DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
441 G Street, NW
Washington, DC 20314-1000

CECW-EC

Technical Letter
No. 1110-2-586

24 May 2023

EXPIRES 30 JUNE 2026
Engineering and Design
DEWATERING: METHODS, EVALUATION, DESIGN, INSTALLATION, AND PERFORMANCE MONITORING

1. This Change 1 to Technical Letter 1110-2-586, 24 May 2021, revises/adds/deletes technical figures and equations for accuracy and clarity.

2. ETL 1110-2-586, 24 May 2021, is changed as follows:

a. Page 82. Replaced Figure 37.

b. Page 90. Figure 45 - changed 'R' to 'OR' in Equation 2.

c. Page 111. Figure 60 - changed the x-axis on part (b) to start at zero.

d. Page 119. Section 5.5.1.1 – Replaced the second sentence.

e. Page 128. Section 5.7.1.1 – Replaced equation 10.

f. Page 133. Section 5.8.2.1.2.f - Added three new sentences.

g. Page 176. A.1.10 – Updated reference link

h. Page 176. A.1.15 – Updated reference link

i. Page 176. A.2.3 – Updated reference link

j. Page 177. A.3.2 – Updated reference link

k. Page 177. A.3.3 – Updated reference link

l. Page 177. A.3.4 – Updated reference link

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PETE G. PEREZ, P.E., SES
Chief, Engineering and Construction

DEPARTMENT OF THE ARMY ETL 1110-2-586
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DEWATERING: METHODS, EVALUATION, DESIGN, INSTALLATION, AND PERFORMANCE MONITORING

1. Purpose. This Engineer Technical Letter (ETL) provides guidance for the planning, design, supervision, construction, and operation of construction phase dewatering and pressure relief systems, and of seepage cutoffs for deep excavations for structures.

2. Applicability. This ETL applies to all Headquarters US Army Corps of Engineers (USACE) elements and all USACE elements having responsibility for planning, engineering design, construction, operations, and maintenance responsibilities associated with dewatering of Civil Works.

3. Distribution Statement. Approved for public release, distribution is unlimited.

4. References. References are included in Appendix A.

5. Records Management (Recordkeeping) Requirements. The records management requirement for all record numbers, associated forms, and reports required by this Engineer Technical Letter are addressed in the Army Records Retention Schedule (RRS-A). Detailed information for all related record numbers are located in ARIMS/RRS-A at https://www.arims.army.mil. If any record numbers, forms, and reports are not current, addressed, and/or published correctly in ARIMS/RRS-A, see Department of the Army (DA) Pamphlet 25-403, Guide to Recordkeeping in the Army.

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7 Appendixes
(See Table of Contents)

PETER L. STURDIVANT, P.E., P.G.
Acting Chief, Engineering and Construction

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Chapter 1
Introduction

1.1 Purpose. This ETL provides guidance for the planning, design, supervision, construction, and operation of construction phase dewatering and pressure relief systems, and of seepage cutoffs for deep excavations for structures.

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1.6 Introduction. This ETL provides guidance for the planning, design, supervision, construction, and operation of construction phase dewatering and pressure relief systems, and of seepage cutoffs for deep excavations for structures. There is currently no USACE specific dewatering guidance; this ETL provides guidance for any dewatering required on USACE projects. It presents:

a. Descriptions of various methods of dewatering and pressure relief;

b. Techniques for evaluating groundwater conditions;

c. Characteristics of pervious aquifers, and dewatering requirements;

d. Procedures for designing, installing, operating, and checking the performance of dewatering systems for various types of excavations; and

e. Descriptions of various types of cutoffs and bottom seals are sometimes used as the primary strategy but most typically used in combination with dewatering and pressure relief for controlling groundwater in excavations.

1.7 General.

1.7.1 The responsibility to design, install, and operate construction dewatering and groundwater control systems is often borne by the construction contractor. The principal purposes of this document are to provide guidance in selecting dewatering and groundwater control systems and designing such systems for cost estimating. The portions of the document dealing with design considerations should facilitate review of the contractor's plans for achieving the desired results.

1.7.2 This document can also be used when the owner/engineer is responsible for the design of a dewatering system, particularly when safety of the general public are of concern (safety of critical structures such as dams/levees, adjacent structures, etc.) and based on construction schedule criticality (i.e., when there is no time for contractor trial and error). In these cases, it may be desirable to design and specify the equipment and procedures to be used and to have the owner accept responsibility for results obtained. See Chapter 8 for additional discussion.

1.7.3 Most of the analytical procedures set forth in this document for groundwater flow are for "steady-state" flow, which is the most common application, and not for "unsteady-state" flow, which is a more unique situation. Empirical approximations for radii of influence and distance to an equivalent line source of seepage are presented that permit non-steady flow problems to be analyzed using steady-state flow equations.

1.7.4 This document presents dewatering and groundwater control procedures that are not commonly used by general construction contractors for subsurface construction. This document also includes cases where the dewatering system may be sufficiently critical as to affect the competency of the foundation and design of the substructure.

1.7.5 This ETL is largely based on information found in TM 5-818-5 (Army, Air Force, Navy, 1983). The 1983 version of TM 5-818-5 was based on a guidance document produced by Charles I. Mansur of Fruco & Associates, Inc. under contract with the USACE Waterways Experiment Station (WES). WES is now called the U.S. Army Engineer Research and Development Center.

1.7.6 This ETL has been developed to provide redrafted versions of many of the previously illegible figures, as well as provide updates on current practices in the dewatering industry and correct any errors made in the 1983 version of TM 5-818-5.

Chapter 2
Basics of Groundwater Control

2.1 Need for Groundwater Control.

2.1.1 Proper control of groundwater can greatly facilitate construction of subsurface structures founded in, or underlain by, pervious soil strata below the water table by:

a. Intercepting seepage that would otherwise emerge from the slopes or bottom of an excavation.

b. Increasing the stability of excavated slopes and bottom of excavations.

c. Preventing the loss of material from the slopes or bottom of the excavation.

d. Reducing lateral loads on cofferdams.

e. Improving the excavation and backfill characteristics of sandy soils.

f. Enabling construction of structures in dry conditions.

2.1.2 Uncontrolled or improperly controlled groundwater can, by hydrostatic pressure and seepage, cause internal erosion (piping), heave, or reduce the stability of excavation slopes or foundation soils so as to make them unsuitable for supporting the structure. For these reasons, subsurface construction (including excavation below the groundwater table) should not be attempted or permitted without first providing adequate control of the groundwater and (subsurface) hydrostatic pressure.

2.2 Influence of Excavation Characteristics.

2.2.1 The location of an excavation, its size, depth, and type, such as open cut, shaft, or tunnel, and the type of soil to be excavated are important considerations in the design and selection of a dewatering system. For most granular soils, the groundwater table during construction should be maintained at least 2 to 3 feet below the excavated slope surfaces and bottom of an excavation in order to ensure "dry" working conditions. The groundwater table may need to be maintained at greater depths for silts (more than 5 feet below subgrade) to prevent water pumping to the surface (from equipment operating on subgrade) and making the bottom of the excavation wet and unstable. Where such deep dewatering provisions are necessary, they should be explicitly required by the specifications, as they greatly exceed normal requirements and would not otherwise be anticipated by contractors.

2.2.2 Where the bottom of an excavation is underlain by a clay, silt, or shale stratum that is underlain by a pervious formation under artesian pressure (Figure 1), the upward pressure or seepage may rupture the bottom of the excavation or keep it wet even though the slopes have been dewatered. Factor of safety considerations with regard to artesian pressure are discussed in Section 5.7.

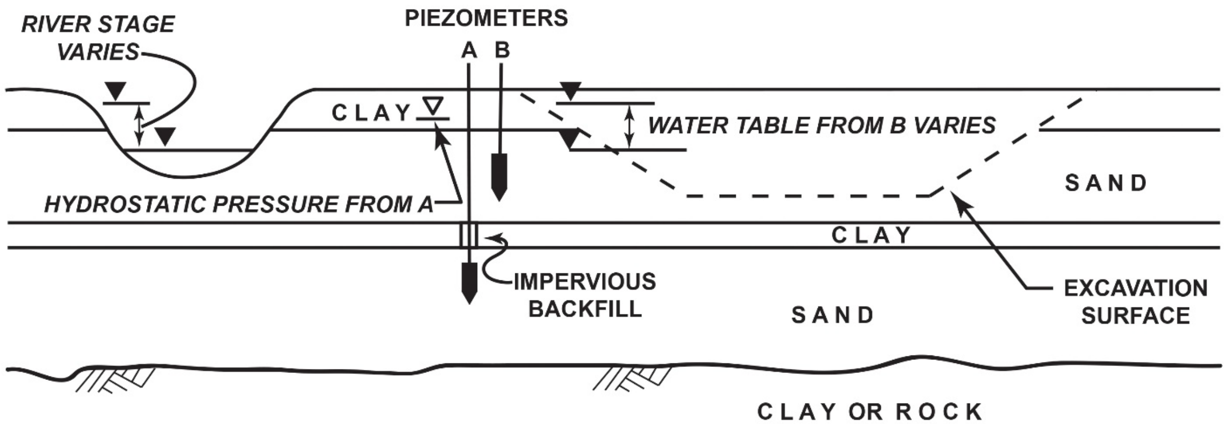

Figure 1. Installation of piezometers for determining water table and artesian hydrostatic pressure (Adapted from Leonards, 1962 and TM 5-818-5)

2.2.3 Special dewatering measures may be required for excavations extending through weathered rock where substantial water inflow can be accommodated without severe erosion. If the groundwater has not been controlled by dewatering and there is appreciable flow through fractures, lateral support of the rock may be required using rock bolts or other methods including internally or externally braced wales, soldier beams and lagging. If there are excessive hydrostatic pressures within the underlying rock deposit, rock anchors may be required to prevent uplift. Rock support is discussed further in EM 1110-1-2907.

2.2.4 An important facet of dewatering an excavation is the relative risk of damage that may occur to the excavation, cofferdam, or foundation of a structure or nearby structures in the event of failure of the dewatering system. The method of excavation and reuse of the excavated soil may also have a bearing on the need for dewatering. These factors, as well as the construction schedule, must be determined and evaluated before proceeding with the design of a dewatering system.

2.3 Groundwater Control Methods. Methods for controlling groundwater may be divided into three categories:

2.3.1 Interception and removal of groundwater from the site using sumps/ditches or drains. This type of control must include consideration of a filter to prevent migration of foundation fines and possible development of internal erosion (piping) in the soil being drained.

2.3.2 Interception and removal of groundwater from the site by pumping using wells or wellpoints. This method can also be used to reduce artesian pressures beneath the bottom of an excavation. This type of control must also include consideration of a filter to prevent migration of foundation fines and possible development of internal erosion (piping) in the soil being drained.

2.3.3 Isolation of the excavation from groundwater inflow using cutoff walls (sheet-piles, grout curtains, secant piles, deep soil mixing, jet-grouting, soil-bentonite, or cement-bentonite), or by freezing. A variation of this category is provision of a bottom seal in combination with watertight vertical shoring to isolate the excavation from groundwater inflow and to resist uplift pressure. This method can also be used to reduce artesian pressures beneath the bottom of an excavation.

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Chapter 3
Seepage Types/Sources and Dewatering Methods

3.1 General

3.1.1 Dewatering and control of groundwater during construction may be accomplished by one or a combination of methods described in the following paragraphs. The applicability of different methods to various types of excavations, groundwater lowering, and soil conditions is also discussed in these paragraphs. Analysis and design of dewatering, pressure relief and groundwater control systems are described in Chapter 5 below.

3.1.2 For some stratigraphy and drawdown conditions, the flow may be artesian in some areas and gravity in other areas, such as near wells or sumps where drawdown occurs. The type of seepage flow to a dewatering system can be estimated from a study of the groundwater table and soil formations in the area and the drawdown required to dewater the excavation.

3.2 Types and Sources of Seepage.

3.2.1 Types of Seepage Flow. The two types of seepage flow are:

3.2.1.1 Artesian - Seepage through the pervious aquifer is confined between two or more impervious strata, and the piezometric head within the pervious aquifer is above the top of the pervious aquifer (Figure 1).

3.2.1.2 Gravity - The surface of the water table is below the top of the pervious aquifer (Figure 2).

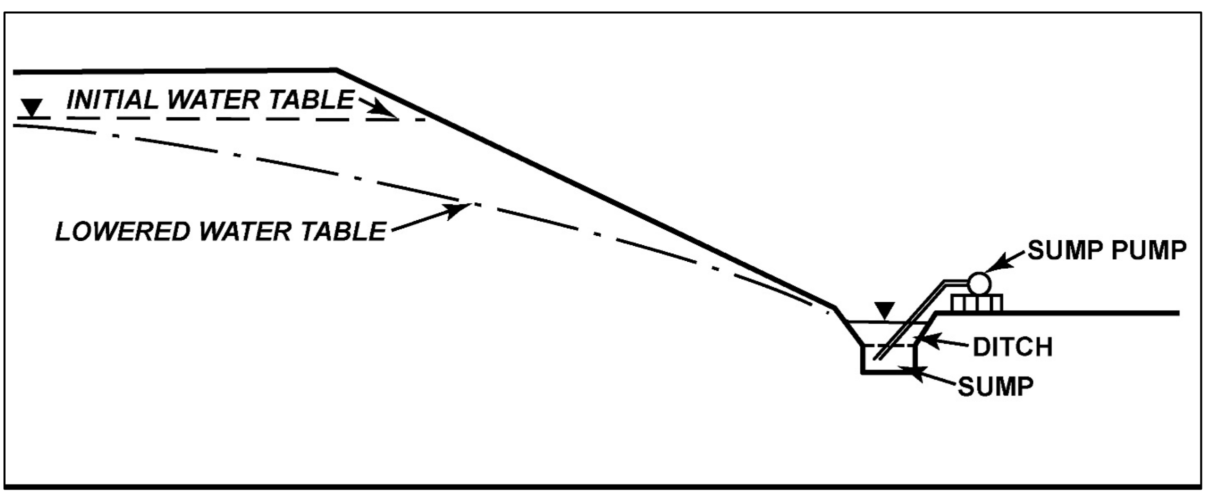


Figure 2. Dewatering open excavation by ditch and sump (Adapted from Leonards, 1962 and TM 5-818-5)

3.2.2 Sources of Seepage Flow.

3.2.2.1 The sources of seepage and distance (L) or radius of influence (R) to these sources must be estimated or determined prior to designing or evaluating a dewatering or drainage system.

3.2.2.2 The sources of seepage depend on the geological features of the area, the existence of adjacent streams or bodies of water, the perviousness of the formation, recharge, amount of drawdown, and duration of pumping. The sources of seepage may be a nearby stream, reservoir or lake, the aquifer being drained, or both an adjacent body of water and storage in the aquifer.

3.2.2.3 Where the site is not adjacent to a river or lake, or a reservoir is empty, the source of seepage will be from storage within the formation being drained and recharged from rainfall over the area. Where this condition exists, flow to the area being dewatered can be computed on the assumption that the source of seepage is circular and at a distance R. The radius of influence, R, is defined as the radius of the circle beyond which pumping of a dewatering system has no significant effect on the original groundwater level or piezometric surface (see Chapter 5).

3.2.2.4 Where an excavation is located close to a river or shoreline in contact with the aquifer to be dewatered, the distance to the effective source of seepage L, if less than R/2, may be considered as being approximately near the bank of the river; if the distance to the riverbank or shoreline is equal to about R/2 or greater, the source of seepage can be considered a circle with a radius somewhat less than R. The rationale for this is that the formulas for steady state flow to a well from circular and line sources of seepage are identical except for the distance to the source (equal to 2L for a line source or R for a circular source).

3.2.2.5 Where a line or two parallel lines of wells are installed in an area that is not close to a river or other line source, and the expected effective radius of influence R for a single well is a fraction or a small multiple of the length(s) of the well line(s), the source of seepage may be considered as a line paralleling each line of wells at a distance equal to R/2. At the ends of a long well line, the source of seepage may be considered as circular with radius R.

3.3 Dewatering Methods. There are three basic dewatering methods:[1]

[1] Another groundwater control method (ground freezing) is also discussed. Each of these methods is discussed in this section. The applicability of types of groundwater control methods to various subsurface conditions are included in Table 1, which is discussed later in this document.

a. sumps and ditches;

b. wellpoints, wells, and other pre-drainage systems; and

c. cutoffs and bottom seals.

3.3.1 Sumps and Ditches.

3.3.1.1 An elementary dewatering procedure involves the installation of ditches, blanket drains, French drains, and sumps within an excavation, from which water entering the excavation can be pumped (Figure 2). This method is generally effective in soil or rock that is not easily erodible and in semi-pervious or pervious soils where there is no continuous source of recharge (e.g., minimal perched groundwater in sand or gravel with limited recharge above a clay stratum). Figure 3 shows open pumping from an undisclosed local pocket of gravel at the downstream toe of an existing dam with a reduced upstream pool in preparation for a larger excavation to construct an aggregate drainage layer along the toe.

Figure 3. Use of 2-inch submersible pump to dewater localized gravel pocket at downstream toe of an existing earth dam in preparation for a larger excavation to construct an aggregate drain along the toe (Lake Oneida Dam, Butler County, Pennsylvania, Courtesy of Keller)

3.3.1.2 However, the open pumping method has also been used successfully on major projects in pervious soils with continuous recharge when there was thorough engineering analysis and design performed in advance by experienced geotechnical engineers. The excavations into highly permeable alluvial sand extending well below the planned subgrade for the original locks and dams on the Mississippi River were successfully completed using open pumping methods.[2] Figure 4 shows schematic details of berms, drains and dewatering ditches constructed circa 1936 for Lock and Dam 26 on the Mississippi River at Alton, IL. The pumping systems for dewatering were designed and operated for a head differential of as much as 35 feet between the river stage and the subgrade level within the cofferdam.

[2] For example see case histories for Lock No. 6, Tremplealeau, WI, and Lock and Dam 26, Alton, IL, in *Cofferdams*, by White, L. and Prentis, E.A., Second Edition, 1950, Columbia University Press, New York, NY, 611 pages. The authors state on page 39 that ". . . in the authors' experience a good system of open ditches with efficient centrifugal pumps has proved far more efficient and safer than drainage by well points."

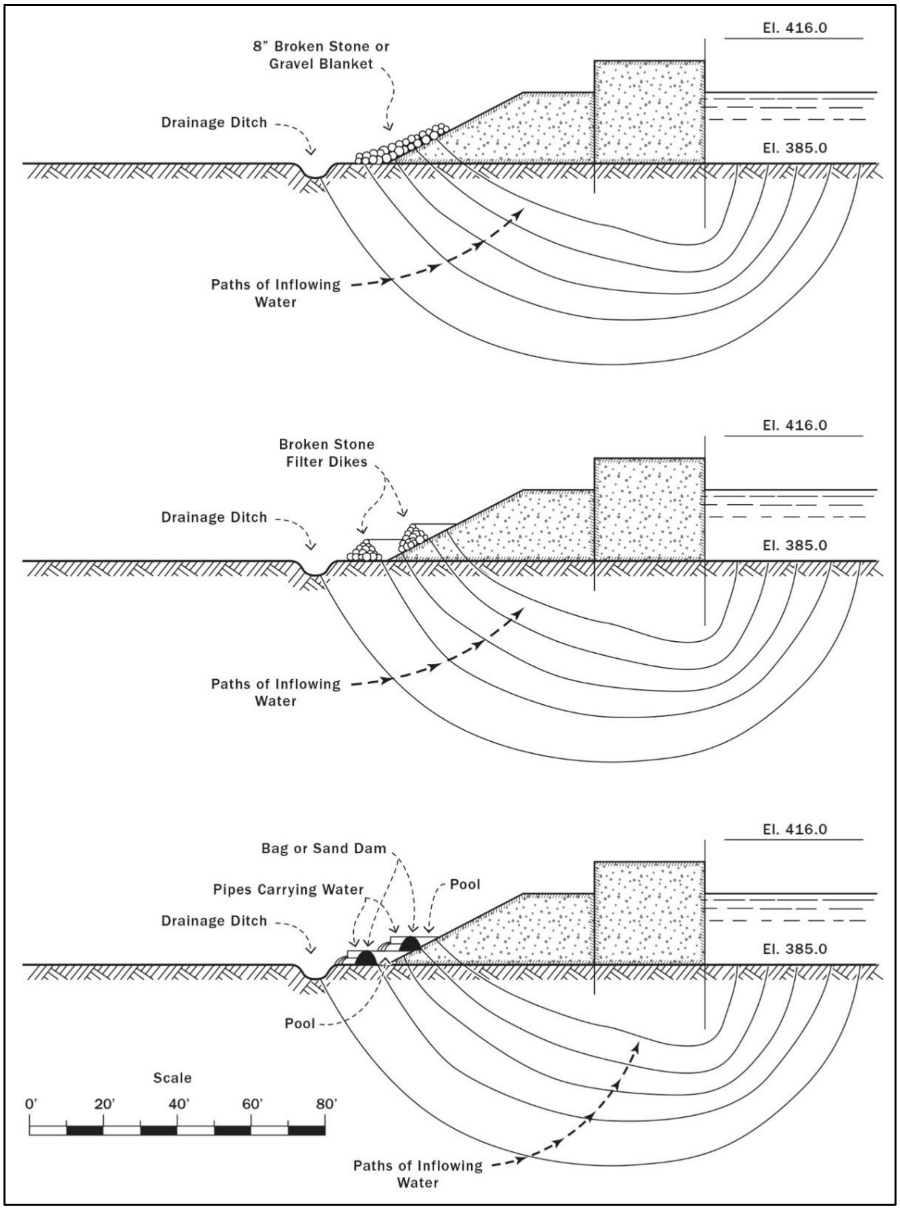


Figure 4. Open dewatering methods used at Dam 26, Mississippi River, circa 1936 (adapted from White and Prentis, 1950)

3.3.1.3 For small projects in erodible, semi-pervious or pervious soils with a continuous source of recharge, this method of dewatering should generally not be considered where the groundwater head must be lowered more than a few feet, as seepage into the excavation may impair the stability of excavation slopes or have a detrimental effect on the integrity of the foundation soils.

3.3.1.4 For some soils (e.g., nonplastic silts, uniform silty fine sands) even excavating a few feet below the groundwater table without predrainage may cause bottom instability. Filter blankets or drains may be included in a sump-and-ditch system to overcome minor slope raveling and facilitate collection of seepage.

3.3.1.5 Disadvantages of a sump dewatering system include the relative slowness in drainage of the slopes; potentially wet conditions during excavation and backfilling, which may impede construction and adversely affect the subgrade soil; space required in the bottom of the excavation for drains, ditches, sumps, and pumps.

3.3.1.6 An improperly operated or designed sump system, as shown in Figure 5, is one that does not control the ground water to allow construction on a stable, dry foundation.

Figure 5. Improperly constructed sump (Courtesy of Keller)

3.3.2 Wellpoints, Wells, and Other Pre-Drainage Systems. The term 'well' is a universal term for a feature that connects or accesses a supply of water (or other liquid or gas), while wellpoint and deep-well systems are specific types of wells that are used in relation to dewatering. Deep-wells are typically larger diameter, and deeper systems than wellpoints with well casings that are installed similar to water wells. Wellpoints are typically smaller diameter, shallower systems than deep wells.

3.3.2.1 Wellpoints. Wellpoint systems are a commonly used dewatering method as they are applicable to a wide range of excavations and groundwater conditions and are typically used in foundation materials ranging from silts and sands to gravels.

3.3.2.2 Conventional Wellpoint Systems.

3.3.2.2.1 A conventional wellpoint system consists of one or more stages of wellpoints having 1½ or 2 inch-diameter riser pipes, installed in a line or ring at spacings between about 3 to 10 feet, with the risers connected to a common header pumped with one or more wellpoint pumps. Wellpoint screens typically consist of polyvinyl chloride (PVC) or stainless steel well screens with machined (factory) cut slots, mesh over perforated pipe, or continuous slots achieved using spirally wrapped PVC, steel, or stainless steel shaped wire over vertical support rods. Wellpoint screens generally range in size from 2 to 4 inches in diameter and 2 to 5 feet in length and are constructed with either closed ends or self-jetting tips as shown in Figure 6. The wellpoint screens often require a filter, depending upon the type of soil being drained. Granular filter materials are the most common type of filters, and they need to be designed to be filter compatible with the foundation material, and the wellpoint screens. Refer to EM 1110-2-1901 for guidance on filter compatibility. Geotextile fabric and woven socks are less commonly used as filters since they may clog over time, and typically have lower flow rates and produce lower water quality than granular filters. A wellpoint system is shown in Figure 7.

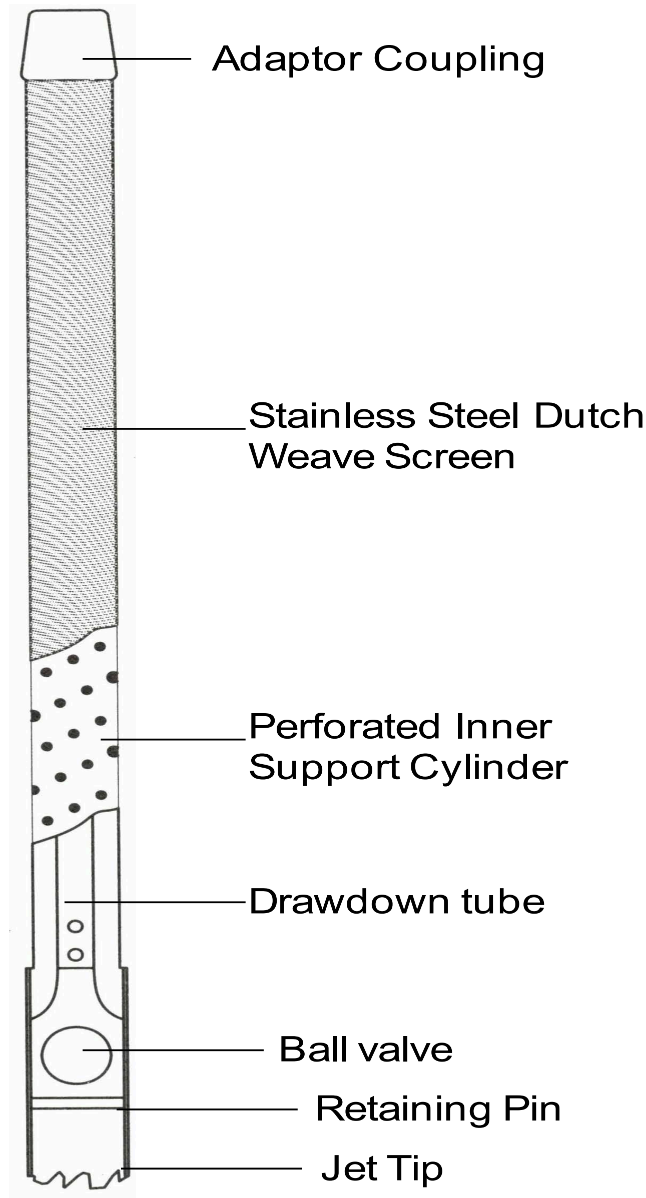


Figure 6. Self-jetting wellpoint (Courtesy of Keller and adapted from TM 5-818-5)

Figure 7. Typical wellpoint system (Courtesy of Keller)

3.3.2.2.2 Wellpoint screens and riser pipes may be as large as 6 inches in diameter and as long as 25 feet in high volume flow applications. In some areas where the transmissivity of the aquifer is very high, achieving drawdown exceeding a few feet using conventional commercially available 2-inch diameter wellpoints with 3-foot long screens is not practical, even if the wellpoints are closely spaced. In such areas, larger diameter wellpoints with longer screens can be used effectively to achieve the same drawdown as that achieved with deep wells, provided all components of the system, including wellpoints, pumps, swing connections, vacuum headers, and discharge lines (see Figure 8) are designed for the higher transmissivity of the aquifer.[3] A wellpoint pump, which typically combines a vacuum pump for air removal and a centrifugal water pump, produces a vacuum in the header system to remove the water. High capacity rotary

[3] Such high capacity wellpoint systems were used extensively for the dewatering of excavations for the Melvin Price Locks and Dam near Alton, Illinois in the 1980s and also for a test excavation near this project in the 1970s.

positive displacement pumps are also used and are capable of pumping both air and water. One or more supplementary vacuum pumps may be added to the main pumps where additional air handling capacity is required or desirable. Generally, a stage of wellpoints (wellpoints connected to a header at a common elevation) is capable of lowering the groundwater table a maximum of about 15 feet near sea level, depending on the soil type being dewatered. Lowering the groundwater more than 15 feet generally requires a multistage installation of wellpoints as shown in Figures 9 and 10. Submergence (as shown in Figure 9) is defined as the distance between the groundwater level at the wellpoint or well to an impervious layer below the bottom of the well screen when the groundwater level has been lowered by the dewatering system. Submergence is typically not less than 4 feet since it is impractical to lower the phreatic level closer than about 4 feet above the top of a laterally extensive horizontal impervious stratum in or underlying the pervious stratum being dewatered.

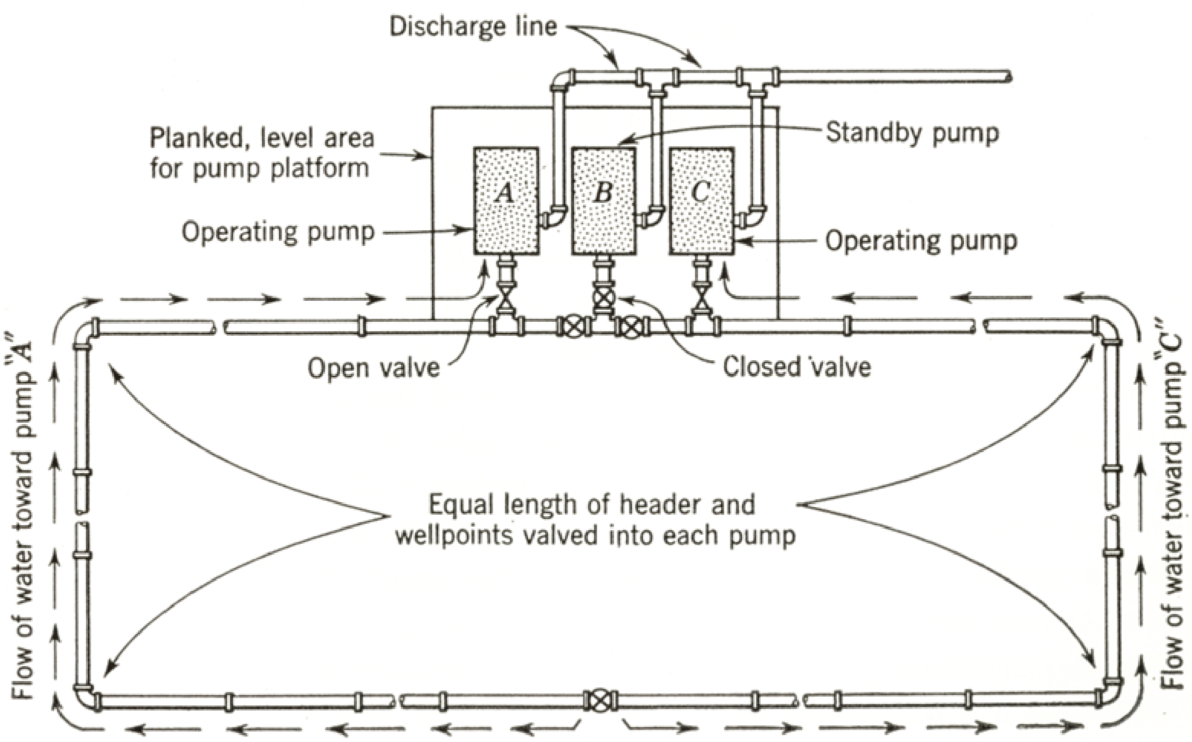
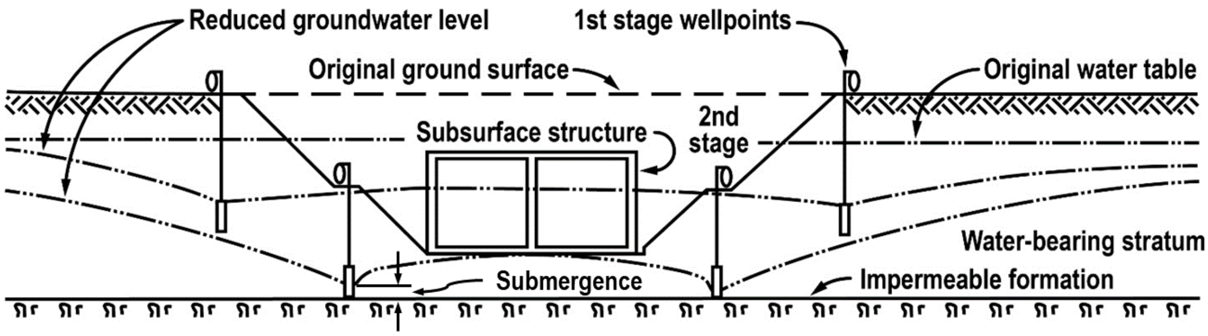


Figure 8. Plan of typical wellpoint system (Adapted from Leonards, 1962 and TM 5-818-5). Wellpoints not shown.



(After "Foundation Engineering," G.A. Leonards, ed, 1962, McGraw-Hill Book Company. Used with permission of McGraw-Hill Book Company)

Figure 9. Use of wellpoints where submergence is small (less than about 4 feet) (Adapted from Leonards, 1962 and TM 5-818-5)

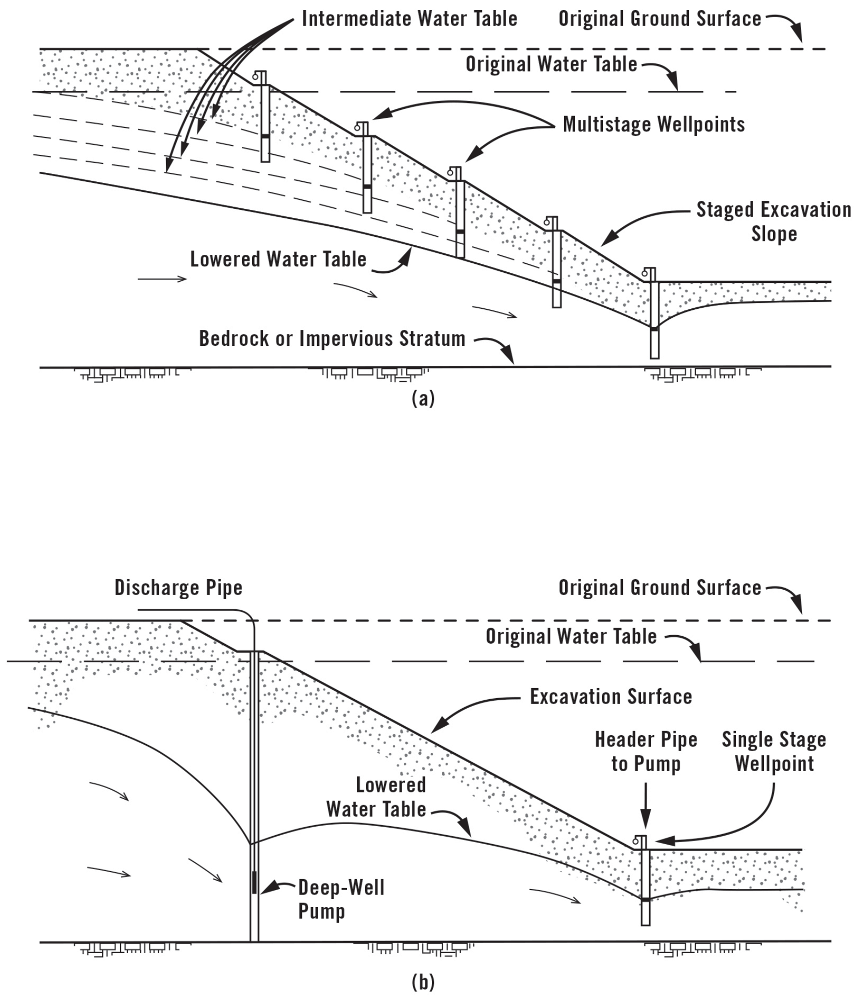


Figure 10. Predrainage of a deep excavation: (a) using multiple stages of wellpoints; (b) using deep wells supplemented by a single stage of wellpoints at the bottom of the excavation (Adapted from Terzaghi and Peck and Mesri, 1996 and TM 5-818-5)

3.3.2.2.3 Because maximum vacuum is limited by atmospheric pressure, the 15-foot rule of thumb should be reduced for work at elevations considerably above sea level to account for lower atmospheric pressure. For example, the air pressure at an altitude of 5,000 feet is about 25 inches of mercury, compared to about 30 inches of mercury at sea level, a difference equivalent to almost 6 feet of water head. Therefore, at 5,000 feet above mean sea level the 15-foot rule of thumb becomes a 9-foot rule of thumb.

3.3.2.2.4 A wellpoint system is usually the most practical method for dewatering where the site is accessible and where the excavation and water-bearing strata to be drained are not too deep. For large or deeper excavations where the depth of excavation is more than 30 to 40 feet, or where artesian pressure in a deep aquifer must be reduced, it may be more practical to use eductor-type wellpoints or deep wells (discussed subsequently) with turbine or submersible pumps, using wellpoints as a supplementary method of dewatering if needed. Wellpoints are more suitable than deep wells where the submergence available for the well screens is small (Figure 9) and close spacing is required to intercept seepage.

3.3.2.2.5 Silts and sandy silts (D10 ≤0.05 millimeters) with a low hydraulic conductivity (k = 0.1 x 10-4 to 10 x 10-4 centimeters per second [cm/sec]) cannot be drained successfully using wellpoints without a vacuum applied to a sand filter. Such soils can often be stabilized by applying a vacuum to the sand filter around the wellpoint and riser pipe (Figure 11). This vacuum will increase the hydraulic gradient producing flow to the wellpoints and will improve drainage and stabilization of the surrounding soil. For a wellpoint system, the net vacuum at the wellpoint and in the filter is the vacuum in the header pipe minus the lift or length of the riser pipe. Therefore, relatively little vacuum effect can be obtained with a wellpoint system if the lift is more than about 15 feet, (and even less than 15 feet at site elevations that are considerably above sea level). If there is much air loss, it may be necessary to provide additional vacuum pumps to ensure maintaining the maximum vacuum in the filter column. Due to the low hydraulic conductivity of the formation, the required capacity of the water pump is, of course, small.

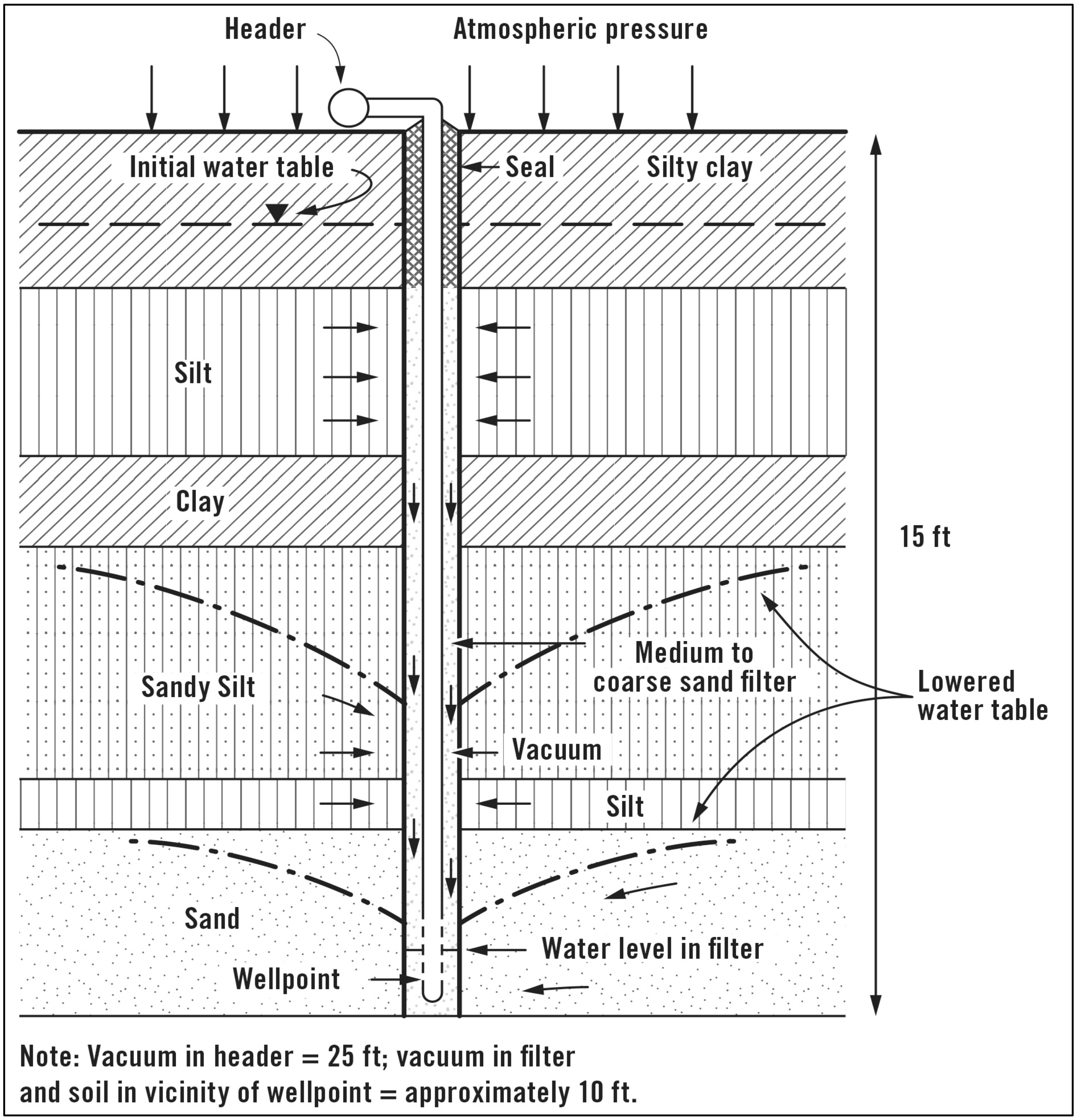


Figure 11. Vacuum wellpoint system (Adapted from Leonards, 1962 and TM 5-818-5)

3.3.2.3 Jet-eductor Wellpoint Systems.

3.3.2.3.1 Another type of dewatering system is the jet-eductor wellpoint system (Figure 12 and Figure 13), which consists of an eductor installed in a small diameter well or a wellpoint screen attached to a jet-eductor installed at the end of double riser pipes, a pressure pipe to supply the jet-eductor and another pipe for the discharge from the eductor pump. These systems are also call ejector systems (Powers et al. 2007). Eductor wellpoints may also be pumped with a pressure pipe, which includes a smaller return pipe in the center of the pressure pipe. Eductors can pump both air and water. A variation of this type of wellpoint is an eductor well, which uses a two-pipe eductor inside a well screen, usually 4-inches in diameter (Figure 14). The advantage of the jet-eductor well is that the entire water-bearing layer can be screened, and a vacuum can be developed inside the well casing for the full length of screen above the pumping water level if the annular space around the riser pipe is sealed against a relatively impervious overlying stratum with clay, grout, or bentonite chips.

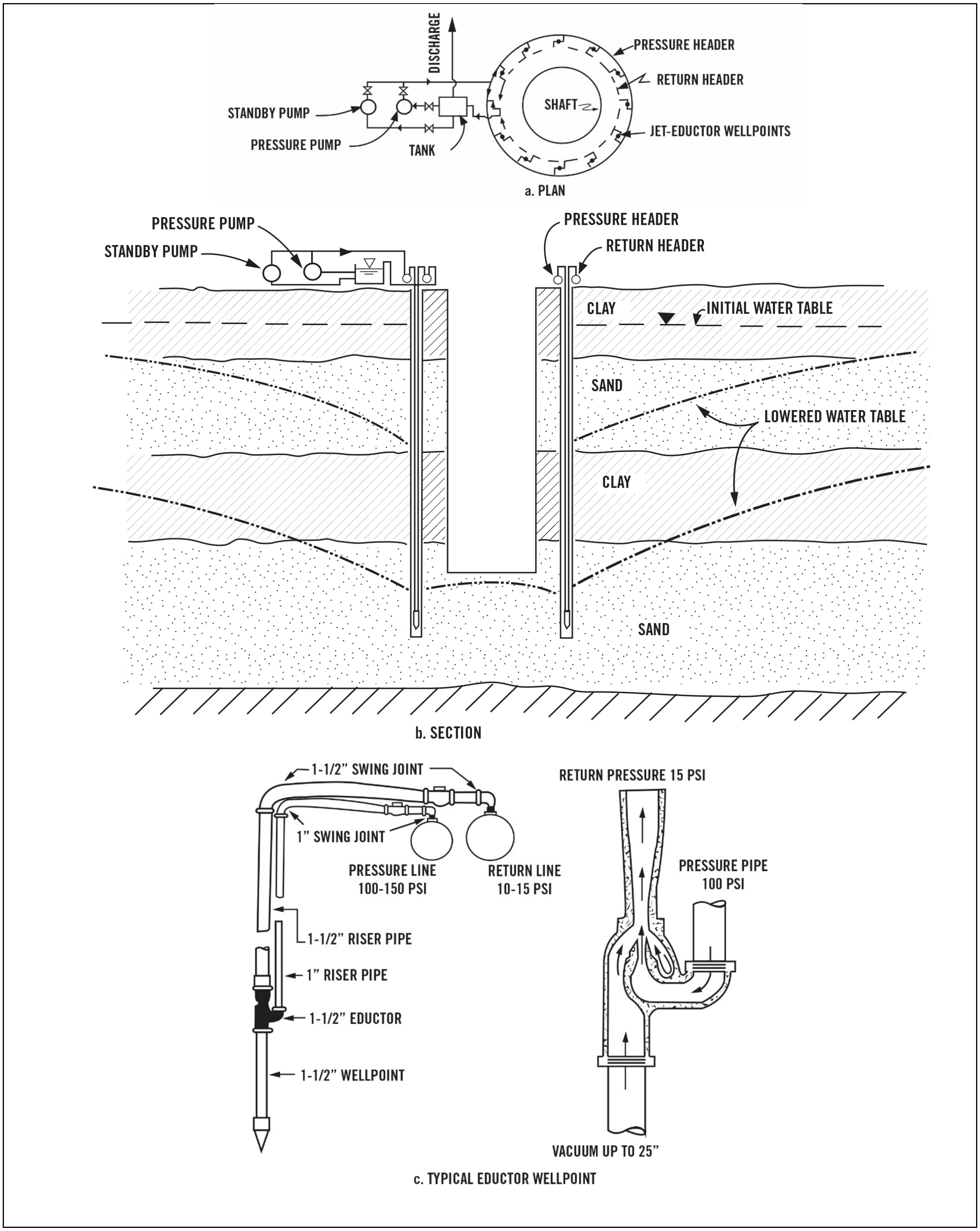


Figure 12. Jet-eductor wellpoint system for dewatering a shaft (Adapted from TM 5-818-5)

Figure 13. Single pipe jet-eductor (Courtesy of Keller)

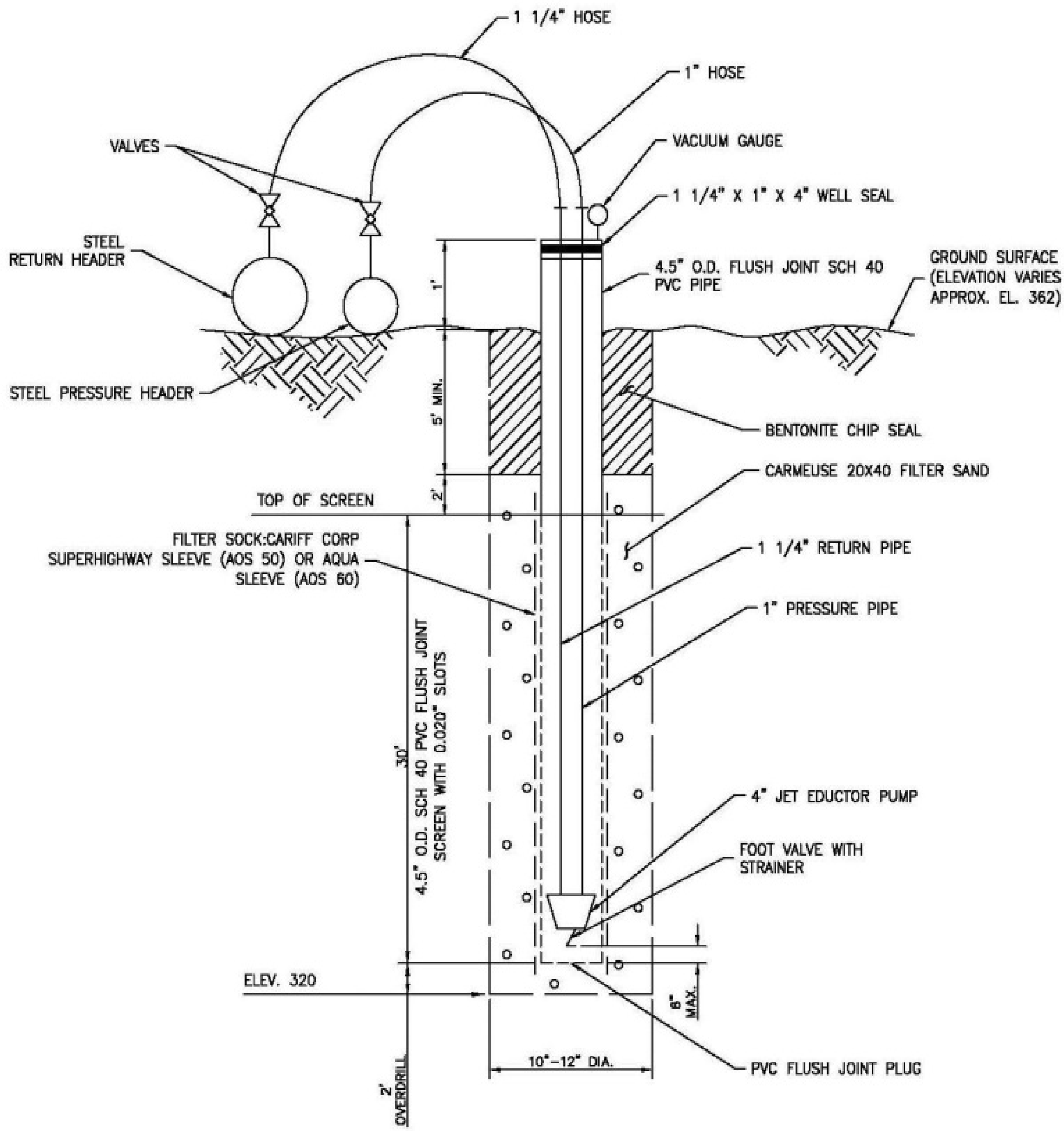


Figure 14. Typical schematic of a jet-eductor well (Courtesy of AECOM)

3.3.2.3.2 Jet-eductor wellpoints are installed in the same manner as conventional wellpoints, generally with a filter as required by the foundation soils, and are typically more expensive to install, operate and maintain. However, an eductor system has the advantage over a conventional wellpoint system of being able to lower the water table as much as 100 feet from the top of the excavation, thus potentially eliminating the need for a multi-stage wellpoint system. Jet-eductor wells are also installed in the same manner as dewatering wells. The pressure and return riser pipes are connected to separate headers, one to supply water under pressure to the eductors and the other for return of flow from the wellpoints and eductors (Figure 12). Jet-eductor well and wellpoint systems are most advantageously used to dewater deep excavations where the volume of water to be pumped is relatively small because of the low hydraulic conductivity of the aquifer and where application of vacuum to the formation is desirable.

