oaks Avenue
Pasadena, California 91105

A. A. Weiss and Son, Inc.
261 Vanderbilt Avenue
Brooklyn, New York 11205

C.9.2 Electric Transducers

Bell and Howell
360 Sierra Madre Villa
Pasadena, California 91109

Schaevitz Engineering
Pennsauken
New Jersey 08110

Tyco Instruments
4 Hartwell Place
Lexington, Massachusetts 02173

APPENDIX D

PACKER TEST SUPPLEMENTARY

Packer test apparatus can be either commercially obtained or shop-fabricated. This Appendix discusses two packer designs which can be constructed by the user.

Gale (1975) constructed a double-packer assembly which was used in fractured crystalline and metamorphic rocks. The packer shells are approximately three-quarters of an inch smaller than the borehole diameter, and are constructed from PVC (polyvinyl chloride) pipe. PVC pipe combines light weight and high tensile and compressive strengths with easy machining properties. The shells are cut into 2-foot lengths. Two 1/4-inch holes are drilled into the pipe walls approximately eight inches from each end (see Figure D-1). The lower shell requires only one hole. An air line consisting of 1/4-inch high strength plastic tubing or 1/8-inch copper tubing is seated in the holes using a water-proof epoxy. The injection pipe, constructed of 3/4-inch PVC, and cables for the pressure transducer are placed in the shells (see Figure D-1) and the space inside the shells is filled with Urethane (Flexane-95, Devon Corporation) or any other non-shrinking epoxy.

After the urethane has hardened, a two-foot length of 40-45 durometer natural gum rubber tubing with a fabric backing and a 1/8-inch wall thickness is placed over the PVC pipe. A 3- to 4-inch epoxy seal is created at each end of the packer shells and is then bonded with fiberglass tape. The 3/4-inch PVC injection pipe is perforated with 1/4-inch diameter injection holes (see Figure D-1).

Another system is discussed by Harper and Brown (1972). This packer assembly incorporates a three-layer rubber hose (Ductube) with a thickness of about 1/3-inch. The hose is used in building construction, where it is inflated in freshly-placed concrete to form voids for conduit passages. The layered-hose consists of an outer and inner tube with a diagonally braided inner cotton core, which allows expansion to a specified diameter. The hose ends are sealed by metal plugs. A machined groove around the plug provides seating for a metal strap which, when tightened, locks the tube in the groove.

The injection pipe and transducer cables are routed through axial holes in the end plugs, and a stiff nylon tube is strapped to nipple flanges on the inner end of each end plug.

The two construction assemblies above pertain to the double packer test, where two packers are used to isolate a single pressurized test cavity. One assumption in this test is that flow out of the test cavity into the surrounding rock mass is directed perpendicularly to the borehole axis, i.e., no flow component parallel to the borehole exists. This may be an accurate assumption in homogeneous strata, or in heavily fractured rock.