3.3.2.4 Deep-well Systems.

3.3.2.4.1 Deep wells can be used to dewater pervious soil or rock formations or to relieve artesian pressure beneath an excavation. They are particularly suited for dewatering large excavations in pervious soils (cobbles, gravel, and sand) requiring high rates of pumping, and for dewatering deep excavations for dams, tunnels, locks, powerhouses, and shafts. Excavations and shafts as deep as 300 feet can be dewatered by pumping from deep wells with lineshaft turbine or submersible turbine pumps. The principal advantage of deep wells is that they can be installed around the periphery of an excavation and thus leave the construction area unencumbered by dewatering equipment, as shown in Figure 15, and the excavation can be predrained for its full depth with one dewatering system. Figure 16 is an aerial view of the replacement outlet works excavation dewatered using a combination of high capacity deep wells and a sheet pile cutoff at Deer Flat Dam near Boise, ID.

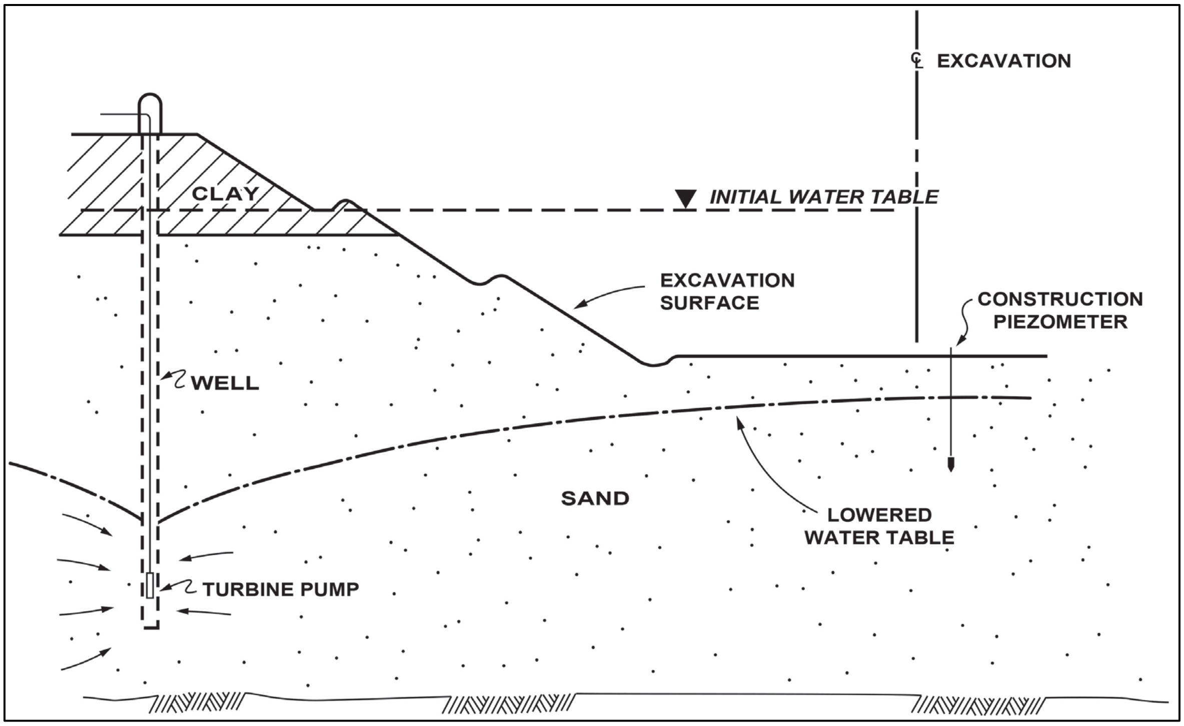


Figure 15. Deep-well system for dewatering an excavation in sand (Adapted from TM 5-818-5)

Figure 16. Aerial view of excavation for replacement outlet works for Deer Flat Dam, Boise, ID (Courtesy of Reclamation Project Files)

3.3.2.4.2 Deep wells for dewatering are similar in type and construction to commercial, municipal, and irrigation water supply wells. They commonly have a screen with a diameter of 6 to 24 inches with lengths up to 300 feet and are generally installed with a filter around the screen to prevent the infiltration of foundation materials into the well and to improve the yield of the well.

3.3.2.4.3 Deep wells may be used in conjunction with a vacuum system to dewater small, deep excavations for tunnels, shafts, or caissons sunk into relatively fine-grained or stratified pervious soils or rock below the groundwater table. The addition of a vacuum to the well screen and filter will increase the hydraulic gradient to the well and will create a vacuum within the surrounding soil that will prevent or minimize seepage from perched water into the excavation. Installations of this type, as shown in Figure 17, require adequate vacuum capacity to handle air flowing into the pervious formations or into the well filter annulus from the ground surface or the face of the adjacent excavations.

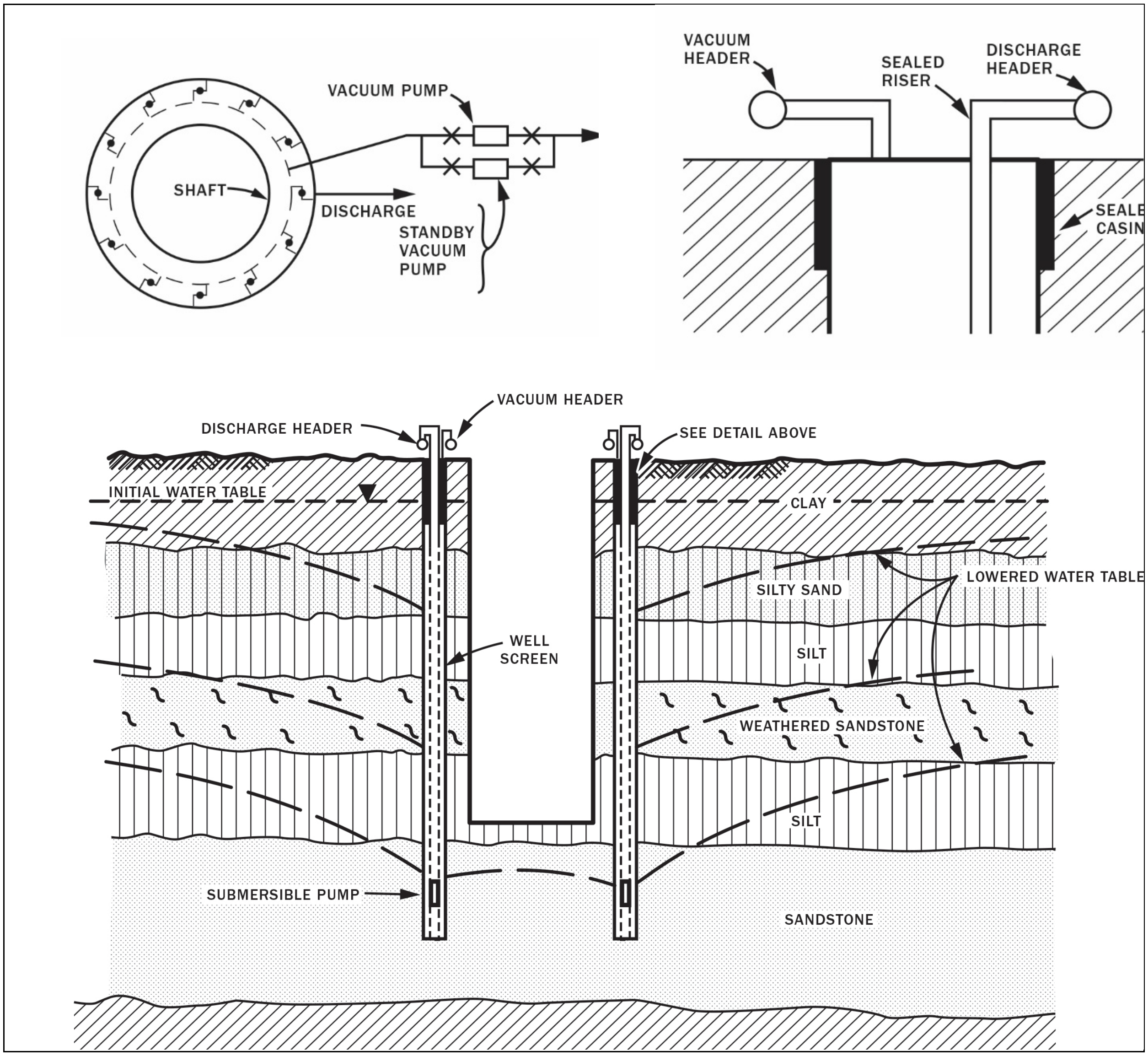


Figure 17. Deep wells with auxiliary vacuum system for dewatering a shaft in stratified materials (Adapted from TM 5-818-5)

3.3.2.5 Other Pre-Drainage Systems.

3.3.2.5.1 Drainage Trenches with Perforated Collector Pipe and Pumps. In some areas of the United States, especially Florida, drainage trenches (with or without aggregate backfill) with perforated collector pipes installed below planned excavation depths, have proved effective as a construction dewatering method for relatively shallow structures (pipelines and other structures installed in similar linear excavations) in mostly pervious foundation materials. This method employs a continuous trenching machine equipped with a trailing shield to install a flexible perforated collector pipe encased in a woven geotextile sock. A wellpoint or centrifugal pump is connected to the collector pipe where it emerges from the ground to pump groundwater from the collector pipe and lower the groundwater level below planned subgrade level. Because of the pumping system, this method has the same drawdown limitation as a conventional wellpoint system (i.e., about 15 feet below the pump suction elevation). Figure 18 shows a typical trencher equipped to install a sock-encased perforated collector pipe. Collector pipe diameter is typically limited to about 8 inches for high density polyethylene (HDPE), although HDPE pipe as large as 24 inches in diameter has reportedly been installed to a depth of about 10 feet. The maximum trenching depth and pipe diameter depend greatly upon the bending radius of the pipe to be installed. Although the drawdown is limited by the suction lift limitations of the pump and is generally used for shallow structures, trenches can be installed 40+ feet deep, depending on the design of the trenching machine and the bending radius of the collector pipe.

Figure 18. Trencher installing sock-encased 5-inch diameter extra-heavy perforated corrugated HDPE pipe 20 feet deep (Courtesy of DeWind Trenching)

3.3.2.5.2 Vertical Drains. Where a stratified semi-pervious stratum with a low vertical hydraulic conductivity overlies a pervious stratum and the groundwater table has to be lowered in both strata, the water table in the upper stratum can be lowered by means of vertical drains, examples of which are shown in Figures 19 and 20.

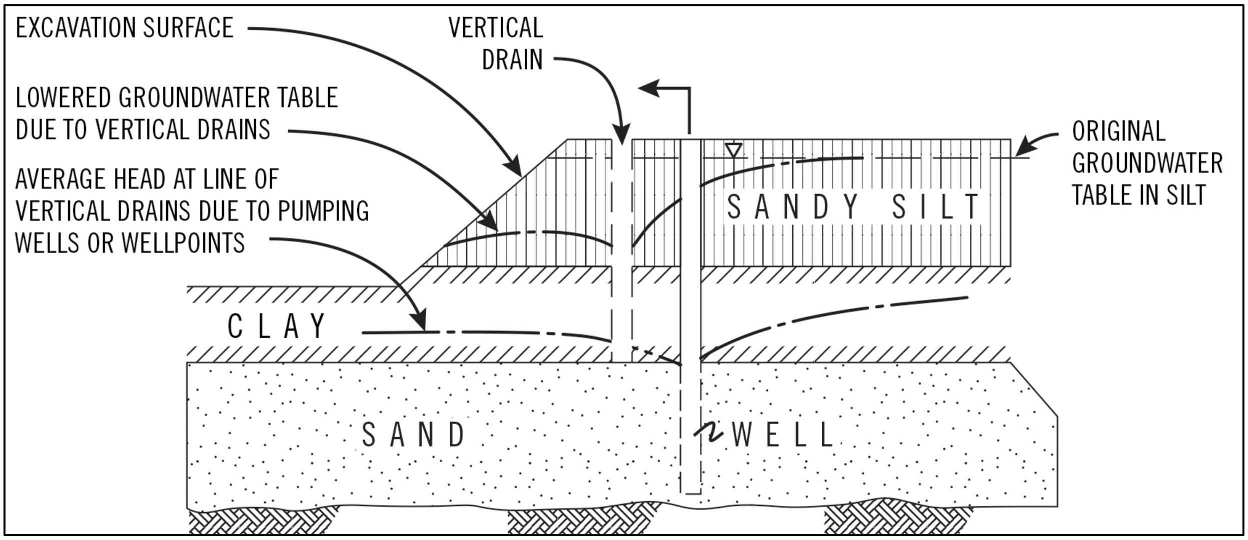


Figure 19. Vertical drains for dewatering a slope (Adapted from TM 5-818-5)

Figure 20. Installing 3-inch diameter drain using hollow tube advanced with vibratory hammer. Primary purpose of this drain was for rapid dissipation of excess pore pressures during an earthquake. (Courtesy of Keller)

3.3.2.5.3 If properly designed and installed, vertical drains will intercept seepage in the upper stratum and conduct it into the lower, more permeable stratum being dewatered with deep wells or wellpoints. Vertical drains may consist of a column of pervious sand placed in a cased hole, either driven or drilled through the soil, with the casing subsequently removed, or a prefabricated vertical drain (PVD), commonly known as a wick drain, which consists of a plastic strip with molded channels wrapped in a geotextile. Sand column vertical drains typically have a diameter of 12 to 18 inches and are spaced from 15 to 20 feet apart depending on the thickness of the perched water layer and undulations in the top elevation of the perching clay. PVDs are installed using a mandrel that is vibrated or pushed into the ground. The capacity of sand drains can be significantly increased by installation of a slotted 1- or 2-inch diameter pipe in the sand drain to conduct the water down to the more pervious stratum. If the anticipated flow for each vertical drain is low enough, prefabricated wick drains that are used to accelerate consolidation of soft clay may have adequate vertical flow capacity for use in dewatering.

3.3.2.5.4 Electro-osmosis. Some soils, such as clay-silt-sand mixtures, cannot always be dewatered by pumping from wellpoints or deep wells. However, such soils can usually be drained by impressing a direct current electrical field using anodes and cathodes installed in the soil. The electrical current through the soil causes ions in the water contained in the soil voids to migrate from the positive electrode (anode) to the negative electrode (cathode). By making the cathode a wellpoint, the water that migrates to the cathode can be removed by either vacuum or eductor pumping (Figure 21).

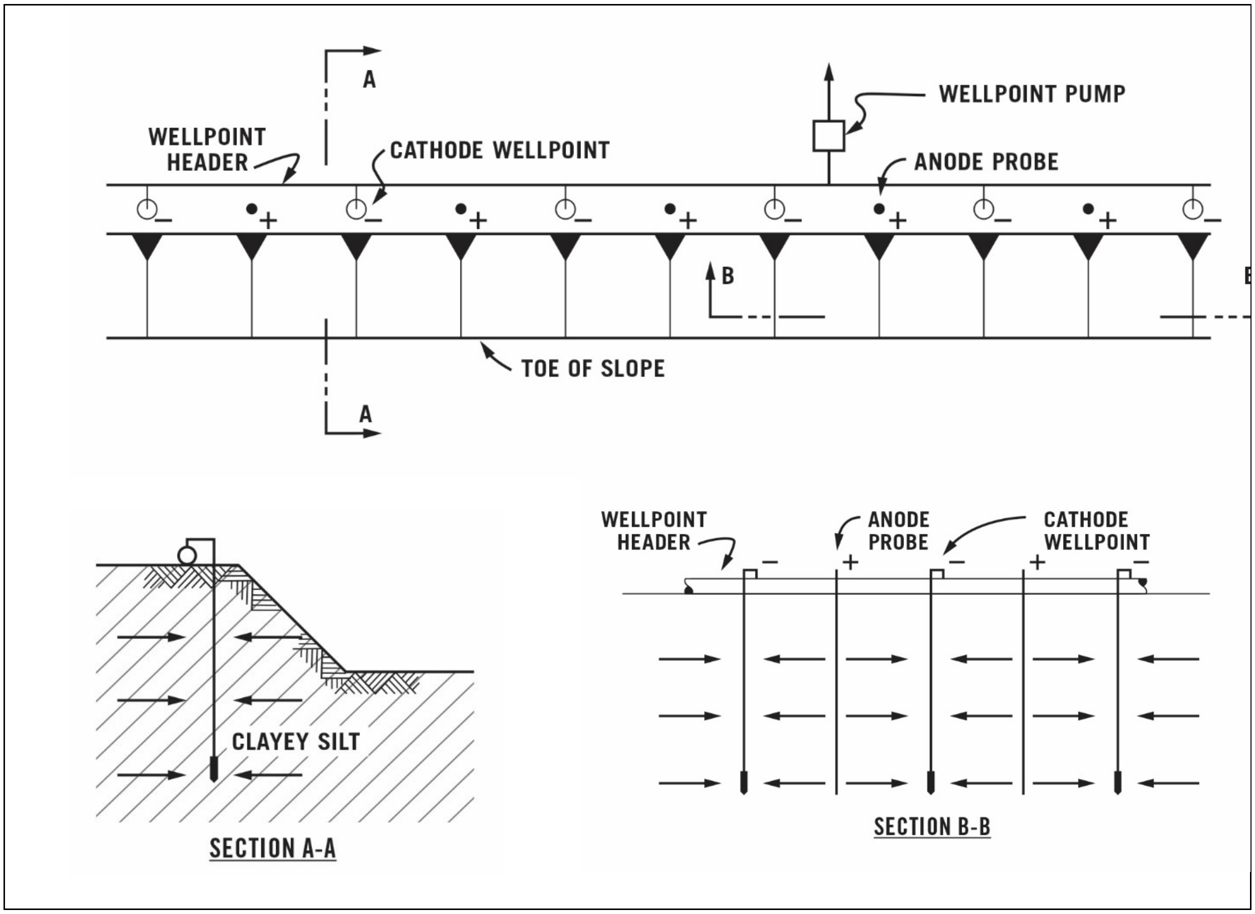


Figure 21. Electro-osmotic wellpoint system for stabilizing an excavation slope (Adapted from TM 5-818-5)

3.3.3 Cutoffs and Bottom Seals. Cutoff curtains can be used to reduce[4] seepage into an excavation where the cutoff can be installed down to an impervious formation. Such cutoffs can be constructed by driving steel sheet piling, grouting existing soil with cement or chemical grout, excavating by means of a slurry trench and backfilling with a plastic mix of bentonite and soil, installing a concrete or mixed soil wall (such as a secant pile wall or jet-grouted columns), or

[4] Note that in practice, cutoffs are rarely 100% effective in stopping flow, due to seepage through the barrier itself, an incomplete seal at the bottom of the barrier, or an inaccurate assessment of the hydraulic conductivity of the stratum into which the barrier extends

freezing. Refer to Reclamation's Design Standard No. 13 Chapter 16 on Cutoff Walls (2014) for details regarding design and construction of various types of cutoff walls. A new USACE Engineer Manual for the design and construction of cutoff walls is pending publication as of February 2020. However, groundwater within the area enclosed by a cutoff curtain, or leakage through or under such a curtain, will have to be pumped out with a deep well or wellpoint system as shown in Figure 22. Bottom seals are used in conjunction with rigid watertight shoring (usually steel sheet piles) to prevent vertical seepage into an excavation and to resist hydrostatic uplift pressures in pervious strata underlying the seal by a combination of weight and transfer of uplift forces to vertical piles and/or to the soil mass adjacent to the excavation through friction and arching. Types of bottom seals include tremie concrete seals and jet grouting. Figure 23 schematically illustrates a jet-grout bottom seal and anchorage system successfully completed beneath a sheet pile cofferdam for an excavation at a power plant near Jacksonville, FL.

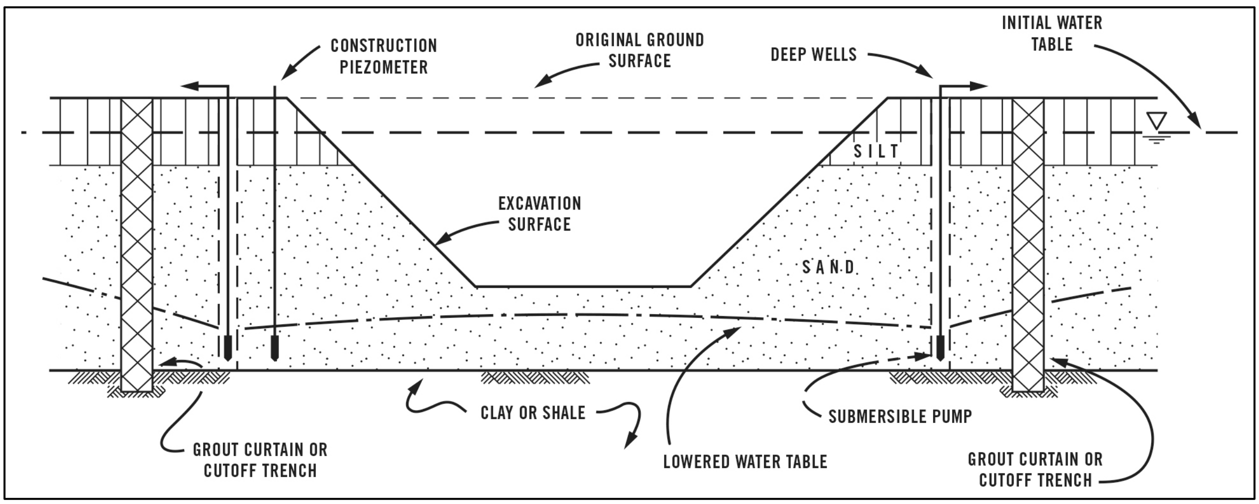


Figure 22. Grout curtain or cutoff trench around an excavation (Adapted from TM 5-818-5)

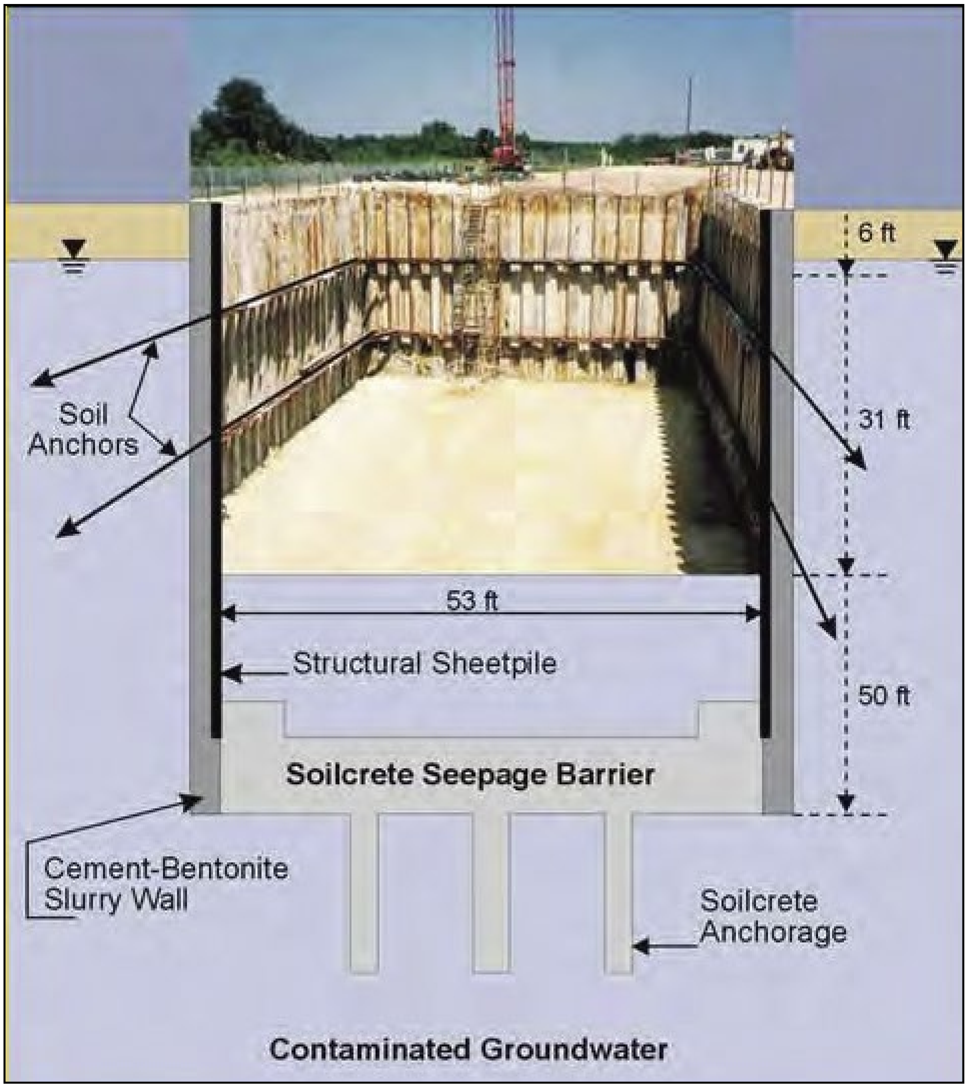


Figure 23. Jet-grout bottom seal and anchorage system for sheet pile cofferdam near Jacksonville, FL (Courtesy of Keller)

3.3.3.1 Cement and Chemical Grout Curtains. A cutoff around an excavation in coarse sand and gravel or porous rock can be created by injecting cement or chemical grout into the voids of the soil or rock. For grouting to be effective, the voids in the rock or soil must be large enough to accept the grout and the grout holes must be close enough together so that a continuous grout curtain is created. The type of grout depends upon the size of voids in the sand and gravel or rock to be grouted. Grouts commonly used for this purpose are Portland cement and water; cement, bentonite, an admixture to reduce surface tension, and water; silica gels; or a commercial product. Generally, grouting of fine or medium sand is not very effective in blocking seepage. Single lines of grout holes are also generally ineffective as seepage cutoffs;

typically, at least two lines are used. Detailed information on chemical grouting and grouting methods is contained in EM 1110-2-3506.

3.3.3.2 Slurry Walls. A cutoff to prevent or minimize seepage into an excavation can also be formed by digging a narrow trench around the area to be excavated and backfilling it with an impervious soil. Such a trench can be constructed in almost any soil, either above or below the water table, by keeping the trench filled with a bentonite slurry and backfilling it with a suitable impervious soil. Generally, the trench is backfilled with a well-graded clay–sand-gravel mixed with bentonite slurry. Another type of slurry wall that requires less space to construct is a cement-bentonite wall, in which the slurry hardens and becomes the trench backfill.

3.3.3.3 Concrete, Jet-grouted and Mixed Soil Walls. Techniques have been developed for constructing concrete, plastic concrete, jet-grouted soil, and mixed soil cutoff walls by overlapping cylinders or columns and also as continuous walls excavated and concreted in panels or mixed-in-place. Continuous trenching machines have proved effective in constructing mixed-in-place cutoff walls (so-called “one-pass” cutoff walls) by adding dry bentonite and water or slurry to the soil as the trencher progresses. The maximum depth of such trenches is limited by the design of the equipment used, but is typically up to 50 feet, although contractors are continually developing new equipment to meet the requirements of new projects. Specially designed larger machines, especially those used for deep mixing methods (DMM), can install mixed-in-place walls much deeper than 50 feet. Figure 24 shows a trenching machine constructing a 2-foot thick mixed-in-place soil-bentonite cutoff wall. Concrete walls can be reinforced and are sometimes incorporated as a permanent part of a structure.

Figure 24. Trencher installing mixed-in-place cutoff wall (Courtesy of DeWind Trenching)

3.3.3.4 Steel Sheet Piling. The effectiveness of sheet piling driven around an excavation to reduce seepage depends upon the perviousness of the soil, the tightness of the interlocks, and the length of the seepage path. Some seepage through the interlocks should be expected. Some seepage reduction may be achieved by using various sealants in the interlocks prior to driving the sheetpiles. When constructing structures in open water (e.g., a bridge pier in a river), it may be desirable to drive steel sheet piling around the structure, excavate the soil underwater, and then tremie in a concrete seal. The concrete tremie seal must withstand uplift pressures or pressure relief measures must be used. In restricted areas, it may be necessary to use a combination of sheeting and bracing with deep wells or wellpoints installed just inside or outside of the sheeting. Sheet piling is not very effective in blocking seepage where boulders or other hard obstructions may be encountered because of driving out of interlock or inability to drive the piling through the obstructions.

3.3.4 Other Groundwater Control Methods. Seepage into an excavation or shaft can be prevented by freezing the surrounding soil. Frozen soil can also be designed as part of the support for the excavation. However, freezing is expensive and requires careful engineering design, installation and operation by an experienced contractor. If the soil around the excavation is not completely frozen, seepage can cause rapid enlargement of an unfrozen zone, which is difficult to remedy. Freezing is most advantageous in fine-grained soils that are difficult or impractical to dewater. For a comprehensive discussion of the design, installation, and operation of ground freezing systems see Powers et al. (2007).

3.4 Summary of Groundwater Control Methods. A brief summary of groundwater control methods discussed in this section is given in Table 1.

Table 1
Summary of Groundwater Control Methods

Method	Applicability	Remarks
Sumps and ditches	Collect water entering an excavation or structure.	Generally, water level can be lowered only a few feet. Used to collect water within cofferdams and excavations. Sumps are usually only successful in relatively stable gravel or well-graded sandy gravel, partially cemented materials, or porous rock formations.
Conventional wellpoint system	Dewater soils that can be drained by gravity flow.	Most commonly used dewatering method. Drawdown limited to about 15 feet per stage (less at high elevations); however, several stages may be used. Can be installed quickly.
Vacuum wellpoint system	Dewater or stabilize soils with low hydraulic conductivity. (Some silts, sandy silts).	Vacuum increases the hydraulic gradient causing flow. Little vacuum effect can be obtained if lift is more than 15 feet.
Jet-eductor wellpoints and wells	Dewater both soils that can be drained by gravity flow and soils with low hydraulic conductivity. Usually for deep excavations where small flows are required.	Can lower water table as much as 100 feet from top of excavation. Jet-eductors are particularly suitable for dewatering shafts and tunnels. Two header pipes and two riser pipes, or a pipe within a pipe, are required.
Deep-well systems	Dewater soils that can be drained by gravity flow. Usually for large, deep excavations where large flows are required.	Can be installed around periphery of excavation, thus removing dewatering equipment from within the excavation. Deep wells are particularly suitable for dewatering shafts and tunnels. Can improve interception of perched water by sealing wells and adding a vacuum pumping system.
Vertical drains	Usually used to conduct water from an upper stratum to a lower more pervious stratum.	Sand drains not effective in highly pervious soils. Vertical flow capacity can be greatly improved using concentric slotted well screen in sand drains or by using prefabricated vertical drains.
Electro-osmosis	Dewater soils that cannot be drained by gravity. (Some silts, clayey silts, and clay-silt-sand mixtures).	Direct electrical current causes ions in groundwater to migrate toward cathodes. Generally, very expensive and requires expert design, installation and operation.
Cutoffs and bottom seals	Cutoffs minimize seepage into an excavation when installed down to an impervious stratum or used in combination with a bottom seal.	When cutoff is part of a cofferdam, a bottom seal can be used to stop vertical seepage and resist hydrostatic uplift. Cutoffs walls have been used in cases where settlement due to dewatering had the potential to damage adjacent structures. Typically, cutoffs are more expensive than dewatering systems. Additional details about cutoff walls will be available in a new USACE engineer manual on cutoff walls, pending publication.
Ground freezing	Stops seepage when installed down to an impervious stratum or when installed horizontally surrounding a tunnel.	Most advantageous in less pervious soils that are difficult to dewater. Generally, very expensive and requires careful engineering design, installation and operation. Frozen soil mass can be designed as part or all of the excavation support. Inappropriate if there is high flow across the site in permeable strata. Some problems have been reported of damage to soils and adjacent structures when frozen soils thawed.

3.5 Selection of Dewatering System.

3.5.1 General. The method most suitable for dewatering an excavation depends upon the location, type, size, and depth of the excavation; thickness, stratification, and hydraulic conductivity of the foundation soils below the water table into which the excavation extends or which underlie the excavation; potential damage resulting from failure of the dewatering system; and the cost of installation and operation of the system. The cost of a dewatering method or system will depend upon:

a. Type, size, and pumping requirements of the project.

b. Type and availability of power.

c. Labor requirements to install, maintain, and operate the system.

d. Duration of required pumping.

e. Treatment requirements of pumped water.

3.5.2 Factors Controlling Selection. Factors that control the selection of open pumping dewatering systems (sumps and ditches) are described in Powers et al. (2007) and are shown in Tables 2 and 3.

Table 2

Conditions Favorable to Open Pumping (Adapted from Powers et al. 2007)

Condition	Explanation
Soil Characteristics	
Dense, well-graded granular soils, especially those with some degree of cementation or cohesive binder	Such soils are low in hydraulic conductivity and seepage is likely to be low to moderate in volume. Slopes can bleed reasonable quantities of water without becoming unstable. Lateral seepage and boils in the bottom of an excavation will often become clear in a short time, avoiding the transport of excessive fines from soils so that foundation properties are not impaired.
Stiff clays with no more than a few lenses of sand, which are not connected to a significant water source	Only small quantities of water can be expected from the sand lenses, and it should diminish quickly to a negligible value. No water is expected from the clay.
Hard fissured rock	If the rock is hard, even moderate to large quantities of water can be controlled by open pumping, as in typical quarry operations. (For soft rock and rock with blocked fissures, see guidance in Powers et al. (2007))
Hydrology Characteristics	
• Low to moderate dewatering head • Remote source of recharge • Low to moderate hydraulic conductivity • Minor storage depletion	These characteristics indicate that groundwater seepage will be low, minimizing problems with slope stability and subgrade deterioration, and facilitating the construction and maintenance of sumps and ditches.

Table 3

Conditions Unfavorable to Open Pumping (Predrainage or Cutoff Usually Advisable) (Adapted from Powers et al. 2007)

Condition	**Explanation**
Soil Characteristics	
Loose, uniform granular soils without plastic fines	Such soils have moderate to high hydraulic conductivity and are very sensitive to seepage pressures. Slope instability and loss of strength at subgrade are likely when open pumping.
Cohesionless silts, and soft clays or cohesive silts with moisture contents near or above the liquid limit	Such soils are inherently unstable, and slight seepage pressures in permeable lenses can trigger massive slides.
Soft rock; rock with large fissures filled with granular soft soils, erodible materials or soluble precipitates; sandstone with uncemented sand layers	If substantial quantities of water are open pumped, soft rock may erode. Soft materials in the fissures of hard rock may be leached out. Uncemented sand layers can wash away. The quantity of water may progressively increase, and massive blocks of rock may shift.
Hydrology Characteristics	
• Moderate to high dewatering head • Proximate source of recharge • Moderate to high hydraulic conductivity	These characteristics indicate the potential for high water quantities. Even well-graded gravels can become quick if the seepage gradient is high enough. Problems with construction and maintenance of ditches and sumps are aggravated.
Large quantity of storage water	If the aquifer to be dewatered is high in hydraulic conductivity and porosity, large quantities of water from aquifer storage must be expected during the early phase of lowering the water table. This higher flow can greatly aggravate problems with open pumping. With predrainage, pumping can be started some weeks or months before excavation, the pumping rate will decrease, and the problem can be mitigated.
Artesian pressure below subgrade	Open pumping cannot cope with pressure from below subgrade since, if water reaches the excavation, damage from heave or piping has already occurred. Predrainage with relief wells is advisable.

3.5.3 Predrainage. Where foundations must be constructed on soils below the groundwater level, it will generally be necessary to dewater the excavation by means of predrainage systems rather than by trenching and sump pumping. Predrainage is defined as lowering the groundwater table in an unconfined aquifer to below the planned bottom of an excavation before excavating below the groundwater level or relieving hydrostatic pressure in a confined aquifer below the bottom of an excavation to a safe level before excavation. Predrainage methods typically include deep wells or wellpoints installed around the perimeter of an excavation and are pumped before excavating below the groundwater level. Dewatering by predrainage methods is usually essential to prevent damage to foundation soils caused by equipment operations and sloughing or sliding of the side slopes or bottom heave due to unrelieved hydrostatic pressure.

3.5.4 System Design. Conventional deep-well and wellpoint systems designed and installed by companies specializing in this work are generally satisfactory, and therefore would not require a detailed design to be prepared by the owner or owner's engineer. However, the contract documents should include a specification requiring submittal of a detailed design by the specialty contractor for review by the owner. Where unusual pressure relief or dewatering requirements must be achieved, or when dam safety, public safety, or schedule constraints are critical, the engineer should make detailed analyses and design the dewatering system or specify in the contract documents the detailed results to be achieved. Alternatively, the contractor can design these critical systems provided that the specifications include design requirements (e.g., only specialty dewatering contractors with a minimum number of years of dewatering design experience on similar projects, prepared by a registered professional engineer), with the owners' engineer preparing an independent check of the contractor's design. The owner's engineer should have dewatering design and construction experience in order to adequately review the contractor's proposed design. For projects that have life safety potential, the owner should require that the contractor's proposed design be reviewed by a dewatering specialist with extensive dewatering experience. Where unusual equipment and procedures are required to achieve desired results, they should be described in detail in the contract documents. Major factors affecting selection of dewatering and groundwater control systems are discussed in the following sections.

3.5.5 Potential for Hydraulic Fracturing During Installation. If hydraulic fracturing during installation cannot be tolerated, methods of dewatering or their installation will have to be changed to preclude this. When designing a dewatering system that involves drilling in or near an earthfill dam or levee, ER 1110-1-1807 (Drilling in Earth Embankment Dams and Levees) must be followed for USACE earthen dams and levees.

3.5.6 Type of Excavation. Small open excavations, or excavations where the depth of water table lowering is small, can generally be dewatered most economically and safely by means of a conventional wellpoint system. If the excavation requires that the water table or artesian pressure be lowered more than 20 to 30 feet, a system of jet-eductor type wellpoints or deep wells may be more suitable. Either wellpoints, deep wells, or a combination thereof can be used to dewater an excavation. Excavations for deep shafts, caissons, or tunnels that penetrate stratified pervious soil or rock can generally best be dewatered with either a deep-well system (with or without an auxiliary vacuum) or a jet-eductor wellpoint system depending on the soil formation and required rate of pumping, but cutoff walls and ground freezing should be evaluated as alternative procedures. Other factors relating to selection of a dewatering system are interference of the system with construction operations, space available for the system, sequence of construction operations, durations of dewatering, and cost of installation and operation.

3.5.7 Subsurface Conditions.

3.5.7.1 The geologic and soil formations at a site and their positions relative to planned subgrade will govern the type of dewatering or drainage system to be used. If the soil below the water table is a thick, more or less homogeneous, free-draining sand extending relatively deep below planned subgrade, it can be effectively dewatered with either a conventional deep well or wellpoint system. If, on the other hand, the formation is highly stratified, or the saturated soil to be dewatered is underlain by an impervious stratum of clay, shale, or rock either above or immediately below the planned subgrade, wellpoints or deep wells on relatively close centers may be required. Where soil and groundwater conditions require only the relief of artesian pressure beneath an excavation, pressure relief can be accomplished by means of relatively few deep wells or jet-eductor wells or wellpoints installed around and at the top of the excavation.

3.5.7.2 For relatively thin or stratified aquifers, wellpoints may be preferred. For thick aquifers, a few long-screened deep-wells may be preferred to installing numerous wellpoints that don't penetrate the aquifer deep enough. If wellpoints are used in thick aquifers, the length of the wellpoints should be increased, and the wellpoints set deep into the aquifer and surrounded by a high-capacity filter.

3.5.7.3 The perviousness and drainability of a soil or rock may dictate the general type of dewatering system to be used for a project. A guide for the selection of a dewatering system related to the grain size of soils is presented in Figure 25. Some gravels and rock formations may be so permeable that a cutoff wall, or ground freezing, may be necessary to reduce the rate of flow to the dewatering system. Drainage of sandy silts and silts will usually require jet-eductor wells, the application of vacuum to deep well systems, or wellpoint dewatering systems. Freezing or the use of the electro-osmotic method of dewatering may be necessary for clayey silt and clay. However, where thin sand layers are present, such special dewatering methods may be unnecessary. Electro-osmosis or freezing should not be used until a test of a conventional system of wellpoints, wells with vacuum, or jet-eductor wells or wellpoints has been attempted.

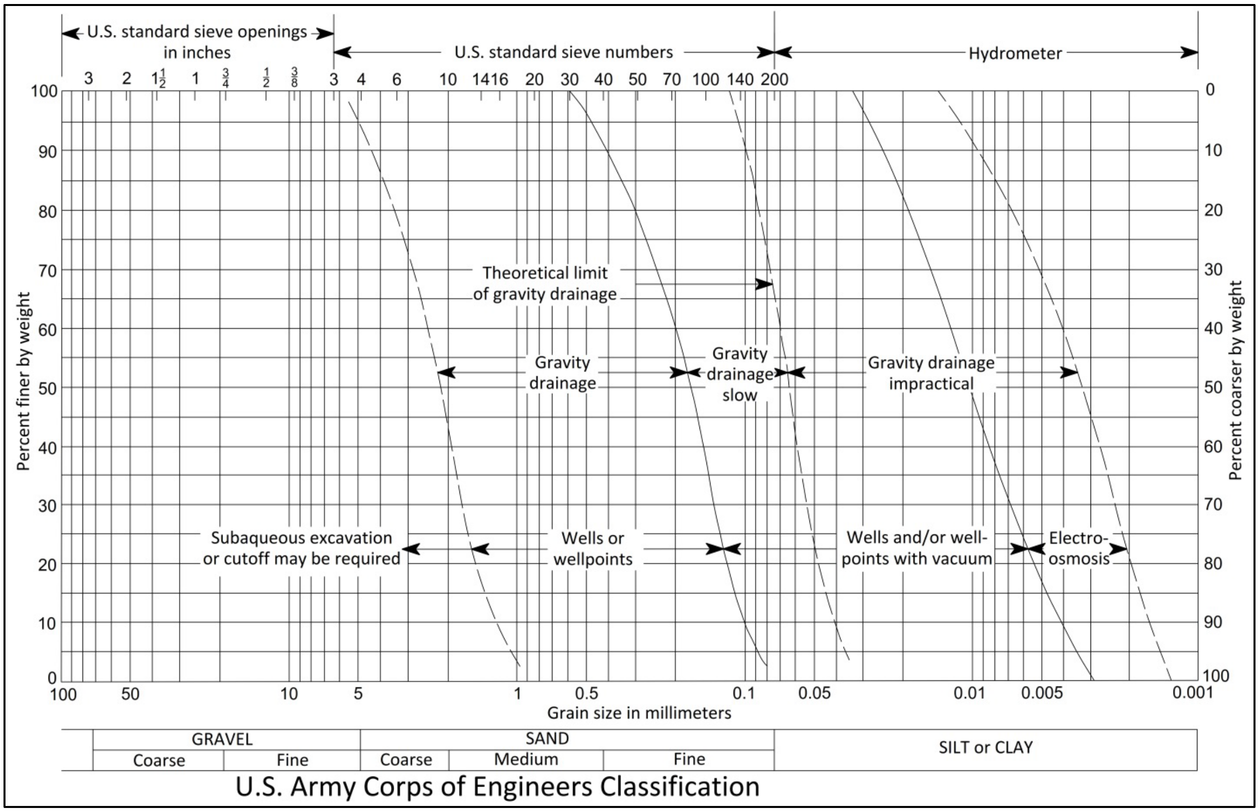


Figure 25. Dewatering systems applicable to different soils (Courtesy of Keller). Ground freezing is not shown on this figure since it can be used essentially for the full range of soils shown in this figure.