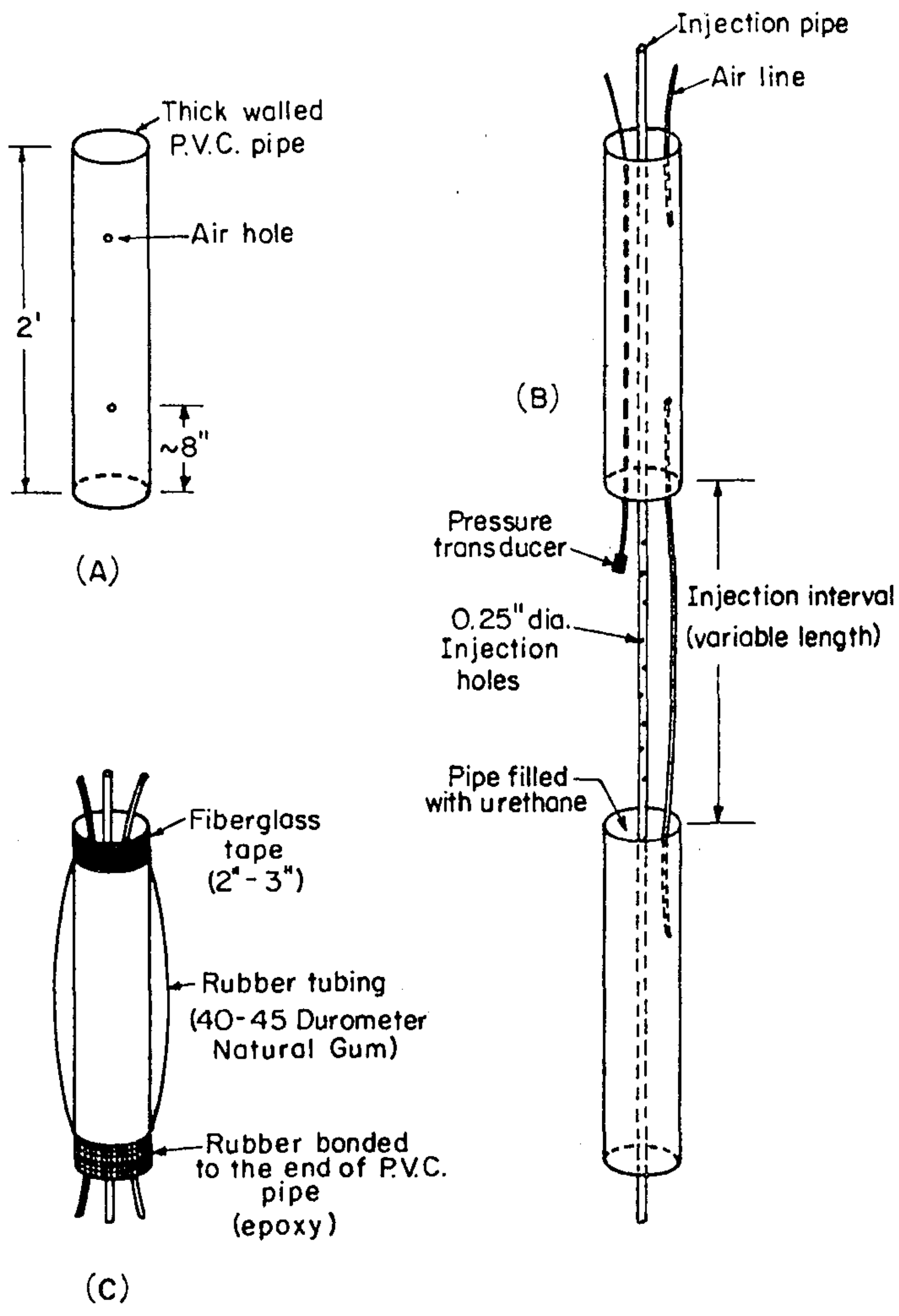
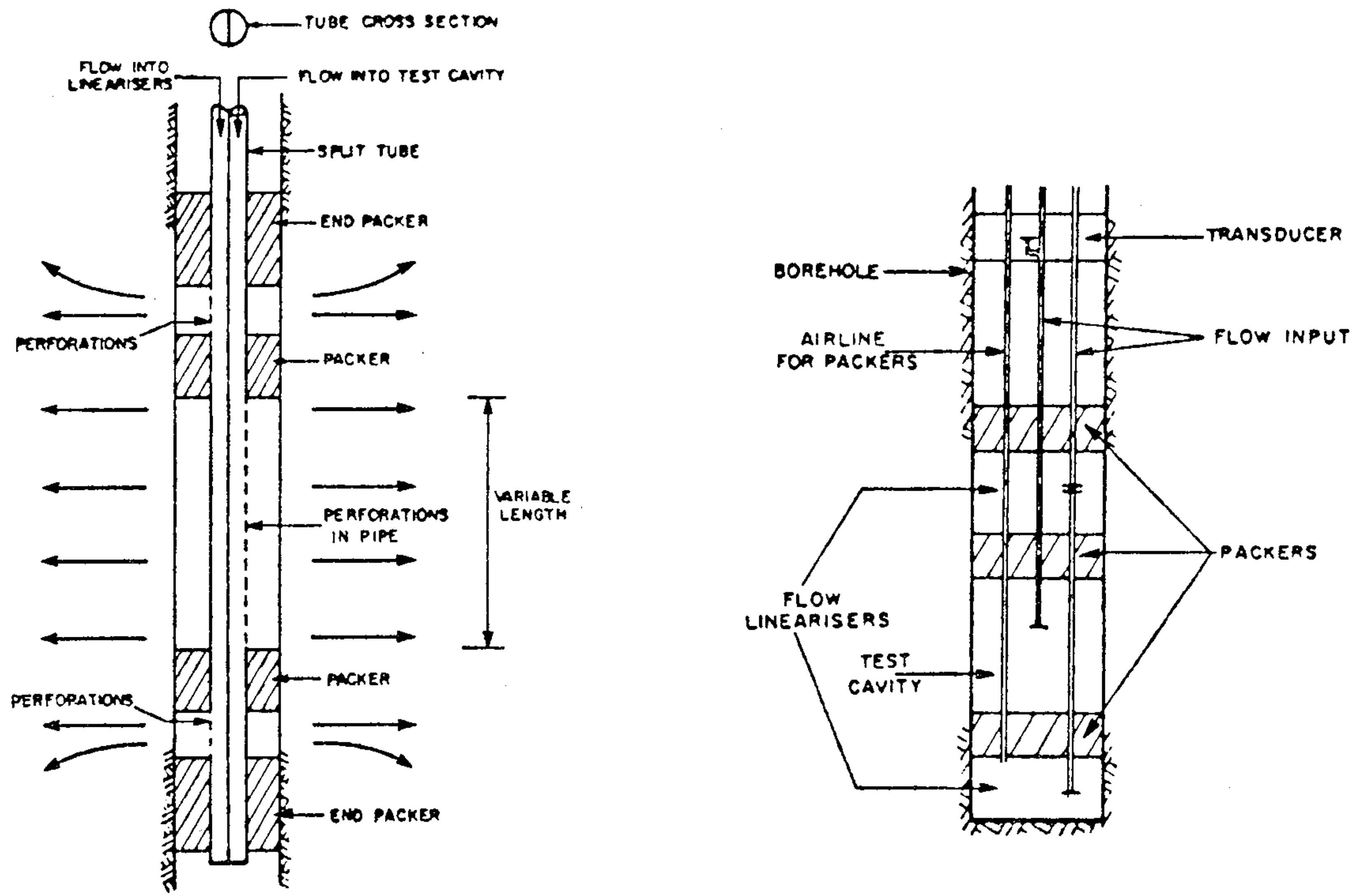


Figure D-1 CONSTRUCTION OF INFLATABLE PACKERS; A) PACKER SHELL, B) DOUBLE PACKER ASSEMBLY AND C) BONDING RUBBER TUBING TO PACKER SHELL (Gale, 1975)

However, in moderately- to lightly-fractured rock, the validity of this assumption is questionable. A system has been proposed (Sharp, 1970) and tested (Maini, 1971) whereby flow from the test cavity is controlled such that the planar flow assumption is satisfied. The method consists of having two additional cavities immediately above and below the two packers through which water is injected to conform the test cavity flow to a single plane. Figure D-2 illustrates its application in both continuous and stage packer tests.

An additional modification of the packer test may be useful in assessing the permeability due to vertical "parallel" flow in the immediate vicinity of the borehole. The method consists of two test cavities which are isolated by three packers (see Figure D-3). Water is injected into one test cavity (source) while water is pumped from the other (sink). A flow pattern is established between the two test sections, and the flow rates observed in the two test cavities can be used to estimate the permeability. Additional study is required for this method to determine mathematical relationships between the observed flow and the permeability of the flow region. The method can also be used as a profiling device in sedimentary formations. An impermeable boundary located between the two test cavities would prohibit flow between them. This would be reflected as an observed decrease in flow from the pumped-out test cavity.