3.5.7.4 Table 4 includes selection criteria from Powers et al. (2007) for pre-drainage systems based on soil type, anticipated flow rates, construction schedule and excavation depths. This table also includes typical well and wellpoint spacings and estimated ranges of pump flow rates.

Table 4

Checklist for Selection of Predrainage Methods (Adapted from Powers et al. 2007)

Conditions	Wellpoint Systems	Vacuum Wells	Deep Wells	Jet Eductor Systems
Soil				
Silty and clayey sands	Good	Poor	Poor to fair	Good
Clean sands and gravels	Good	Good	Good	Poor
Stratified Soil	Good	Poor	Poor to fair	Good[a]
Clay or rock at subgrade	Fair to good	Poor	Good	Fair to good
Hydrology				
High hydraulic conductivity	Good	Good	Good	Poor
Low hydraulic conductivity	Good	Poor	Poor to fair	Good
Proximate recharge	Good	Poor	Poor	Poor to good
Remote recharge	Good	Good	Good	Good
Schedule				
Rapid drawdown	OK	OK	Unsatisfactory	OK
Slow drawdown	OK	OK	OK	OK
Excavation				
Shallow (<20 feet below water table	OK	OK	OK	OK
Deep (>20 feet below water table	Multiple stages required	Multiple stages required	OK	OK
Cramped	Interferences	Interferences	OK	OK
Characteristics				
Normal spacing	5-10 feet (1.5-3 m)	20-40 feet (6-12 m)	>50 feet (>15 m)	10-20 feet (3-6 m)
Range of capacity				
Per unit	0.1-25 gpm (0.4-95 L/min)	50-600 gpm (190-2270 L/min)	0.1-3000 gpm (0.4-11360 L/min)	0.1-40 gpm (0.4-150 L/min)
Total system	Low-5000 gpm (Low-18930 L/min)	2000-25,000 gpm (7570-94635 L/min)	Low-60,000 gpm (Low- 227125 L/min)	Low-1000 gpm (Low-3785 L/min)
Efficiency with accurate design	Good	Good	Fair	Poor
[a]Double pipe eductors with wellscreen full length				

3.5.8 Potential Adverse Impacts on Adjacent Structures and Facilities.

3.5.8.1 Where unacceptable surface settlement and/or downdrag forces on nearby deep foundations due to increases in effective stress on compressible soil strata will be caused by lowering the groundwater level, either groundwater recharge and/or isolation of the excavation from groundwater by cutoff and bottom seal methods will be necessary. Groundwater recharge is rarely employed in practice, and it is neither simple nor inexpensive to accomplish.

3.5.8.2 Lowering the groundwater table increases the load on foundation soils below the original groundwater table. As most soils consolidate upon application of additional load, structures, pavements and utilities located within the radius of influence of a dewatering system may settle and be damaged as a result of settlement or downdrag forces on deep foundations. The risk that settlement will occur due to dewatering is reduced if the deposits have been preconsolidated or have been previously dewatered. These factors should be carefully considered by the project geotechnical engineer to evaluate settlement and downdrag before a dewatering system is specified. Establishing and surveying reference points on adjacent structures, utilities and pavements prior to the start of dewatering operations will permit measuring any settlement that occurs during dewatering and provides a warning of possible distress or failure of a structure, utility or pavement that might be affected. Methods of surveying and measuring parameters related to concrete dams, such as joint movement, uplift pressure, strain, stress, and leakage are outside the scope of this manual but are presented in the following USACE publications:

a. EM 1110-1-1002, Survey Markers and Monumentation

b. EM 1110-1-1003, NAVSTAR Global Positioninga System Surveying

c. EM 1110-2-1009, Structural Deformation Surveying

d. EM 1110-2-1908, Instrumentation and Monitoring of Embankment Dams and Levees

e. EM 1110-2-4300, Instrumentation for Concrete Structures.

3.5.8.3 Recharge of the groundwater, as illustrated in Figure 26, may be necessary to reduce or eliminate distress to adjacent structures, or it may be necessary to use positive cutoffs and excavation bottom seals to avoid lowering the groundwater level outside of an excavation. As a rule of thumb, twice as many wells are needed to recharge water than are used to extract water. In addition, recharge water is typically required to be very clean. Most potable filter systems will produce total suspended solids that are too high; therefore, the recharge water will need to be filtered through a fine filter medium (typically down to 1 micron). Recharge is usually inherently more difficult and expensive than dewatering and requires careful design, installation and operation by an experienced specialist.

3.5.8.4 Existing water supply wells may also be adversely affected if a dewatering system is installed and operated in an aquifer that is also used for water supply.

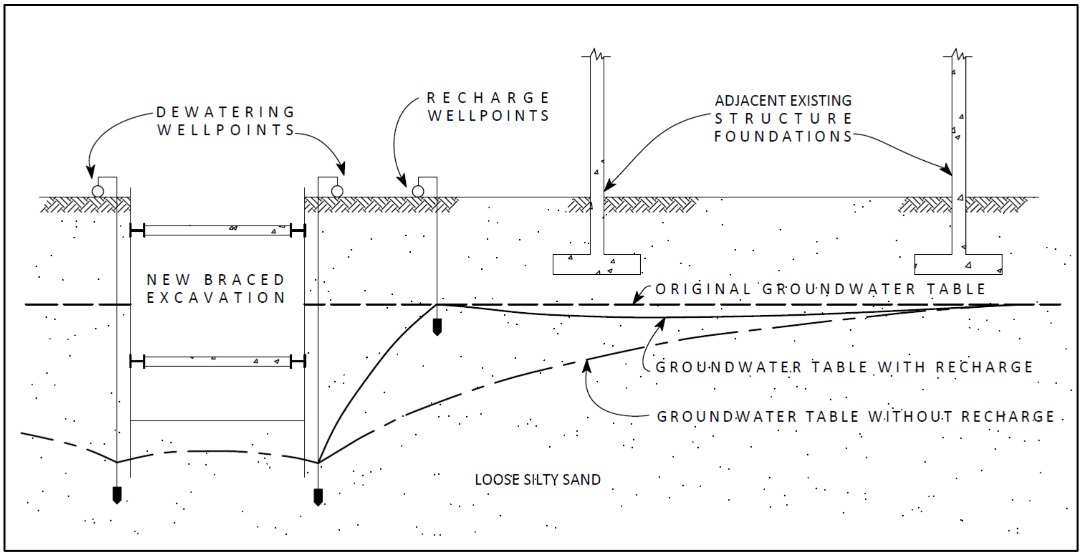


Figure 26. Recharge of groundwater to prevent settlement of a building as a result of dewatering operations (Adapted from TM 5-818-5)

3.5.9 Depth of Groundwater Lowering. The magnitude of the drawdown required is an important consideration in selecting a dewatering system. If the drawdown required is large, deep wells or jet-eductor wells or jet-eductor wellpoints may be the best option because of their ability to achieve large drawdowns from the top of an excavation, whereas many stages of conventional wellpoints would be required to accomplish the same drawdown. Deep wells can be used for a wide range of flows by selecting pumps of appropriate size, but jet-eductor wells and wellpoints are not as flexible. Since jet-eductor pumps are relatively inefficient, they are most applicable where well flows are small, as could be expected in silt to silty fine sand formations.

3.5.10 Reliability Requirements and Type of Dewatering Specification.

3.5.10.1 The reliability of groundwater control required for a project will have a significant bearing on the design of the dewatering pumps, power supply, and standby power and equipment. If the dewatering problem is one involving the relief of artesian pressure to prevent uplift or heave of the bottom of an excavation, the rate of water table rebound, in event of failure of the system, is in most cases extremely rapid. If an excavation is shored and there are adjacent structures and utilities, a failure of the dewatering system could lead to collapse of the shoring. Sloped excavations could also fail due to heave and loss of toe support. More importantly, for excavations at the toe of a dam or in an embankment dam, failure of the excavation and loss of life can occur. Such a situation may influence the type of pressure relief system selected and require inclusion of 100% standby power with automatic switching. Standby requirements for diesel driven pumps, because each pump has its own power source, are much less and may be covered by specifying a degree of redundancy in the primary pressure relief components of the system. Even if a groundwater control system is designed for high reliability, careful planning of the construction (e.g., limiting the extent of an excavation that can be made before it is backfilled) and requiring the implementation of emergency measures (such as intentional flooding or backfilling of the excavation) may be prudent and necessary. Planning is essential in such cases because such emergency measures require time and resources to implement. If the groundwater flow to the dewatering system for a critical excavation is unconfined, additional time can be obtained for emergency responses by lowering the groundwater level to a greater depth below excavation subgrade, and the amount of time for recovery of the groundwater to subgrade level can be evaluated before the excavation is started by observing the response of a critical piezometer to a system outage. Where an excavation is to be dewatered downstream of an existing dam with a pool that cannot be drained, the flow is confined and rapid recovery time does not allow enough time for flooding or backfilling (e.g., most pressure relief situations), 100% redundancy of all components of the dewatering system (i.e., wells or wellpoints, power, standby power, and discharge piping) is required.

3.5.10.2 Design by the owner’s engineer as opposed to a performance specification for dewatering should be considered:

a. Where safety of an existing dam with a pool that cannot be drained is at stake;

b. On projects where subsurface construction requires dewatering or other groundwater control procedures that are not commonly used by construction contractors;

c. Inadequate dewatering would reduce the competency of the foundation or affect the design of the substructure; and

d. The construction schedule is critical (i.e., when there is no time for contractor trial and error).

e. In cases where a dewatering system is designed by the owner's engineer, it may be desirable to design and specify the equipment and procedures to be used and for the owner to accept responsibility for results obtained. A variation of this approach is to specify a minimum design and hold the contractor responsible for the ultimate performance of the system.

3.5.11 Required Rate of Pumping. The rate of pumping required to dewater an excavation may vary from 5 to 50,000 gallons per minute (gpm) or more. Thus, flow to a drainage system will have an important effect on the design and selection of the wells, pumps, and piping system. See subsequent Chapter 5 for an extensive discussion of the capacities of available pumps and the sizes of wells that pumps fit into, as well as wellpoint pumps. Lineshaft turbine or submersible turbine pumps for pumping deep wells are available in bowl sizes from 4 to 16 inches with capacities ranging from 5 to 5,000 gpm at heads up to 500 feet. Wellpoint pumps are available in suction diameters from 4 to 18 inches with capacities ranging from 250 to 5,000 gpm depending upon suction conditions and discharge heads. Jet-eductor pumps are available that will pump from 3 to 40 gpm for lifts up to 100 feet, although generally it will be more economical to use small submersible pumps when flows are 5 gpm or more. Where soil conditions dictate the use of vacuum or electro-osmotic wellpoint systems, the rate of pumping will be very small. The rate of pumping will depend largely on the distance to the effective source of seepage, amount of drawdown or pressure relief required, and thickness and hydraulic conductivity of the aquifer through which the flow is occurring.

3.5.12 Pumping Versus Other Methods of Groundwater Control.

3.5.12.1 While dewatering is generally the most expeditious and economical procedure for controlling groundwater, it is sometimes possible to excavate more economically in the ‘wet’ inside of a cofferdam or caisson and then seal the bottom of the excavation with a tremie seal, or use a combination of barrier wall or other type of cutoff and dewatering. Where subsurface construction extends to a considerable depth or where high uplift pressures or large flows are anticipated, it may occasionally be advantageous to: (1) substitute a caisson for a conventional foundation and sink it to the design elevation without lowering the groundwater level; (2) use a combination of concrete cutoff walls constructed in slurry-supported trenches, and a tremied concrete foundation slab, in which case the cutoff walls may serve also as part of the completed structure; (3) use large rotary drilling machines for excavating purposes, without lowering the groundwater level; or (4) use freezing techniques. Cofferdams, caissons, and cutoff walls may have difficulty penetrating formations containing numerous boulders. Foundation designs requiring compressed air will rarely be needed, although compressed air may be economical or necessary for some tunnel construction work. The rapid development of slurry and other types of cutoff walls has made this method of groundwater control, combined with a certain amount of pumping, a practical and economical alternative for some projects, especially those where pumping costs would otherwise be great.

3.5.12.2 Powers et al. (2007) provides selection criteria based on foundation conditions for various types of cutoff methods. A USACE Engineer Manual for the design and construction of cutoff walls is pending publication as of February 2020.

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Chapter 4
Investigations

4.1 General. Before selecting or designing a system for dewatering an excavation, it is necessary to consider or investigate subsurface soils, groundwater conditions, power availability, and other factors as listed in Table 5. The extent and detail of these investigations will depend on the effect groundwater and hydrostatic pressure will have on the construction of the project and the complexity of the dewatering problem. Additional investigation of various types, test well(s) and pumping tests may be necessary when aquifer characteristics are unknown or poorly understood and the volume of groundwater to be pumped has a large impact on the cost of dewatering. Defining the position of clay layers where such layers impede vertical drainage to the screens of dewatering devices or are key to designing seepage barriers may also require further investigation.

Table 5

Preliminary Investigations

Item	Investigate	Reference
Geologic and soil conditions	Type, stratification, and thickness of soil involved in excavation and dewatering.	Section 4.2; EM 1110-1-1804
Criticality	Damage to excavation or foundation in event of failure, rate of rebound, etc.	
Groundwater or piezometric pressure characteristics	Groundwater table or hydrostatic pressure in area and its source. Variation with river stage, season of year, etc. Type of seepage (confined, unconfined, combined). Chemical characteristics and temperature of groundwater.	Section 4.3
Hydraulic conductivity	Estimate hydraulic conductivity and transmissivity from visual, field, and/or laboratory tests, preferably by field tests.	Section 4.4
Power	Availability, reliability, type and capacity of power at site.	Section 4.5
Degree of possible flooding	Rainfall in area. Runoff characteristics. High water levels in nearby bodies of water.	Section 4.6
Adjacent structures	Proximity of nearby structures to the area to be dewatered. Type of structure.	Section 4.7

4.2 Evaluation of Geologic Conditions. An understanding of the geology of the area is necessary to plan any subsurface investigation. Information obtained from the geologic and soil investigations, as outlined in EM 1110-1-1804, should be used in evaluating a dewatering or groundwater control problem. Depending on the completeness of information available, it may be possible to postulate the general characteristics and stratification of the soil and rock formations in the area. With this information, and the size of and depth of the excavation to be dewatered, the remainder of the geologic and soil investigations can be planned. Sufficient subsurface investigations are required to adequately design and/or prepare a cost estimate for a dewatering system. Insufficient subsurface information may result in under designed or over designed systems and may result in costly modifications during construction. There are a variety of methods that can be used to characterize the subsurface conditions including borings (logging soil samples and rock cores), cone penetrometer tests, downhole imaging and geophysics, and geophysical surveys. Subsurface conditions can be highly variable in lateral extent and depth at a site as shown in Figure 27. Refer to EM 1110-2-1421 (Groundwater Hydrology) for discussions of the use of borehole and surface geophysical exploration methods in the evaluation of groundwater problems. Other references on borehole geophysical methods include Maliva and Missimer (2012) and Fell, Stapleton, Bell and Foster (2015). Refer to EM 1110-1-1802 Geophysical Exploration for Engineering and Environmental Investigations for a comprehensive discussion of surface geophysical methods.

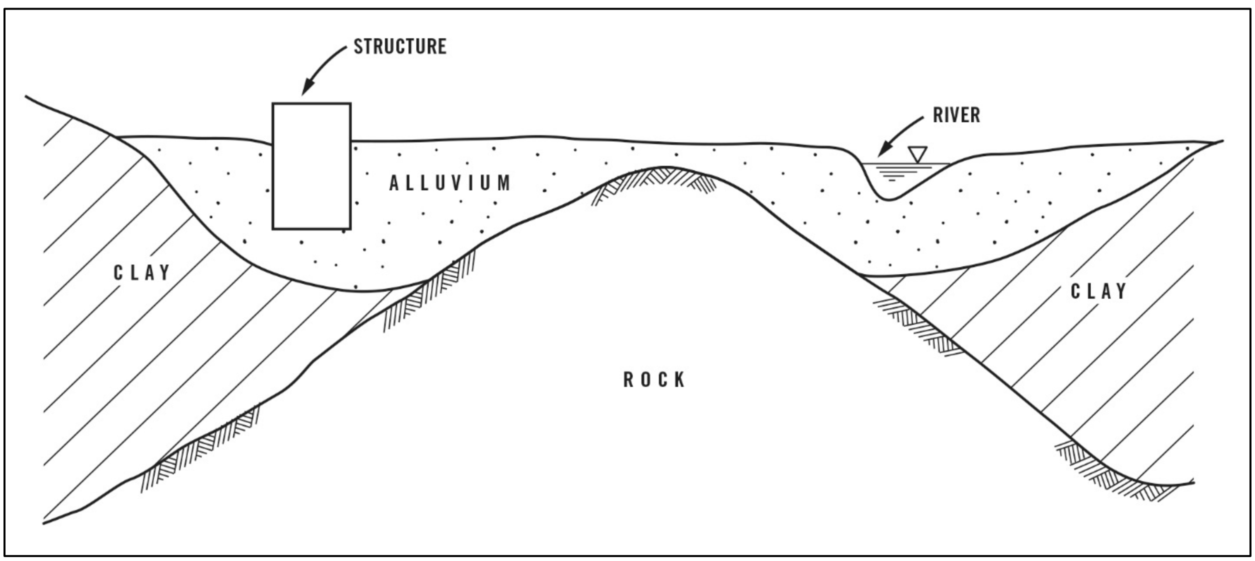


Figure 27. Geologic profile developed from geophysical explorations (Adapted from TM 5-818-5)

4.2.1 Borings and Cone Penetrometer Soundings.

4.2.1.1 A thorough knowledge of the extent, thickness, stratification, and seepage characteristics of the subsurface soil or rock adjacent to and beneath an excavation is required to analyze and design a dewatering system. These factors are generally determined during the normal field exploration that is required for most structures. Borings should not only be made in the immediate vicinity of the excavation, but some borings should be made on lines out to the source of groundwater flow or to the estimated “effective” radius of influence. Samples of the soil or rock formation obtained from these borings should be suitable for classifying and testing for grain size and hydraulic conductivity, if the complexity of the project warrants. All of the information gathered in the investigation should be presented on soil or geologic profiles of the site. For large, complex dewatering or drainage projects, it may be desirable to construct a three-dimensional numerical model to depict the different geologic or soil formations at the site.

4.2.1.2 The depth and spacing of cone penetrometer soundings and borings (and samples) depend on the characteristics of the materials, and on the type and configuration of the formations or deposits as discussed in EM 1110-1-1804. Cone penetrometer soundings, because they are fast, are best completed before sampled test borings are performed. Analysis of cone penetrometer sounding logs should be performed during the planning of test borings and sampling, and borings should be drilled adjacent to selected cone penetrometer soundings to correlate the stratification and soil behavior types indicated by the cone penetrometer soundings. Care must be taken that the borings accomplish the following:

a. Identify all soils or rocks that would affect or be affected by seepage or hydrostatic pressure.

b. Delineate the soil stratification. Borings need to be field logged by an experienced geotechnical engineer or geologist who is intimately familiar with the project’s dewatering requirements.

c. Identify any significant variation in soil and rock conditions that would have a bearing on seepage flow, interruption of vertical seepage, location and depth of wells, or depth of cutoff. Continuous wash or auger boring samples (i.e., borings advanced without taking representative samples with a split-barrel sampler or Shelby tubes) are not considered satisfactory for dewatering exploration. If samples are unable to be obtained, downhole imaging, with an optical televiewer, and borehole geophysics may be used to characterize subsurface materials.

d. Estimate the groundwater level during drilling.

e. Are according to “Do No Harm” for critical structures (earth dams and levees), and are in strict accordance with ER 1110-1-1807, Drilling in Earth Embankment Dams and Levees.

4.2.2 Rock Coring.

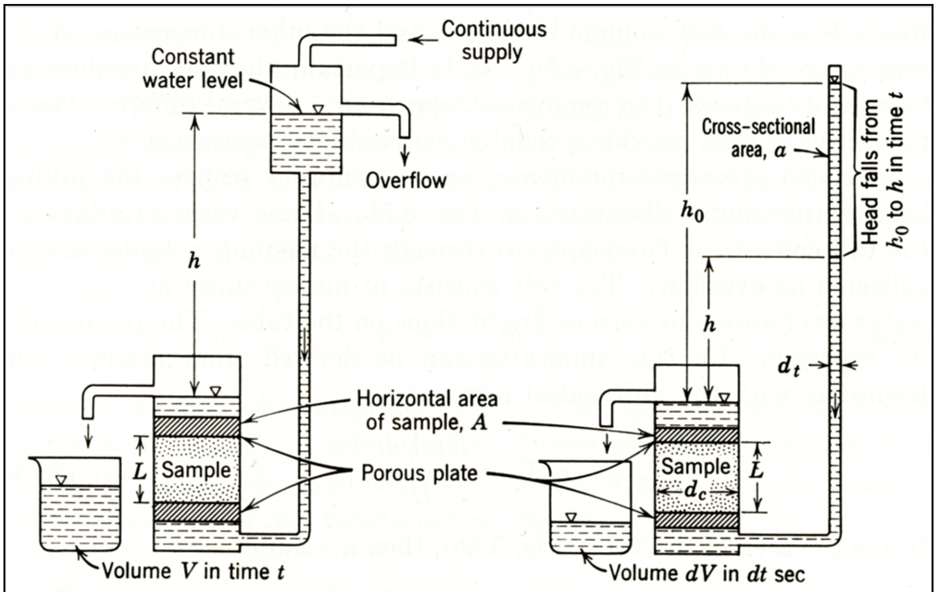
4.2.2.1 Rock samples, to be meaningful for groundwater studies, should be intact samples obtained by core drilling. Although identification of rock types can be made from drill cuttings, the determination of characteristics of rock formations, such as frequency, orientation, and width of joints or fractures, that affect groundwater flow requires core samples. To characterize the rock mass properly, sufficient inclined core borings must be drilled in order to intersect vertical joints. The percent of core recovery and any voids or loss of drill water encountered while core drilling should be recorded. Optical televiewer profiles can provide a wealth of information regarding material type, joints (magnitude, dip, dip direction), bedding planes, etc., and should be included as an option if coring cannot be obtained or to verify the in-situ conditions. Acoustic televiewers and other geophysical methods can also be used to supplement rock core data, especially where core samples are difficult to retrieve.

4.2.2.2 The approximate mass hydraulic conductivity of rock strata can be measured by making pressure or pumping tests of the various strata encountered. Without pressure or pumping tests, important details of a rock formation can remain undetected, even with extensive boring and sampling. For instance, open channels or joints in a rock formation can have a significant influence on the hydraulic conductivity of the formation, yet core samples may not clearly indicate these features where the core recovery is less than 100 percent. For critical structures (earth dams and levees), rock coring must be performed to greatly reduce the potential of hydraulic fracturing the rock and/or overlying foundation soils. Rock coring in critical structures must be performed according to ER 1110-1-1807.

4.2.3 Soil Testing.

4.2.3.1 All soil and rock samples should be carefully classified, noting particularly those characteristics that have a bearing on the perviousness and stratification of the formation. Soil samples should be classified according to the Unified Soil Classification System described in ASTM D2487. Particular attention should be given to the existence and amount of fines (material passing the No. 200 sieve) in cohesionless samples, as fines content has a pronounced effect on the hydraulic conductivity of these materials. Sieve analyses should be made on at least the representative samples of the aquifer deposits to determine their gradation and effective grain size (for example, D_{10} is the effective diameter of which 10 percent of the total sample has particles that have an effective diameter that are less than D_{10}). Preferably, most of the cohesionless samples recovered below the groundwater table should be tested for grain size distribution according to ASTM D6913 for sizes larger than the No. 200 sieve. To estimate grain sizes smaller than the No. 200 sieve, testing should be done according to ASTM D7928. The D_{10} size may be used to estimate the hydraulic conductivity, k. The gradation is required to design filters for wells, wellpoints, or permanent drainage systems to be installed in the formation. Correlations between k and D_{10} are presented in Section 4.4.

4.2.3.2 Laboratory tests depicted in Figure 28 can be used to estimate the approximate hydraulic conductivity of a soil or rock sample; however, conductivities obtained from such tests may have little relation to field values even when carefully conducted under controlled conditions. When samples of cohesionless materials are distributed and repacked/reconstituted in a laboratory, the porosity and orientation of the grains are significantly changed, with resulting change in the hydraulic conductivity. Also, any air entrapped in these samples during testing will significantly reduce its hydraulic conductivity. Laboratory tests on samples of cohesionless materials that have been segregated or contaminated with drilling mud during sampling operations do not provide reliable results. In addition, the hydraulic conductivity of remolded samples of cohesionless materials is usually considerably less than the horizontal hydraulic conductivity k_h of a formation, which is generally more significant in estimating seepage flow to a dewatering or pressure relief system.



$$k = \frac{VL}{hAt} \quad (1)$$

$$k = \frac{aL}{At}\ln\frac{h_o}{h} \quad (2)$$

(a) (b)

Figure 28. Permeameters: (a) constant head and (b) falling head (adapted from Todd, 1980 and TM 5-818-5)

4.2.3.3 Where a non-equilibrium type of pumping test (described in Appendix B) is to be conducted, it is useful in estimating the required duration of the test to estimate the specific yield, Sy, of the formation (volume of water that is free to drain out of a material under natural conditions as a percentage of total volume). Sy can be determined in the laboratory by:

a. Saturating the sample and allowing it to drain. Care must be taken to assure that capillary stresses on the surface of the sample do not cause an incorrect conclusion regarding the drainage.

b. Estimating Sy from the soil type or by laboratory tests. The specific yield can be computed from a laboratory drainage test as follows:

$$S_y = \frac{100V_y}{V} \tag{1}$$

Where:

V_y = volume of water drained from sample

V = gross volume of sample

4.2.3.4 The specific yield may also be estimated from the soil type (Table 6), but laboratory tests are more reliable.

Table 6

Representative Values of Specific Yield

Material	Specific Yield, percent
Gravel, coarse	23
Gravel, medium	24
Gravel, fine	25
Sand, coarse	27
Sand, medium	28
Sand, fine	23
Silt	8
Clay	3
Sandstone, fine-grained	21
Sandstone, medium-grained	27
Limestone	14
Dune sand	38
Loess	18
Peat	44
Schist	26
Siltstone	12
Till, predominantly silt	6
Till, predominantly sand	16
Till, predominantly gravel	16
Tuff	21

(Recreated from "Groundwater Hydrology" (Second Edition) by D.K. Todd, 1980, Wiley & Sons, Inc.)

4.3 Groundwater Characteristics

4.3.1 An investigation of groundwater at a site should include a study of the source of groundwater that will flow to the dewatering or drainage system and determination of the elevation of the water table and its variation with changes in river or tide stages, seasonal effects, and pumping from nearby water wells. Groundwater and artesian pressure levels at a construction site are best determined from piezometers and/or observation wells. Piezometers (vibrating wire, pneumatic, etc.) should be installed in all pervious strata, both those that may require dewatering and pressure relief as well as those that may not be affected by construction dewatering and/or pressure relief. Piezometers in pervious soils and fine-grained soils (silts and clays) may be standpipe type piezometers with commercially available slotted pipe sections or vibrating wire pressure transducers (installed in a borehole, in a standpipe with a slotted pipe, or grouted in-place). Piezometers may be installed with or without a filter, if required, to be filter compatible with the foundation material. Grouted in-place piezometers should be used only below the lowest existing phreatic surface because anomalous readings may occur if these instruments are installed in the unsaturated zone (vadose zone). If there are compressible fine-grained soil strata at a site, particularly if such strata are below the phreatic surface, it may also be necessary and advisable to install piezometers in those strata in order to evaluate the effects of groundwater lowering on pore pressures in the compressible strata. Piezometers should be installed early in the investigation and monitored at a sufficient frequency as to establish a baseline condition prior to construction. The effect of seasonal impacts and subsurface water level fluctuations on the piezometric levels should be considered when establishing monitoring frequencies. The stage of nearby rivers, streams and lakes should be monitored at least daily to permit evaluation of the effect of variations in surface water elevation on piezometric levels. Refer to EM 1110-2-1908 for details on piezometer types and installation methods, and for details on planning instrumentation and monitoring programs.

4.3.2 The groundwater regime should be observed for an extended period of time to establish variations in the groundwater level likely to occur during the construction or operation of a project. General information regarding the groundwater table and river or tide stages in the area is often available from public agencies and may serve as a basis of establishing approximate water levels. Specific conditions at a site can then be predicted by correlating the long-term recorded observations in the area with more detailed short-term observations at the site. Precipitation data from nearby weather stations should be collected to evaluate the influence that precipitation has on groundwater levels and on piezometers readings.

4.3.3 The chemical composition of the groundwater and the presence of bacteria are of concern, because some groundwater is highly corrosive to metal screens, pipes, and pumps, or may contain bacteria and dissolved metals or carbonates that will form incrustations or bacterial film in the wells, pumps, discharge piping and filters that will, with time, cause clogging and reduced efficiency of the dewatering or drainage system. Indicators of corrosive and incrusting waters are given in Table 7. Predicting whether or not incrustation, bacterial action, and corrosion will be a problem during dewatering is difficult, but it is generally worthwhile to engage a specialist to evaluate laboratory tests on the groundwater and report on the potential for such problems before a project is advertised for bids, if for no other reason than to inform bidding contractors that such problems may develop. See references listed below for comprehensive discussions of corrosion and incrustation in wells and recommendations for laboratory testing to evaluate the potential for corrosion and incrustation, including bacterial causes. Following the recommendations of specialists is advisable when either corrosion or incrustation is expected to be a serious problem.

Table 7

Indicators of Corrosive and Incrusting Waters

Indicators of Corrosive Water	Indicators of Incrusting Waters
1. A pH less than 7	1. A pH greater than 7.5
2. Dissolved oxygen in excess of 2 parts per million (ppm)	2. Total iron (Fe) in excess of 2 ppm
3. Hydrogen sulfide (H_2S) in excess of 1 ppm, detected by a rotten egg odor	3. Total manganese (Mn) in excess of 1 ppm in conjunction with a high pH and the presence of oxygen
4. Total dissolved solids in excess of 1,000 ppm indicates an ability to conduct electric current great enough to cause serious electrolytic corrosion	4. Total carbonate hardness in excess of 300 ppm
5. Carbon dioxide (CO_2) in excess of 50 ppm	
6. Chlorides (Cl) in excess of 500 ppm	

(Courtesy of the EPA - recreated)

4.3.4 Laboratory evaluation of groundwater samples collected from one or more wells is a useful technique for identifying fouling mechanisms. Each well selected should be pumped or bailed for approximately 5 minutes and a 1-liter sample collected for shipment to a laboratory. The temperature, total dissolved solids (conductivity method), oxygen reduction potential (ORP) and pH of the well water should be measured and recorded in the field when samples are collected. The following tests are recommended (as a minimum) for evaluation of the water chemistry:

a. pH

b. Temperature

c. Oxidation Reduction Potential (ORP)

d. Hydrogen Sulfide (H2S)

e. Carbon Dioxide (CO2)

f. Chlorides (Cl)

g. Total Alkalinity as CaCO3

h. Total dissolved solids (TDS)

i. Hardness (including carbonate and non-carbonate)

j. Calcium ion concentration as CaCO3

k. Silica ion concentration (as SiO2)

l. Iron concentrations (including Fe+2, Fe+3, and total)

m. Manganese concentration

4.3.5 These parameters will aid in two ways. First, they can be used in the calculation of the Langelier Saturation Index (LSI) to characterize the base water chemistry. The saturation index is a measure of the saturation of calcium carbonate and as such, is a predictor of whether a scale will form or not. For scale formation, the water must have a saturation index greater than 0.0. The LSI is estimated using the following formula:

$$\mathrm{LSI} = \mathrm{pH} - \mathrm{pH_s} \tag{2}$$

$$\mathrm{pH_s} = (9.3 + \mathrm{A} + \mathrm{B}) - (\mathrm{C} + \mathrm{D}) \tag{3}$$

Where:

A = (Log10 [TDS] - 1) / 10

B = -13.12 x Log10 (°C + 273) + 34.55

C = Log10 [Ca^{+2} as $CaCO_3$] - 0.4

D = Log10 [Alkalinity as $CaCO_3$]

4.3.6 There are a number of online calculators for the LSI. The parameters will also allow for the monitoring of changes in key ion concentrations that may reflect accumulation or dissolution occurring down-hole.

4.3.7 Bacteriological analyses are useful in predicting the probability of bacterial as well as mineral plugging being a problem. Chapter 13 of Groundwater and Wells (Schnieders 2007) recommends performing the heterotrophic plate count (HPC) test to determine the number of colony-forming units per unit volume of water and the adenosine triphosphate test (ATP) to determine the number of bacteria per unit volume. The HPC allows correlation with other work in the industry while the ATP accounts for a much better assessment of the actual population numbers as it is not dependent on culturability as with the HPC. More than 90% of all bacteria are non-culturable. Chapter 13 of Groundwater and Wells (Schnieders 2007) also recommends microscopic examination of water samples to identify bacteria that can be identified visually as well as iron-oxide accumulation, sand infiltration, presence of protozoa, and other abnormalities.

4.3.8 The ORP (oxidation-reduction potential) measurement provides a reasonable way to distinguish between aerobic vs. anaerobic bacterial activity.[5] This statement is true where high population numbers are present or verified by analysis. Schnieders (URS 2010) provided the following discussion on the interpretation of the ORP measurements:

> "ORP is a measurement of the dominant chemical reactions in an environment with a negative reading indicating a more reducing environment and a positive reading signifying an oxidative condition. It can indicate the presence of oxygen since oxygen is oxidative, but other oxidation reactions can also be present. In closed water environments such as a well sitting idle, or slowly pumped, bacterial activity is usually the dominating force in developing either oxidative or reducing conditions. The ORP is then often used as an indicator of bacterial activity or growth. Anaerobic growth is known for the production of reducing conditions (Example: sulfur reducing bacteria). The aerobic bacteria of course require oxygen and many of the aerobes are capable of oxidizing many metals and non-metal species. (Examples: iron-oxidizing bacteria, or the oxidation of sulfides to sulfates by sulfate oxidizers etc.)"

4.3.9 While most significant bacterial activity produces readings between -50 millivolts (mV) to +200 mV with aerobic growth dominating at +150mv, it is not so much the level but the change or swing in the reading that you should notice. For instance: if the reading has been +75 to +130 mV and the following month drops to a negative 30 mV, (one would) expect an increase in anaerobic conditions near the well bottom. This could signal increasing accumulation near the well bottom (anaerobic conditions usually develop lower in the well) and more fouling in that area. Conversely, a change from a negative reading to a positive 150 mV would signal an increase in aerobic activity, which could indicate fouling higher up in the well."

[5] Dr. Paul Sturman, professor of civil engineering at Montana State University, commented on ORP measurements in a personal communication on 05 Nov 2009: "It is necessary to measure ORP at the well to get a useful measurement. Lab-measured field samples typically become 'contaminated' with atmospheric oxygen prior to measurement, thereby increasing the ORP. A good rule of thumb is that ORP measurements less than zero signify an anaerobic environment while measurements between zero and approx. +200 mV indicate some oxygen is present. There is not a firm correlation between ORP and dissolved oxygen concentration because ORP can be influenced by other ionic species in solution, notably iron and sulfur species, but it is a useful measurement to get an idea of the processes that are taking place." John Schnieders added that this statement is true where high population numbers are present or verified by analysis.

4.3.10 The following are references with extensive discussion of corrosion and incrustation problems in groundwater: Powers et al. (2007), Schnieders (2003), Roscoe Moss Company (1990) and Sterrett (2007).

4.3.11 Changes in the temperature of the groundwater will result in minor variations of the rate of water flowing to a dewatering system. The change in viscosity associated with temperature changes will result in a change in flow of about 1.5 percent for each 1-degree Fahrenheit of temperature change in the water. Only large variations in temperature need be considered in design because groundwater temperatures are usually relatively constant and the accuracy of determining other parameters does not warrant excessive refinement.

4.4 Hydraulic Conductivity of Pervious Strata.

4.4.1 General. The rate at which water can be pumped from a dewatering system is directly proportional to the hydraulic conductivity of the formation being dewatered; thus, this parameter should be estimated reasonably accurately prior to the design of any drainage system. The term hydraulic conductivity is used generically and assumes a material is homogeneous and isotropic. For analysis, isotropy or anisotropy of the formations through which water is flowing must be clarified. The use of the variable k implies the hydraulic conductivity of a homogeneous isotropic aquifer while the terms k_h and k_v denote horizontal and vertical hydraulic conductivity, respectively. Methods that can be used to estimate the hydraulic conductivity of a pervious aquifer are presented in the following sections.

4.4.2 Visual Classification. The simplest approximate method for estimating the hydraulic conductivity of a soil is by visual examination and classification, and comparison with similar soils of measured hydraulic conductivity. An approximation of the hydraulic conductivity of homogeneous materials (silty sand, clean sand, and sand with gravel) can be estimated from Table 8.

Table 8
Approximate Range of Hydraulic Conductivity Values (k) for Granular Soils (Adapted from TM 5-818-5)

Soil Description (Unified Soil Classification System)	**Hydraulic Conductivity (k)**	
	x 10^{-4} cm/sec	**x 10^{-4} ft/min**
Sandy silt (ML)	5 – 20	10 - 40
Silty sand (SM)	20 – 50	40 - 100
Very fine sand (SP)	50 – 200	100 - 400
Fine sand (SP)	200 – 500	400 - 1,000
Fine to medium sand (SW)	500 - 1,000	1,000 - 2,000
Medium sand (SP)	1,000 - 1,500	2,000 - 3,000
Medium to coarse sand (SW)	1,500 - 2,000	3,000 - 4,000
Coarse sand and gravel (SP)g	2,000 - 2,500	4,000 - 10,000

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4.4.3 Empirical Relations Between Grain Size and k or k_h. The horizontal hydraulic conductivity of a clean sand can be estimated from empirical relations between D_{10} and k_h (Figure 29), which were developed from laboratory sieve analyses and field pumping tests for sands in the Mississippi and Arkansas River valleys. The correlation curve shown in Figure 29 is an average based on numerous pumping tests in the Mississippi and Arkansas River Valleys, and it is strongly recommended to perform site specific tests at sites outside of this area. If no pumping test data are available when a dewatering system is designed, consideration should be given to factoring the mean k_h calculated from D_{10} values using the correlation curve. Testing of relief wells in 2014 at various sites in Wood River, IL (Bird and Andersen 2014) indicated that the actual average formation kh ranged between 1.0 to 2.0 times the average k_h estimated from the curve in Figure 29 using D_{10} values from extensive sieve analysis testing on representative samples from multiple fully penetrating test borings in the immediate vicinity of the relief wells that were tested. Three empirical correlation charts (Powers et al. 2007) developed by Byron Prugh (Figures 30, 31, and 32) correlate k with D_{50} and also account for the effects of relative density and uniformity. The authors of Powers et al. (2007) state that in their experience, these charts give good results if the samples selected for analysis are representative.

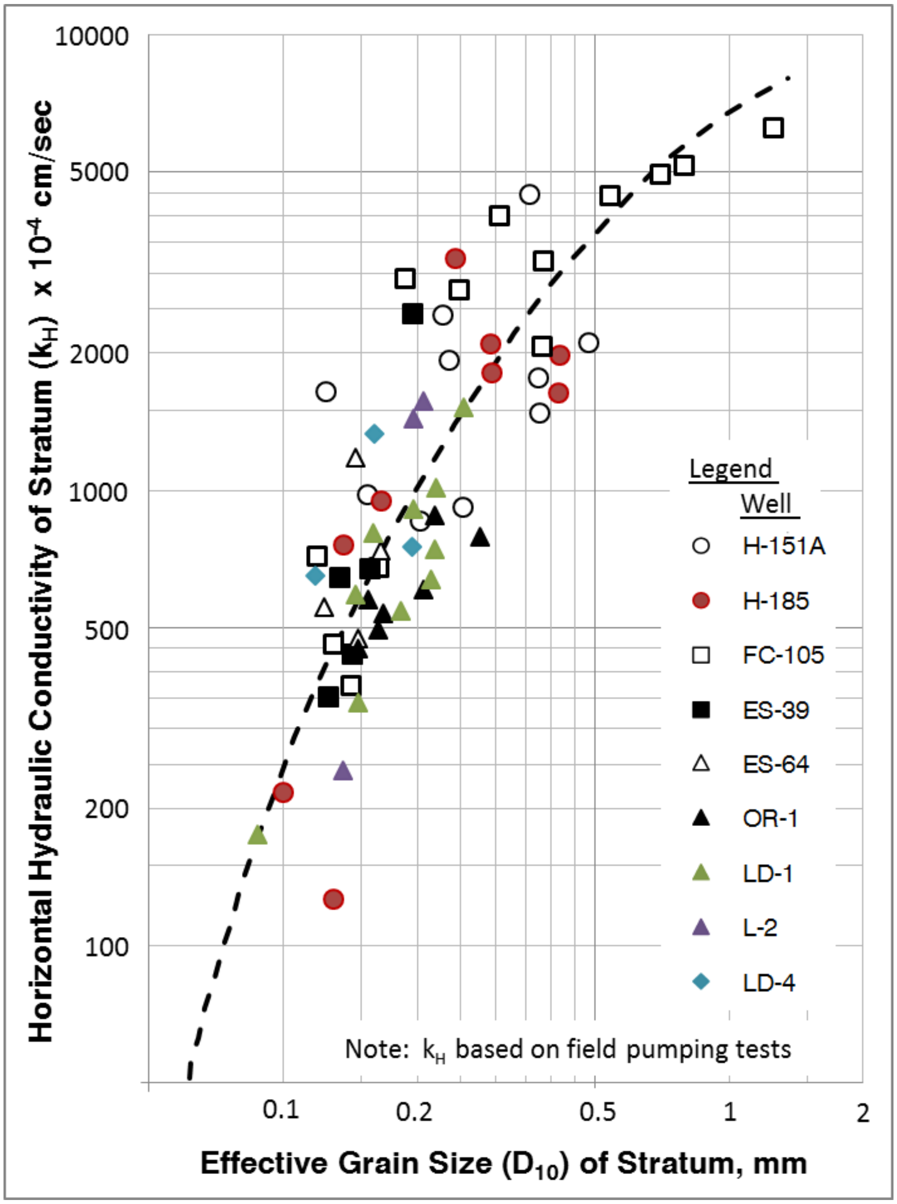


Figure 29. D_{10} versus in situ horizontal hydraulic conductivity- Mississippi River valley and Arkansas River valley (Adapted from TM 5-818-5)

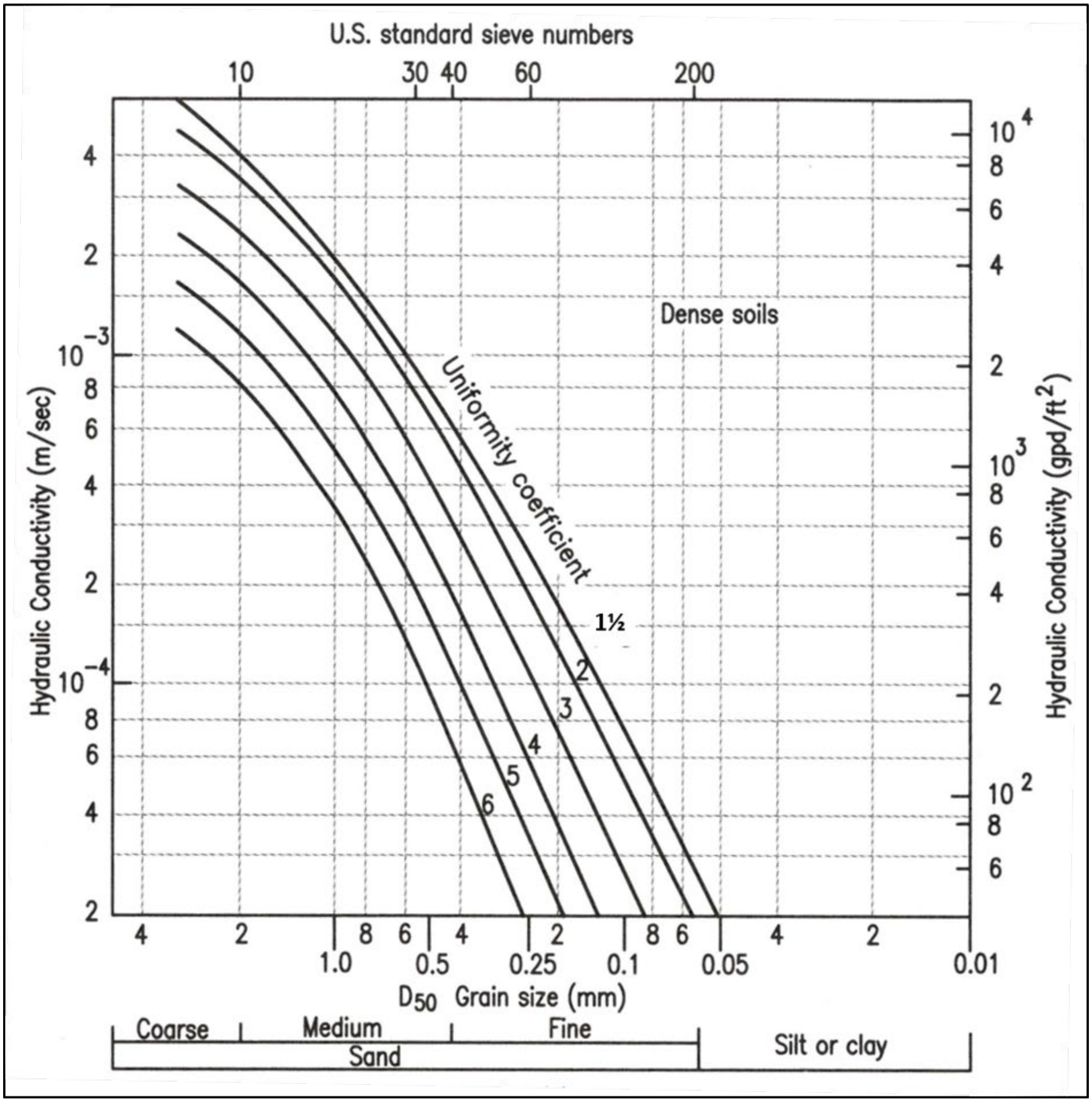


Figure 30. Prugh k chart for dense soils (Courtesy of Keller)

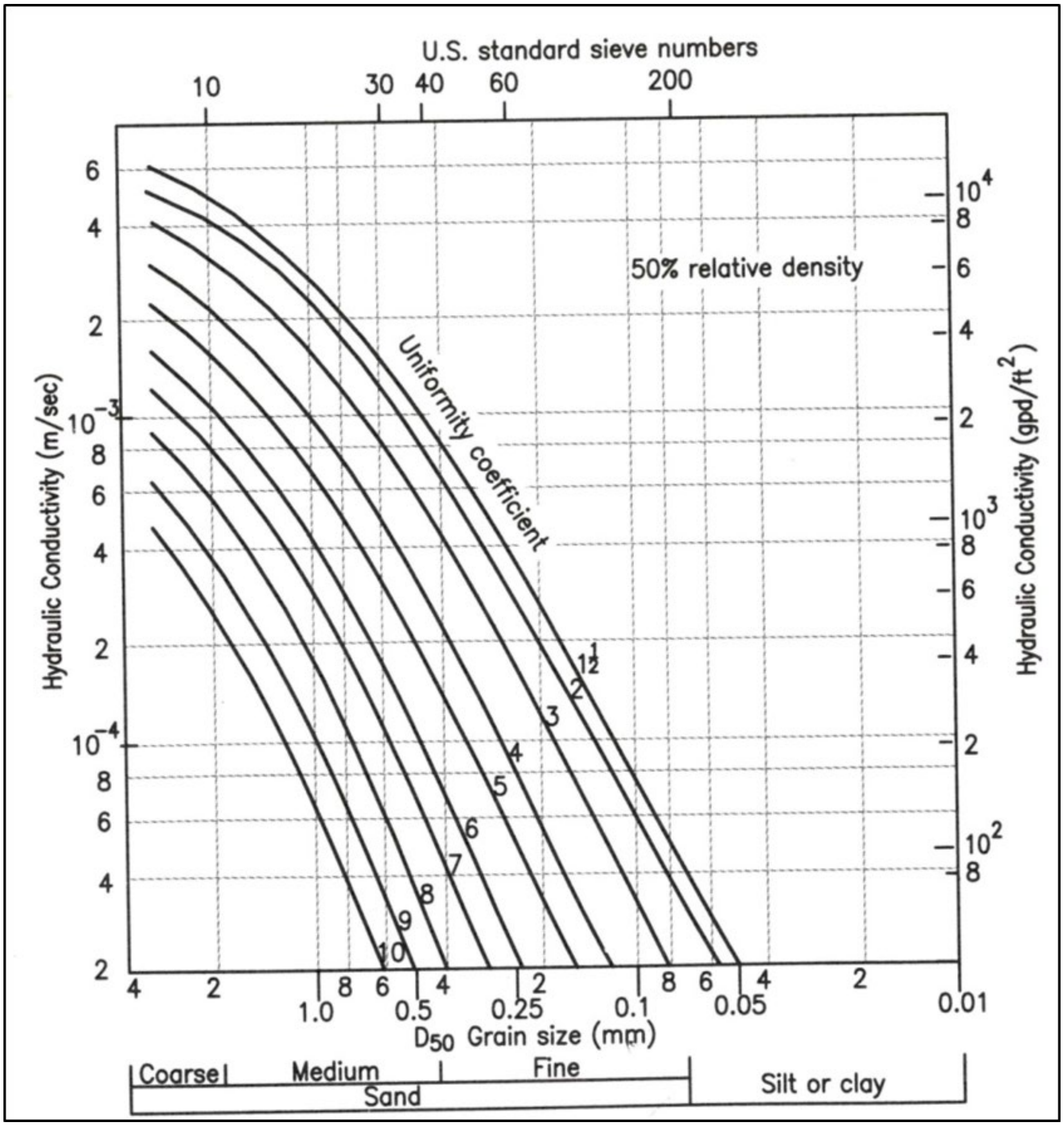


Figure 31. Prugh k chart for soils with 50% relative density (Courtesy of Keller)

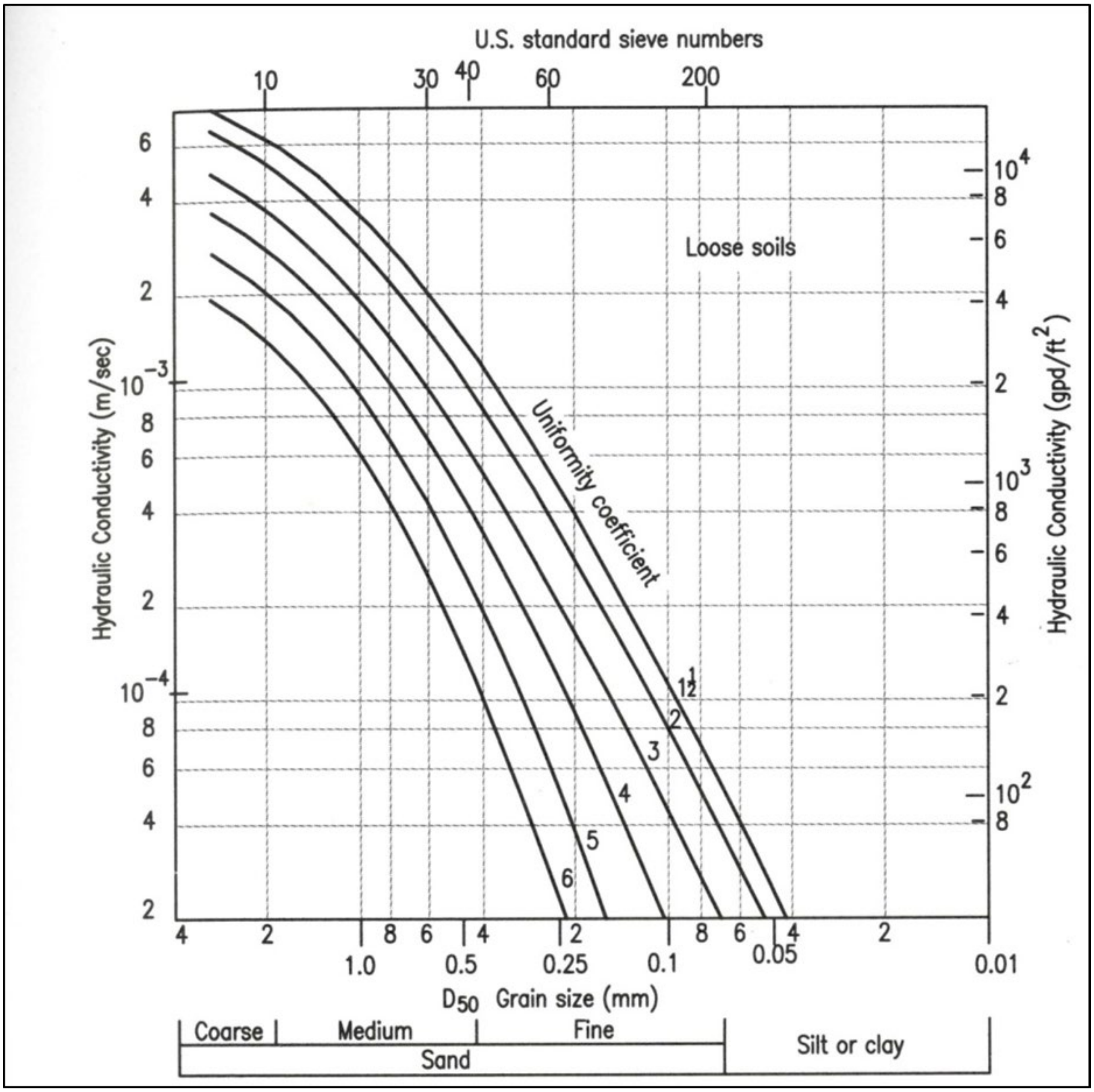


Figure 32. Prugh k chart for loose soils (Courtesy of Keller)

4.4.4 Hazen Equation. Hazen's investigation of the hydraulic conductivity of filter sands revealed that the hydraulic conductivity of clean, relatively uniform, remolded sand could be estimated from the empirical relation:

$$k = C \times (D_{10})^2 \tag{4}$$

Where:

$k =$ hydraulic conductivity (cm/sec)

$C \cong 1.0$ (may vary from 0.4 to 1.5)

$D_{10} =$ effective grain size of filter (mm)

4.4.5 Other Empirical Relationships.

4.4.5.1 There are several other empirical relationships between k and grain size in addition to Hazen's. The most complex is Kozeny-Carman (such as the Chapuis and Aubertin 2003 version of Kozeny-Carman). A very useful, relatively reliable empirical relationship for filter sands was developed for the median value of k by the Natural Resource Conservation Service and reported by Sherard, Dunnigan and Talbot (1984a):

$$k = 0.35(D_{15})^2 \tag{5}$$

Where:

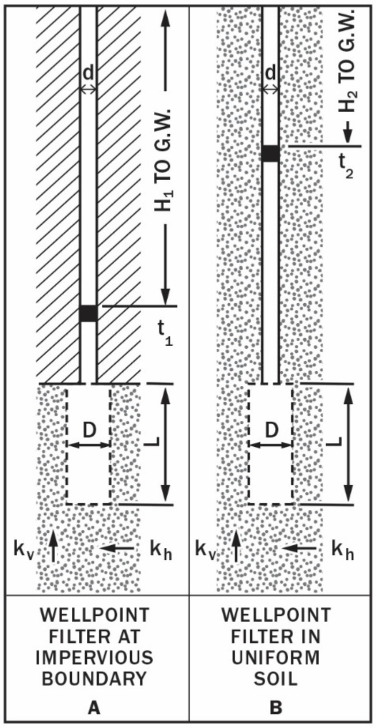
$k =$ hydraulic conductivity (cm/sec)

$D_{15} =$ 15% size of filter (mm)

4.4.5.2 Empirical relations between grain size and k are only approximate and should be used with reservation until a correlation based on a field pumping test or local experience is available. Empirical relationships between grain size and k only represent a very small sample of the larger aquifer. The permeability of the aquifer is likely controlled by gravel seams, silt intrusions, and other geologic features.

4.4.6 Field Pumping Tests. Field pumping tests are the most reliable procedure for estimating the in-situ hydraulic conductivity of a water-bearing formation. For large dewatering projects, a pumping test on a well that fully penetrates the sand stratum to be dewatered is warranted; such tests should be made during the design phase so that results can be used for design purposes and will be available to bidders. However, for small dewatering projects, it may be more economical to select a more conservative value of k based on empirical relations than to perform a field pumping test. Pumping tests are discussed in detail in Appendix B.

4.4.7 Slug Testing and Other Simple Field Tests in Wells or Piezometers. The hydraulic conductivity of a water-bearing formation can be estimated from constant, rising or falling head tests made in wells or piezometers in a manner similar to laboratory permeameter tests. Figure 33 presents formulas for determining the hydraulic conductivity using various types and installations of well screens. As these tests are sensitive to details of the installation and execution of the test, exact dimensions of the well screen, casing, and filter surrounding the well screen, and the rate of inflow or fall in water level must be accurately measured. Transducers with data loggers provide a convenient way to acquire sufficient water level data from slug tests to permit analysis. Kruseman and de Ridder (1990) is a good reference for different methods of analyzing slug test results, and Batu (1998) provides a comprehensive discussion of several methods of analysis of slug testing. Disturbance of the soil adjacent to a borehole or filter, leakage up the borehole around the casing, clogging or removal of the fine-grained particles of the aquifer or the accumulation of gas bubbles in or around the well screen can make the test completely unreliable. Performing slug tests in boreholes advanced by hollow stem augers can underestimate hydraulic conductivity due to the augers densifying materials adjacent to the borehole and smearing the borehole walls. Other methods (jetting or direct rotary methods) are preferred to hollow stem augers, provided that these methods do not damage the foundation of critical structures (e.g., dams and levees). In addition, the test likely only measures the permeability of a small volume of soil immediately surrounding the wellpoint or piezometer. The results should be interpreted as part of the overall geologic understanding of the water-bearing formation.



NOTATION	
D	= DIAM, INTAKE, SAMPLE, CM
d	= DIAM, STANDPIPE, CM
L	= LENGTH, INTAKE, SAMPLE, CM
H_c	= CONSTANT PIEZ HEAD, CM
H_1	= PIEZ HEAD FOR $t = t_1$, CM
H_2	= PIEZ HEAD FOR $t = t_2$, CM
q	= FLOW OF WATER, CM^3/SEC
t	= TIME, SEC
k_v	= VERT HYDRAULIC CONDUCTIVITY GROUND, CM/SEC
k_h	= HORIZ HYDRAULIC CONDUCTIVITY GROUND, CM/SEC
k_m	= MEAN HYDRAULIC CONDUCTIVITY, CM/SEC
m	= TRANSFORMATION RATIO

$$k_m = \sqrt{k_h k_v} \qquad m = \sqrt{k_h/k_v}$$

$$\ln = \log_e = 2.3 \log_{10}$$

CASE	CONSTANT HEAD	VARIABLE HEAD
A	$k_h = \frac{q\ln\left[\frac{2mL}{D} + \sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{2\pi L H_c}$	$k_h = \frac{d^2\ln\left[\frac{2mL}{D} + \sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{8L(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_h = \frac{d^2\ln\left(\frac{4mL}{D}\right)}{8L(t_2-t_1)}\ln\frac{H_1}{H_2} \quad \text{FOR } \frac{2mL}{D} > 4$
B	$k_h = \frac{q\ln\left[\frac{mL}{D} + \sqrt{1+\left(\frac{mL}{D}\right)^2}\right]}{2\pi L H_c}$	$k_h = \frac{d^2\ln\left[\frac{mL}{D} + \sqrt{1+\left(\frac{2mL}{D}\right)^2}\right]}{8L(t_2-t_1)}\ln\frac{H_1}{H_2}$ $k_h = \frac{d^2\ln\left(\frac{2mL}{D}\right)}{8L(t_2-t_1)}\ln\frac{H_1}{H_2} \quad \text{FOR } \frac{mL}{D} > 4$

ASSUMPTIONS

SOIL AT INTAKE, INFINITE DEPTH AND DIRECTIONAL ISOTROPY (k_v and k_h CONSTANT) – NO DISTURBANCE, SEGREGATION, SWELLING, OR CONSOLIDATION OF SOIL – NO SEDIMENTATION OR LEAKAGE – NO AIR OR GAS IN SOIL, WELLPOINT, OR PIPE – HYDRAULIC LOSSES IN PIPES, WELLPOINT, OR FILTER NEGLIGIBLE.

Figure 33. Formulas for determining hydraulic conductivity from field falling head tests. (Adapted from TM 5-818-5)

4.5 Power. The availability, reliability, type, and capacity of power available at a site should be investigated prior to selecting or designing the pumping units for a dewatering system. Types of power used for dewatering systems include electric, natural gas, butane, diesel, and gasoline engines. Electric motors and diesel engines are most commonly used to drive dewatering equipment. Noise may be an issue for engine-driven pumps and diesel generators in some settings. Silencer enclosures for engines are very effective in noise reduction and may be advisable to utilize in instances where noise is objectionable, and engines have to be used to drive pumps. The reliability of the power source and the criticality of the dewatering system should also be evaluated to determine whether a backup power sources will be required. A backup power source will generally be required to provide capacity equivalent to the main source of power, and the backup source should be configured to automatically switch over from the main power source.

4.6 Surface Water

4.6.1 Investigations for the control of surface water at a site should be performed by an engineer with sufficient experience in hydraulics and hydrology, and should include a study of precipitation data for the locality of the project and determination of runoff conditions that will exist within the excavation. Precipitation data for various localities and the frequency of occurrence are available online from the National Oceanic and Atmospheric Administration's (NOAA's) National Weather Service website. The most convenient way to obtain rainfall frequency and duration data for a particular site is to use NOAA's interactive point-and-click interface: Precipitation Frequency Data Server (PFDS) at http://hdsc.nws.noaa.gov/hdsc/pfds/. Data for all states (except OR, WA, ID, MT, and WY), Puerto Rico, U.S. Virgin Islands and selected Pacific islands are currently accessible on this interface. Example tabulations from the PFDS of the amounts and durations of rainfall at North Little Rock, Arkansas that can be expected at various frequencies is shown in Figure 34. Refer to Figure C.10 in Appendix C which uses data from this figure to solve a practical surface water drainage problem for an excavation.

NOAA Atlas 14, Volume 9, Version 2
Location name: North Little Rock, Arkansas, US*
Latitude: 34.7664°, Longitude: -92.2200°
Elevation: 250 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, Geoffery Bonnin

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.434 (0.370-0.515)	0.492 (0.419-0.584)	0.585 (0.496-0.696)	0.660 (0.557-0.788)	0.761 (0.617-0.927)	0.835 (0.663-1.03)	0.908 (0.697-1.15)	0.979 (0.720-1.26)	1.07 (0.757-1.42)	1.14 (0.785-1.53)
10-min	0.636 (0.541-0.754)	0.721 (0.613-0.855)	0.857 (0.727-1.02)	0.967 (0.815-1.15)	1.11 (0.904-1.36)	1.22 (0.971-1.51)	1.33 (1.02-1.68)	1.43 (1.05-1.85)	1.57 (1.11-2.08)	1.67 (1.15-2.24)
15-min	0.775 (0.660-0.919)	0.879 (0.748-1.04)	1.04 (0.886-1.24)	1.18 (0.994-1.41)	1.36 (1.10-1.66)	1.49 (1.19-1.84)	1.62 (1.24-2.04)	1.75 (1.29-2.26)	1.91 (1.35-2.53)	2.03 (1.40-2.74)
30-min	1.16 (0.984-1.37)	1.31 (1.12-1.56)	1.56 (1.33-1.86)	1.77 (1.49-2.11)	2.03 (1.65-2.48)	2.23 (1.77-2.76)	2.43 (1.86-3.06)	2.61 (1.92-3.37)	2.85 (2.02-3.77)	3.02 (2.09-4.08)
60-min	1.55 (1.32-1.84)	1.76 (1.49-2.08)	2.08 (1.77-2.48)	2.35 (1.98-2.80)	2.71 (2.20-3.30)	2.98 (2.37-3.68)	3.24 (2.49-4.09)	3.50 (2.58-4.53)	3.84 (2.72-5.09)	4.09 (2.83-5.51)

Figure 34. Point Precipitation Frequency Estimates for North Little Rock, AR (from NOAA Precipitation Frequency Data Server)

4.6.2 The coefficient of runoff, C, within the excavation will depend on the characteristics of soils present or the treatment, if any, of the slopes. Except for excavations in clean sands, the coefficient of runoff, C, generally ranges from 0.8 to 1.0. The rate of runoff can be determined as follows:

$$Q_{sw} = CiA \tag{6}$$

Where:

Q_{sw} = rate of runoff (cfs)

C = coefficient of runoff

i= intensity of rainfall (inches per hour)

A = drainage area (acres)

4.7 Adjacent Structures

4.7.1 The investigation should include developing a list of structures that could be influenced by the dewatering. Lowering of the groundwater table through dewatering can lead to settlement of, and damage to, adjacent structures. To limit these impacts, each component of the dewatering system must be carefully designed, including filters, cut-off walls, and recharge wells. Careful investigations should be carried out in the design phase of a project to evaluate the need for such methods, because dewatering by pumping wells or wellpoints is usually much less costly.

4.7.2 For structures that are located in the vicinity of a planned dewatering system, collect structure information including structure type, structure use, and structure foundation type. If evaluations indicate a structure could be affected by dewatering, a more detailed preconstruction survey should be performed. This survey should include detailed mapping of the structure by a licensed structural engineer. The engineer should locate all existing signs of cracking or other distress and document the existing condition of the structure. This survey should include photographs and crack measurements. The engineer should also determine locations to place monitoring points to monitor movement during the dewatering process as discussed in Section 3.5. This information will provide a basis for evaluating any claims that may be made for potential damages to nearby structures.

4.7.3 Dewatering can also adversely impact existing water supply wells if a dewatering system is installed and operated in an aquifer that is also used for water supply. Collect water well information for wells located nearby the planned dewatering system and in the same aquifer, including depth of well and installation records. Observations should be made of the water level in nearby water supply wells (refer to EM 1110-2-1908 for methods to determine water levels in wells) and the collection of historic yield of these wells, before and during dewatering to evaluate the effects of dewatering on these wells, and before the issuance of bid documents. This information will provide a basis for evaluating any claims that may be made for decreases in the capacity of nearby water supply wells.

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Chapter 5
Design of Dewatering, Pressure Relief, and Groundwater Control Systems.

5.1 Analysis of Groundwater Flow

5.1.1 Design of a dewatering and pressure relief or groundwater control system first requires definition of the type of groundwater flow (artesian, gravity, or combined) to be expected and of the type of system that will be required. Also, a reasonably complete picture of the groundwater and subsurface conditions is necessary. Then the number, size, spacing, and penetration of wellpoints or wells and the rate at which the water must be removed to achieve the required groundwater lowering or pressure relief must be estimated.

5.1.2 In the analysis of any dewatering system, the source of seepage must be estimated and the boundaries and seepage flow characteristics of geologic and soil formations at and adjacent to the site must be generalized into a form that can be analyzed. In some cases, the dewatering system and soil and groundwater flow conditions can be generalized into rather simple configurations. For example, the source of seepage can be reduced to a line or circle; the aquifer to a homogeneous, isotropic formation of uniform thickness; and the dewatering system to one or two parallel lines or a circle of wells or wellpoints. Analysis of these conditions can generally be made by means of mathematical formulas for flow of groundwater. Complicated configurations of wells, sources of seepage, and soil formations can, in most cases, be solved or at least approximated by means of mathematical formulas, numerical models, method of fragments and flow nets, or a combination of these methods.

5.1.3 Any analysis, either mathematical, flow net, method of fragments, or numerical model, is no better than the validity of the formation boundaries and material characteristics used in the analysis. The solution obtained, regardless of the rigor or precision of the analysis, will be representative of actual behavior only if the problem situation and boundary conditions are adequately represented. An approximate solution to the right problem is far more desirable than a precise solution to the wrong problem. The importance of formulating correct groundwater flow and boundary conditions, as presented in Chapter 4, cannot be over emphasized.

5.1.4 Methods for dewatering and pressure relief, and their suitability for various types of excavations and soil conditions were described in Chapter 3. Mathematical, graphical, and numerical methods of analyzing seepage flow through generalized soil conditions and boundaries to various types of dewatering or pressure relief systems are presented in Sections 5.2, 5.3, and 5.4, respectively.

5.1.5 Other factors that have a bearing on the actual design of dewatering, pressure relief, and surface-water control systems are considered in this section.

5.1.6 The formulas and flow net procedures presented in Sections 5.2, 5.3, and 5.4, and Figures 35 through 56 are for a steady state condition of groundwater flow. During the initial stages of dewatering an excavation, water is removed from storage and the rate of flow is larger than required to maintain the specified drawdown. Therefore, initial pumping rates will probably be about 30 percent larger than computed values. Refer to Section 5.2 for additional explanation of the use of the equations presented in this section.

5.1.7 Examples of design for dewatering and pressure relief systems are given in Appendix C.

5.2 Mathematical and Numerical Model Analyses

5.2.1 General. Design of a dewatering system requires the estimation of the number, size, spacing and penetration of wells or wellpoints, and the rate at which water must be removed from the pervious strata to achieve the required groundwater lowering or pressure relief. The size and capacity of pumps and collectors also depend on the required discharge and drawdown, as well as the electrical system requirements. The fundamental relationships between well and wellpoint discharge and corresponding drawdown are presented in this section and Section 5.3. The equations presented assume that the flow is laminar, the pervious stratum is homogeneous and isotropic, the water draining into the system is pumped out at a constant rate, and flow conditions have stabilized. Procedures for transforming an anisotropic aquifer (required for flow net construction) with respect to hydraulic conductivity to an isotropic section are presented in Cedergren's Seepage, Drainage and Flow Nets (1997). Equations and example problems for analyses using the method of fragments are presented in Harr's Groundwater and Seepage (1962) and Mechanics of Particulate Media (1977).

5.2.2 Equations for Steady Flow to and Drawdown in Slots[6] and Wells.

5.2.2.1 General. The equations referenced in this section are in two groups: flow to and drawdown at slots (Section 5.2.2.2 and Figures 35 through 43) and flow to and drawdown in wells (Section 5.2.2.3 and Figures 44 through 56). Equations for slots are applicable for flow to trenches, French drains, and similar drainage systems. They may also be used where the drainage system consists of closely spaced wells or wellpoints. It is usually assumed that a well system equivalent to a slot simplifies the analysis; however, corrections must be made to consider that the drainage system consists of wells or wellpoints rather than the more efficient slot. These corrections are given with the well formulas discussed in Section 5.2.2.3 below. When the well system cannot be simulated with a slot, well equations must be used. The figures

[6] The term "slot," as used in this document, is a geometrical concept used in theoretical analyses of groundwater flow to represent similar physical features in the field and also to approximate a line of closely spaced wells.

in which equations for flow to slots and wells are indexed in Table 9. The equations for slots and wells do not consider the effects of hydraulic head losses, H_w, in wells or wellpoints; procedures for accounting for these effects are presented separately.

5.2.2.2 Flow to a Drainage Slot.

5.2.2.2.1 Line Drainage Slots. Equations presented in Figures 35 through 39 can be used to compute flow and head produced by pumping either a single or a double continuous slot of infinite length. These equations assume that the source of seepage and the drainage slot are infinite in length and parallel, and that seepage enters the pervious stratum from a vertical line source. In actuality, the slot will be of finite length, the flow at the ends of the slot for a distance of about L/2 (where L equals distance between slot and source) will be greater, and the drawdown will be less than for the central portion of the slot. Flow to the ends of a fully penetrating slot can be estimated, if necessary, from flow-net or numerical analyses presented later.

5.2.2.2.2 Circular and Rectangular Slots. Equations for flow and head or drawdown produced by circular and rectangular slots supplied by a circular seepage source are given in Figures 40 through 43. Equations for flow from a circular seepage source assume that the slot is located in the center of an island of radius R. For many dewatering projects, R is the radius of influence rather than the radius of an island, and procedures for determining the value of R are discussed in Section 5.2.2.7. Dewatering systems of relatively short lengths are considered to have a circular source where they are far removed from a line source such as a river or shoreline.

5.2.2.2.3 Use of Slots for Designing Well Systems. Wells can be substituted for a slot; and the flow Q_w, drawdown at the well (H-h_w) neglecting hydraulic head losses at and in the well, and head midway between the wells above that in the wells Δh_m can be computed from the equations given in Figures 54 through 56 for a (single) line source for artesian and gravity flow for both "fully" and "partially" penetrating wells where the well spacing, a, is substituted for the length of slot, x.

5.2.2.2.4 Partially Penetrating Slots. The equations for gravity flow to partially penetrating slots are considered valid only for slot penetrations of 50% or greater.

5.2.2.3 Flow to Wells.

5.2.2.3.1 Flow to Wells from a Circular Source.

a. Equations for flow and drawdown produced by a single well supplied by a circular source are given in Figures 44 through 46. It is apparent from Figure 45 that considerable computation is required to determine the height of the phreatic surface and resulting drawdown in the immediate vicinity of a gravity well (r/h less than 0.3). The drawdown in this zone usually is not of special interest in dewatering systems and seldom needs to be computed. However, it is always necessary to compute the water level in the well for the selection and design of the pumping equipment.

b. The general equations for flow and drawdown produced by pumping a group of wells supplied by a circular source are given in Figure 47. These equations are based on the principle of superposition, meaning that the drawdown at any point is the summation of drawdowns produced at that point by each well in the system. The drawdown factors, F, to be substituted into the general equations in Figure 47 appear in the equations for both artesian and gravity flow conditions. Consequently, the factors given in Figure 48 for commonly used well arrays are applicable for either condition.

c. Flow and drawdown for circular well arrays can also be computed in a relatively simple manner, by first considering the well system to be a slot, as shown in Figure 49 or 50. However, the piezometric head in the vicinity of the wells (or wellpoints) will not correspond exactly to that determined for the slot due to convergence of flow to the wells. The piezometric head in the vicinity of the well is a function of well flow Qw; well spacing a; well penetration W; effective well radius rw; aquifer thickness D, or gravity head H; and aquifer hydraulic conductivity k. The equations given in Figures 49 and 50 consider these variables.

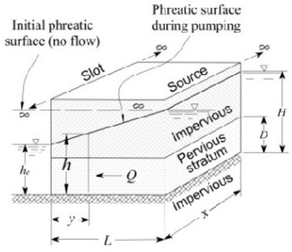
5.2.2.3.2 Flow to Wells from a Line Source.

a. Equations given in Figures 51 through 53 for flow and drawdown produced by pumping a single well or group of fully penetrating wells supplied from an infinite line source were developed using the method of image wells. The image well (a recharge well) is located as the mirror image of the real well with respect to the line source and supplies the pervious stratum with the same quantity of water as that being pumped from the real well.

b. The equations given in Figures 52 and 53 for multiple-well systems supplied by a line source are based on the principle of superposition, meaning that the drawdown at any point is the summation of drawdowns produced at that point by each well in the system. Consequently, the drawdown at a point is the sum of the drawdowns produced by the real wells and the negative drawdowns produced by the image or recharge wells.

c. Equations are given in Figures 54 through 56 for flow and drawdown produced by pumping an infinite line of wells supplied by a (single) line source. The equations are based on the equivalent slot assumption. Where twice the distance to a single line source or 2L is greater than the radius of influence R, the value of R as determined from a pumping test or from Figure 58 should be used in lieu of L unless the excavation is quite large or the tunnel is long, in which case equations for a line source or a flow-net or numerical analysis should be used. The assumption that a line of wells is infinite cannot be replicated by the dewatering system installed in the field. Additional wells or closer well spacing may be required beyond what is calculated using the equations in Figures 54 through 56. Three-dimensional end effects present at the edge of "infinite" installations should also be evaluated and will likely required additional wells, deeper wells, or a longer well reach to create an "infinite" line of wells at the area requiring dewatering.

d. Equations for computing the head midway between wells above that in the wells (Δhw) are not given in this document for two line sources adjacent to a single line of wells. However, such can be readily determined from (plan) numerical and flow-net analyses.



(a)

FLOW

$$Q = \frac{kDx}{L}(H - h_e) \qquad (1)$$

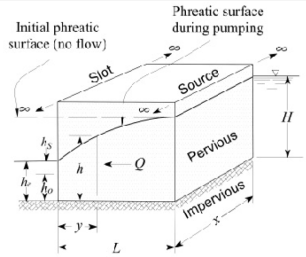
DRAWDOWN

AT ANY DISTANCE y FROM SLOT

$$H - h = \frac{Q}{kDx}(L - y) \qquad (2)$$

$$= \frac{L - y}{L}(H - h_e) \qquad (2a)$$

ARTESIAN FLOW



(b)

FLOW

$$Q = \frac{kx}{2L}(H^2 - h_o^2) \qquad (3)$$

DRAWDOWN

AT ANY DISTANCE y FROM SLOT

$$H^2 - h^2 = \frac{L - y}{L}(H^2 - h_e^2) \qquad (4)$$

WHERE $h_e = h_o + h_s$ AND h_s IS OBTAINED FROM FIG. 36

GRAVITY FLOW

FLOW

$$Q = \frac{kx(D^2 - h_o^2)(2DH - D^2 - h_e^2)}{2L(D^2 - h_e^2)} \qquad (5)$$

WHERE $h_e = h_o + h_s$ AND h_s IS OBTAINED FROM FIG. 36

DRAWDOWN

AT ANY DISTANCE y FROM SLOT

FOR $y \leq L_G$

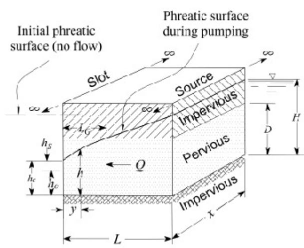
$$H - h = H - \sqrt{\left\{D^2 - \left(\frac{L_G - y}{L_G}\right)[D^2 - (h_o + h_s)^2]\right\}} \qquad (6)$$

FOR $y \geq L_G$

$$H - h = H - \left[\left(\frac{H - D}{L - L_G}\right)(y - L_G) + D\right] \qquad (7)$$

WHERE L_G IS THE DISTANCE FROM THE SLOT TO THE POINT AT WHICH THE FLOW CHANGES FROM ARTESIAN TO GRAVITY, AND IS COMPUTED FROM

$$L_G = \frac{L[D^2 - (h_o + h_s)^2]}{2DH - D^2 - (h_o + h_s)^2} \qquad (8)$$



(c)

COMBINED ARTESIAN-GRAVITY FLOW

Figure 35. Flow and head for fully penetrating line slot; single-line source; artesian, gravity, and combined flows (Adapted from Leonards, 1962 and TM 5-818-5)

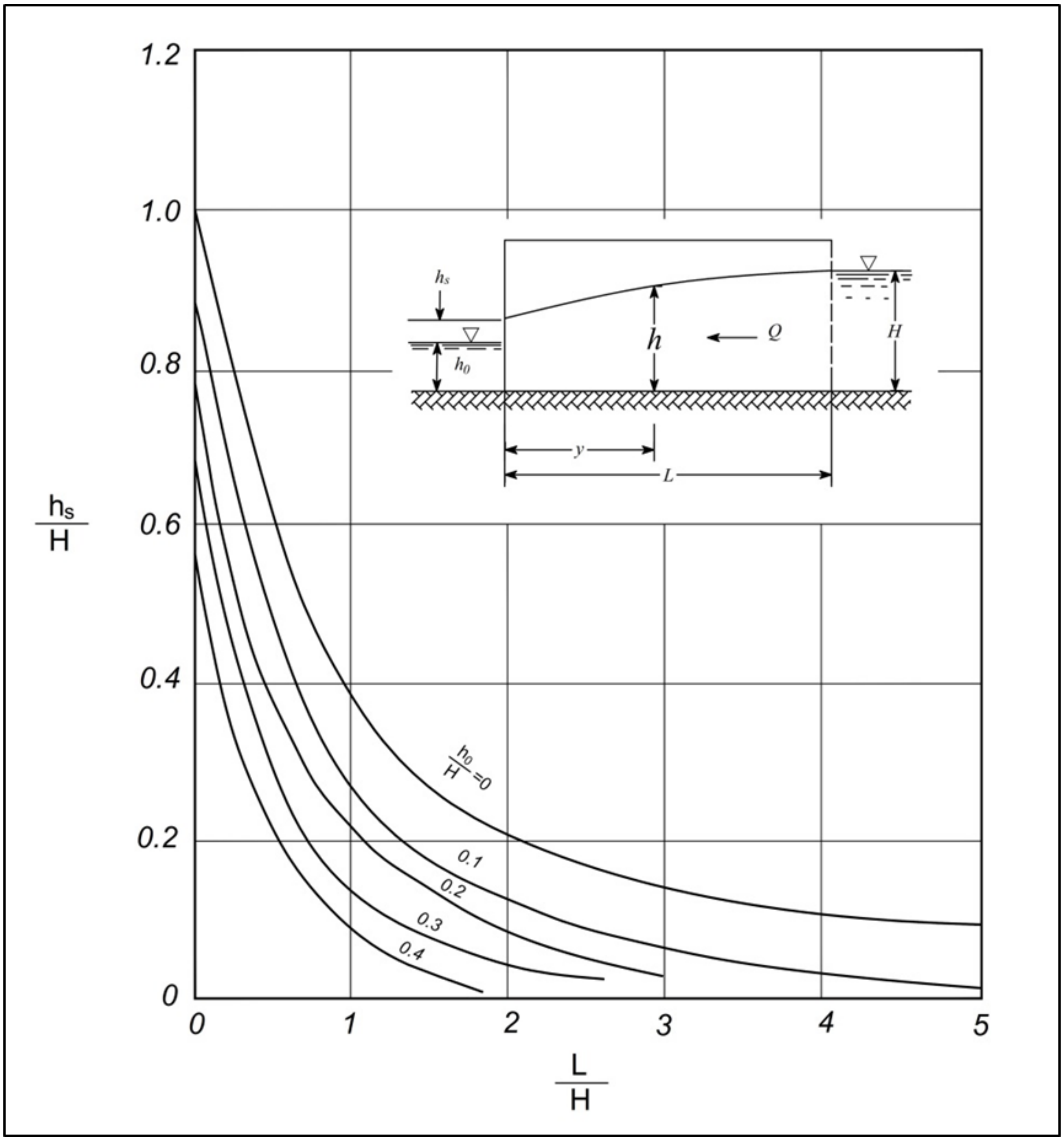
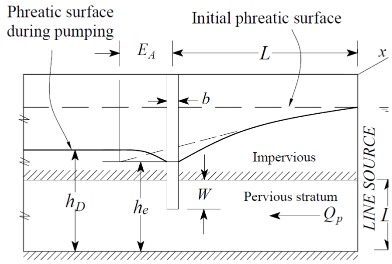


Figure 36. Height of free discharge surface h_s; gravity flow (Adapted from Leonards, 1962 and TM 5-818-5)



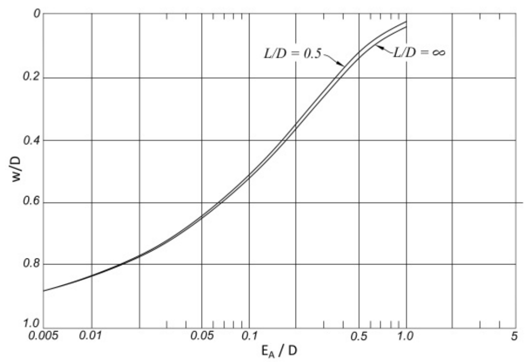
FLOW

$$Q_p = \frac{kDx(H - h_e)}{L + E_A} \quad (1)$$

MAX RESIDUAL HEAD DOWNSTREAM OF SLOT

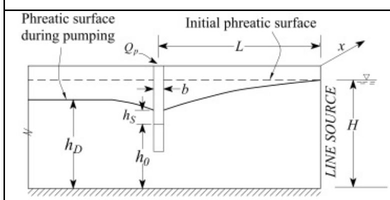
$$h_D = \frac{E_A(H - h_e)}{L + E_A} + h_e \quad (2)$$

WHERE E_A IS AN ADDITIONAL LENGTH FACTOR OBTAINED FROM THE FIGURE BELOW



(b)

ARTESIAN FLOW



h_s IS OBTAINED FROM FIG. 36

(c)

FLOW

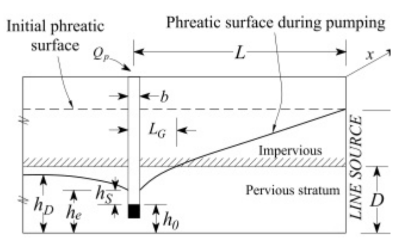
$$Q_p = \left(0.73 + 0.27\frac{H - h_o}{H}\right)\frac{kx}{2L}(H^2 - h_o^2) \quad (3)$$

WHERE $L \geq 3H$

MAX RESIDUAL HEAD DOWNSTREAM OF SLOT

$$h_D = h_o\left[\frac{1.48}{L}(H - h_o) + 1\right] \quad (4)$$

GRAVITY FLOW



h_s IS OBTAINED FROM FIG. 36

(d)

FLOW

$$Q_p = \frac{kDx(H - D)}{L - L_G} \quad (5)$$

MAX RESIDUAL HEAD DOWNSTREAM OF SLOT

$$h_D = h_o\left[\frac{1.48}{L}(D - h_o) + 1\right] \quad (6)$$

PROVIDED $h_D \leq D;\ L_G \geq 3D$

WHERE

$$L_G = \frac{L(D^2 - h_o^2)\left(0.73 + 0.27\frac{D - h_o}{D}\right)}{2D(H - D) + (D^2 - h_o^2)\left(0.73 + 0.27\frac{D - h_o}{D}\right)} \quad (7)$$

COMBINED ARTESIAN AND GRAVITY FLOWS

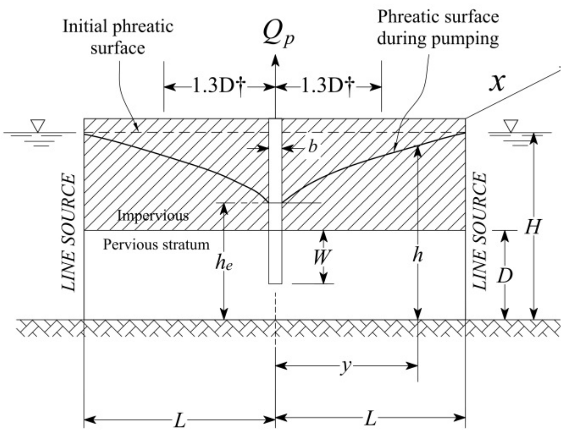
Figure 37. Flow and head for partially penetrating line slot; single-line source; artesian, gravity, and combined flows (Adapted from Leonards, 1962 and TM 5-818-5)

FULLY PENETRATING SLOT

THE FLOW TO A FULLY PENETRATING SLOT FROM TWO LINE SOURCES, BOTH OF INFINITE LENGTH (AND PARALLEL), IS THE SUM OF THE FLOW FROM EACH SOURCE, WITH REGARD TO THE APPROPRIATE FLOW BOUNDARY CONDITIONS, AS DETERMINED FROM THE FLOW EQUATIONS IN FIG. 35. LIKEWISE, THE DRAWDOWN FROM EACH SOURCE CAN BE COMPUTED FROM THE DRAWDOWN EQUATIONS IN FIG. 35 AS IF ONLY ONE SOURCE EXISTED.

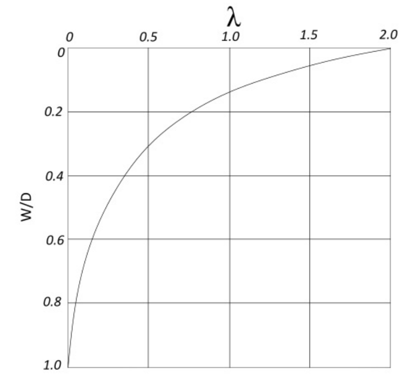
PARTIALLY PENETRATING SLOT

ARTESIAN FLOW



NOTE: WIDTH OF SLOT, b, ASSUMED = 0
† WITHIN THIS DISTANCE (1.3d) THE PIEZOMETRIC SURFACE IS NONLINEAR DUE TO CONVERGING FLOW.

(a)



(b)

FLOW

$$Q_p = \frac{2kDx(H - h_e)}{L + \lambda D} \qquad (1)$$

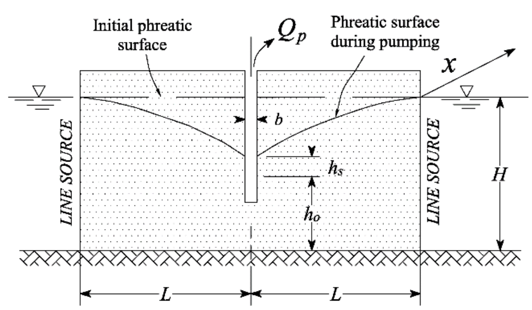
DRAWDOWN

AT ANY DISTANCE $y > 1.3D$ FROM SLOT ‡

$$H - h = H - \left[h_e + (H - h_e)\frac{y + \lambda D}{L + \lambda D}\right] \qquad (2)$$

‡ DRAWDOWN WHEN $y < 1.3D$ CAN BE ESTIMATED BY DRAWING A FREEHAND CURVE FROM h_e TANGENT TO THE SLOPE OF THE LINEAR PART AT $y = 1.3D$.

GRAVITY FLOW



(c)

FLOW

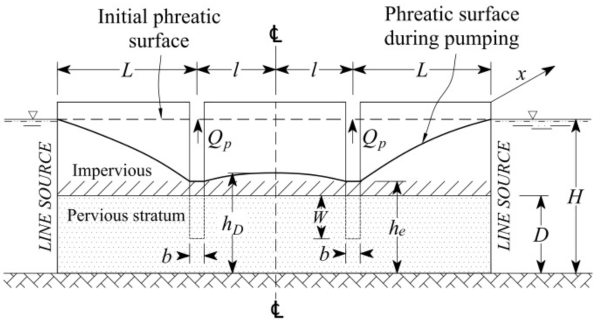
APPROXIMATELY, BUT SOMEWHAT LESS THAN, TWICE THAT COMPUTED FROM A SINGLE SOURCE, EQ 3, FIG. 37.

DRAWDOWN

APPROXIMATELY THAT COMPUTED FROM A SINGLE SOURCE, EQ 4, FIG. 35.

Figure 38. Flow and head for full and partially penetrating line slot; two-line source; artesian and gravity flows (Adapted from Leonards, 1962 and TM 5-818-5)

A FREQUENTLY ENCOUNTERED DEWATERING SYSTEM IS ONE WITH TWO LINES OF PARTIALLY PENETRATING WELLPOINTS ALONG EACH SIDE OF A LONG EXCAVATION, WHERE THE FLOW CAN BE ASSUMED TO ORIGINATE FROM TWO EQUIDISTANT LINE SOURCES.



(a)

FLOW

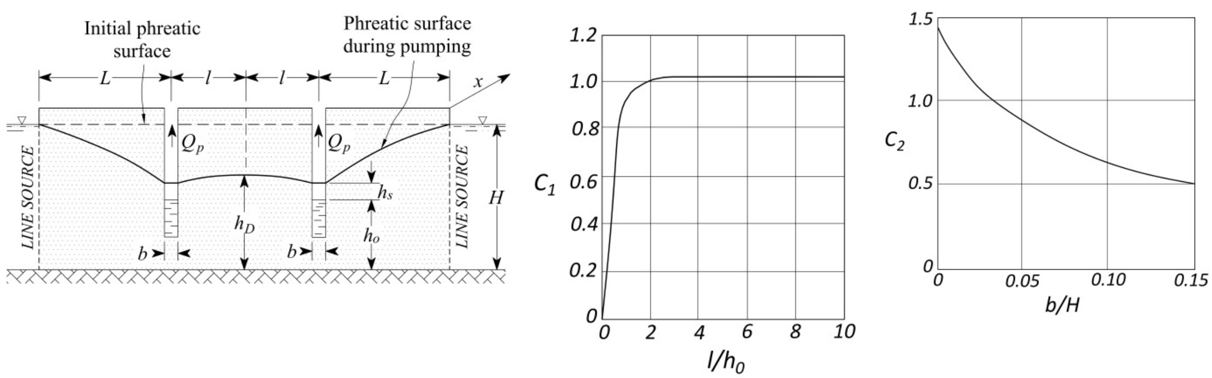
FLOW FOR EACH SLOT CAN BE ESTIMATED AS FOR ONE SLOT WITH ONE LINE SOURCE, EQ 1, FIG. 37.

h_D†

VALUE OF h_D CAN BE ESTIMATED AS FOR ONE SLOT AND ONE LINE SOURCE EQ 2, FIG. 37.

† MAXIMUM RESIDUAL HEAD MIDWAY BETWEEN THE TWO SLOTS.

ARTESIAN FLOW



(b) (c) (d)

FLOW

FLOW TO EACH SLOT APPROXIMATELY THAT OF ONE SLOT WITH ONE LINE SOURCE, EQ 3, FIG. 37.

h_D†

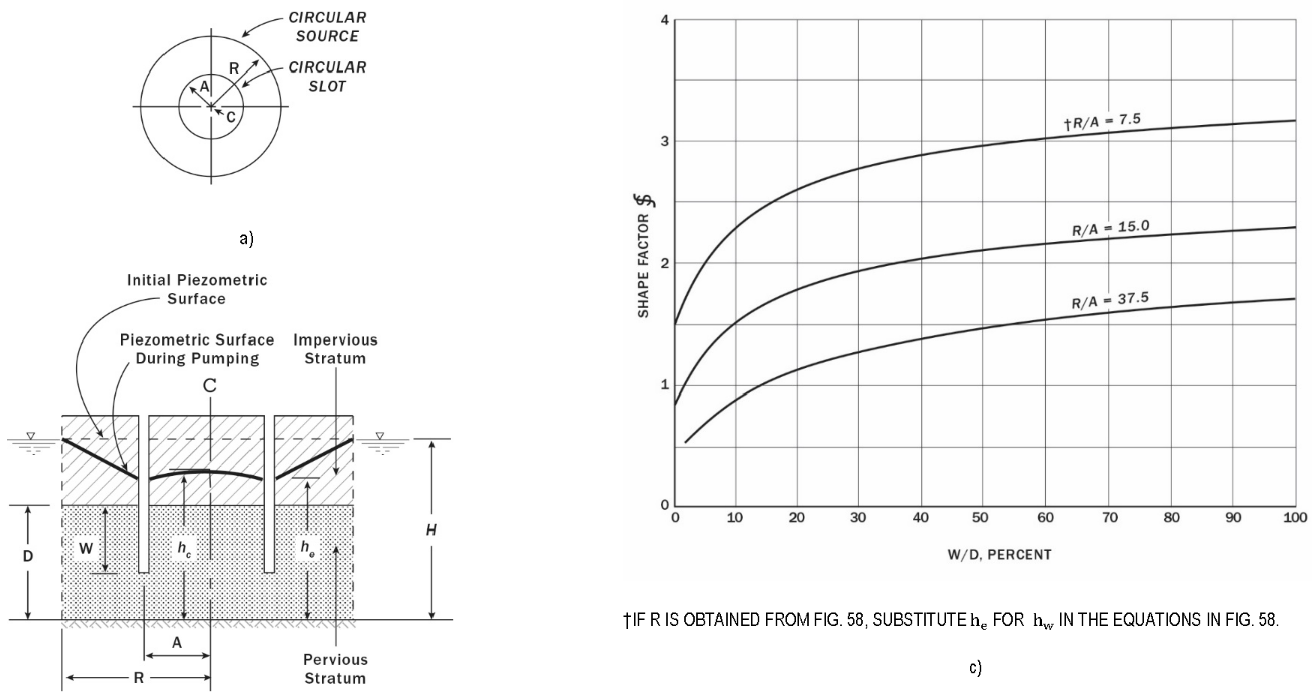
$$h_D = h_o\left[\frac{C_1 C_2}{L}(H - h_o) + 1\right] \quad (1)$$

WHERE C_1 AND C_2 ARE OBTAINED FROM FIG (c) AND (d) ABOVE.