(a) CONTINUOUS TESTING

(b) STAGE TESTING

Figure D-2 SCHEMATIC OF FLOW LINEARIZERS IN PACKER TESTS

(Maini et al, 1972)

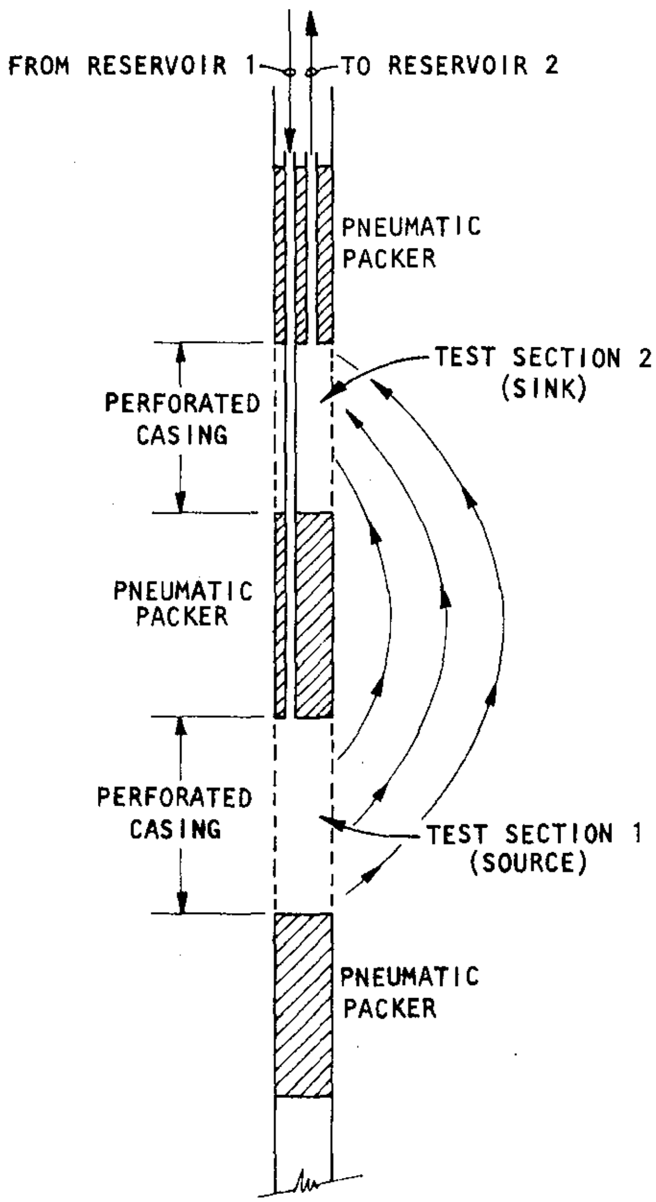


Figure D-3 SCHEMATIC FOR SOURCE-SINK TESTS
(Maini et al, 1972)

Discharge rate

	l/sec	m³/day	m³/sec	Imp. gal/day	U.S. gal/day	ft³/day
1 l/sec	1.000	86.40	1.000×10^{-3}	1.901×10^{4}	2.282×10^{4}	3.051×10^{3}
1 m³/h	0.2777	24.00	2.777×10^{-4}	5.279×10^{3}	6.340×10^{3}	8.476×10^{2}
1 m³/day	1.157×10^{-2}	1.000	1.157×10^{-5}	2.200×10^{2}	2.642×10^{2}	35.32
1 m³/sec	1.000×10^{3}	8.640×10^{4}	1.000	1.901×10^{7}	2.282×10^{7}	3.051×10^{6}
1 Imp. gal/day	5.262×10^{-5}	4.546×10^{-3}	5.262×10^{-8}	1.000	1.201	0.1605
1 U.S. gal/day	4.381×10^{-5}	3.785×10^{-3}	4.381×10^{-8}	0.8327	1.000	0.1337
1 ft³/day	0.3277	2.832×10^{-2}	3.277×10^{-7}	6.229	7.481	1.000