† MAXIMUM RESIDUAL HEAD MIDWAY BETWEEN THE TWO SLOTS.

GRAVITY FLOW

Figure 39. Flow and head (midway) for two partially penetrating slots; two-line source; artesian and gravity flows (Adapted from Leonards, 1962 and TM 5-818-5)



FLOW, Q_w, OR DRAWDOWN $(H - h_e)$ CAN BE ESTIMATED FROM

$$Q_w = (H - h_e)kD\$$$

WHERE

$\$$ = SHAPE FACTOR FOR ARTESIAN FLOW OBTAINED FROM (c)

h_c CAN BE OBTAINED FROM PLOTS IN FIG. 41

Figure 40. Flow and head for fully and partially penetrating circular slots; circular source; artesian flow (Adapted from TM 5-818-5)

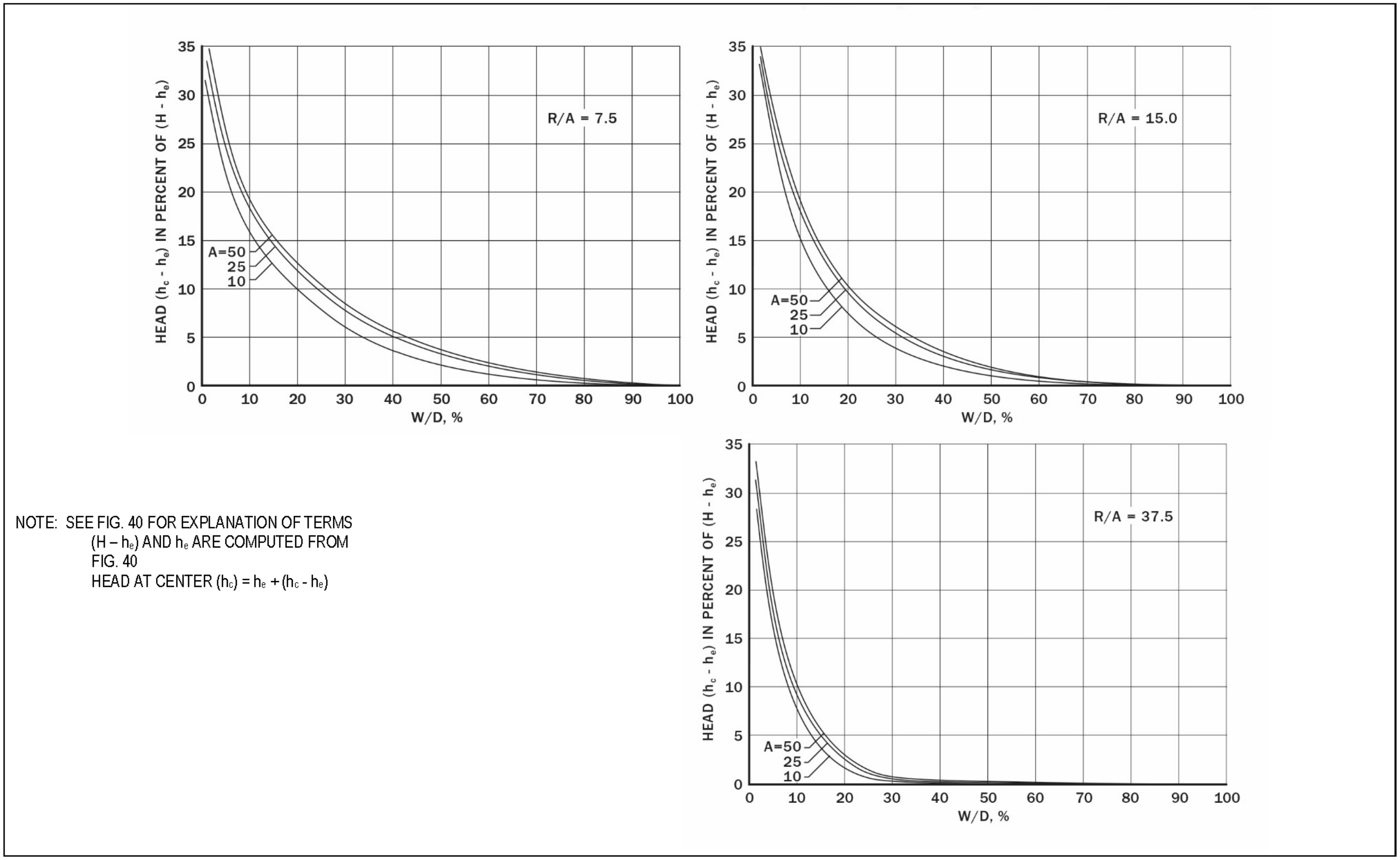


Figure 41. Head at center of fully and partially penetrating circular slots; circular source; artesian flow (Adapted from TM 5-818-5)

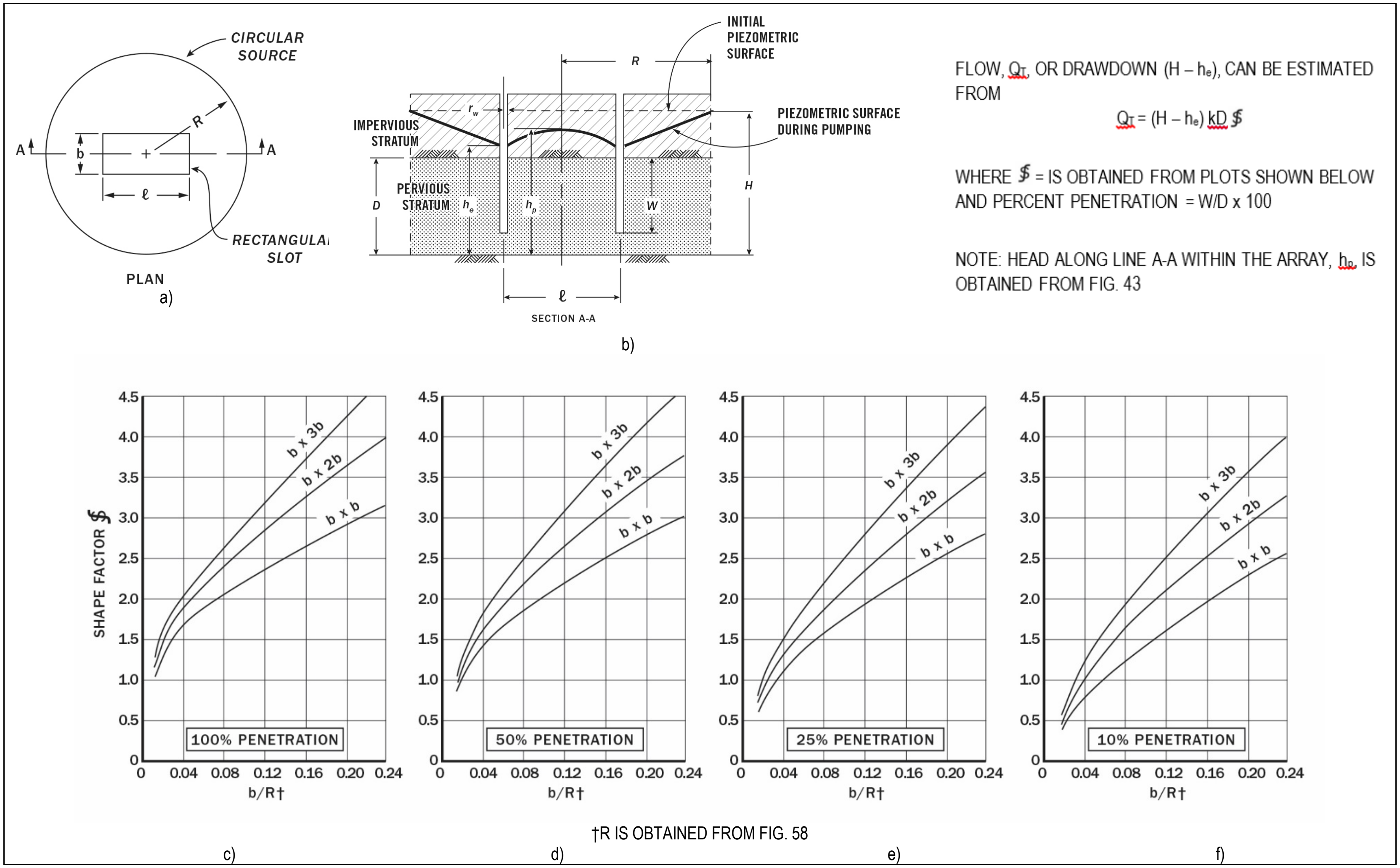


Figure 42. Flow and drawdown at slot for fully and partially penetrating rectangular slots; circular source; artesian flow (Adapted from TM 5-818-5)

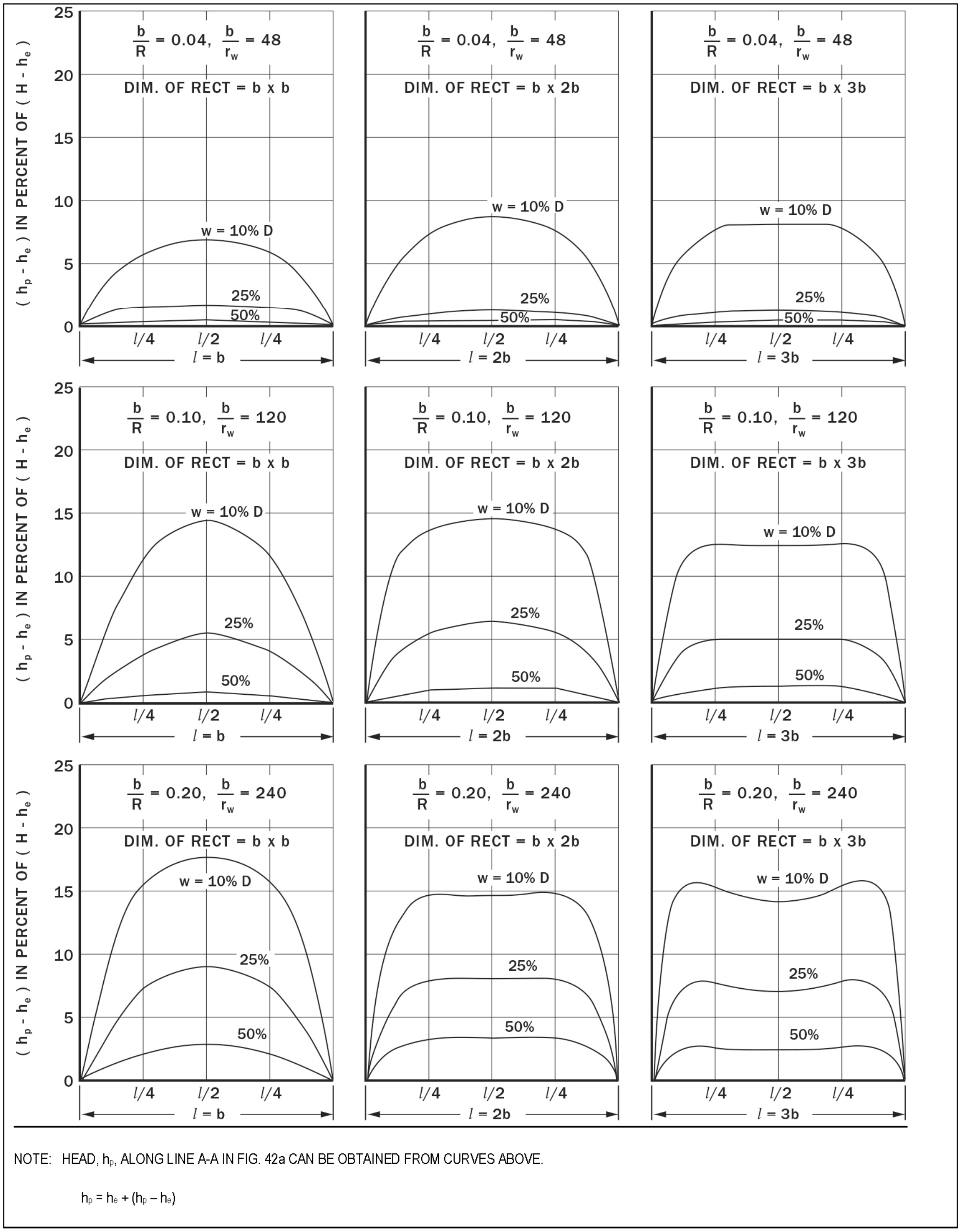
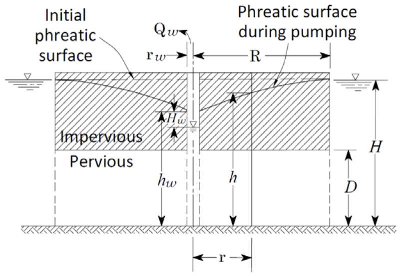
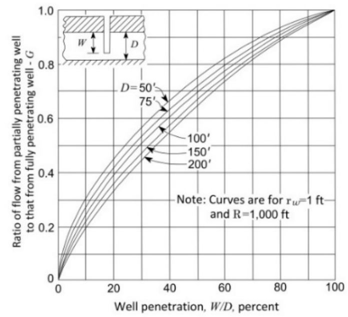


Figure 43. Head within a partially penetrating rectangular slot; circular source; artesian flow (Adapted from TM 5-818-5)



HYDRAULIC HEAD LOSS, H_w, IS OBTAINED FROM FIG. 59
RADIUS OF INFLUENCE, R, IS OBTAINED FROM FIG. 58
(a)



(b)

FULLY PENETRATING WELL

FLOW, Q_w

$$Q_w = \frac{2\pi kD(H-h)}{\ln(R/r)} \quad (1) \quad \text{OR} \quad Q_w = \frac{2\pi kD(H-h_w)}{\ln(R/r_w)} \quad (2)$$

DRAWDOWN, H – h

$$H - h = \frac{H-h_w}{\ln(R/r_w)}\ln\left(\frac{R}{r}\right) \quad (3)$$

PARTIALLY PENETRATING WELL

FLOW, Q_{wp}

$$Q_{wp} = \frac{2\pi kD(H-h_w)G}{\ln(R/r_w)} = Q_{w-100\%} \times G \quad (4)$$

WHERE G IS EQUAL TO THE RATIO OF FLOW FROM A PARTIALLY PENETRATING WELL, Q_{wp}, TO THAT FOR A FULLY PENETRATING WELL FOR THE SAME DRAWDOWN, H - h_w, AT THE PERIPHERY OF THE WELLS.
APPROXIMATE VALUES OF G CAN BE COMPUTED FROM THE FORMULA:

$$G = \frac{W}{D}\left(1 + 7\sqrt{r_w/2W}\cos\frac{\pi W/D}{2}\right) \quad (5)$$

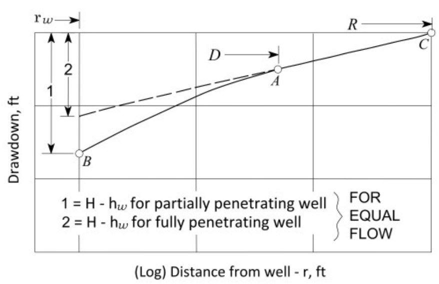
MORE EXACT VALUES CAN BE COMPUTED FROM THE FORMULA:

$$G = \frac{\ln(R/r_w)}{\frac{D}{2W}\left[2\ln\frac{4D}{r_w} - \ln\frac{\Gamma(0.875W/D)\Gamma(0.125W/D)}{\Gamma(1-0.875W/D)\Gamma(1-0.125W/D)}\right] - \ln\frac{4D}{R}} \quad (6)$$

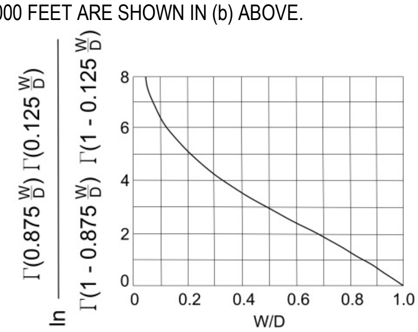
WHERE Γ IS THE GAMMA FUNCTION; W = WELL PENETRATION.
VALUES OF G FOR A TYPICAL LARGE-DIAMETER WELL (r_w = 1.0 FEET) WITH A RADIUS OF INFLUENCE OF 1,000 FEET ARE SHOWN IN (b) ABOVE.

DRAWDOWN, H – h

THE SHAPE OF THE DRAWDOWN CURVE IN THE VICINITY OF A PARTIALLY PENETRATING WELL CANNOT BE DETERMINED DIRECTLY FROM EQ 4 BUT CAN BE APPROXIMATED BY ASSUMING THE EFFECT OF WELL PENETRATION, W, IS INSIGNIFICANT BEYOND A DISTANCE, r, THAT IS GREATER THAN D. THE DRAWDOWN IS APPROXIMATED AS FOLLOWS:



(c)

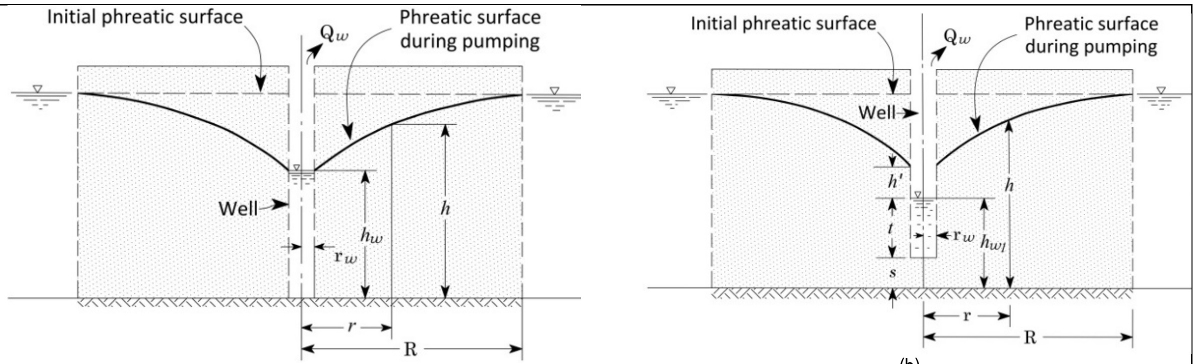


(d)

1. COMPUTE Q_{wp} FROM EQ 4 FOR A GIVEN DRAWDOWN OF 1 ON (c).
2. COMPUTE H – h_w FROM EQ 2 FOR A FULLY PENETRATING WELL FOR A DISCHARGE OF Q_{wp} (2 ON (c)).
3. PLOT DRAWDOWN FOR FULLY PENETRATING WELL VS (LOG)r AS SHOWN BY LINE AC IN (c).
4. DRAW A CURVED LINE FROM THE POINT (h_w, r_w) – POINT B IN ILLUSTRATION – FOR THE PARTIALLY PENETRATING WELL TO POINT A.

THE COMBINED CURVE, BAC, REPRESENTS AN APPROXIMATION OF THE DRAWDOWN CURVE FOR A PARTIALLY PENETRATING ARTESIAN WELL.

Figure 44. Flow and drawdown for fully and partially penetrating single wells; circular source; artesian flow (Adapted from Leonards, 1962 and TM 5-818-5)



(a) (b)

(SEE FIG. 58 FOR DETERMINING R.)

FULLY PENETRATING WELL

FLOW, Q_w, OR DRAWDOWN, $H^2 - h^2$; NEGLECTING HEIGHT OF FREE DISCHARGE, h' (CONDITION (a))

$$Q_w = \frac{\pi k(H^2 - h^2)}{\ln(R/r)} \quad 1) \qquad \textbf{\textit{OR}} \qquad Q_w = \frac{\pi k(H^2 - h_w^2)}{\ln(R/r_w)} \quad 2)$$

FLOW, Q_w; TAKING h' INTO ACCOUNT (b) CAN BE ESTIMATED ACCURATELY FROM EQ 2 USING HEIGHT OF WATER, t + s (s = 0 FOR FULLY PENETRATING WELL), FOR THE TERM h_w.

FULLY OR PARTIALLY PENETRATING WELL

FLOW, Q_w; FOR ANY GRAVITY WELL WITH A CIRCULAR SOURCE

$$Q_w = \frac{\pi k[(H-s)^2 - t^2]}{\ln(R/r_w)}\left[1 + \left(0.30 + \frac{10r_w}{H}\right)\sin\frac{1.8s}{H}\right] \quad 3)$$

DRAWDOWN, H – h OR $H^2 - h^2$; WHERE h' IS ACCOUNTED FOR (OBTAIN Q_w FROM EQ 3)

WHERE r > 1.5H,

$$H^2 - h^2 = \frac{Q_w}{\pi k}\ln\frac{R}{r} \quad 4)$$

WHERE r < 1.5H,

FOR r / h > 1.5, USE EQ 4

FOR r / h < 1.5,

$$H - h = \frac{Q_w P \ln(10R/H)}{\pi k H[1 - 0.8(s/H)^{1.5}]} \quad 5)$$

FOR 0.3 < r / h < 1.5,

$$P = 0.13 \ln R/r \quad 6)$$

FOR r / h < 0.3,

$$P = \overline{C_x} + \Delta C \quad 7)$$

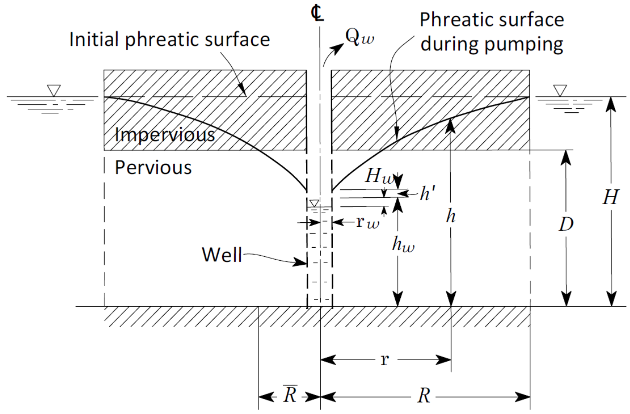
WHERE

$$\overline{C_x} = 0.13\ln\frac{R}{r} - 0.0123\ln^2\frac{R}{10r} \quad 8)$$

AND

$$\Delta C = \frac{s}{h}\left[\left(\frac{1}{2.3}\ln\frac{R}{10r}\right)\left(1.2\frac{s}{H} - 0.48\right) + 0.113\ln\frac{2.4H}{R}\ln\frac{R}{34r}\right] \quad 9)$$

Figure 45. Flow and drawdown for fully and partially penetrating single wells; circular source; gravity flow (Adapted from Leonards, 1962 and TM 5-818-5)



<u>FLOW, Q_w;</u> CAN BE COMPUTED FROM

$$Q_w = \frac{\pi k(2DH - D^2 - h_w^2)}{\ln(R/r_w)} \tag{1}$$

<u>DRAWDOWN, H – h;</u> CAN BE COMPUTED AT ANY DISTANCE FROM

$$H - h = H - \left(\frac{H-D}{\ln(R/r_w)}\ln\frac{r}{r_w} + \sqrt{D^2 - \frac{D^2 - h_w^2}{\ln(R/r_w)}\ln\frac{R}{r}}\right) \tag{2}$$

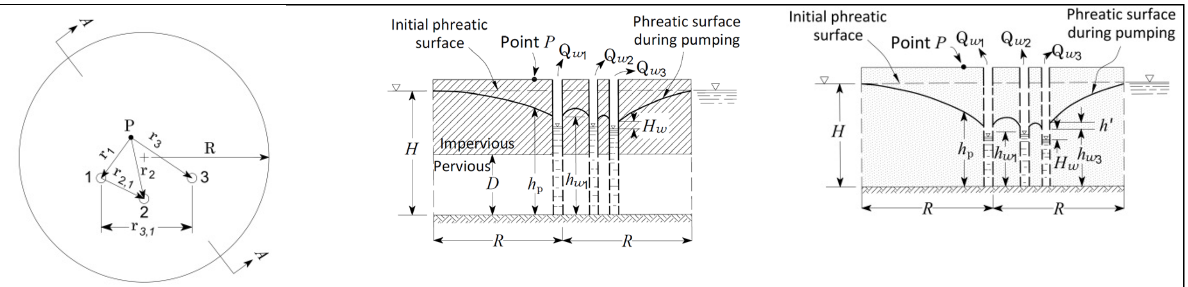
$\overline{R}$; DISTANCE FROM WELL AT WHICH FLOW CHANGES FROM GRAVITY TO ARTESIAN CAN BE COMPUTED FROM

$$\ln\overline{R} = \frac{(D^2 - h_w^2)\ln R + 2D(H-D)\ln r_w}{2DH - D^2 - h_w^2} \tag{3}$$

R IS DETERMINED FROM FIG 58.

EQUATIONS 1 AND 2 ARE BASED ON THE ASSUMPTION THAT THE HEAD h_w AT THE WELL IS AT THE SAME ELEVATION AS THE WATER SURFACE IN THE WELL. THIS WILL NOT BE TRUE WHERE THE DRAWDOWN IS RELATIVELY LARGE. IN THE LATTER CASE, THE HEAD AT AND IN THE CLOSE VICINITY OF THE WELL CAN BE COMPUTED FROM EQ 4 THROUGH 9 (FIG 45). IN THESE EQUATIONS, THE VALUE OF Q_w USED IS THAT COMPUTED FROM EQ 1, ASSUMING h_w IS EQUAL TO THE HEIGHT OF WATER IN THE WELL, AND THE VALUE OF $\overline{R}$ COMPUTED FROM EQ 3 IS USED IN LIEU OF R.

Figure 46. Flow and drawdown for fully penetrating single well; circular source; combined artesian and gravity flows (Adapted from Leonards, 1962 and TM 5-818-5)



(a) (b) ARTESIAN FLOW (SECTION A-A) (c) GRAVITY FLOW (SECTION A-A)

ARTESIAN FLOW

DRAWDOWN ($H - h_p$) AT ANY POINT P

$$H - h_p = \frac{F}{2\pi kD} \qquad (1)$$

WHERE

$$F \dagger = \sum_{i=1}^{i=n} Q_{wi} \ln\left(\frac{R_i}{r_i}\right) \qquad (2)$$

AND Q_{wi} = FLOW FROM WELL i

r_i = DISTANCE FROM WELL i TO POINT P

R_i = RADIUS OF INFLUENCE FOR WELL i ‡

n = NUMBER OF WELLS IN THE ARRAY

GRAVITY FLOW

DRAWDOWN ($H^2 - h_p^2$) AT ANY POINT P

$$H^2 - h_p^2 = \frac{F}{\pi k} \qquad (3)$$

WHERE F IS COMPUTED FROM EQ 2

ARTESIAN OR GRAVITY FLOW

DRAWDOWN AT ANY WELL, j, FOR ARTESIAN OR GRAVITY FLOW CAN BE COMPUTED FROM EQ 1 OR 3 RESPECTIVELY, SUBSTITUTING F_w FOR F

WHERE

$$F_w = Q_{wj} \ln\left(\frac{R_j}{r_{wj}}\right) + \sum_{i=1}^{i=n-1} Q_{wi} \ln\left(\frac{R_i}{r_{ij}}\right) \qquad (4)$$

AND Q_{wj} = FLOW FROM WELL j

R_j = RADIUS OF INFLUENCE FOR WELL j

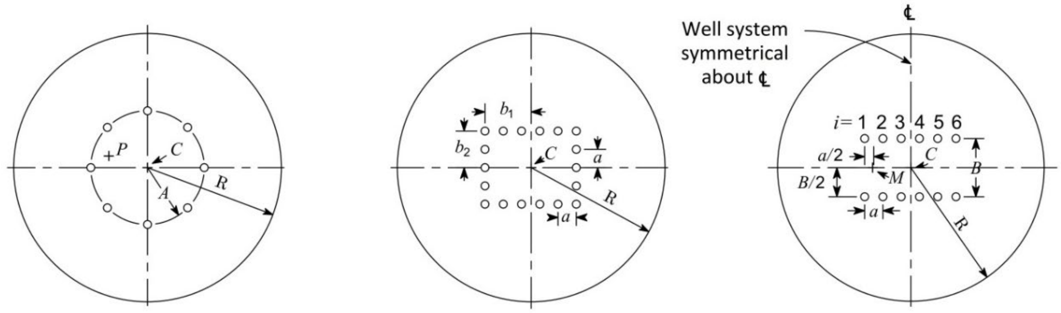
r_{wj} = EFFECTIVE WELL RADIUS OF WELL j

r_{ij} = DISTANCE FROM EACH WELL TO WELL j

† DRAWDOWN FACTORS, F, FOR SEVERAL COMMON WELL ARRAYS ARE GIVEN IN FIG. 48.

‡ FOR RELATIVELY SMALL DEWATERING SYSTEMS AND WHERE NO UNUSUAL BOUNDARY CONDITIONS EXIST, THE RADIUS OF INFLUENCE FOR ALL WELLS CAN BE ASSUMED CONSTANT AS IN (a) ABOVE. SEE FIG. 58 FOR DETERMINING THE VALUE OF R.

Figure 47. Flow and drawdown for fully penetrating multiple wells; circular source; artesian and gravity flows (Adapted from Leonards, 1962 and TM 5-818-5)



ARRAY 1 ARRAY 2 ARRAY 3

ALL WELLS ARE <u>FULLY PENETRATING</u> WITH A CIRCULAR SOURCE. THE FLOW, Q_w, FROM ALL WELLS IS <u>EQUAL</u>.

F_w = DRAWDOWN FACTOR FOR ANY WELL IN THE ARRAY.
F_m = DRAWDOWN FACTOR AT POINT M IN ARRAY 3.
F_c = DRAWDOWN FACTOR FOR CENTER OF THE ARRAY.

r_w = EFFECTIVE WELL RADIUS
h_c = HEAD AT POINT C
n, R, Q_w, H, h_p, h_w, r_i, r_{wj}, r_{ij} ARE DEFINED IN FIG. 47.

ARRAY 1. CIRCULAR ARRAY OF EQUALLY SPACED WELLS

$$F_w = Q_w \ln \frac{R^n}{n r_w A^{(n-1)}} \qquad (1)$$

$$F_c = nQ_w \ln R/A \qquad (2)$$

WHERE A = DIMENSION SHOWN IN ARRAY 1 ABOVE.

DRAWDOWN AT POINTS P AND C FOR <u>ARTESIAN</u> FLOW CAN BE COMPUTED FROM

$$\text{Point P: } (H - h_p) = \frac{(H - h_w)\left(n \ln R \sum_{i=1}^{i=n} \ln r_i\right)}{\ln \frac{R^n}{n r_w A^{(n-1)}}} \qquad (3)$$

$$\text{Point C: } (H - h_c) = \frac{(H - h_w) n \ln(R/A)}{\ln \frac{R^n}{n r_w A^{(n-1)}}} \qquad (4)$$

DRAWDOWN AT C FOR <u>GRAVITY</u> FLOW CAN BE COMPUTED FROM

$$(H - h_c) = H - \sqrt{H^2 - \frac{n(H^2 - h_w^2)\ln(R/A)}{\ln \frac{R^n}{n r_w A^{(n-1)}}}} \qquad (5)$$

ARRAY 2. RECTANGULAR ARRAY OF EQUALLY SPACED WELLS

F_w AND F_c MAY BE APPROXIMATED FROM EQ 1 AND 2, RESPECTIVELY, IF A_e IS SUBSTITUTED FOR A AND

$$A_e = \frac{4}{\pi}\sqrt{b_1 b_2} \qquad (6)$$

F_w AND F_c CAN BE COMPUTED MORE EXACTLY FROM

$$F_w = Q_w \ln \frac{R}{r_{wj}} + \sum_{i=1}^{i=n-1} Q_w \ln \frac{R}{r_{ij}} \qquad (7)$$

$$F_c = \sum_{i=1}^{i=n} Q_w \ln \frac{R}{r_i} \qquad (8)$$

ARRAY 3. TWO PARALLEL LINES OF EQUALLY SPACED WELLS

$$F_c = 4Q_w \sum_{i=1}^{i=n/4} \ln \frac{R}{\frac{1}{2}\sqrt{a^2(2i-1)^2 + B^2}} \qquad (9)$$

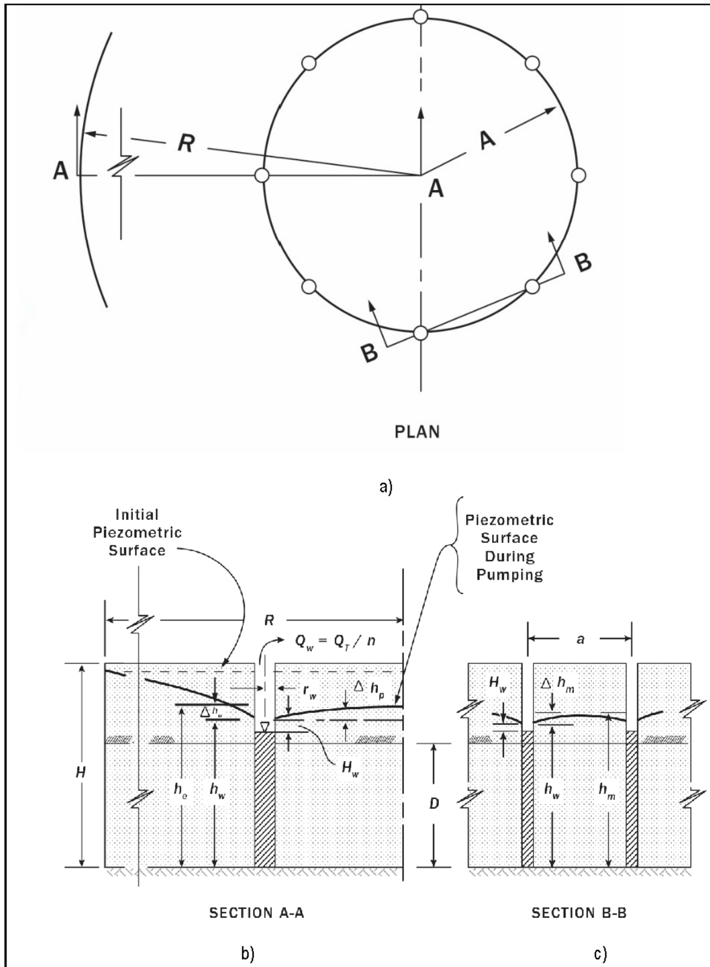
$$F_m = 2Q_w \sum_{i=1}^{i=n/2} \ln \frac{R}{\frac{1}{2}\sqrt{a^2(2i-3)^2 + B^2}} \qquad (10)$$

WHERE i = WELL NUMBER AS SHOWN IN THE ARRAY ABOVE.

NOTE THAT THE LOCATION OF M IS MIDWAY BETWEEN THE TWO LINES OF WELLS AND CENTERED BETWEEN THE END TWO WELLS OF THE LINE. THIS POINT CORRESPONDS TO THE LOCATION OF THE MINIMUM DRAWDOWN WITHIN THE ARRAY.

VALUES DETERMINED FOR F_w, F_c, AND F_m ARE SUBSTITUTED FOR F IN EQ 1 AND 3 (FIG. 47) TO COMPUTE DRAWDOWN AT THE RESPECTIVE POINTS.

Figure 48. Drawdown factors for fully penetrating circular, rectangular and two-line well arrays; circular source; artesian and gravity flows (Adapted from Leonards, 1962 and TM 5-818-5)



FULLY PENETRATING WELL

DRAWDOWN, $H - h_e$, PRODUCED BY PUMPING A FLOW OF Q_T FROM AN EQUIVALENT SLOT IS COMPUTED FROM EQ 1 (FIG.40 OR FIG. 44) ($Q_T = nQ_W$), n = NUMBER OF WELLS; Q_w = FLOW FROM A WELL)

HEAD LOSS DUE TO CONVERGING FLOW AT WELL

$$\Delta h_w = \frac{Q_w}{2\pi kD} \ln \frac{a}{2\pi r_w} = \frac{(H - h_e)\mathcal{S}}{2\pi n} \ln \frac{a}{2\pi r_w} \qquad (1)$$

TOTAL DRAWDOWN AT WELL (NEGLECTING HYDRAULIC HEAD LOSS, H_w)

$$H - h_w = H - h_e + \Delta h_w = \frac{Q_T}{kD}\left(\frac{1}{\mathcal{S}} + \frac{1}{2\pi n} ln \frac{a}{2\pi r_w}\right) \qquad (2)$$

HEAD INCREASE MIDWAY BETWEEN WELLS

$$\Delta h_m = \frac{Q_w}{2\pi kD} \ln \frac{a}{\pi r_w} = \frac{(H - h_e)\mathcal{S}}{2\pi r_w} \ln \frac{a}{\pi r_w} \qquad (3)$$

DRAWDOWN MIDWAY BETWEEN WELLS

$$H - h_m = H - h_w + \Delta h_m = \frac{Q_T}{kD}\left(\frac{1}{\mathcal{S}} - \frac{0.110}{n}\right) \qquad (4)$$

HEAD INCREASE IN CENTER OF A RING OF WELLS, Δh_D, IS EQUAL TO Δh_w AND CAN BE COMPUTED FROM EQ 1. DRAWDOWN AT THE CENTER OF THE RING OF WELLS, $H - h_D$, IS EQUAL TO $H - h_w - \Delta h_w$ OR $H - h_e$ AND, CONSEQUENTLY, CAN BE COMPUTED FROM FIG. 40.

FOR EQ 1 THROUGH 4:

- FLOWS FROM ALL WELLS ARE EQUAL
- SHAPE FACTOR $\mathcal{S}$ IS OBTAINED FROM FIG. 40c.
- k = HYDRAULIC CONDUCTIVITY
- ALL OTHER TERMS ARE EXPLAINED IN a, b, and c

HYDRAULIC HEAD LOSS IN WELL (H_W) IS OBTAINED FROM FIG. 59.

Figure 49. Flow and drawdown for fully penetrating circular well arrays; circular source; artesian flow (Adapted from TM 5-818-5)

SEE FIGURE 49 a, b, AND c FOR EXPLANATION OF TERMS NOT DEFINED IN THIS FIGURE.

DRAWDOWN $H - h_e$, PRODUCED BY PUMPING A FLOW OF Q_T FROM AN EQUIVALENT SLOT, IS COMPUTED FROM FIG. 40 FOR CIRCULAR SLOT AND FROM FIG 42 FOR RECTANGULAR SLOT.
HEAD LOSS DUE TO CONVERGING FLOW AT WELL

$$\Delta h_W = \frac{Q_w \theta_a}{kD} = \frac{(H - h_e) \mathcal{F} \theta_a}{n} \quad (1)$$

TOTAL DRAWDOWN AT WELL (NEGLECTING H_W)

$$H - h_w = H - h_e + \Delta h_w = \frac{Q_T}{kD}\left(\frac{1}{\mathcal{F}} + \frac{\theta_a}{n}\right) \quad (2)$$

HEAD INCREASE MIDWAY BETWEEN WELLS

$$\Delta h_m = \frac{Q_w \theta_m}{kD} = \frac{(H - h_e) \mathcal{F} \theta_m}{n} \quad (3)$$

DRAWDOWN MIDWAY BETWEEN WELLS

$$H - h_m = H - h_w + \Delta h_m = \frac{Q_T}{kD}\left[\frac{1}{\mathcal{F}} + \frac{1}{n}(\theta_a - \theta_m)\right] \quad (4)$$

HEAD INCREASE IN CENTER OF A RING OF WELLS, Δh_D, IS EQUAL TO Δh_W AND CAN BE COMPUTED FROM EQ 1.

DRAWDOWN AT THE CENTER OF A RING OF WELLS, $H - h_D$, IS EQUAL TO $H - h_w - \Delta h_w$ OR $H - h_e$ AND, CONSEQUENTLY, CAN BE COMPUTED FROM FIG. 40.

FOR EQ 1 THROUGH 4: $h_e = h_w + \Delta h_w$

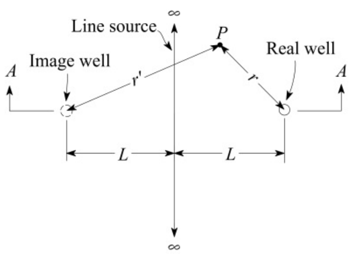
FLOWS FROM ALL WELLS ARE EQUAL.

θ_a AND θ_m ARE DRAWDOWN FACTORS OBTAINED FROM FIG. 55 (a AND b, RESPECTIVELY)

$\mathcal{F}$ FROM FIG. 40 AND 42

Figure 50. Flow and drawdown for partially penetrating circular and rectangular well arrays; circular source; artesian flow (Adapted from TM 5-818-5)

EQUATIONS FOR FLOW AND DRAWDOWN FOR A FULLY PENETRATING WELL WITH A LINE SOURCE OF INFINITE LENGTH WERE DEVELOPED UTILIZING THE METHOD OF IMAGE WELLS. THE IMAGE WELL IS CONSTRUCTED AS SHOWN IN (a) BELOW.



(a)

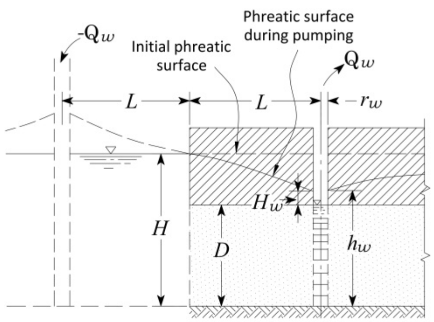
ARTESIAN FLOW

FLOW, Q_w

$$Q_w = \frac{2\pi k D(H - h_w)}{\ln(2L/r_w)} \tag{1}$$

DRAWDOWN AT ANY POINT, P, LOCATED A DISTANCE, r, FROM THE WELL.

$$H - h = \frac{Q_w}{2\pi k D}\ln\left(\frac{r'}{r}\right) \tag{2}$$



(b) ARTESIAN FLOW

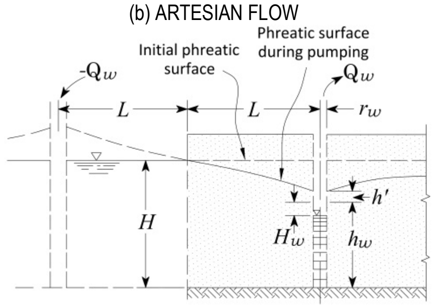
GRAVITY FLOW

FLOW, Q_w

$$Q_w = \frac{\pi k(H^2 - h_w^2)}{\ln(2L/r_w)} \tag{3}$$

DRAWDOWN AT ANY POINT, P, LOCATED A DISTANCE, r, FROM THE WELL.

$$H^2 - h^2 = \frac{Q_w}{\pi k}\ln\left(\frac{r'}{r}\right) \tag{4}$$



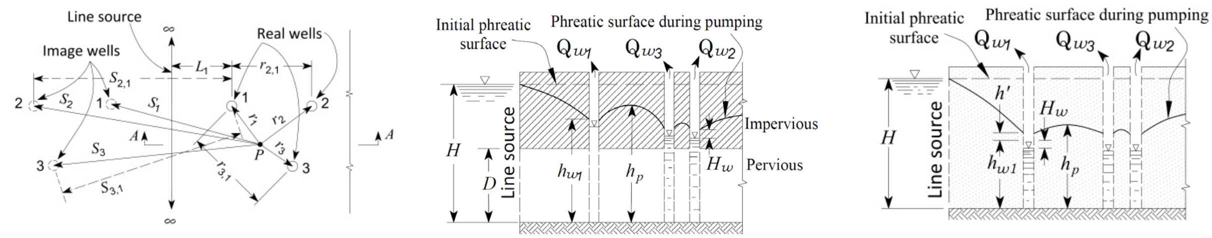
(c) GRAVITY FLOW

IN THE EQUATIONS ABOVE, THE DISTANCE TO THE LINE SOURCE MUST BE COMPARED TO THE CIRCULAR RADIUS OF INFLUENCE, R, FOR THE WELL. IF 2L IS GREATER THAN R, THE WELL WILL PERFORM AS IF SUPPLIED BY A CIRCULAR SOURCE OF SEEPAGE, AND SOLUTIONS FOR A LINE SOURCE OF SEEPAGE ARE NOT APPLICABLE.

SEE FIG. 57 FOR DETERMINING THE VALUE OF R.

SEE FIG. 58 FOR DETERMINING THE VALUE OF H_w.

Figure 51. Flow and drawdown for fully penetrating single well; line source; artesian and gravity flow (Adapted from Leonards, 1962 and TM 5-818-5)



(a) (b) ARTESIAN FLOW (c) GRAVITY FLOW

ARTESIAN FLOW

DRAWDOWN ($H - h_p$) AT ANY POINT P

$$H - h_p = \frac{F'_p}{2\pi kD} \quad (1)$$

WHERE

$$F'_p \dagger = \sum_{i=1}^{i=n} Q_{wi} \ln\left(\frac{S_i}{r_i}\right) \quad (2)$$

AND Q_{wi} = FLOW FROM WELL i

S_i = DISTANCE FROM IMAGE WELL i TO POINT P

r_i = DISTANCE FROM WELL i TO POINT P

n = NUMBER OF REAL WELLS

GRAVITY FLOW

DRAWDOWN ($H^2 - h_p^2$) AT ANY POINT P

$$H^2 - h_p^2 = \frac{F'_p}{\pi k} \quad (3)$$

WHERE $F'_p\dagger$ IS COMPUTED FROM EQ 2.

ARTESIAN OR GRAVITY FLOW

DRAWDOWN AT ANY WELL, j, FOR ARTESIAN OR GRAVITY FLOW CAN BE COMPUTED FROM EQ 1 OR 3 RESPECTIVELY, SUBSTITUTING F'_w for F'_p

WHERE

$$F'_w = Q_{wj} \ln\left(\frac{2L_j}{r_{wj}}\right) + \sum_{i=2}^{i=n} Q_{wi} \ln\left(\frac{S_{ij}}{r_{ij}}\right) \quad (4)$$

AND Q_{wj} = FLOW FROM WELL j

Q_{wi} = FLOW FROM WELL i

L_j = DISTANCE FROM LINE SOURCE TO WELL j

S_{ij} = DISTANCE FROM IMAGE WELL i TO WELL j

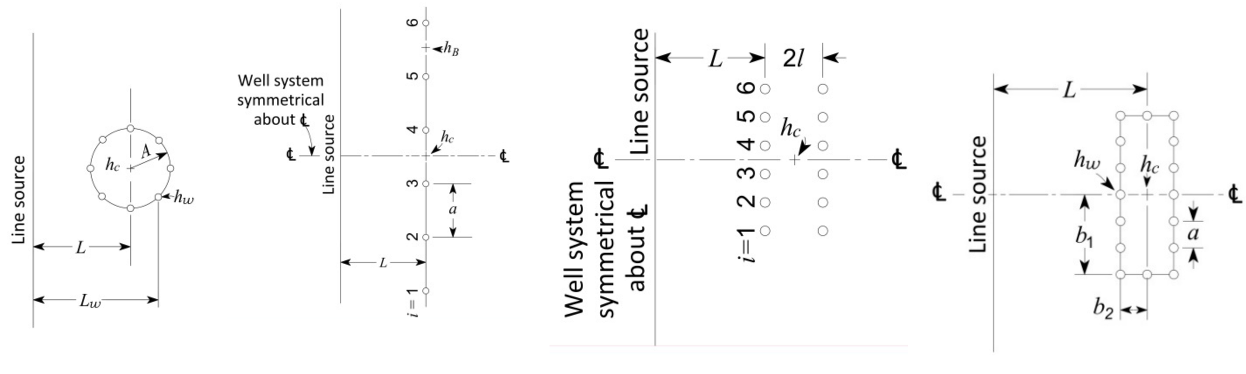
r_w = RADIUS OF WELL

n = NUMBER OF REAL WELLS

r_{ij} = DISTANCE FROM EACH WELL TO WELL j

† DRAWDOWN FACTORS, F', FOR SEVERAL COMMON WELL ARRAYS ARE GIVEN IN FIG. 53.

Figure 52. Drawdown for group of fully penetrating wells; line source; artesian and gravity flows (Adapted from Leonards, 1962 and TM 5-818-5)



ARRAY 1 ARRAY 2 ARRAY 3 ARRAY 4

F'_c = DRAWDOWN FACTOR FOR CENTER OF ARRAY.
F'_w = DRAWDOWN FACTOR FOR ANY WELL OF ARRAY.
F'_B = DRAWDOWN FACTOR FOR MIDWAY BETWEEN LAST TWO WELLS (ARRAY 2).

} SEE EQ 1 AND 3 (FIG. 47) FOR DEFINITION OF F.

VALUES DETERMINED FOR DRAWDOWN FACTORS ARE SUBSTITUTED INTO EQ 1 OR 3 (FIG. 52).
ALL WELLS ARE FULLY PENETRATING. FLOWS FROM ALL WELLS ARE EQUAL.
SEE FIG. 52 FOR EXPLANATION OF TERMS NOT DEFINED IN THIS FIGURE.

ARRAY 1. CIRCULAR ARRAY OF EQUALLY SPACED WELLS

$$F'_c = \frac{Q_w}{2}\sum_{i=1}^{i=n} \ln\left[1+4\left(\frac{L}{A}\right)^2 - 4\left(\frac{L}{A}\right)\cos(i-1)\frac{2\pi}{n}\right] \quad (1)$$

$$\text{IF } \frac{L}{A}\ge 2 \qquad F'_c = Q_w n \ln\frac{2L}{A} \quad (2)$$

$$F'_w = Q_w\left(n\ln\frac{2L_w}{A} + \ln\frac{A}{nr_w}\right) \quad (3)$$

ARRAY 2. SINGLE LINE OF EQUALLY SPACED WELLS

$$F'_c = 2Q_w\sum_{i=1}^{i=n/2}\ln\sqrt{1+\left[\frac{2L}{(a/2)(n+1-2i)}\right]^2} \quad (4)$$

$$F'_B = Q_w\sum_{i=1}^{i=n}\ln\sqrt{1+\left[\frac{2L}{(a/2)(2i-3)}\right]^2} \quad (5)$$

WHERE n = ∞ USE EQUATIONS GIVEN IN FIG. 53, 54, AND 55.

ARRAY 3. TWO PARALLEL LINES OF EQUALLY SPACED WELLS

$$F'_c = 2Q_w\sum_{i=1}^{i=n/4}\left\{\ln\sqrt{1+\left[\frac{2L+l}{(a/4)(n+2-4i)}\right]^2}+\ln\sqrt{1+\left[\frac{2L+3l}{(a/4)(n+2-4i)}\right]^2}\right\} \quad (6)$$

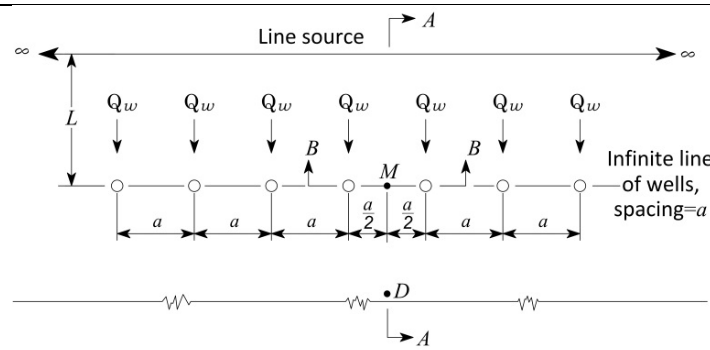
ARRAY 4. RECTANGULAR ARRAY OF EQUALLY SPACED WELLS

APPROXIMATE METHOD. COMPUTE F'_w AND F'_c FROM EQ 1 or 2 AND 3 RESPECTIVELY, WHERE A_e IS SUBSTITUTED FOR A AND

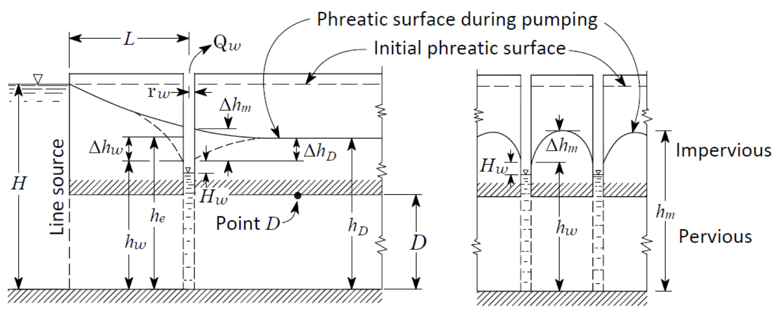
$$A_e = \frac{4}{\pi}\sqrt{b_1 b_2} \quad (7)$$

EXACT METHOD. COMPUTE F'_p AND F'_w FROM EQ 2 AND 4 (FIG. 52), RESPECTIVELY.

Figure 53. Drawdown for fully penetrating circular, single-line, two-line, and rectangular well arrays; line source; artesian and gravity flows (Adapted from Leonards, 1962 and TM 5-818-5)



(a)



HYDRAULIC HEAD LOSS, H_w, IS OBTAINED FROM FIG. 59.

A-A (b)

B-B (c)

DRAWDOWN, $H - h_e$, PRODUCED BY PUMPING Q_w FROM AN EQUIVALENT CONTINUOUS SLOT IS COMPUTED FROM $\frac{Q_w L}{kDa}$.

HEAD LOSS DUE TO CONVERGING FLOW AT WELL

$$\Delta h_w = \frac{Q_w}{2\pi kD} \ln \frac{a}{2\pi r_w} \quad 1)$$

TOTAL DRAWDOWN AT WELL (NEGLECTING HYDRAULIC HEAD LOSS, H_w)

$$H - h_w = H - h_e + \Delta h_w = \frac{Q_w L}{kDa} + \frac{Q_w}{2\pi kD} \ln \frac{a}{2\pi r_w} \quad 2)$$

HEAD INCREASE MIDWAY BETWEEN WELLS

$$\Delta h_m = \frac{Q_w}{2\pi kD} \ln \frac{a}{\pi r_w} \quad 3)$$

DRAWDOWN MIDWAY BETWEEN WELLS

$$H - h_m = H - h_w - \Delta h_m = \frac{Q_w L}{kDa} - 0.11 \frac{Q_w}{kD} \quad 4)$$

HEAD INCREASE Δh_D DOWNSTREAM OF WELLS IS EQUAL TO Δh_w, EQ 1.

DRAWDOWN, $H - h_D$, DOWNSTREAM OF WELLS IS EQUAL TO $H - h_w - \Delta h_w$ OR $H - h_e$ AND, CONSEQUENTLY, CAN BE COMPUTED FROM EQ 1 (FIG. 35), WHERE $x = a$ and $Q = Q_w$. $H - h_D$ CAN ALSO BE COMPUTED FROM

$$H - h_D = \frac{h_D - h_w}{\left(\frac{a}{2\pi L}\right) \ln \frac{a}{2\pi r_w}} \quad 5)$$

Figure 54. Flow and drawdown for fully penetrating infinite line of wells; line source; artesian flow (Adapted from Leonards, 1962 and TM 5-818-5)

SEE DRAWING IN FIG. 40 AND FIGURES (a) AND (b) BELOW FOR DEFINITIONS OF TERMS IN EQUATIONS.

DRAWDOWN, $H - h_W$, PRODUCED BY PUMPING Q_w FROM AN EQUIVALENT CONTINUOUS SLOT IS COMPUTED FROM EQ 1 (FIG. 37).

HEAD LOSS DUE TO CONVERGING FLOW AT WELL

$$\Delta h_w = \frac{Q_w \theta_a}{kD} \quad (1)$$

TOTAL DRAWDOWN AT WELL (NEGLECTING H_w)

$$H - h_w = H - h_e + \Delta h_w = \frac{Q_w}{kD}\left(\frac{L}{a} + \theta_a\right) \quad (2)$$

HEAD INCREASE MIDWAY BETWEEN WELLS

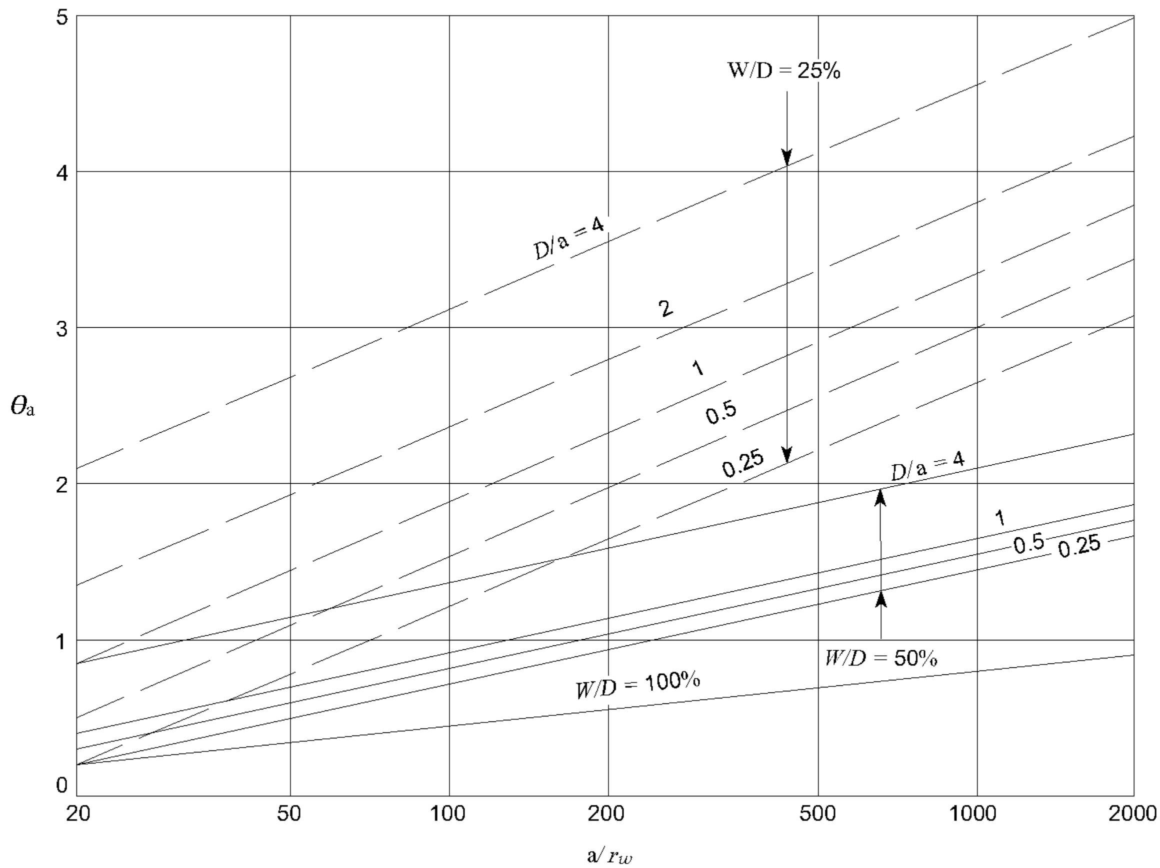
$$\Delta h_m = \frac{Q_w \theta_m}{kD} \quad (3)$$

DRAWDOWN MIDWAY BETWEEN WELLS

$$H - h_m = H - h_w - \Delta h_m = \frac{Q_w}{kD}\left(\frac{L}{a} + \theta_a - \theta_m\right) \quad (4)$$

HEAD INCREASE Δh_D DOWNSTREAM OF WELLS IS EQUAL TO Δh_w, EQ 1.

DRAWDOWN, $H - h_D$, DOWNSTREAM OF WELLS IS EQUAL TO $H - h_w - \Delta h_w$ OR $H - h_e$ AND CONSEQUENTLY, CAN BE COMPUTED FROM EQ 1 (FIG. 37).



(a)

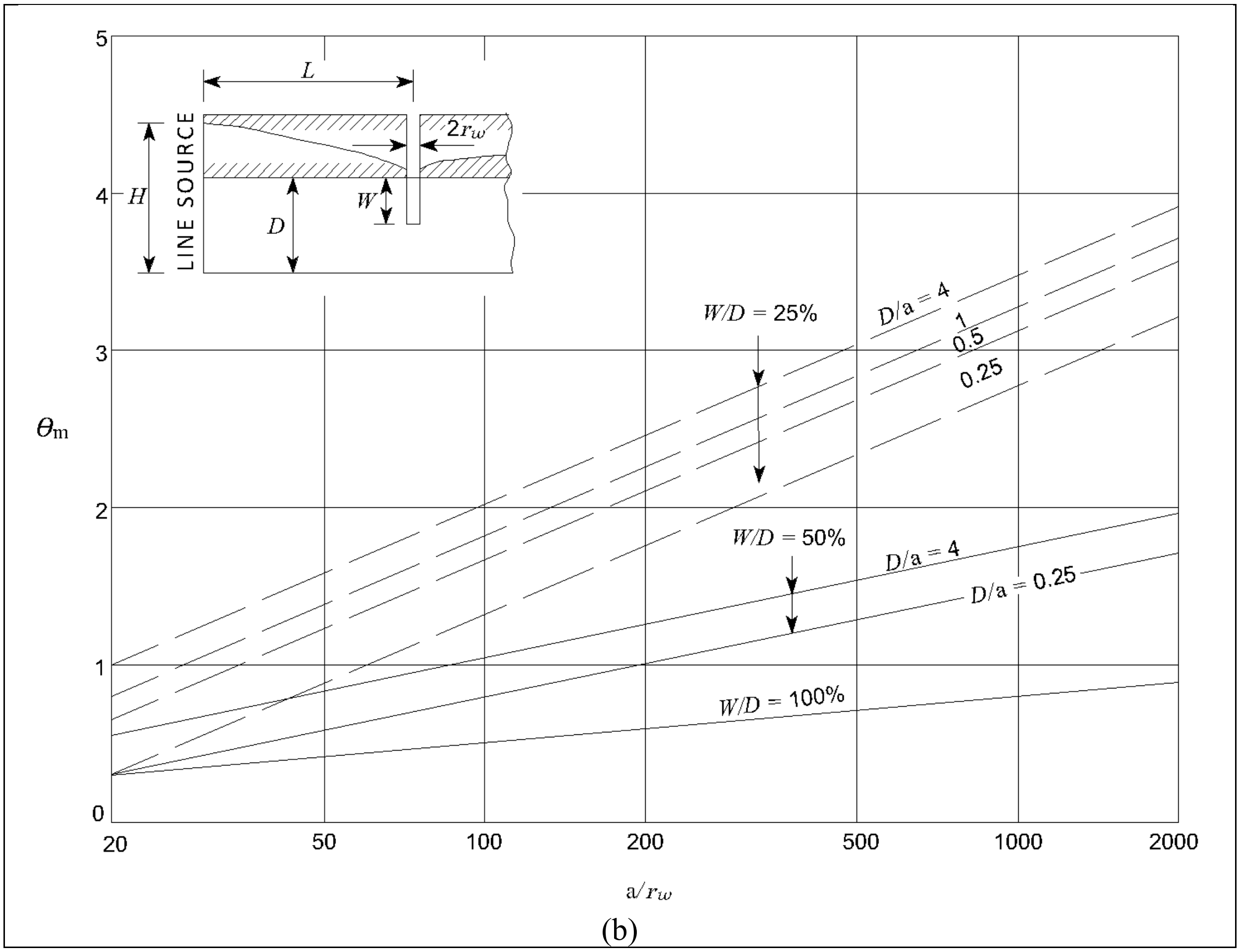
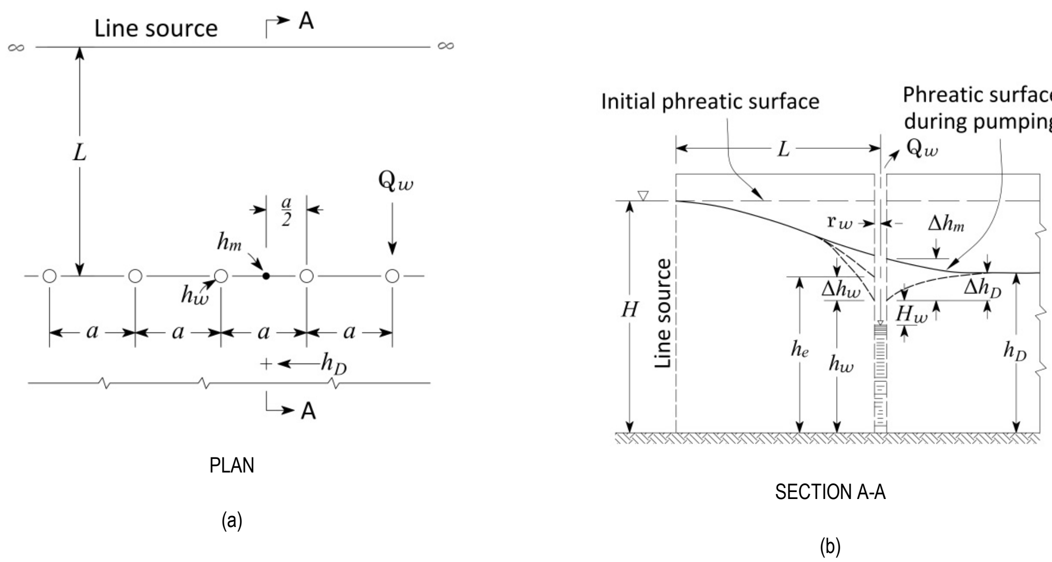


Figure 55. Flow and drawdown for fully and partially penetrating infinite line of wells; line source; artesian flow (Adapted from Leonards, 1962 and TM 5-818-5)



DRAWDOWN, $H^2 - h_e^2$, PRODUCED BY PUMPING Q_w FROM AN EQUIVALENT CONTINUOUS SLOT IS COMPUTED FROM $\frac{2QL}{ka}$.

HEAD LOSS DUE TO CONVERGING FLOW AT WELL

$$h_e^2 - h_w^2 = \frac{Q_w}{\pi k} \ln \frac{a}{2\pi r_w} \tag{1}$$

TOTAL DRAWDOWN AT WELL

$$H^2 - h_w^2 = H^2 - h_e^2 = \frac{2Q_w L}{ka} + \frac{Q_w}{\pi k} \ln \frac{a}{2\pi r_w} \tag{2}$$

HEAD INCREASE MIDWAY BETWEEN WELLS

$$h_m^2 - h_w^2 = \frac{Q_w}{\pi k} \ln \frac{a}{\pi r_w} \tag{3}$$

DRAWDOWN MIDWAY BETWEEN WELLS

$$H^2 - h_m^2 = H^2 - h_w^2 - (h_m^2 - h_w^2) = \frac{Q_w}{k}\left(\frac{2L}{a} - \frac{\ln 2}{\pi}\right) \tag{4}$$

HEAD INCREASE Δh_D DOWNSTREAM OF WELLS IS EQUAL TO Δh_w, EQ 1.

DRAWDOWN, $H^2 - h_D^2$, DOWNSTREAM OF WELLS IS EQUAL TO

$$H^2 - h_D^2 = \frac{h_D^2 - h_w^2}{\left(\frac{a}{2\pi L}\right) \ln \frac{a}{2\pi r_w}} \tag{5}$$

Figure 56. Flow and drawdown for fully penetrating infinite line of wells; line source; gravity flow (Adapted from Leonards, 1962 and TM 5-818-5)

5.2.2.4 Table 9 summarizes the figures related to flow, head or drawdown equations.

Table 9
Index to Figures for Flow, Head, or Drawdown Equations for Given Corrections

Index	Assumed Source of Seepage	Drainage System	Type of Flow	Penetration	Figure
Flow to a slot	Line	Line slot	A, G, C	F	35, 36
	Line	Line Slot	A, G, C	P	36, 37
	Two-line	Line Slot	A, G	P, F	38
	Two-line	Two-line slots	A, G	P	39
	Circular	Circular slots	A	P, F	40, 41
	Circular	Rectangular slots	A	P, F	42, 32
Flow to wells	Circular	Single well	A	P, F	44
	Circular	Single well	G	P, F	45
	Circular	Single well	C	F	46
	Circular	Multiple wells	A, G	F	47
	Circular	Circular, rectangular, and two-line arrays	A, G	F	48
	Circular	Circular array	A	F	49
	Circular	Circular and rectangular array	A	P	50
	Single line	Single well	A, G	F	51
	Single line	Multiple wells	A, G	F	52
	Single line	Circular, line, two-line, and rectangular arrays	A, G	F	53
	Single line	Infinite line	A	F	54
	Single line	Infinite line	A	P, F	55
	Single line	Infinite line	G	F	56
Other	Approximate radius of influence				58
	Hydraulic head loss in a well				59
	Hydraulic head loss in various wellpoints				60
	Shape factors for wells of various penetrations centered inside a circular source				57
	Flow and drawdown for slots from flow-net analyses				61
	Flow and drawdown to wells from flow-net analyses				62

Note: A = artesian flow; G = gravity flow; C = combined artesian-gravity flow;
F = fully penetrating; P = partially penetrating

5.2.2.4 Limitations on Flow to a Partially Penetrating Well.

5.2.2.4.1 Theoretical boundaries for a partially penetrating well (for artesian flow) are approximate relations intended to present in a simple form the results of more rigorous but tedious computations. The rigorous computations were made for ratios of R/D = 4.0 and 6.7 and a ratio $R/r_w = 1000$. As a consequence, any agreement between experimental and computed values cannot be expected except for the cases with these particular boundary conditions. In model studies at the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi (TM 5-818-5), the flow from a partially penetrating well was based on the formula:

$$Q_{wp} = \frac{2\pi kD(H - h_w)G}{\ln\left(\frac{R}{r_w}\right)} \tag{9}$$

or

$$Q_{wp} = kD(H - h_w)(\text{shape factor}) \tag{9a}$$

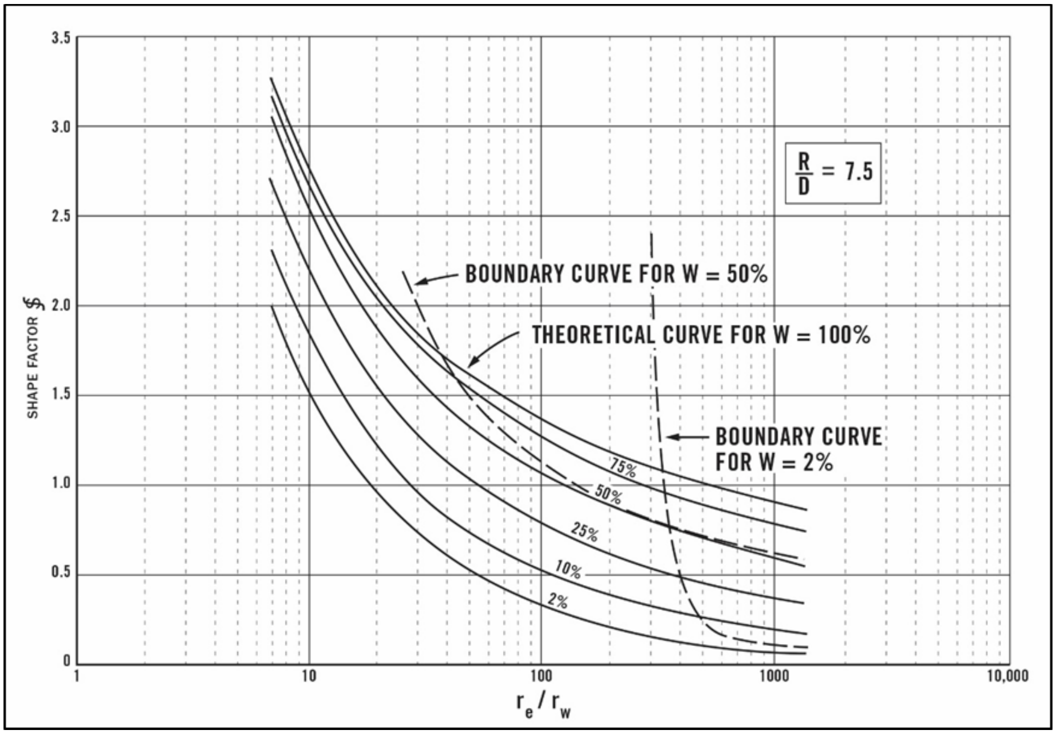
with

$$\text{shape factor} = \frac{2\pi G}{\ln\left(\frac{R}{r_w}\right)}$$

Where:

G = value shown in equation (6) on Figure 44

5.2.2.4.2 Figure 57 shows some of the results obtained at the WES for shape factors for wells of various penetrations centered inside a circular source. Also presented in Figure 57 are boundary curves computed for well-screen penetrations (based on percent W or W/D, see Figure 40) of 2 and 50 percent, as well as the theoretical curves for 100 percent fully penetrating wells (W = 100%). Comparison of shape factors computed from WES model data with shape factors computed from the boundary formulas indicates fairly good agreement for well penetrations > 25 percent and values of R/D between about 5 and 15 where $R/r_w > 200$ to 1000. Other empirical formulas for flow from a partially penetrating well may suffer from similar limitations.



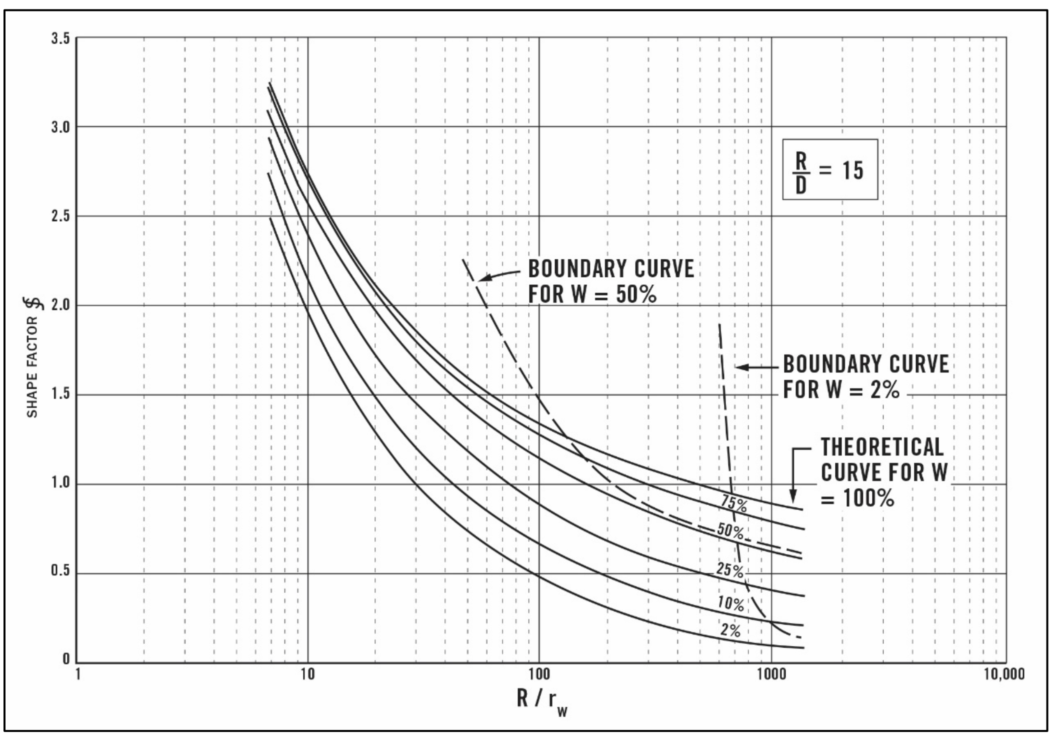
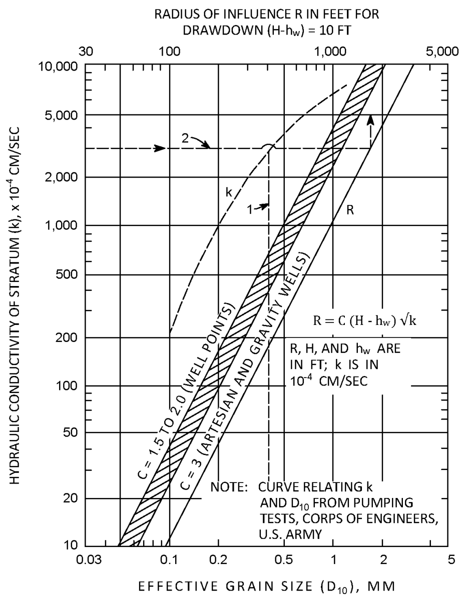


Figure 57. Shape factors for wells of various penetrations centered inside a circular source (Adapted from TM 5-818-5)

5.2.2.5 Partially Penetrating Wells. The equations for gravity flow to partially penetrating wells are only considered valid for well penetrations with W/D = 50 percent or greater.

5.2.2.6 Radius of Influence R.

5.2.2.6.1 Equations for flow to drainage systems from a circular seepage source are based on the assumption that the system is centered on an island of radius R. Generally, R is the radius of influence that is defined as the radius of a circle beyond which pumping of a dewatering system has no significant effect on the original groundwater level or piezometric surface. The value of R can be estimated from the Sichardt and Kyrieleis (1930) empirical equation and is plotted in Figure 58. Where there is little or no recharge to an aquifer, the radius of influence will become greater with pumping time and with increased drawdown in the area being dewatered. Generally, R is greater for very pervious sands than for finer soils. If the value of R is large relative to the size of the excavation, a reasonably good approximation of R will serve adequately for design because flow and drawdown for such a condition are not especially sensitive to the actual value of R. As it is usually impossible to determine R accurately, the value should be selected conservatively from pumping test data or, if necessary, from Figure 58. The radius of influence calculated using the empirical formula and chart presented in Figure 58 is an approximation of a reasonable value of R to use with steady-state equations to estimate flow and drawdown. Almost all dewatering problems are actually transient problems, but the empirical estimate of R presented in Figure 58 has been found to produce reasonable, conservative values in most cases for use in analyzing construction dewatering problems.



1. R DETERMINED WHEN ONLY D_{10} IS KNOWN.
2. R DETERMINED WHEN k IS KNOWN.

RADIUS OF INFLUENCE, R, CAN BE ESTIMATED FOR BOTH ARTESIAN AND GRAVITY FLOWS BY

$$R = C(H - h_w)\sqrt{k}$$

W R, H, AND h_w ARE DEFINED PREVIOUSLY AND EXPRESSED IN FEET. COEFFICIENT OF PERMEABILITY, k, IS EXPRESSED IN 10^{-4} CM/SEC.

A C = 3 FOR ARTESIAN AND GRAVITY FLOWS TO A WELL.

C = 1.5 TO 2.0 FOR A SINGLE LINE OF WELL POINTS.

THE VALUE OF R FOR $(H - h_w)$ = 10 FEET CAN BE DETERMINED FROM THE PLOT HEREIN WHEN EITHER THE D_{10} SIZE OR HYDRAULIC CONDUCTIVITY OF THE MATERIAL IS KNOWN. THE VALUE OF R WHEN $(H - h_w)$ ≠ 10 CAN BE DETERMINED BY MULTIPLYING THE R VALUE OBTAINED FROM THE PLOT BY THE RATIO OF THE ACTUAL VALUE OF $(H - h_w)$ TO THE DRAWDOWN OF 10 FEET.

Figure 58. Approximate radius of influence R (Adapted from Leonards, 1962 and TM 5-818-5)

5.2.2.6.2 Terzaghi (1943) cites a rigorously correct but approximate theoretical equation developed by Steinbrenner (1937) for the radius of influence in an unconfined sand aquifer of thickness H with a horizontal phreatic surface and a horizontal impermeable base for elapsed pumping time t:

$$R = 1.5\sqrt{\frac{Hkt}{G_a n}} \tag{7}$$

Where:

R = Radius from pumped well to zero drawdown

H = Thickness of unconfined aquifer below initial phreatic surface

k = Hydraulic conductivity of sand

t = Elapsed pumping time

G_a = Air space ratio (percentage of voids that will drain by gravity)

n= porosity

5.2.2.6.3 The product Gan is the effective porosity and is the same as the specific yield Sy. This equation may be used to approximate R in confined aquifers by substituting the aquifer thickness D for H and the storage coefficient S for Gan. A typical value of S for unconfined clean sand and confined aquifers is 0.2 and 0.001, respectively. For construction dewatering, the pumping time t can usually be assumed to be between 15 and 30 days for this calculation; for projects that involve rapidly moving trench excavations, the pumping time could be as low as 5 days. Note that according to this equation, the radius R at a given time t is independent of the drawdown at the well or the flow Q, whereas the Sichardt and Kyrieleis equation (1930) shown on Figure 58 indicates that R is proportional to drawdown at the well. Terzaghi (1943) states that several other independent solutions of the problem, including Kozeny (1933) and Weber (1928), also indicate that R is independent of the drawdown or flow. In general, this equation will yield values of R that are typically greater than those selected using the chart or equation in Figure 58 for drawdown values of 20 feet or less. Therefore, use of the Steinbrenner equation (1937) will typically yield less conservative values than the Sichardt and Kyrieleis empirical equation (1930).

5.2.2.7 Wetted Screen. There should always be sufficient well and screen length below the required drawdown in a well in the formation being dewatered so that the design or required pumping rate does not produce a gradient at the interface of the formation and the well filter (or screen) or at the screen and filter that starts to cause the flow to become turbulent. Therefore, the design of a dewatering system should always be checked to see that the well or wellpoints have adequate "wetted screen length h_{ws}" or submergence to pass the maximum computed flow. According to the Sichardt and Kyrieleis empirical equation (1930), the limiting flow q_c into a filter or well screen is approximately equal to:

$$q_c = \frac{2\,\pi\, r_w \sqrt{k}}{1.07} \text{ x } 7.48 = \text{gpm per foot of filter screen} \tag{8}$$

Where:

r_w = radius of well filter (feet)

k= hydraulic conductivity of filter or aquifer sand (ft/min)

5.2.2.8 Hydraulic Head Loss H_w. The equations in Figures 35 through 56 do not consider hydraulic head losses that occurs in the filter, screen, collector pipes, etc. These losses cannot be neglected, however, and must be accounted for separately. The hydraulic head loss through a filter and screen will depend upon: (1) the diameter of the screen, slot width, and opening per foot of screen, hydraulic conductivity and thickness of the filter; (2) any clogging of the filter or screen by incrustation, drilling fluid, or bacteria; (3) migration of soil or sand particles into the filter; and (4) rate of flow per foot of screen. Graphs for estimating hydraulic head losses in pipes, wells, and screens are shown in Figures 59 and 60. The hydraulic head loss through various sizes and types of header or discharge pipes, and for certain well screens and (clean) filters, as determined from laboratory and field tests, are given in Figures 59 and 60. Head losses in the screened section of a well, H_s, are calculated from Figure 59b. This head loss is based on equal inflow per unit of screen surface and turbulent flow inside the well and is equivalent to the entire well flow passing through one-half the screen length. Other head losses can be determined directly from Figure 59. Hydraulic head loss within a wellpoint system can be estimated from Figure 60. As stated in the first paragraph of this section above, flow into a well can be impeded by the lack of "wetted screen length," in addition to hydraulic head losses in the filter or through the screens and/or chemical or biological clogging of the aquifer and filter.

TOTAL HYDRAULIC HEAD LOSS IN A WELL (H_W) IS

$$H_w = H_e + H_s + H_r + H_v \quad (1)$$

WHERE H_e = ENTRANCE HEAD LOSS (SCREEN AND FILTER); ESTIMATE FROM CURVE a

H_s = HEAD LOSS IN SCREENED SECTION OF WELL; ESTIMATE FROM CURVE b for A DISTANCE OF ONE-HALF THE SCREEN LENGTH.

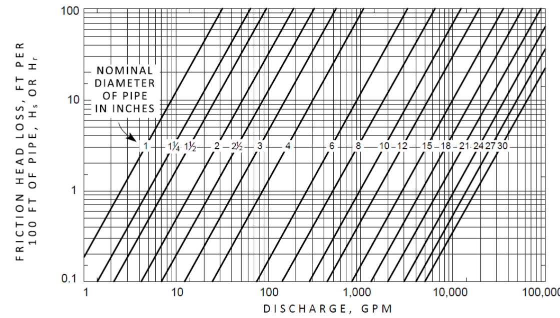
H_r = HEAD LOSS WITHIN THE RISER AND CONNECTIONS; ESTIMATE FROM CURVE b

H_v = VELOCITY HEAD LOSS;

ESTIMATE FROM CURVE c

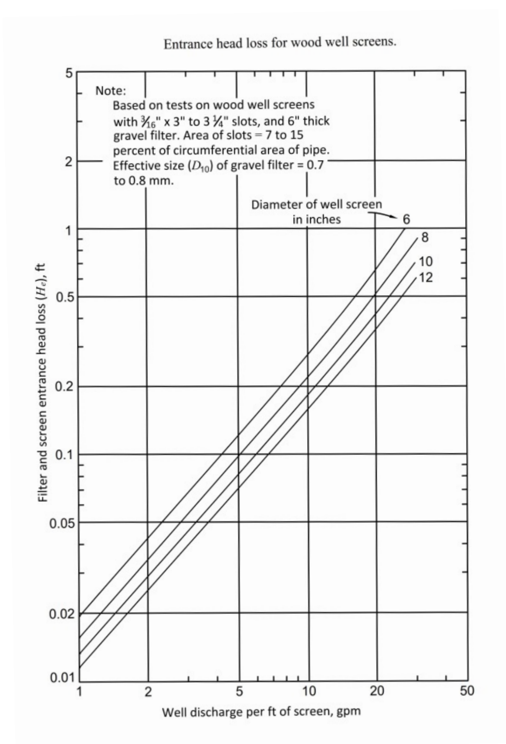
THE VALUE OF H_w MUST BE SUBTRACTED FROM THE COMPUTED VALUE OF h_w TO OBTAIN THE LIFT OR WATER LEVEL IN A WELL.

TYPE OF PIPE	C	$(100/C)^{1.85}$
STEEL (NEW)	125	0.67
STEEL (AVG. CONDITION)	110	0.83
PVC (POLYVINYL CHLORIDE)	150	0.47
CORRUGATED METAL	70	1.92

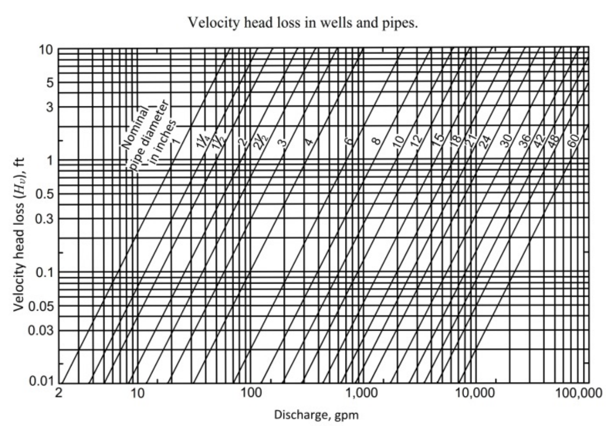


BASED ON HAZEN-WILLIAMS EQUATION WITH C=100; MULTIPLY LOSSES BY $(100/C)^{1.85}$ FOR VALUES OF C OTHER THAN 100

(b)



(a)



(c)

Figure 59. Hydraulic head loss in a well (Adapted from Leonards, 1962 and TM 5-818-5)

TOTAL HYDRAULIC HEAD LOSS IN A WELLPOINT (H_w) IS

$$H_w = H_e + H_s + H_r + H_v$$

WHERE

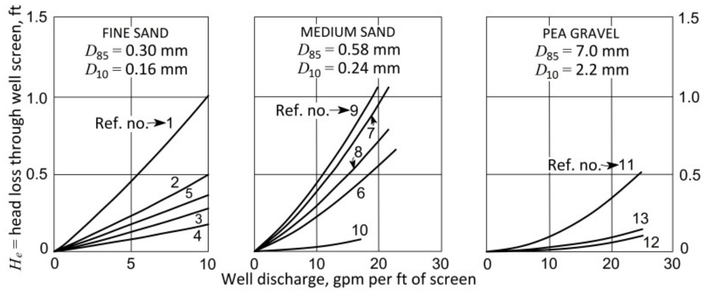
H_e = ENTRANCE HEAD LOSS (WELLPOINT AND FILTER)

H_s = FRICTION HEAD LOSS WITHIN THE WELLPOINT

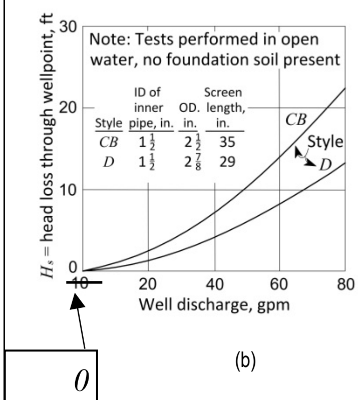
H_r = FRICTION HEAD LOSS IN RISER, SWING CONNECTION, AND VALVE

H_v = VELOCITY HEAD LOSS IN RISER, SWING CONNECTION, AND VALVE

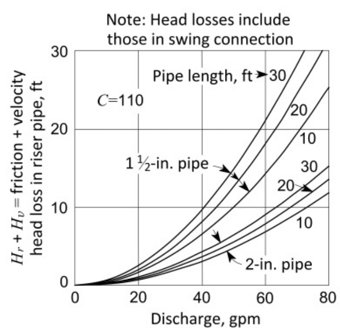
HYDRAULIC HEAD LOSSES FOR TYPICAL WELLPOINTS AND RISERS CAN BE ESTIMATED FROM THE PLOTS BELOW.



(a)



(b)



(c)

REF NO.	WELLPOINT †	SLOT OR MESH NO.	SLOT OR MESH OPENING IN MM
1	A AND B, GROOVED SLOT	12	0.30
2	C, WIRE WRAP ON PERFORATED PIPE	20	0.51
3	D, WIRE MESH ON PERFORATED PIPE	28	0.59
4	E, WIRE MESH ON PERFORATED PIPE	28	0.59
5	B, GROOVED SLOT	25	0.63
6	A, GROOVED SLOT	25	0.63
7	D, WIRE MESH ON PERFORATED PIPE	28	0.59
8	A, GROOVED SLOT	50	1.27
9	B, GROOVED SLOT	30	0.76
10	F, PERFORATED PIPE WITH 6-IN PEA-GRAVEL FILTER	5/32 IN.	3.97
11	A, GROOVED SLOT	12	0.30
12	A, GROOVED SLOT	100	2.54
13	E, PERFORATED PIPE	5/32 IN.	3.97
CB	MESH SF, COMMERCIAL BRONZE, SELF-JETTING	40 × 45	0.31 × 0.38
D	MESH E, STAINLESS STEEL STYLE D, SELF-JETTING	12 × 68	0.30 × 1.73

† EXCEPT FOR C, B, AND D, WELLPOINTS ARE PLAIN-TIP, 2-1/2-IN ID.

Figure 60. Hydraulic head loss in various wellpoints (Adapted from Leonards, 1962 and TM 5-818-5)

5.2.2.9 Well or screen penetration W/D. Most of the equations and graphs presented in this document for flow and drawdown to slots or well systems were basically derived for fully penetrating drainage slots or wells. Equations and graphs for partially penetrating slots or wells are generally based on fully penetrating drainage systems modified by model studies and, in some instances, mathematical derivations. The amount or percent of screen penetration required for effective pressure reduction or interception of seepage depends upon many factors, such as thickness of the aquifer, distance to the effective source of seepage, well or wellpoint radius, stratification, required "wetted screen length," type and size of excavation, and whether or not the excavation penetrates alternating pervious and impervious strata or the bottom is underlain at a shallow depth by a less pervious stratum of soil or rock. Where a sizeable open excavation or tunnel is underlain by a fairly deep stratum of sand and wells are spaced rather widely, the well screens should penetrate at least 25 percent of the thickness of the aquifer to be dewatered below the bottom of the excavation and more preferably 50 to 100 percent. Where the aquifer(s) to be dewatered is stratified, the drainage slots or well screens should fully penetrate all the strata to be dewatered. If the bottom of an excavation in a pervious formation is underlain at a shallow depth by an impervious formation and the amount of "wetted screen length" available is limited, the drainage trench or well screen should penetrate to the top of the underlying impervious stratum.

5.3 Flow Net Analyses.

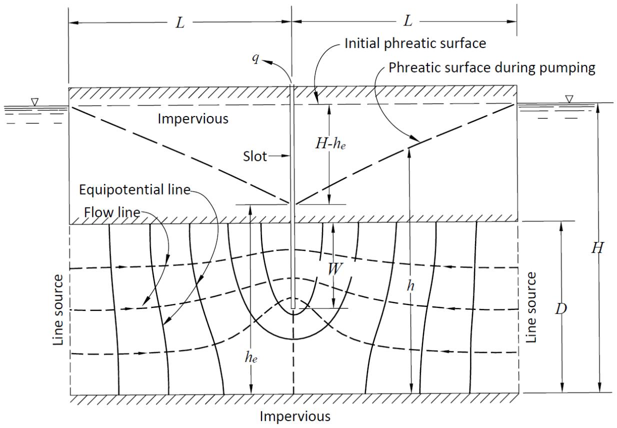
5.3.1 Flow nets (see EM 1110-2-1901 based on work by A. Casagrande in "Seepage Through Dams," 1937) are valuable where irregular configurations of the source of seepage or of the dewatering system make mathematical analyses complex or nearly impossible. A flow net is a graphical representation of flow of water through an aquifer and defines paths of seepage (flow lines) and contours of equal piezometric head (equipotential lines). Considerable practice in drawing and studying properly constructed flow nets is required before accurate flow nets can be constructed. Flow nets are still a very useful tool to evaluate seepage flow conditions. Today, the standard of practice is to use finite element seepage models. The numerical models are no more accurate than flow nets, but they have a number of advantages. The most important advantages include the greatly reduced time for evaluating complex seepage problems and the ease of performing calibrations to known conditions, revisions of boundary conditions, and parametric sensitivity analyses. Guidance for the use of finite element seepage models is presented in Section 5.4. The study of flow net construction and flow net examples can be instructive in understanding groundwater flow behavior associated with dewatering and to more effectively use computer numerical analysis results.

5.3.2 Flow nets are limited to analysis in two dimensions; the third dimension in each case is assumed infinite in extent. An example of a sectional flow net showing artesian flow from two line sources to a partially penetrating drainage slot is given in Figure 60a. An example of a plan flow net showing artesian flow from a river to a line of relief wells is shown in Figure 60b.