Hydraulic conductivity

	m/day	m/sec	cm/h	Darcy*	U.S.gal/day-ft²	Imp.gal/min-ft²	U.S.gal/min-ft²
1 m/day	1.000	1.157×10^{-5}	4.167	1.209	24.54	1.419×10^{-2}	1.704×10^{-2}
1 m/sec	8.640×10^{4}	1.000	3.600×10^{5}	1.045×10^{5}	2.121×10^{6}	1.226×10^{3}	1.472×10^{3}
1 cm/h	0.2400	2.777×10^{-6}	1.000	0.290	5.890	3.406×10^{-3}	4.089×10^{-3}
1 Darcy*	0.827	0.957×10^{-5}	3.45	1.000	0.585		0.843×10^{3}
1 U.S. gal/day-ft²	4.075×10^{-2}	4.716×10^{-7}	0.1698	1.708	1.000	5.783×10^{-4}	6.944×10^{-4}
1 Imp. gal/min-ft²	70.46	8.155×10^{-2}	2.936×10^{2}		1.729×10^{3}	1.000	1.201
1 U.S gal/min-ft²	58.67	6.791×10^{-2}	2.445×10^{2}	1.186×10^{-3}	1.440×10^{3}	0.8326	1.000

Transmissivity

	m²/day	m²/sec	Imp.gal/day-ft	U.S.gal/day-ft	Imp.gal/min-ft	U.S.gal/min-ft
1 m²/day	1.000	1.157×10^{-5}	67.05	80.52	4.656×10^{-2}	5.592×10^{-2}
1 m²/sec	8.64×10^{4}	1.000	5.793×10^{6}	6.957×10^{6}	4.023×10^{3}	4.831×10^{3}
1 Imp. gal/day-ft	1.491×10^{-2}	1.726×10^{-7}	1.000	1.201	6.944×10^{-4}	8.339×10^{-4}
1 U.S. gal/day-ft	1.242×10^{-2}	1.437×10^{-7}	0.8326	1.000	5.783×10^{-4}	6.944×10^{-4}
1 Imp. gal/min-ft	21.48	2.486×10^{-4}	1.440×10^{3}	1.729×10^{3}	1.000	1.201
1 U.S. gal/min-ft	17.88	2.070×10^{-4}	1.199×10^{3}	1.440×10^{3}	0.8326	1.000

Abbreviations: ft = foot; in = inch; l = liter; Imp. gal = Imperial gallon; U.S. gal = U.S. gallon; h = hour.
*for water @ 20°C

*Conversion coefficients**

Length:

	m	cm	ft	in
1 m	1.000	1.000×10^{2}	3.281	39.37
1 cm	1.000×10^{-2}	1.000	3.281×10^{-2}	0.3937
1 ft	0.3048	30.48	1.000	12.00
1 in	2.540×10^{-2}	2.540	8.333×10^{-2}	1.000

Area:

	m²	ft²
1 m²	1.000	10.76
1 ft²	9.290×10^{-2}	1.000

Volume:

	m³	l	Imp. gal.	U.S. gal.	ft³
1 m³	1.000	1.000×10^{3}	2.200×10^{2}	2.642×10^{2}	35.32
1 l	1.000×10^{-3}	1.000	0.2200	0.2642	3.532×10^{-2}
1 Imp.gal	4.546×10^{-3}	4.546	1.000	1.200	0.1605
1 U.S.gal	3.785×10^{-3}	3.785	0.8326	1.000	0.1337
1 ft³	2.827×10^{-2}	28.27	6.229	7.480	1.000

Time:

	day	h	min	sec
1 day	1.000	24.00	1.440×10^{3}	8.640×10^{4}
1 h	4.167×10^{-2}	1.000	60.00	3.600×10^{3}
1 min	6.944×10^{-4}	1.667×10^{-2}	1.000	60.00
1 sec	1.157×10^{-5}	2.777×10^{-4}	1.667×10^{-2}	1.000

Time reciprocals:

	day⁻¹	h⁻¹	min⁻¹	sec⁻¹
1 day⁻¹	1.000	4.167×10^{-2}	6.944×10^{-4}	1.157×10^{-5}
1 h⁻¹	24.00	1.000	1.667×10^{-2}	2.777×10^{-4}
1 min⁻¹	1.440×10^{3}	60.00	1.000	1.667×10^{-2}
1 sec⁻¹	8.640×10^{4}	3.600×10^{3}	60.00	1.000

* *Abbreviations*: ft = foot; in = inch; l = liter; Imp.gal = Imperial gallon; U.S.gal = U.S. gallon; h = hour.