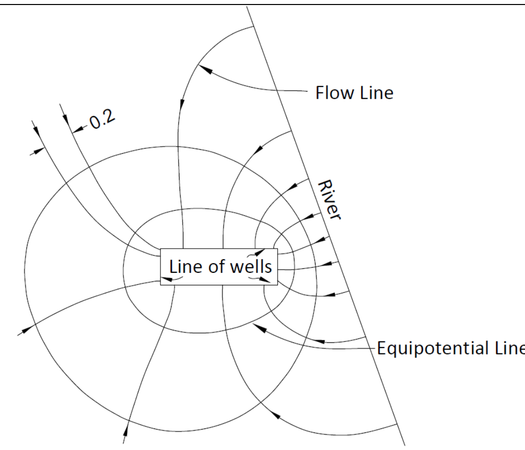
5.3.3 The flow per unit length (for sectional flow nets) or depth (for plan flow nets) can be computed by means of equations (1) and (2), and (5) and (6), respectively (Figure 60). Drawdowns from either sectional or plan flow nets can be computed from equations (3) and (4) (Figure 60). In plan flow nets for artesian flow, the equipotential lines correspond to various values of H-h, whereas for gravity flow, they correspond to H^2-h^2. Since section equipotential lines for gravity flow conditions are curved rather than vertical, plan flow nets for gravity flow conditions give erroneous results for large drawdowns and should always be used with caution.

5.3.4 Plan flow nets give erroneous results if used to analyze partially penetrating drainage systems, the error being inversely proportional to the percentage of penetration. They give fairly accurate results if the penetration of the drainage system exceeds 80 percent and if the heads are adjusted as described in the following paragraph.

5.3.5 In previous analyses of well systems by means of flow nets, it was assumed that dewatering or drainage wells were spaced sufficiently close to be simulated by a continuous drainage slot and that the drawdown (H-h_D) required to dewater an area was equal to the average drawdown at the drainage slot or in the lines of wells (H-h_c). These analyses give the amount of flow Q_T that must be pumped to achieve H-h_D, but do not give the drawdown at the wells. The drawdown at the wells is required to produce H-h_o downstream or within a ring of wells that can be computed (approximately) for artesian flow from plan flow nets by the equations shown in Figure 61 and if the wells have been spaced proportional to the flow lines as shown in Figure 62. The drawdown at fully penetrating gravity wells can also be computed from equations given in Figure 61.



SECTIONAL FLOW NET (ARTESIAN)
(a)



PLAN FLOW NET
(b)

SECTIONAL FLOW NET

FLOW

ARTESIAN $q = kH' \mathcal{S}$ (1) GRAVITY $q = kH'' \mathcal{S}$ (2)

DRAWDOWN AT ANY POINT

ARTESIAN $H - h = H' \frac{n_e}{N_e}$ (3) GRAVITY $H^2 - h^2 = \frac{n_e}{N_e}[H^2 - (h_o + h_s)^2]$ (4)

PLAN FLOW NET

FLOW

ARTESIAN $Q_T = kDH' \mathcal{S}$ (5) GRAVITY $Q_T = \frac{kH'' \mathcal{S}}{2}$ (6)

DRAWDOWN AT ANY POINT

USE EQ 3 AND 4 FOR ARTESIAN AND GRAVITY FLOW CONDITIONS, RESPECTIVELY.

$H' = H - h_e$ $H'' = H^2 - h_o^2$ $\mathcal{S}$ = SHAPE FACTOR = $\frac{N_f}{N_e}$ N_f = NUMBER OF FLOW CHANNELS IN NET

N_e = TOTAL NUMBER OF EQUIPOTENTIAL DROPS BETWEEN FULL HEAD, H, AND HEAD AT EXIT, h_e

n_e = NUMBER OF EQUIPOTENTIAL DROPS FROM EXIT TO POINT AT WHICH HEAD, h, IS DESIRED

h_o IS SHOWN IN FIG. 35.

Figure 61. Flow and drawdown to slots computed from flow nets (Adapted from Leonards, 1962 and TM 5-818-5)

CONSTRUCT PLAN FLOW NET. SPACE WELLS PROPORTIONAL TO FLOW LINES. COMPUTE TOTAL FLOW TO SYSTEM FROM EQ 5 (FIG. 61) FOR ARTESIAN FLOW OR EQ 6 (FIG. 61) FOR GRAVITY FLOW. ASSUME IN EQ 5 H' = H – h_D. SEE FIG. 54, b, c; FIG. 56, b; AND FIG. 57 FOR EXPLANATION OF TERMS.

ARTESIAN FLOW

FLOW TO EACH WELL

$$Q_W = \frac{Q_T}{n} \qquad (1)$$

WHERE n = NUMBER OF WELLS IN THE SYSTEM

DRAWDOWN AT WELLS

FULLY PENETRATING

$$H - h_w = \frac{Q_W}{kD}\left(\frac{n}{\mathscr{S}} + \frac{1}{2\pi}\ln\frac{a}{2\pi r_w}\right) \qquad (2)$$

PARTIALLY PENETRATING

$$H - h_w = \frac{Q_W}{kD}\left(\frac{n}{\mathscr{S}} + \theta_a\right) \qquad (3)$$

WHERE θ_a IS OBTAINED FROM FIG. 55.

HEAD INCREASES MIDWAY BETWEEN AND DOWNSTREAM OF WELLS MAY BE COMPUTED FROM EQUATIONS GIVEN IN FIG. 54 AND 55.

GRAVITY FLOW

FLOW TO EACH WELL

USE EQ 1

DRAWDOWN AT FULLY PENETRATING WELL

$$H^2 - h_w{}^2 = \frac{Q_W}{k}\left(\frac{n}{\mathscr{S}} + \frac{1}{\pi}\ln\frac{a}{\pi r_w}\right) \qquad (4)$$

HEAD INCREASES MIDWAY BETWEEN AND DOWNSTREAM OF WELLS MAY BE COMPUTED FROM EQUATIONS GIVEN IN FIG. 56.

Figure 62. Flow and drawdown to wells computed from flow-net analyses (Adapted from TM 5-818-5)

5.4 Numerical Analyses. Many complex seepage problems, including such categories as steady confined, steady unconfined, and transient unconfined can be solved using the finite element method. Commercial finite element and finite difference software packages for analysis of seepage in both two and three dimensions are now widely used throughout the geotechnical engineering and geology professions. These codes can handle most cases of nonhomogeneous and anisotropic media. Refer to EM 1110-2-1421 for a comprehensive overview and guidance for the development of numerical models for groundwater flow, as well as a list of publications pertinent to groundwater modeling.

5.4.1 Application of Numerical Methods to Practical Problems. Numerical methods are useful for problems such as estimating seepage inflow for dewatering system design, the effectiveness of a dewatering system, and other aspects of dewatering system design. Most commercial software packages solve seepage problems using Darcy's law, in the same manner as the simplified methods, but allow for complex geometries, material properties, and boundary conditions to be modeled. As with all analyses, the results of numerical analyses are only as good as the input values and assumptions. See Figures C.3 and C.7 in Appendix C for examples of two-dimensional numerical methods applied to practical dewatering problems as well as comparisons of the results of numerical analyses with the results of mathematical closed form solutions to these problems.

5.4.2 Calibration of Models to Ranges of Existing Conditions.

5.4.2.1 Numerical analyses should be calibrated to observed existing conditions, whenever possible. Calibrating the numerical analysis will lead to a higher degree of confidence in the results of the analysis, relative to an analysis that is not calibrated.

5.4.2.2 Calibration in a numerical seepage model typically involves developing the model geometry, material properties, and boundary conditions using best-estimate parameters based on laboratory and field testing, and engineering judgment. The analysis should then be performed, and pore water pressures measured by piezometers are typically compared to the model's estimated pore water pressures at the same location.

5.4.2.3 If the pore pressures differ significantly (e.g., by more than a few feet of head), the seepage model input parameters are adjusted, within reason, and the seepage analysis is re-performed. The process is repeated until the pore pressures predicted by the seepage analysis reasonably match pore pressures measured in the field by piezometers. With the advent of numerical methods, inclusion of anisotropy is easier and hence greater consideration should be given to the effect of varying this parameter.

5.4.3 Calibration of Models to Pumping Tests and Sensitivity Analyses.

5.4.3.1 As discussed in Appendix B, pumping tests that are properly instrumented with piezometers and are conducted for a sufficient duration can be used to accurately estimate the transmissivity and storativity of an aquifer and also leakage through aquitards from other aquifers above and below the aquifer of interest. A two-dimensional plan view model of a confined aquifer with a uniform thickness can be calibrated in transient mode to piezometric and flow measurements made during and after completing a pumping test. By iterating values of aquifer hydraulic conductivity and storativity in the model, the model can be used to refine the selection of these parameters. If a three-dimensional local model is developed, it is possible to use such a model in transient mode as a calibration tool to refine the selection of pertinent aquifer parameters for unconfined aquifers, including anisotropy and hydraulic conductivity. Further refinements can be made in a three-dimensional model by iterating the vertical hydraulic conductivity and thickness assumed for aquitards above and below the aquifer of interest.

5.4.3.2 In some instances, groundwater or piezometric data may not be available. In these cases, it may not be possible to calibrate the seepage analysis until adequate data are acquired. In such cases, sensitivity analyses may be appropriate. By varying one property and keeping all other properties the same, it is possible to evaluate the significance of a single parameter and how accurately it should be defined.

5.4.4 Verification of Models Using System Test Data. Plan view two-dimensional models can be used in exactly the same way that plan flow nets are used to verify system tests of a dewatering system. Plan view models are especially useful in checking the effective radius of influence, the position of effective recharge boundaries and the average transmissivity of an aquifer. Three-dimensional models are being used in industry and should be considered for complex foundation conditions and critical structures.

5.4.5 Two-Dimensional Models. Two-dimensional seepage analyses are typically most useful in plan view for analyzing flow to dewatering systems from irregular recharge boundaries in the same way that plan flow nets have been used in the past. Dewatering systems and effective recharge boundaries are approximated as fully penetrating slots in such models.

5.4.5.1 Geometry and Meshing.

5.4.5.1.1 Model geometry should be based on ground surface survey data, subsurface exploratory information such as borings or cone penetration tests, and other data. The geometry of the seepage model should be simplified as much as practical but should capture the essential elements of the surface and subsurface. Excessive detail in the ground surface geometry and subsurface materials will lengthen computing times and can cause singularities in the model that do not match actual conditions. Additionally, soil strata should not be extrapolated over long distances between borings when only limited subsurface information is available. The extrapolation of strata will often lead to unrealistic results, if the actual subsurface conditions differ from those assumed.

5.4.5.1.2 Most commercial finite-element seepage analysis software allows the user to select and vary mesh sizes. Mesh size is important for performing an accurate seepage analysis, as an overly large mesh size may lead to numerical issues in the model and fail to capture details in seepage flow and quantity, especially around buried structures. Therefore, the user should select a mesh size where each material and/or geometric zone has at least two vertical elements across the zone. At edges or corners of structures, the mesh size should be decreased to allow for more accurate modeling of seepage flow. Excessively fine mesh sizes should be avoided, as they tend to increase computing time inordinately. With modern computers, this is typically less of an issue than in the past.

5.4.5.2 Boundary Conditions.

5.4.5.2.1 Boundary conditions are important for numerical seepage analysis. Selection of appropriate boundary conditions requires a great deal of engineering judgment, and improper boundary conditions can invalidate the results of the seepage analysis.

5.4.5.2.2 Numerical seepage analyses allow the user to specify two types of boundary conditions: head and flow. Typically, head boundary conditions can be defined either by total head or by pore pressure. These are useful for modeling a body of water, such as a reservoir, when the head is known.

5.4.5.2.3 Flow boundary conditions are useful when the amount of flow is known. A typical use would be when an impermeable boundary is present in the subsurface, such as the top of bedrock or a sheet pile wall. A boundary condition of zero flow can be applied to model this surface. Other requirements for the flow boundary condition include the analysis of relief wells or pumped wells, where the wells remove a known rate of flow from the aquifer.

5.4.5.3 Analysis of Results.

5.4.5.3.1 The results of a numerical seepage analysis are typically used to evaluate two factors: (1) the quantity and distribution of flow in both plan view and sectional models, and (2) seepage gradients in sectional models.

5.4.5.3.2 Flow rates can be estimated by including flux sections within the model. The flux sections should be placed at a location in the model where the rate of flow is desired, such as around an excavation or relief well. The analysis will then estimate the rate of flow passing through the flux section. See EM 1110-2-1901 for further discussion of using flux sections. A plan view model can be used to design the well spacing to match the flow to an equivalent slot representing the dewatering system.

5.4.5.3.3 In sectional models, seepage gradients are typically calculated to evaluate the potential for quick conditions, or if heave, blowout or uplift of the bottom of an excavation will be problematic. Most commercial seepage software will calculate and display nodal gradients, which are gradients over a very small zone. These gradients are affected by the node placement in the numerical analysis and may not represent true seepage gradients. The mesh size should be fine in regions where it is desired to calculate heads, pore pressures, and hydraulic gradients.

5.4.5.3.4 Gradients in sectional models should be manually calculated by dividing the change in head by the distance between two points where the user desires to estimate the gradient. Such a location could be from the bottom to the top of an impermeable confining layer. These average gradients are more useful than nodal gradients.

5.4.6 Three-Dimensional Models.

5.4.6.1 Three-dimensional models are being used to design more complex dewatering systems. However, for the simple dewatering projects, two-dimensional models or hand computations can be used. Partially penetrating wells are one common example of three-dimensional flow, and many solutions have been developed for partially penetrating wells and well systems screened in a single homogeneous aquifer (see Section 5.2 and Table 9). A method for designing a linear array of partially-penetrating wells using a two-dimensional finite element sectional model of flow to a partially penetrating slot in conjunction with well factors presented in EM 1110-2-1914 is included in the appendix detailing relief well design in EM 1110-2-1913.

5.4.6.2 Three-dimensional models are useful in evaluating the transient influence of a dewatering system where the recharge is complex, and drawdown may cause excessive settlement or affect the pumping level of existing water supply wells or leakage from surface water bodies.

5.4.6.3 Practical guidance for the selection of software and the development and calibration of three-dimensional numerical groundwater models is presented in EM 1110-2-1421.

5.5 Wellpoints, Wells and Filters. Wells and wellpoints should be of a type that will prevent infiltration of filter material or foundation sand (if the screens are not surrounded by a filter pack), offer little resistance to the inflow of water, and resist corrosion by water and soil. Wellpoints must also have sufficient penetration of the principal water-carrying strata to intercept seepage without excessive residual head between the wells or within the dewatered area.

5.5.1 Wellpoints. Where large flows are anticipated, a high-capacity type of wellpoint should be selected. The inner suction pipe of self-jetting wellpoints should permit inflow of water with a minimum hydraulic head loss. Self-jetting wellpoints should also be designed so that most of the jet water will go out the tip of the point, with some backflow to keep the screen flushed clean while jetting the wellpoint in place.

5.5.1.1 Wellpoint Screens. Generally, wellpoints are covered with 30- to 60-mesh screen or have an equivalent slot opening (0.010 to 0.025 inch). *The mesh should meet filter criteria given in Section 5.7.1 below*. Screens generally used for wellpoints are slotted (or perforated) steel pipe, perforated steel pipe wrapped with galvanized wire, galvanized wire wrapped and welded to longitudinal rods, and slotted PVC pipe. Riser pipes for most dewatering wells consist of mild steel or PVC pipe. Where the soil to be drained is silty or fine sand, the yield of the wellpoint and its efficiency can be greatly improved by placing the material with a relatively uniform, medium sand filter around the wellpoint. A sand filter is considered to be a current best practice for wellpoint filters. The sand filter should be designed according to criteria subsequently set forth in Section 5.5.3. A filter will permit the use of screens or slots with larger openings and provide a more pervious material around the wellpoint, thereby increasing its effective radius (Section 5.5.4). Geotextiles have been used but can clog quickly compared to a properly designed sand filter. Therefore, geotextiles should only be considered for very short duration (less than 2 months) of dewatering. The consequences of plugging wellpoints (lost production time, damaging foundations, and risk of failure of critical structures) must be carefully considered against the small increase in costs to install sand filters.

5.5.1.2 Wellpoint Hydraulics. The hydraulic head losses in a wellpoint system must be considered in designing a dewatering system. These losses can be estimated from Figure 60.

5.5.2 Wells. Wells for temporary dewatering systems typically have diameters ranging from 4 to 18 inches or more with a screen 20 to 75 feet long depending on the flow and pump size requirements.

5.5.2.1 Well Screens. Screens and riser pipes for wells are generally constructed of the same materials as for wellpoints. Good practice dictates the use of a filter around dewatering wells, which permits the use of fairly large slots or perforations, usually 0.025 to 0.100 inch in equivalent slot opening. As with wellpoints, sand filters are considered to be a best practice for well filters, and geotextiles should only be considered for very short duration dewatering projects. The slots in well screens should be as wide as possible but should meet criteria given in the following sections

5.5.2.2 Open Screen Area.

5.5.2.2.1 The open area of a well screen should be sufficient to keep the entrance velocity for the design flow low to reduce head losses and to minimize incrustation of the well screen in certain types of water. Figure 63, based on model research performed by Williams (1985) and funded by Roscoe Moss Company (1990), indicates that 3% to 5% open screen area is a reasonable design value for efficient wells. The American Water Works Association (AWWA), which had previously suggested an upper limit of 1.5 feet per second (ft/sec) in its Standard for Water Wells, made major revisions to the standard in 2006 (AWWA A100-06). This revision included a statement that “the standard no longer endorses the use of screen entrance velocity as the sole criterion for determining the minimum length of well screen.” Powers et al. (2007) recommends entrance velocities ranging from 0.03 to 0.2 ft/sec based on hydraulic conductivities of the surrounding soils.

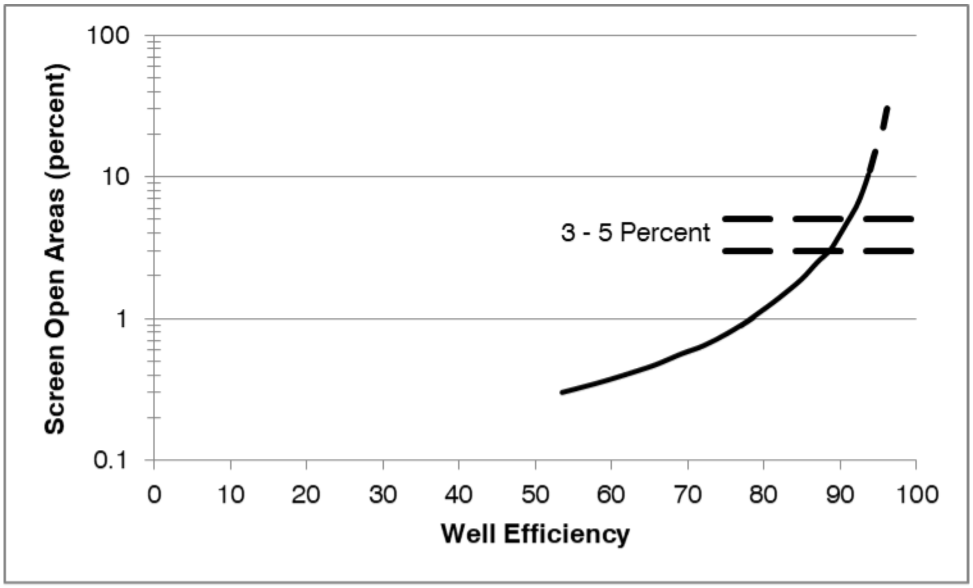


Figure 63. Relationships between screen open area and well efficiency (Williams, 1985)

5.5.2.2.2 As a practical matter, well screen openings and percentage open area should be as large as reasonably possible to minimize the effects of mineral incrustation and bacterial fouling during the operation of a dewatering system. As the flow to and length of a well screen is usually dictated by the characteristics of the aquifer and drawdown requirements, the required open screen area can be obtained by using a screen of appropriate diameter with a maximum amount of open screen area.

5.5.2.3 Well Hydraulics. Head losses within the well system (H_w) discussed in Section 5.2 can be estimated from Figures 59 and 60.

5.5.3 Effective Well Radius. The "effective" radius r_w of a well is the well radius that would have no hydraulic entrance loss H_w. If well entrance losses are considered separately in the design of a well or system of wells, r_w for a well or wellpoint without a filter may be considered to be one-half the outside diameter of the well screens; where a filter has been placed around a wellpoint or well screen, r_w may generally be considered to be one-half the outside diameter or the radius of the filter.

5.5.4 Well Penetration. In a stratified aquifer, the effective well penetration usually differs from that computed from the ratio of the length of well screen to total thickness of the aquifer.

5.5.5 Screen Length, Penetration and Diameter. The length and penetration of the screen depends on the thickness and stratification of the strata to be dewatered. The length and diameter of the screen and the area of perforations should be sufficient to permit the inflow of water without exceeding the entrance velocity given in Section 5.5.2 above. The "wetted screen length, h_{ws}" (or h_w for each stratum to be dewatered) is equal to or greater than Q_w/q_c. The diameter of the well screen should be at least 2 to 4 inches larger than the nominal diameter of the pump bowl, but smaller screens have been used successfully on many projects.

5.6 Pumps, Headers and Discharge Pipes. The capacity of pumps and piping should allow for a possible reduction in efficiency because of incrustation or mechanical wear caused by prolonged operation. This equipment should also be designed with appropriate valves, crossovers, and standby units so that the system can operate continuously, regardless of interruption for routine maintenance or breakdown.

5.6.1 Centrifugal and Wellpoint Pumps.

5.6.1.1 Centrifugal pumps can be used as sump pumps, jet pumps, or in combination with an auxiliary vacuum pump as a wellpoint pump. The selection of a pump and power unit depends on the discharge, suction lift, hydraulic head losses, including velocity head and discharge head, air-handling requirement, power available, fuel economy, and durability of unit. A wellpoint pump, usually consisting of a self-priming centrifugal pump with an auxiliary vacuum pump, should have adequate air-handling capacity and be capable of producing a vacuum of at least 22 to 25 feet of water in the headers. The suction lift of a wellpoint pump is dependent on the vacuum available at the pump bowl, and the required vacuum must be considered in determining the pumping capacity of the pump. Characteristics of a typical 8-inch wellpoint pump are shown in Figure 64. If the site is at a high elevation, atmospheric pressure can be materially lower than at sea level, and the maximum suction lift will be reduced. Powers et al. (2007) suggests reducing the theoretical suction lift 1 foot for every 1000 feet of elevation above sea level. Characteristics of a typical wellpoint pump vacuum unit are shown in Figure 65. Sump pumps of the centrifugal type should be self-priming and capable of developing at least 20 feet of vacuum. Jetting pumps are high head centrifugal pumps; typical characteristics of a typical 6-inch jetting pump are shown in Figure 66.

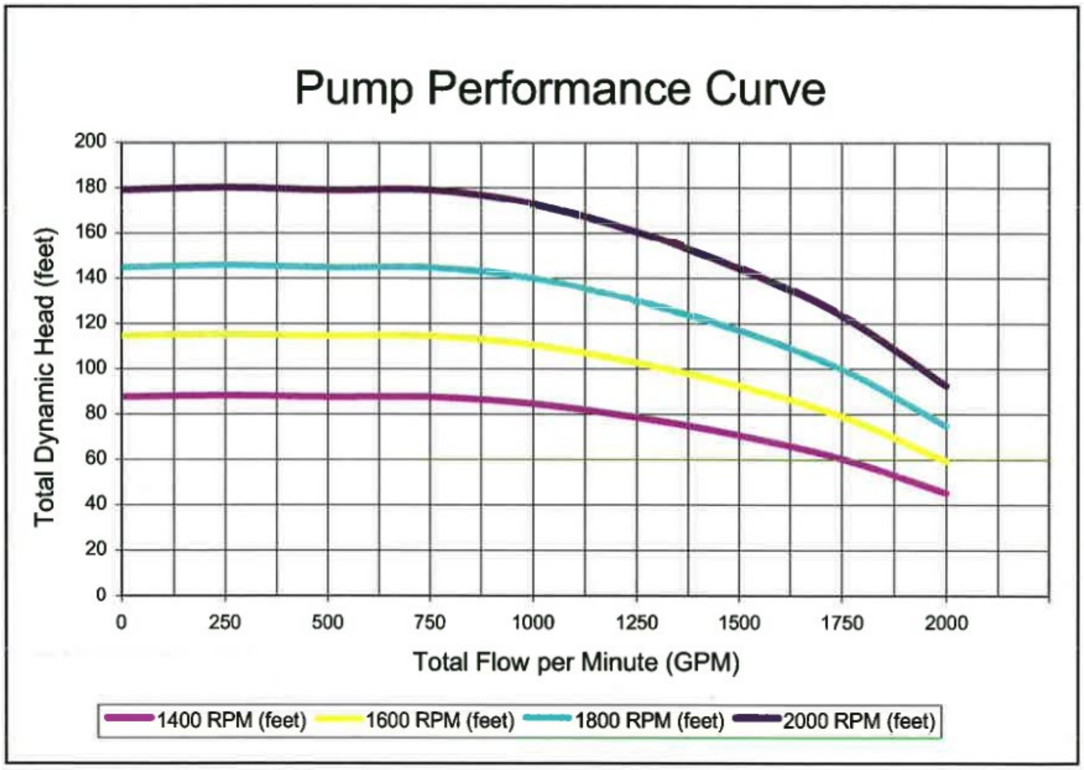


Figure 64. Characteristics of 8-inch Griffin wellpoint pump (Courtesy Griffin Dewatering, LLC.)

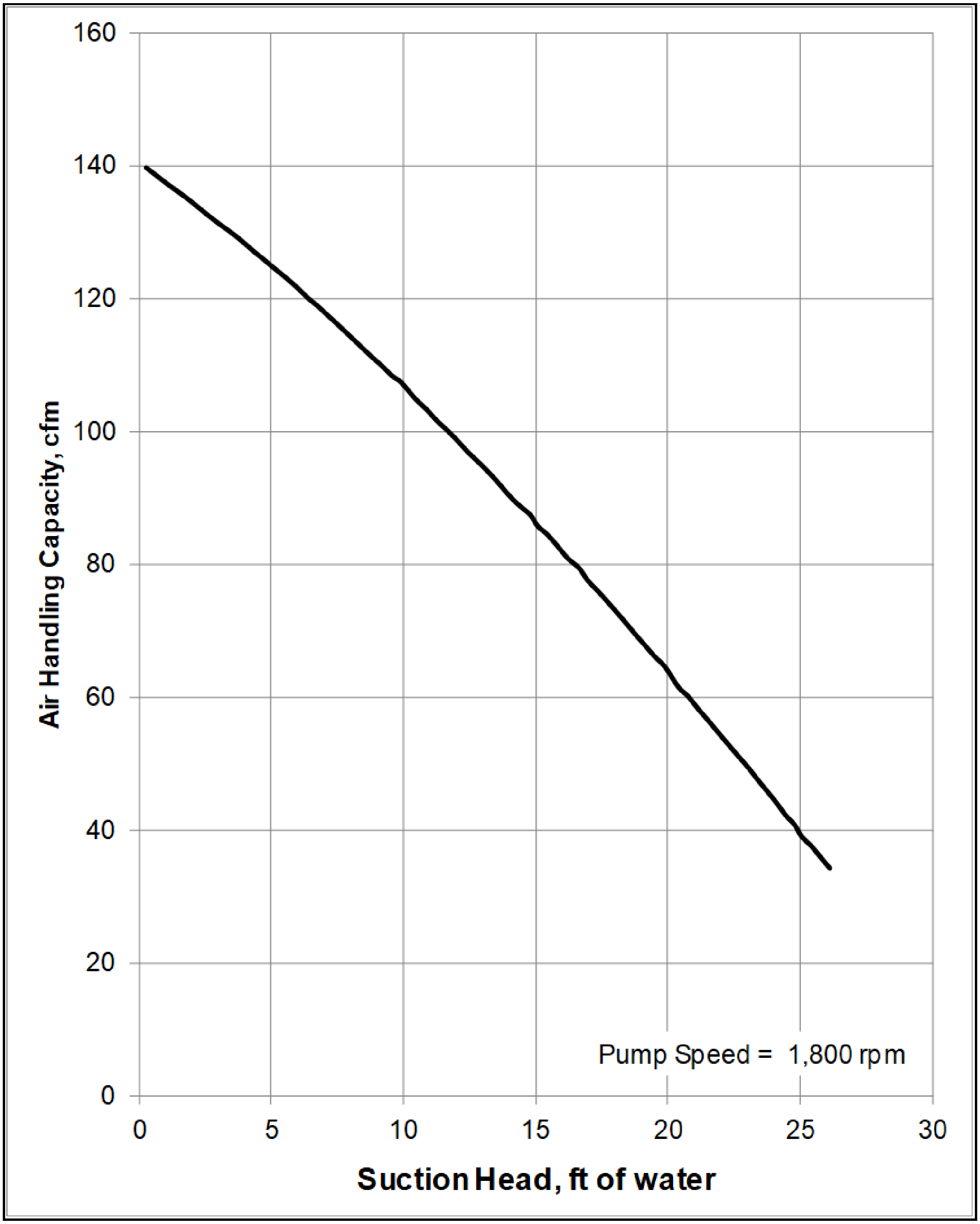


Figure 65. Characteristics of typical vacuum unit for wellpoint pumps (Adapted from TM 5-818-5)

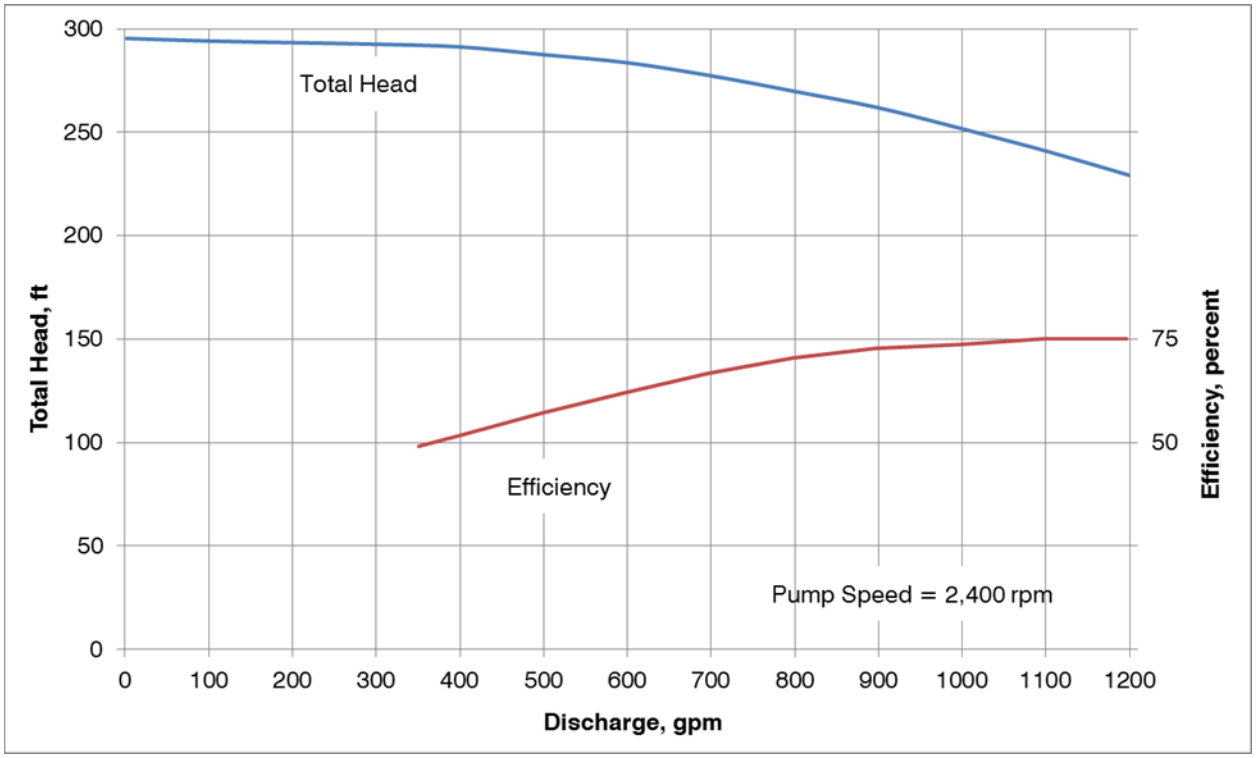


Figure 66. Characteristics of 6-inch jet pump (Adapted from TM 5-818-5)

5.6.1.2 Each operational wellpoint pump should be provided with one standby pump (connected to system, ready to operate) so as to ensure continuity of operation in event of pump or engine failure, or for repair or maintenance, particularly for pressure relief systems, recharge systems, and critical structures. For other, less critical projects, a few on-site stand-by pumps that are ready to be placed in service if a primary pump fails may be adequate.

5.6.1.3 By overdesigning the header pipe system and proper placement of valves, it may be possible to install only one standby pump for every two operational pumps. If electric motors are used for driving the normally operating pumps, the standby pumps should be powered with diesel, natural or LP gas, or gasoline engines. The type of power selected will depend on the power facilities at the site and the economics of installation, operation, and maintenance. It is also advisable to have spare power units on site in addition to the standby pumping units. Automatic switches, starters, and valves may be required if failure of the system is critical; it is good practice to test automatic switching one hour per week during operation of the dewatering system. On projects where risks are very high, it may be advisable to include a contract provision that allows the contracting officer's representative to test automatic switching at any time without the contractor's involvement. Such unannounced testing should obviously only be performed when a failure of the automatic switching system would not endanger the project and people working on it.

5.6.2 Deep-well Pumps.

5.6.2.1 Lineshaft turbine or submersible pumps are generally used to pump large-diameter deep wells and consist of one or more stages of impellers on a vertical shaft. Lineshaft turbine pumps can also be used as sump pumps, but adequate stilling basins and trash racks are required to ensure that the pumps do not become clogged. Motors of most large-capacity turbine pumps used in deep wells are mounted at the ground surface, but submersible pumps with capacities of up to about 1700 gpm are available and are commonly used in dewatering applications.

5.6.2.2 In the design of deep-well pumps, consideration must be given to required capacity, size of well screen and riser pipe, total pumping head, and the lowered elevation of the water in the well. The diameter of the pump bowl must be determined before the wells are installed, as the inside diameter of the well casing should be at least 2 to 4 inches larger in diameter than the pump bowl. Approximate capacities of various turbine pumps are presented in Table 10.

5.6.2.3 Electrically powered pumps require either power from a commercial source or one or more motor generators. If commercial power is used, 100 percent standby power should be provided for the system using motor generators equipped with automatic transfer switches. The standby generators and automatic switching should be tested for about an hour every week that the system is operated. Spare pumps, generally 10 to 20 percent of the number of operating pumps, as well as spare starters, switches, heaters, and fuses, should also be kept at the site.

5.6.2.4 Lineshaft turbine pumps can be powered with either electric motors or diesel engines with power takeoff clutches and gear drives. Where electric motors are used, 50 to 100 percent of the pumps should be equipped with combination gear drives connected to diesel (standby) engines. The number of pumps equipped would depend upon the redundancy designed into the system and the criticality of the dewatering or pressure relief needs. Motor generators may also be used as standby for commercial power. For some excavations and subsurface conditions, automatic starters and weather protection may be required for the diesel engines or motor generators being used as backup for commercial power.

Table 10

Capacity of Various Size Submersible and Lineshaft Turbine Pumps

Maximum Pump Bowl or Motor Size (inches)	Inside Diameter of Well (inches)	Approximate Maximum Capacity (gallons per minute)	
		Lineshaft	Submersible
4	5-6	90	90
5	6-8	160	-
6	8-10	450	350
8	10-12	600	1,100
10	12-14	1,200	1,400
12	14-16	1,800	1,800
14	16-18	2,400	-
16	18-20	3,000	-

U.S. Army Corps of Engineers, American-Marsh Pumps, and Grundfos

5.6.3 Turbovacuum Pumps. For some wellpoint systems requiring high pumping rates, it may be desirable to connect the header pipe to a 30- or 36-inch collection tank about 20 to 30 feet deep, seal at the bottom and top, and pump the flow into the tank with a high-capacity deep well turbine pump using a separate vacuum pump connected to the top of the tank to produce the necessary vacuum in the header pipe for the wells or wellpoints.

5.6.4 Header Pipe.

5.6.4.1 Hydraulic head losses caused by flow through the header pipe, reducers, tees, fittings, and valves should be computed and kept to a minimum by using large enough pipes. Minor head losses can be computed using equivalent pipe lengths for various fittings, valves and bends

5.6.4.2 Wellpoint header pipes should be installed as close as practical to the prevailing groundwater elevation and in accessible locations. Wellpoint pumps should be centrally located so that head losses to the ends of the system are balanced and as low as possible. If suction lift is critical, the pump should be placed low enough so that the pump suction is level with the header, thereby achieving a maximum vacuum in the header and the wellpoints. If construction is to be performed in stages, sufficient valves should be provided in the header to permit addition or removal of portions of the system without interrupting operation of the remainder of the system.

Valves should also be located to permit isolation of a portion of the system in case construction operations should break a swing connection or rupture a header.

5.6.4.3 Discharge lines should be sized so that the head losses do not create excessive back pressure on the pump. Ditches may be used to carry the water from the construction site, but they should be located well back of the excavation and should be reasonably watertight to prevent recharging the groundwater at the excavation.

5.7 Redundancy

5.7.1 Filters.

5.7.1.1 Filters are usually 2 to 3 inches thick for wellpoints and 3 to 6 inches thick for large-diameter wells. To prevent infiltration of the aquifer materials into the filter and of filter materials into the well or wellpoint, without excessive head losses, filters should meet the following criteria:

Screen-filter criteria

$$\textit{Slot or screen openings} \leq \textit{minimum filter } D_{50} \tag{10}$$

Filter-aquifer criteria

$$\frac{\text{Max filter } D_{15}}{\text{Min aquifer } D_{85}} \leq 5 \tag{11}$$

$$\frac{\text{Max filter } D_{50}}{\text{Min aquifer } D_{50}} \leq 4 \text{ to } 5.5 \tag{12}$$

$$\frac{\text{Min filter } D_{15}}{\text{Max aquifer } D_{15}} \geq 2 \text{ to } 5 \tag{13}$$

5.7.1.2 The filter should be poured around the well screen in a heavy continuous stream to minimize segregation. The filter may also be pumped into the well around the screen without causing segregation. Practice has proved that the tremie method is not necessary for a poorly graded filter if the filter is placed in a continuous stream. These criteria are different from criteria for design of permanent and critical seepage control features, such as chimney and blanket drains in dams. However, the above criteria are almost always adequate for the design of dewatering wells or wellpoints, considering that practically all dewatering flow is from granular soils and dewatering is usually a short-term, temporary operation. If it is necessary to protect soils finer than silty sand because of the duration of dewatering or for other reasons, specifications should be modified to require use of non-perforated pipes opposite the fine soils and/or design of filters using more stringent criteria to prevent loss of fines from clay and silt strata. If a performance specification is used for dewatering, filter performance is typically specified by requiring the contractor to demonstrate that filters are effective by limiting the measured sand content of the discharge from wells or wellpoints to some value, usually 5 ppm by volume. On projects with high capacity wells, limiting the maximum allowable sand content in the well (or wellpoint) discharge to 2 ppm should be considered. Changing the discharge sand content specification from 5 ppm to 2 ppm is not likely to make a significant difference in either installation cost or bid prices for dewatering.

5.7.2 General. The stability of soil in areas of seepage emergence is critical in the control of seepage. The exit gradient at the toe of a slope or in the bottom of an excavation must not exceed that which will cause surface raveling or sloughing of the slope, internal erosion (piping), or heave, blowout, or uplift (as these terms are defined in EM 1110-2-1901) of the bottom of the excavation.

5.7.3 Uplift or Blowout.

5.7.3.1 Before attempting to control seepage, an analysis should be made to evaluate the need for pressure relief to prevent blowout or uplift of less pervious strata below the bottom of an excavation that are underlain by a pervious stratum under pressure. The factor of safety against uplift of the less pervious strata should be calculated using the vertical effective stress factor of safety as described in EM 1110-2-1901. The factor of safety against uplift should be 1.25 to 1.5, depending on the criticality of the dewatering system.

5.7.3.2 In stratified subsurface soils, such as a coarse-grained pervious stratum overlain by a finer grained stratum of relatively low hydraulic conductivity, most of the head loss through the entire section will occur through the finer grained material. Consequently, a factor of safety based on the head loss through the top stratum would indicate a more critical condition than if the factor of safety was computed from the total head loss through the entire section.

5.7.4 Dewatering System Factor of Safety.

5.7.4.1 As in the design of any works, the design of a dewatering system should include a factor of safety to cover: (1) the variations in characteristics of the subsurface soils, stratification, and groundwater table; (2) the incompleteness of the data and inaccuracy of the formulations on which the design is based; (3) the reduction in the efficiency of the dewatering system with time; (4) the frailties of machines and operating personnel; and (5) the risk of failure of the system with regard to life safety, economics, and damage to the project. Including this factor of safety will provide important redundancy to the system. If all groundwater flow is unconfined, recovery of the groundwater level in the event of an outage will be much slower than if the flow is confined. In such cases, less redundancy may be required, especially if the groundwater level is drawn down several feet below subgrade level. It is prudent to perform full-scale field testing to predict the recovery time before the excavation is started. All of these factors should be considered in selecting the redundancy to be specified. The less information on which the design is based and the more critical the dewatering is to the success of the project, the higher the required redundancy should be.

5.7.4.2 Suggested factors of safety and design procedures are as follows:

a. Step 1. Determine the design parameters as accurately as possible from the available information.

b. Step 2. Use applicable design procedures and equations set forth in this manual.

c. Step 3. Consider the above enumerated factors in selecting a factor of safety.

d. Step 4. Evaluate the experience of the designer.

e. Step 5. After having considered steps 1-4, the factors of safety in Tables 11 and 12 are considered appropriate for modifying computed design values for flow, drawdown, well spacing, and required “wetted screen length.”

Table 11

Recommended Factors of Safety

Factor	Factor to be added to 1.0
Factor of Safety for Design FS = 1.0 + a factor + b factor + c factor	
a) Design Data	
Poor	0.25
Fair	0.20
Good	0.10
Excellent	0.05
b) Experience of Designer	
Little	0.25
Some	0.20
Good	0.10
Excellent	0.05
c) Consequences of Failure	
Great	0.25
Moderate	0.20
Little	0.15

Table 12

Application of Factor of Safety to Computed Values or System Design Features

<table>
<tr><th>Computed Value
System Design Feature</th><th>Design Procedure</th><th>Remarks</th></tr>
<tr><td>Pump capacity, header, and discharge pipe (Q)</td><td>Increase Q based on FS</td><td>-</td></tr>
<tr><td>Drawdown (Δh)</td><td>Decrease Δh by 10 percent</td><td rowspan="2">Adjust either drawdown or well spacing, but not both</td></tr>
<tr><td>Well spacing (a)</td><td>Decrease a by 10 percent</td></tr>
<tr><td>Wetted screen length (h_{ws})</td><td>Use h_{ws} computed from design procedure</td><td>-</td></tr>
<tr><td colspan="3">Note: In initially computing drawdown, well spacing, and wetted screen, use flow and other parameters unadjusted for factor of safety.</td></tr>
</table>

5.8 Dewatering Open Excavations. An excavation can be dewatered, or the artesian pressure relieved by one or a combination of methods described in Chapter 3. The design of dewatering and groundwater control systems for open excavations, shafts, and tunnels is discussed in the following sections. Examples of design for various types of dewatering and pressure relief systems are given in Appendix C.

5.8.1 Trenching and Sump Pumping.

5.8.1.1 The applicability of ditching and sump pumping for dewatering an open excavation is discussed in Chapter 3. Where soil conditions and the depth of an excavation below the water table permit ditching and sump pumping of seepage (Figure 2), the rate of flow into the excavation can be estimated from plan and sectional flow net analyses (Figure 61) or formulas presented in Sections 5.2 through 5.4.

5.8.1.2 Where an excavation extends into rock and there is a substantial inflow of seepage, perimeter drains can be installed at the foundation level outside of the formwork for a structure. The perimeter drainage system should be connected to a sump in the excavation outside of the planned work, and the seepage water pumped out. After construction, the drainage system should be grouted. Excessive hydrostatic pressures in the rock mass endangering the stability of the excavated face can be relieved by drilling 4-inch diameter horizontal drain holes into the rock at approximately 10-foot centers. For large seepage inflow, supplementary vertical holes for deep-well pumps at 50- to 100-foot intervals may be desirable for temporary lowering of the groundwater level to provide suitable conditions for concrete or earth fill placement.

5.8.2 Wellpoint System. The design of a line of wellpoints pumped with either a conventional wellpoint pump or jet-eductors is generally based on mathematical, flow-net or numerical analysis of flow and drawdown at a continuous slot (Sections 5.2 through 5.4).

5.8.2.1 Conventional Wellpoint System.

5.8.2.1.1 The drawdown attainable per stage of wellpoints (about 15 feet at sea level) is limited by the vacuum that can be developed by the pump (reduced about 1 foot for every 1000 feet above sea level), the height of the pump above the header pipe, and hydraulic head losses in the wellpoint and collector system. Where two or more stages of wellpoints are required, it is customary to design each stage so that it is capable of producing the total drawdown required by that stage with none of the upper stages functioning. However, the upper stages are generally left in so that they can be pumped in the event pumping of the bottom stage of wellpoints does not lower the water table below the excavation slope because of stratification, and so that they can be pumped during backfilling operations and to allow lower wellpoint stages to be removed.

5.8.2.1.2 The design of a conventional wellpoint system to dewater an open excavation, as discussed in Section 5.2, is outlined below.

a. Step 1. Select dimensions and groundwater coefficients (H, L, and k) of the formation to be dewatered based on investigations outlined in Chapter 4.

b. Step 2. Determine the drawdown required to dewater the excavation or to dewater down to the next stage of wellpoints, based on the maximum groundwater level expected during the period of operation.

c. Step 3. Compute the head at the assumed slot (he or ho) to produce the desired residual head h_D in the excavation.

d. Step 4. Compute the flow per lineal foot of drainage system to the slot Q_p.

e. Step 5. Assume a wellpoint spacing, a, and compute the flow per wellpoint, $Q_w = aQ_p$.

f. Step 6. Calculate the required head at the wellpoint hw corresponding to Q_w. *h_w is calculated from equations shown in Figures 44 through 56 for wells. The relevant equations should be selected based on the geometry of system being designed and the geometry of the source of seepage. Additional discussion can be found in Section 5.2.2.3.*

g. Step 7. Check to see if the suction lift that can be produced by the wellpoint pump will lower the water level in the wellpoint to hw as follows:

$$V \geq M - h_w + H_c + H_w \quad (14)$$

Where:

V = vacuum at pump intake (feet of water)

M = vertical distance from base of pervious strata to pump intake (feet)

H_C = average head loss in header pipe from wellpoint (feet)

H_W = head loss in wellpoint, riser pipe, and swing connection to header pipe (feet)

h_W = required head at the wellpoint (feet)

h. Step 8. Set the top of the wellpoint screen at least 1 to 2 feet or more below $hw-Hw$ to provide adequate submergence of the wellpoint so that air will not be drawn into the system. The wellpoint screens and filters should be selected and designed according to the criteria set forth under Section 5.5.

5.8.2.1.3 An example of the design of a single-stage wellpoint system to dewater an excavation is illustrated in Figure C.5, Appendix C.

5.8.2.1.4 If an excavation extends below an aquifer into an underlying impermeable soil or rock formation, some seepage will pass between the wellpoints at the lower boundary of the aquifer. This seepage may be intercepted with ditches or drains inside the excavation and removed by sump pumps. If the underlying stratum is clay, the wellpoints may be installed in holes drilled about 1 to 2 feet into the clay and backfilled with filter material. By this procedure, the water level at the wellpoints can be maintained near the bottom of the aquifer, and thus seepage passing between the wellpoints will be minimized. Sometimes these procedures are ineffective, and a small dike in the excavation just inside the toe of the excavation may be required to prevent seepage from entering the work area. Sump pumping can be used to remove water from within the diked area.

5.8.2.2 Jet-eductor Well or Jet-eductor Wellpoint Systems.

5.8.2.2.1 Flow and drawdown to a jet-eductor well or wellpoint system can be computed or analyzed as discussed in Section 5.2. Jet-eductor dewatering systems can be designed as follows:

a. Step 1. Assume the line of wells or wellpoints to be a drainage slot.

b. Step 2. Compute the total flow to the system for the required drawdown and penetration of the well screens.

c. Step 3. Assume a well or wellpoint spacing that will result in a reasonable flow for the well or wellpoint and jet-eductor pump.

d. Step 4. Compute the head at the well or wellpoint h_w required to achieve the desired drawdown.

e. Step 5. Set eductor pump at $M = h_w - H_w$ with some allowance for future loss of well efficiency. The wells or wellpoint screens and filters should be selected and designed according to the criteria set forth under Section 5.5.

5.8.2.2.2 If the soil formation being drained is stratified and an appreciable flow of water must be drained down through the filter and around the riser pipe to the wellpoint, the spacing of the wellpoints and the hydraulic conductivity of the filter must be such that the flow from formations above the wellpoints does not exceed (from Darcy's Law):

$$Q_w = k_v i A \tag{15}$$

Where:

Q_w = flow from formation above wellpoint

k_v = vertical hydraulic conductivity of filter

A = horizontal area of filter

i = gradient produced by gravity = 1.0

5.8.2.2.3 The filter hydraulic conductivity may be estimated using laboratory tests or reliable correlations with grain size given in Section 4.4. Substitution of smaller diameter well screens for wellpoints may be indicated for stratified formations. Where a formation is stratified or fine-grained, or there is little available submergence for the wellpoints, jet-eductor wells or wellpoints and risers should be provided with a pervious filter, and the wellpoints set at least 10 feet back from the edge of a vertical excavation.

5.8.2.2.4 Jet-eductor pumps may be powered with individual small high-pressure centrifugal or submersible pumps or with one or two large centrifugal or submersible pumps in a recirculation tank pumping into a common pressure header pipe furnishing water to each eductor, connected in turn to a common return header. With a single-pump setup, the water is usually circulated through a stilling tank with an overflow for the flow from wells or wellpoints. Design of jet eductors must consider: (1) the static lift from the wells or wellpoints to the water level in the recirculation tank; (2) head loss in the return riser pipe; (3) head loss in the return header; and (4) flow from the wellpoint. The (net) capacity of a jet-eductor pump depends on the pressure head, input flow, and diameter of the jet nozzle in the pump. Generally, a jet-eductor pump requires an input flow of about 2 to 2½ times the flow to be pumped depending on the operating pressure and design of the nozzle. Consequently, if flow from the wells or wellpoints is large, a deep-well system will be more appropriate and economical than eductor wells or wellpoints. The pressure header supplying a system of jet eductors must be of such size that a fairly uniform pressure is applied to all of the eductors in the system.

5.8.2.3 Vacuum Well or Vacuum Wellpoint System. Vacuum wellpoint systems for dewatering fine-grained soils are similar to conventional wellpoint systems except the wellpoint and riser are surrounded with filter sand that is sealed at the top, and additional vacuum pump capacity is provided to ensure development of the maximum vacuum in the wellpoint and filter regardless of air loss. In order to obtain 8 feet of vacuum in a wellpoint and filter column, with a pump capable of maintaining a 25-foot vacuum in the header, the maximum lift is 17 feet (25 feet minus 8 feet). Where a vacuum type of wellpoint system is required, the required pump capacity is small. The capacity of the vacuum pump will depend on: (1) the air conductivity of the soil, (2) the vacuum to be maintained in the filter, (3) the proximity of the wellpoints to the excavation, (4) the effectiveness of the seal at the top of the filter, and (5) the number of wellpoints being pumped. In very fine-grained soils, pumping must be continuous. The flow may be so small that water may have to be added to the system to cool the pump properly.

5.8.3 Electro-osmosis.

5.8.3.1 An electro-osmosis dewatering system consists of anodes (positive electrodes, usually a pipe or rod) and cathodes (negative electrodes, usually wellpoints or small wells installed with a surrounding filter), across which a d-c voltage is applied. The depth of the electrodes should be at least 5 feet below the bottom of the slope or excavation to be stabilized. The spacing and arrangement of the electrodes may vary, depending on the dimensions of the slope or excavation to be stabilized and the voltage available at the site. Cathode spacing of 25 to 40 feet have been used, with the anodes installed midway between the cathodes. Electrical gradients of 1.5- to 4-volts-per-foot distance between anodes and cathodes have been successful in electro-osmotic stabilization. Applied voltages of 30 to 100 volts are usually satisfactory; a low voltage is usually sufficient if the groundwater has a high mineral content.

5.8.3.2 The discharge of a cathode wellpoint may be estimated from the equation:

$$Q_e = k_e i_e a z \tag{16}$$

Where:

k_e = coefficient of electro-osmotic permeability (assume 0.98 x 10^{-4} feet per second per volt per foot).

i_e = electrical gradient between electrodes (volts per foot)

a = effective spacing of wellpoints (feet)

z = depth of soil being stabilized (feet)

5.8.3.3 Current requirements commonly range between 15 and 30 amperes per well and power requirements are generally high. However, regardless of the expense of installation and operation of an electro-osmotic dewatering system, it may be the only effective means of dewatering and stabilizing certain silts, clayey silts, and clay-silt-sand mixtures. Electro-osmosis may not be applicable to saline soils because of high current requirements, nor to organic soils because of environmentally objectionable effluents, which may be unsightly and have exceptionally high pH values.

5.8.4 Deep-well Systems.

5.8.4.1 The design and analysis of a deep-well system to dewater an excavation depends upon the configuration of the site dewatered, source of seepage, type of flow (artesian or gravity), penetration of the wells, and the submergence available for the well screens with the required drawdown at the wells. Flow and drawdown to wells can be computed or analyzed as discussed in Section 5.2.

5.8.4.2 Methods are presented in Sections 5.2 and 5.3 whereby the flow and drawdown to a well system can be computed either by analysis or by a flow net assuming a continuous slot to represent the array of wells, and the drawdown at and between wells computed for the actual well spacing and location. Examples of the design of a deep-well system using these methods and formulas are presented in Appendix C on Figures C.2 and C.3.

5.8.4.3 The submerged length and size of a well screen should be checked to ensure that the design flow per well can be achieved without excessive screen entrance losses or velocities. The pump intake should be set so that adequate submergence (a minimum of 2 to 5 feet) is provided when all wells are being pumped. Where the type of seepage (artesian or gravity) is not well established during the design phase, the pump intake should be set 5 to 10 feet below the design elevation to ensure adequate submergence. Setting the pump bowl below the expected drawdown level will also facilitate drawdown measurements. The well screens and filters should be selected and designed according to the criteria set forth under Section 5.5.

5.8.5 Combined Systems.

5.8.5.1 Deep Well and Wellpoint Systems. A dewatering system composed of both deep wells and wellpoints may be appropriate where the groundwater table has to be lowered appreciably and near to the top of an impermeable stratum. A wellpoint system alone would require several stages of wellpoints to do the job, and a deep well system alone would not be capable of lowering the groundwater completely to the bottom of the aquifer. A combination of deep wells and a single stage of wellpoints (see Figure C.4) may permit lowering to the desired level. The advantages of a combined system, in which deep wells are essentially used in place of the upper stages of wellpoints, are as follows:

a. The excavation quantity is reduced by the elimination of berms for installation and operation of the upper stages of wellpoints.

b. The excavation can be started and advanced without delays to install any higher stages of wellpoints.

c. The deep wells installed at the top of the excavation will serve not only to lower the groundwater to permit installation of the wellpoint system, but also to intercept a significant amount of seepage and thus reduce the flow to the single stage of wellpoints. A design example of a combined deep well and wellpoint system is shown in Figure C.4.

5.8.5.2 Vertical Drains with Deep Wells and Wellpoints.

5.8.5.2.1 Vertical drains can be used to intercept horizontal seepage from stratified deposits and conduct the water vertically downward into a pervious stratum that can be dewatered by means of deep wells or wellpoints. The limiting feature of dewatering by vertical drains is usually the vertical hydraulic conductivity of the vertical drain itself, which restricts this method of drainage to soils of low hydraulic conductivity that yield only a small flow of water. The vertical capacity can be greatly increased by installing a small diameter well screen in the center of the drain, as discussed below. Vertical drains must be designed so that they will intercept the seepage flow and have adequate capacity to allow the seepage to drain downward without any back pressure. To accomplish this, the drains must be spaced, have a diameter, and be filled with filter sand so that (from Darcy's Law):

$$Q_D \leq k_D i A_D \tag{17}$$

Where:

Q_D = flow per drain

k_D = vertical hydraulic conductivity of sand filter

i = gradient produced by gravity = 1.0

A_D = area of drain

5.8.5.2.2 Generally, sand drains are spaced on 5- to 15-foot centers and have a diameter of 10 to 18 inches. The maximum vertical hydraulic conductivity of a filter that may be used to drain soils for which sand drains are applicable is about 1000 to 3000 x 10^{-4} cm/sec or 0.20 to 0.60 ft/min, thus, the maximum capacity Q_D of a sand drain is only about 1 to 3 gpm for a 10 inch diameter drain, and about 3 to 8 gpm for an 18 inch diameter drain. The capacity of sand drains can be significantly increased by installing a small (1- , 1½-, or 2-inch) slotted PVC pipe in the drain to conduct seepage into the drain downward into underlying more pervious strata being dewatered, or by installing a filter compatible drainage stone column in the center of the sand drain.

5.8.5.2.3 Prefabricated vertical drains (PVDs) or "wick" drains can also be used instead of sand drains. PVDs consist of a plastic strip with molded channels wrapped in a geoxtextile filter. The vertical flow capacity of wick drains, which are typically used in clays, do not have high vertical flow capacity. Nevertheless, wick drains may have adequate vertical capacity for the stratum to be drained if its hydraulic conductivity is not too high. If there are many drains to be installed (say more than 100), they are typically installed using a hollow mandrel that is vibrated into the ground. For fewer than 100 drains, they are usually installed by drilling or by jetting a temporary casing that is withdrawn after the drain is placed. Another dewatering product available consists of a prefabricated two-layer plastic drainage core surrounded by a geotextile envelope into which a pipe (usually PVC) is inserted. This product is typically installed in a drilled or jetted hole either with or without a sand filter. Both of these products overcome the potential vertical capacity problem of conventional sand drains without slotted pipes, but frictional head losses should always be checked for the calculated flows to confirm that the vertical capacity is adequate.

5.8.6 Pressure Relief Systems.

5.8.6.1 Temporary relief of artesian pressure beneath an open excavation is required during construction where the stability of the bottom of the excavation is endangered by artesian pressures in an underlying aquifer. Complete relief of the artesian pressures to a level below the bottom of the excavation is not always required, depending on the thickness, uniformity, and hydraulic conductivity of the materials. For uniform tight shales or clays, an upward seepage gradient i as high as 0.5 to 0.6 may be safe, but clayey silts or silts generally require lowering the groundwater 5 to 10 feet below the bottom of the excavation to provide a dry, stable work area.

5.8.6.2 The flow to a pressure relief system is artesian; therefore, such a system may be designed or evaluated on the basis of the methods presented in Sections 5.2, 5.3, and 5.4. The penetration of the wells or wellpoints need be no more than that required to achieve the required drawdown to keep the flow to the system a minimum. If the aquifer is stratified and anisotropic, the penetration required should be determined by computing the effective penetration into the transformed aquifer. Examples of the design of a wellpoint system and a deep well system for relieving pressure beneath an open excavation are presented in Figures C.5 and C.6 in Appendix C.

5.8.7 Control of Surface Water.

5.8.7.1 Runoff of surface water from areas surrounding the excavation should be prevented from entering the excavation by sloping the ground away from the excavation or by the construction of dikes around the top of the excavation. Ditches and dikes can be constructed on the slopes of an excavation to control the runoff of water and reduce surface erosion. Runoff into slope ditches can be removed by pumping from sumps installed in these ditches, or it can be carried in a pipe or lined ditch to a central sump in the bottom of the excavation where it can be pumped out. Dikes at the top of an excavation and on slopes should be designed with adequate freeboard above the maximum elevation of water to be impounded, adequate crown width, and stable side slopes. The stability of the excavated slope needs to be evaluated if a dike is placed near the top of the slope as the dike could cause slope instability by loading the top of the excavated slope.

5.8.7.2 In designing a dewatering system, provisions must be made for collecting and pumping out surface water so that the dewatering wells and pumps will not be flooded. Control of surface water within the diked area will not only prevent interruption of the dewatering operation, which might seriously impair the stability of the excavation, but also prevent damage to the construction operations and minimize interruption of work. Surface water may be controlled by dikes, ditches, sumps, and pumps; the excavation slope can be protected by seeding or covering with geotextile or asphalt. Items to be considered in the selection and design of a surface water control system include the duration and season of construction, rainfall frequency and intensity, size of the area, and character of surface soils.

5.8.7.3 The magnitude of the rainstorm that should be used for design depends on the geographical location, risk associated with damage to construction or the dewatering system, and probability of occurrence during construction. The common frequency of occurrence used to design surface water control sumps and pumps is a once in 2-to 5-year rainfall. For critical projects, a frequency of occurrence of once in 10 years or longer may be advisable.

5.8.7.4 Impounding runoff on excavation slopes is somewhat risky because any overtopping of an impounding dike on a higher berm could result in overtopping of all impounding dikes on berms at lower elevations with resultant flooding of the excavation.

5.8.7.5 Ditch grades and cross sections should be designed for slow velocity to minimize erosion and sediment transport. White and Prentis (1950) suggest that ditches be about 2 feet deep and be offset from the toe of the excavation slope about 10 to 15 feet. Ample allowance for silting of ditches should be made or regular cleaning required to ensure that adequate capacities are available throughout the duration of construction. Sumps should be designed such that suspended sand and silt in the runoff does not reach the pump easily. Water from sumps should not be pumped into the main dewatering system. Depending on environmental discharge regulations or project discharge permits, it may be necessary to treat water from open pumping to reduce suspended sediment content before discharging it to natural drainage features.

5.8.7.6 The pump and storage requirements for control of surface water within an excavation can be estimated in the following manner this method is illustrated by Figure C.10.

a. Step 1. Select frequency of rainstorm for which pumps, ditches, and sumps are to be designed.

b. Step 2. For selected frequency (e.g., once in 5 years), determine rainfall for 10-, 30-, and 60-minute rainstorms at project site from NOAA's National Weather Service PFDS. See example of PFDS data tabulation for a particular locality in Figure 34.

c. Step 3. Assuming instantaneous runoff, compute volume of runoff VR (for each assumed rainstorm into the excavation or from the drainage area into the excavation) from the equation below. The value of c depends on relative porosity, character, and slope of the surface of the drainage area. For impervious or saturated steep slopes, c values may be assumed to range from 0.8 to 1.0. For unsaturated sand and clay with reasonably flat grades, assume c = 0.55 and 0.70, respectively.

$$V_R = cRA = c\,\frac{R}{12}\,43{,}560A \text{ (cubic feet)} \tag{18}$$

Where:

c = coefficient of runoff

R = rainfall for assumed rainstorm (inches)

A = area of excavation plus area of drainage into excavation, acres

d. Step 4. Plot values of VR versus assumed duration of rainstorm.

e. Step 5. Plot pump rate of the pump to be installed assuming pump is started at onset of rain.

5.8.7.7 The required ditch and sump storage volume $\overline{V}$ is the (maximum) difference between the accumulated runoff for the various assumed rainstorms and the amount of water that the sump pump (or pumps) will remove during the same elapsed period of rainfall. The capacity and layout of the ditches and sumps can be adjusted to produce the optimum design with respect to the number, capacity, and location of the sumps and pumps.

5.8.7.8 Conversely, the required capacity of the pumps for pumping surface runoff depends upon the volume of storage available in sumps, as well as the rate of runoff (see equation 19). For example, if no storage is available, it would be necessary to pump the runoff at the rate it enters the excavation to prevent flooding. This method usually is not practicable. In large excavations, sumps should be provided where practicable to reduce the required pumping capacity. The volume of sumps and their effect on pump size can be determined graphically (as shown in Figure C.10) or can be estimated approximately from the following equation:

$$Q_P = Q - \frac{\overline{V}}{T} \tag{19}$$

Where:

Q_p = total pump capacity (gpm)

Q = average rate of runoff (gpm)

$\overline{V}$ = volume of sump storage (gallons)

T = duration of rainfall (minutes)

Chapter 6
Installation of Dewatering and Groundwater Control Systems.

6.1 General. The successful performance of any dewatering system requires that it be properly installed. Principal installation features of various types of dewatering or groundwater control systems are presented in the following sections.

6.2 Deep-Well Systems. Deep wells may be installed by the reverse-rotary drilling method, by driving and jetting a casing into the ground and cleaning it with a bailer or jet, or with the bucket auger method, and by a variety of full depth casing methods.

6.2.1 Reverse Circulation Rotary Method.

6.2.1.1 In the reverse-rotary method, the hole for the well is made by rotary drilling, using a bit of a size required by the screen diameter and thickness of filter. Soil from the drilling is removed from the hole by the flow of water circulating from the ground surface down the hole and back up the (hollow) drill stem from the bit. The drill water is circulated by air-lifting or a centrifugal or jet-eductor pump that pumps the flow from the drill stem into a sump pit. As the hole is advanced, the soil particles settle out in the sump pit, and the muddy water flows back into the drill hole through a ditch cut from the sump to the hole. The sides of the drill hole are stabilized by seepage forces acting against a thin film of fine-grained soil that forms on the wall of the hole. A sufficient seepage force to stabilize the hole is produced by maintaining the water level in the hole at least 7 feet above the natural water table. No bentonite drilling mud should be used because of gelling in the filter and aquifer adjacent to the well. Organic polymer drilling fluid, e.g., Johnson's Revert (food-grade guar gum) or equivalent, may be added to the drilling water to reduce water loss, if needed. The sump pit should be large enough to allow the sand to settle out, but small enough so that the silt is kept in suspension. Design of such features is typically the responsibility of the contractor.

6.2.1.2 Holes for deep wells should be vertical so that the screen and riser may be installed straight and plumb; appropriate guides should be used to center and keep the screen plumb and straight in the hole. The hole should be about 2 to 3 feet deeper than the well screen and riser to collect sediment in the drilling water or fluid that settles out when drilling is stopped. After the screen is in place, the filter is placed in a heavy, continuous stream around the well screen and casing. The level of drilling fluid or water in a reverse-rotary drilled hole must be maintained at least 7 feet above the natural groundwater level until all the filter material is placed. If a casing is used, it should be pulled as the filter material is placed, keeping the bottom of the casing 2 to 10 feet below the top of the filter material as the filter is placed.

6.2.2 Bucket Auger Method.

6.2.2.1 The bucket auger method is similar to the reverse-rotary method in that hole stability is provided by flooding the hole with water or other fluid and maintaining a certain amount of differential head in the hole above the groundwater level. A typical estimate of the head needed is one foot of head above the groundwater level for every 10 feet of drill depth. Drilling pads may be required to provide artificial head if enough head can't be provided from existing ground. A temporary casing is usually but not always installed to or a few feet below the groundwater level to keep the top of the hole from caving.

6.2.2.2 The drilling bucket has a latched, hinged bottom and two openings with flaps to allow the drilled material to enter the bucket. The bucket is pinned through a Kelly box (usually solid 3.5- to 6-inch square, welded to the top of the bucket) to the bottom of the innermost Kelly. Increased depth is achieved by adding larger telescoping bars (up to 4 bars are common). The bars can be hydraulically crowded with down force to force the bucket to penetrate. Buckets can be used with any rig having a Kelly bar, but the two most common types are bucket auger rigs with a fixed turntable and foundation auger rigs that rotate on a turntable mounted on either a truck or tracked carrier. Rig stability and leveling is provided by 3 to 4 hydraulic outriggers for truck-mounted rigs. When the bucket is full, it is hoisted out of the drill hole, swung or pushed out to the side, and unlatched to dump the drilling spoil. Care should be taken with removal of the bucket auger from the hole as removing it too quickly can create suction that can collapse the hole.

6.2.2.3 The depth capacity of telescoping rigs is commonly 100 to 150 feet, although greater depths can be achieved by "stemming", a procedure that involves adding a Kelly bar extension each time the Kelly is lowered back into the hole and removing the stem when it is hoisted for dumping. The speed of stemming can be increased by having an auxiliary crane to hoist the Kelly plus extension, and bucket to dump the drilling spoil and lower the assembly back into the hole after it is emptied.

6.2.2.4 The maximum diameter of bucket or auger that can be handled with a bucket auger rig is limited by its turntable opening, commonly 51 inches. A swinging foundation auger rig is limited by the distance between the back of the rig and its position when extended fully back, commonly 6 feet or more. Torque capacity ranges from 30,000 to 40,000 ft-lbs (most bucket auger rigs) to 115,000 ft-lbs for an average foundation auger rig, and 180,000 ft-lbs for a heavy-duty foundation rig. Procedures for maintaining hole stability, installing the well assembly, placing the filter, and developing the wells are the same as described above for the reverse-circulation rotary method.

6.2.3 Full-Depth Casing Methods.

6.2.3.1 General. Driving, drilling, or rotating/crowding a temporary steel casing and then cleaning it out for well installation is another effective, safe (especially downstream of existing dams and levees with water against them), economical method for installing efficient wells, particularly if more than a few wells are to be installed. Full-depth casing methods that have been used for well installation include:

a. Cable-tool drilling with casing

b. Top-drive rotary using drill-through casing hammer

c. Hollow stem auger drilling

d. Hole puncher and sanding casing

e. Sonic drilling

f. Foremost Dual Rotary drilling (formerly known as Barber Dual Rotary)

g. Driving, and extracting steel casing using pile hammer

h. Casing rotators and oscillators

6.2.3.2 Cable Tool Drilling with Casing. The cable-tool drilling method has been used continuously in water well construction for many years. This method alternately raises and drops heavy chisel-type tools on the bottom of the borehole to advance it. In unconsolidated formations, a casing is necessary for hole stability. Cuttings are removed with a bailer. Representative formation sample can be obtained during drilling by driving a split barrel sample using driving jars. It is an effective method for installing small diameter wells in deposits with cobbles and boulders. The hole and casing diameter is limited by the weight of the tools, hydraulic jack capacity, and the strength of the wire rope. Although equipment costs are low, the method is relatively slow compared to other methods and often more costly.

6.2.3.3 Top-drive Rotary Using Drill-Through Casing Hammer. The casing hammer method uses a direct air rotary system to simultaneously drill and drive casing. Drivers are available that will drive up to 24-inch diameter casings and will extract as well as drive the casings. This method is effective in formations with boulders. If the formation is well-graded and coarse, the casing itself can be perforated down-hole after installation and will produce sand-free water. If a filter pack is necessary, a smaller diameter well screen backfilled with suitably graded filter sand can be installed and the drive casing extracted after installing the well screen and filter pack. A disadvantage of this method is that most casing hammers are powered using compressed air, which is also used to drill and evacuate drill cuttings. Using air for drilling is a problem due to uncontrolled high pressure compressed air leakage to the ground surface. Using direct rotary wash drilling techniques eliminates the air loss problem. Hydraulically powered casing drivers are also currently manufactured that can be used in conjunction with rigs equipped with conventional piston-type mud pumps and direct rotary wash drilling to remove cuttings.

6.2.3.4 Hollow Stem Auger Drilling. Hollow stem augers (HSAs) are commercially available in inside diameters up to 12.25 inches (outside diameter about 18 inches) that will allow the installation of 8-inch pipe size well screen with a nominal 5-inch thick filter pack. High torque is necessary to turn the augers in sand and gravel formations, so the well depth is limited by the available drilling torque. The effectiveness of HSAs in penetrating formations infested with cobbles and boulders is questionable. Penetrating a clay strata with HSAs have been known to cause well inefficiency due to smearing as the augers drag the clay past higher sand strata as the hole is advanced, and may densify the soil adjacent to the borehole, both of which will impede flow into a well or wellpoint. One advantage of this method is that many rigs set up for HSAs are also equipped to drive barrel-type soil samplers.

6.2.3.5 Hole Puncher and Sanding Casing.

6.2.3.5.1 This method, originally developed for installing filtered small diameter wellpoints, has been widely used since around 1960 to install temporary casings up to 30-inch diameter to depths exceeding 100 feet through sand, gravel, cobbles, boulders, hard clay, compaction shale, and weathered rock. The method, which requires a crane (or other hoisting equipment) with at least two free-spooling independent hoisting lines, consists of simultaneously jetting a heavy-wall steel pipe (the hole puncher) equipped with a flanged heavy steel driving head) and driving a larger diameter heavy-wall steel casing (the sanding casing) with a reinforced flange anvil welded to the top. The hole puncher is alternately raised and dropped with one hoisting line while the casing is held by the second line. It is essential to maintain the bottom of the casing deep enough during driving so that the jetting water returns continually through the annulus between the casing and the hole puncher, or voids will develop outside of the casing.

6.2.3.5.2 Figure 67 shows a 10-inch diameter sanding casing being driven with a 4-inch diameter hole puncher to install wells in the pockets of a sheet pile cofferdam outboard of the wales. After the casing is driven to the desired elevation, the well screen assembly is set, and filter is placed to at least a few feet above the top of the well screen. The method requires a large amount of water for jetting throughout the installation of the casing, so a ready water supply is necessary. If voids have developed due to water return outside of the casing, the filter will move into the voids when the sanding casing is extracted, potentially creating an unfiltered screen contact with the formation, causing potential sanding during pumping and resulting in subsidence close to the well. There would also be a potential for subsidence at some distance from the well if there are stiff layers above the well screen in the soil profile that temporarily bridge above voids.

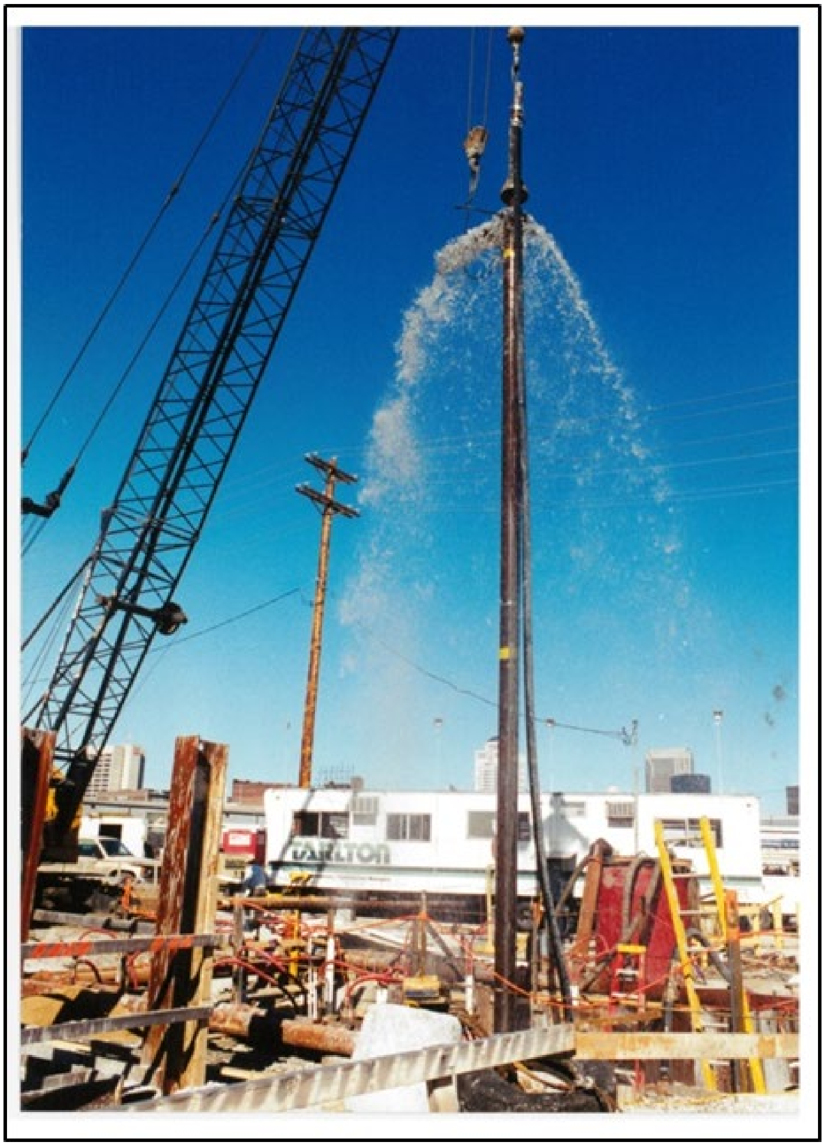


Figure 67. Installing well using 4-inch diameter hole puncher with 10-inch diameter sanding casing about 60 feet long (Courtesy of AECOM)

6.2.3.6 Sonic Drilling. Sonic drilling can currently be used to install casings up to about 12 inches in diameter and at least 100 feet deep. Therefore, the maximum diameter well that can be installed using sonic drilling with a filter pack is about 8 inches, provided a uniform filter sand is used that does not require placement by tremie. The sonic method is effective in penetrating cobbles and produces efficient wells. An inner core barrel between 3 and 8 inches in diameter is advanced 10 to 20 feet below an outer casing up to 12 inches in diameter. Then the outer casing is advanced to the inner barrel depth before the inner barrel is retrieved to extrude the core sample. The stepwise procedure continues until the hole is advance to final depth, which can be as much as several hundred feet, depending on the rig capability. A great advantage of the process is that a continuous sample is recovered for every well. If the formation is dense (dilative), the sample will be longer than the length cored, and vice versa of the formation is loose (contractive).

6.2.3.7 Foremost Dual Rotary Drilling. This method, formerly known as the Barber drilling method in North America, has been used for mineral exploration, water wells, and construction since 1940. It consists of a top head rotary drill combined with an independent lower rotary to turn and advance an outer casing up to 40 inches in diameter. The rig can be configured for flooded reverse circulation, which is appropriate and preferred for drilling holes larger than 12 inches in diameter for relief wells at dams and levees. The Foremost model DR24 is designed for 24-inch diameter outer casing with maximum torque of over 80,000 ft-lbs, and the heavy-duty model DR24HD has a casing drive torque capacity of over 200,000 ft-lbs, plus greater hoisting and pulldown capacity. The outer casing is fitted with a shoe with carbide cutters and can be advanced ahead of the inner drill string. This drill has proven to be effective in constructing wells in boulder-infested formations. Figure 68 shows a photograph of the bottom of 12-inch diameter outer casing shoe that was used with a Foremost model DR24 at Waterbury Dam to drill through sand-and-gravel shell and schist bedrock.

Figure 68. Carbide-studded shoe for 12-inch diameter dual rotary casing (Courtesy of AECOM)

6.2.3.8 Casing Driven and Extracted Using Pile Hammer. This method can be used to install almost any diameter of temporary casing in sand and gravel formations that are not heavily infested with cobbles and boulders. Vibratory, sonic, or hydraulic pile hammers are readily available that are suitable to drive steel casings of up to 6 feet in diameter. The method was successfully used in the late 1990s at Lock and Dam 25 on the Mississippi River north of St. Louis to install 24, 16-inch diameter pressure relief wells in 30-inch diameter holes extending into valley deposits to about 95 feet below the upper pool level (and about 60 to 70 feet below the lock floor) using a barge-mounted crane inside the lock chamber. In 1997, the U.S. Army Corps of Engineers (USACE) St. Louis District had tried but had not been able to unwater the lock since its original construction in the 1930s. The lock floor was cored using a 38-inch diameter diamond core barrel. Although cobbles were noted in one of the test borings, there were no problems in driving the 30-inch diameter by 0.500 inches wall steel casing at any of the 24 locations. A large (minimum eccentric moment 8,000 inch-pounds) vibratory hammer was used to drive and extract 30-inch diameter steel casings (see Figure 69). The casings were cleaned out by jetting as necessary and air-lifting, followed by installation of the well screen and riser pipe and tremie placement of select filter sand backfill.

Figure 69. Driving 30-inch diameter steel casing inside lock chamber with vibratory pile hammer for installation of 16-inch diameter pressure relief wells at Lock and Dam 25, Mississippi River, Winfield, MO (Courtesy of AECOM, St Louis, MO, 1998)

6.2.3.9 Casing Rotators and Oscillators. Casing rotators and oscillators are high torque (typically 2 million ft-lbs or more) rotary rigs, skid-mounted, and are used to install casing equipped with carbide cutting teeth for deep foundations in strata that are very difficult to drill, including cobbles, boulders, bedrock, and man-made obstructions. Figure 70 shows two permanent 12-inch diameter drainage wells being lowered into a 6.5-foot diameter steel casing installed at Waterbury Dam using a Nippon Sharyo RT-300 casing rotator through a sand-and-gravel downstream shell and underlying sharply inclined slabs of competent schist bedrock overlying or within a narrow gorge filled with silt, sand and gravel extending to a total depth of about 170 feet below the drilling level. Select filter sand was placed by tremie around the well screens, followed by sand backfill and grout to the top of the drilling platform. The temporary casing was then removed. The RT300 casing rotator has a continuous torque capacity of about 2 million ft-lbs.

Figure 70. Setting two 12-inch drainage wells into 6.5-foot diameter steel casing installed with Nippon Sharyo SuperTop™ casing rotator at Waterbury Dam, Vermont (October 2004, photograph by Baltimore District, US Army Corps of Engineers)

6.2.4 Well Development, Seal, Test Pumping, Level Controls, and Impeller Settings.

6.2.4.1 After the filter is placed, the well should be developed to obtain the maximum yield and efficiency of the well. The purpose of the development is to remove any film of silt from the walls of the hole and to develop the filter immediately adjacent to the screen to permit an easy flow of water into the well. Development of a well should be as soon as practicable after the filter has been placed. Delay in doing this may prevent a well from being developed to the efficiency assumed in design. A well may be developed by surge pumping or surging it with a loosely fitting surge block that is raised and lowered through the well screen at a speed of about 2 feet/sec. The surge block should be slightly flexible and have a diameter 1 to 2 inches smaller than the inside diameter of the well screen. The amount of material deposited in the bottom of the well should be determined after each cycle (about 15 trips per cycle). Surging should continue until the accumulation of material pulled through the well screen in any one cycle becomes less than about 0.2 feet deep. The well screen should be bailed clean if the accumulation of material in the bottom of the screen becomes more than 1 to 2 feet at any time during surging and re-cleaned after surging is completed. Material bailed from a well should be inspected to see if any foundation silt or sand is being removed. It is possible to over-surge a well, which may breach the filter with resulting infiltration of foundation sand when the well is pumped.

6.2.4.2 After a well has been developed, it should be pumped to clear it of muddy water and sand, and to check it for yield and infiltration. The well should be pumped at approximately the design discharge for 30 minutes to several hours, with periodic measurement of the well flow, drawdown in the well, depth of sand in the bottom of the well, and amount of sand in the discharge. Measurements of well discharge and drawdown may be used to determine the specific capacity of the well, as further discussed in Appendix C. The performance of the well filter may be evaluated by measuring the accumulation of sand in the bottom of the well and in the discharge. A well should be developed and pumped until the amount of sand infiltration is less than 5 to 10 ppm. A Rossum centrifugal sand tester is a convenient method for accurately measuring the sand content in the discharge.

6.2.4.3 Deep wells in which a vacuum is to be maintained require an airtight seal around the well riser pipe from the ground surface to a sufficient depth to limit air flow. The seal may be made with compacted clay, nonshrink grout or concrete, bentonitic mud, or a short length of surface casing capped at the top. Improper or careless placement of this seal will make it nearly impossible to attain a sufficient vacuum in the system, which is required for the dewatering system to operate as designed. The top of the well must also be sealed airtight.

6.2.4.4 After the wells are developed and satisfactorily tested by pumping, the pumps, power units, and discharge piping may be installed.

6.2.4.5 Where drawdown or vacuum requirements in deep wells require that the water level be lowered and maintained near the bottom of the wells, and the pumps cannot be throttled to match the flow or be replaced with smaller pumps, the pumps should be controlled to cycle on and off automatically between two setpoints. It may be necessary to install the wells deeper and increase the well diameter in order to achieve adequate drawdown and at the same time prevent pump starts exceeding the motor manufacturer's recommended start frequency criteria.

6.2.4.6 The impellers of open-impeller deep-well turbine pumps should be set according to the manufacturer's recommendations. Closed-impeller pumps do not require adjustment. Improper impeller settings can significantly reduce the performance of an open-impeller pump.

6.3 Wellpoint Systems.

6.3.1 Wellpoint systems are installed by first installing the wellpoints and then the header at the location and elevation required by the design. After the header pipe is laid, the swing connection should be connected to the header at the spacing called for by the design. Installation of the wellpoints generally follows the alignment of the header pipe.

6.3.2 Self-jetting wellpoints are installed by jetting them into the ground by forcing water out the tip of the wellpoint under high pressure. The jetting action of a typical self-jetting wellpoint is illustrated in Figure 71. Self-jetting wellpoints can be installed in medium to fine sands with water pressures of about 50 pounds per square inch (psi). Wellpoints jetted into coarse sand and gravel require considerably more water and higher water pressures (about 125 psi) to carry out the heavier particles; either a hydrant or a jetting pump of appropriate size for the pressures and quantities of jetting water required can be used. The jetting hose, usually 2 to 3 inches in diameter, is attached to the wellpoint riser, which is picked up either by a crane or by hand and held in a vertical position as the jet water is turned on. The wellpoint is allowed to sink slowly into the ground and is slowly raised and lowered during sinking to ensure that all fine sand and dirt are washed out of the hole. Care should be taken to ensure that a return of jet water to the surface is maintained; otherwise, the point may "freeze" before it reaches grade. If the return of jet water disappears, the point should be quickly raised until circulation is restored and then slowly re-lowered. In gravelly soils, it may be necessary to supplement the jet water with a separate air supply at about 125 psi to lift the gravel to the surface. When the wellpoint reaches grade and before the water is turned off, the swing connection, if used, should be lined up for easy connection when the jet water is turned off and the jetting hose disconnected.

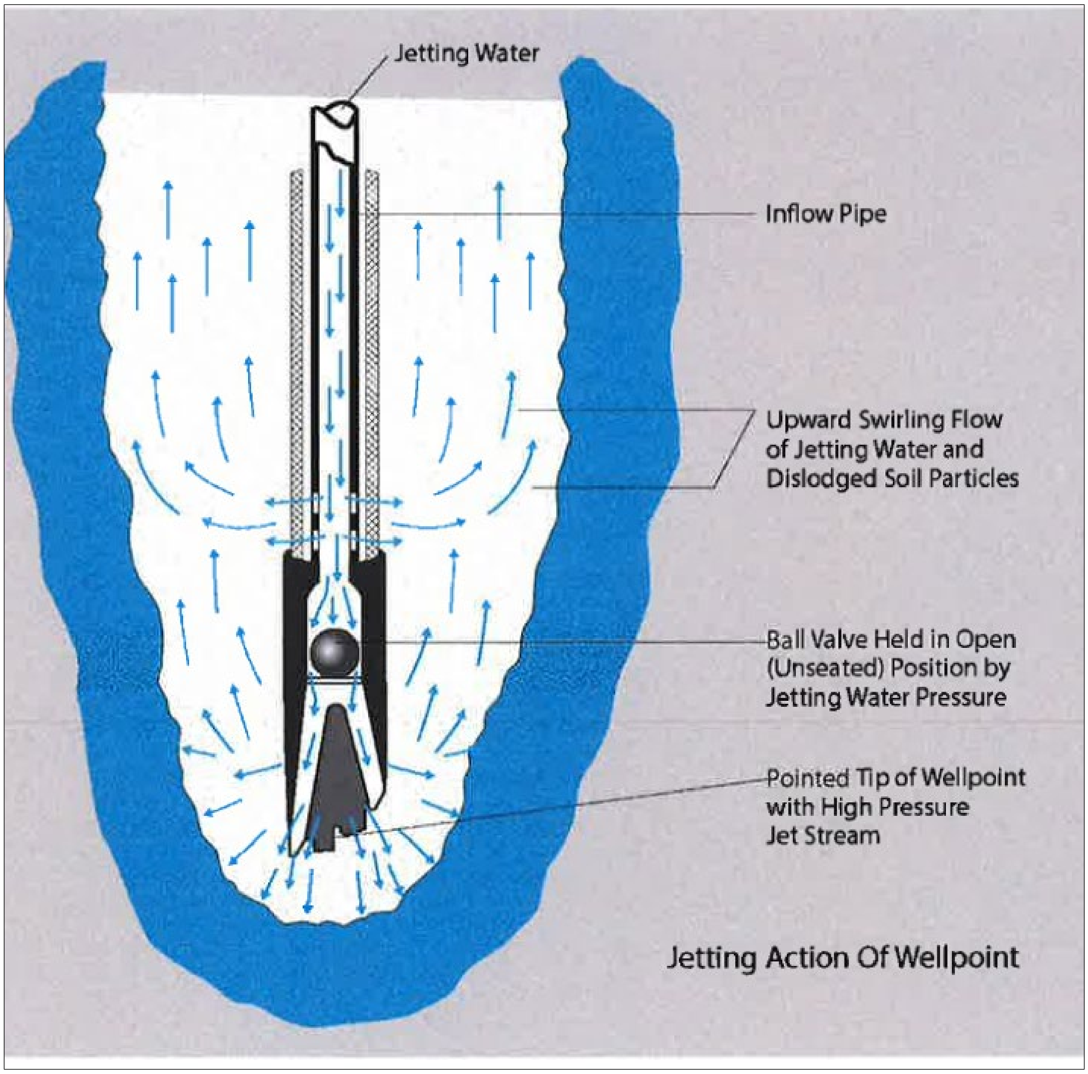


Figure 71. Self-jetting wellpoint (Courtesy Griffin Dewatering, LLC.)

6.3.3 Hydraulic fracturing is possible with this method and use of this method should be carefully considered when working in an area, such as near a dam or levee embankment, where hydraulic fracturing cannot be tolerated.

6.3.4 If filter sand is required around the wellpoint to increase its efficiency or prevent infiltration of foundation soils, the wellpoints generally should be installed using a hole puncher and a sanding casing to form the hole for the wellpoint and filter. In this case, the wellpoint is typically installed in a hole formed by jetting down a 10- to 12-inch heavy steel casing. The casing may be fitted with a removable cap at the top through which air and water may be introduced. The casing is jetted into the ground with a return of air and water along the outside of the casing. Jetting pressures of 125 pounds per square inch are commonly used; where resistant strata are encountered, the casing may have to be raised and dropped with a crane to chop through and penetrate to the required depth. A casing may also be installed using a combination jetting and driving tool, equipped with both water and air lines, which fits inside the casing and extends to the bottom of the casing. Most of the return water from a ‘hole puncher” rises inside the casing, causing considerably less disturbance of the adjacent foundation soils. After the casing is installed to a depth of 1 to 3 feet greater than the length of the assembled wellpoint, the jetting is continued until the casing is flushed clean with clear water. The wellpoint is placed in the casing, the sand filter poured inside the casing, and the casing pulled. Care should be taken to center the wellpoint in the casing so that it is completely surrounded with filter material. Before the wellpoint is connected to the header, it should be pumped to flush it and the filter and to check it for “sanding.” Wellpoints should be developed as discussed for deep-well systems if the wellpoint has a widely graded filter. All joints connecting wellpoints to the header should be made airtight to obtain the maximum vacuum.

6.3.5 Wellpoint pumps are used to provide the vacuum and to remove water flowing to the system. To obtain the maximum possible vacuum, the suction intake of the pump should be set level with the header pipe. Wellpoint pumps should be protected from the weather by a shelter and from surface water or sloughing slopes by ditches and dikes. The discharge pipe should be watertight and supported independently of the pump.

6.3.6 Vacuum wellpoint systems are installed in the same manner as ordinary wellpoint systems using a jetting casing and filter, except the upper 5 feet of the riser is sealed airtight to maintain the vacuum in the filter.

6.3.7 Jet-eductor wellpoints are usually installed using a hole puncher and surrounding the wellpoint and riser pipes with filter sand. Jet eductors are connected to two headers; one for pressure to the eductors and the other for return flow from the eductors and the wellpoints back to the recirculation tank and pressure pump.

6.4 Vertical Drains.

6.4.1 Vertical Sand Drains. Vertical sand drains can be installed by jetting a 12- to 18-inch diameter casing into the soil to be drained; thoroughly flushing the casing with clear water; filling it with clean, properly graded filter sand; and pulling the casing similar to the procedure for installing “sanded” wellpoints. Sand drains should penetrate into the underlying pervious aquifer to be drained by means of wells or wellpoints. A small diameter slotted pipe may be installed to increase the vertical capacity of the sand drain, which is limited by the vertical hydraulic conductivity of the sand filter used.

6.4.2 Prefabricated Vertical Drains. See previous discussion of the installation of prefabricated vertical drains in Chapter 3.

6.5 Piezometers. Piezometers are installed to determine the elevation of the groundwater table in an unconfined aquifer or the piezometric level in a confined aquifer for designing and evaluating the performance of a dewatering system. For most dewatering applications, commercial wells or small screens are satisfactory as piezometers. Refer to EM 1110-2-1908 for piezometer types, methods of installation, and development and testing. If drilling methods with drilling fluids are used to install piezometers, hydraulic fracturing is possible, which requires careful evaluation when working near a dam or levee embankment, where hydraulic fracturing cannot be tolerated.

Chapter 7
Operation and Performance Control.

7.1 General. The success of a dewatering operation hinges on the proper operation, maintenance, and control of the system. If the system is not operated and maintained properly, its effectiveness may be greatly diminished. After a dewatering or pressure relief system has been installed, a full-scale pumping test should be made, and its performance evaluated for adequacy or need for any modification of the system. This test and analysis should include initial and subsequent measurements of the water levels in piezometers, pump discharge, water level in excavation, water levels in wells or the vacuum in the header system, and a comparison of the data with the original design.

7.2 Operation and Maintenance.

7.2.1 Wellpoint and Jet-eductor Systems.

7.2.1.1 The proper performance of a wellpoint system requires continuous maintenance of a steady, high vacuum. After the system is installed, the header line and all joints should be tested for leaks by closing all swing-joint and pump suction valves, filling the header with water under a pressure of 10 to 15 psi, and checking the line for leaks. The next step is to start the wellpoint pump with the pump suction valve closed. The vacuum should rise to a steady 25 to 27 inches of mercury. If the vacuum at the pump is less than this value, there are air leaks or worn parts in the pump itself. If the vacuum at the pump is satisfactory, the gate valve on the suction side of the pump may be opened and the vacuum applied to the header, with the wellpoint swing-joint valves still closed. If the pump creates a steady vacuum of 25 inches or more in the line, the header line may be considered tight. If the vacuum at the pump is less than this value, there are air leaks in the header line. The leaking header line should be inspected to determine the locations of air leaks, and the leaks should be repaired. The swing-joint valves are then opened one by one and the vacuum is applied to the wellpoints. If a low, unsteady vacuum develops, leaks may be present in the wellpoint riser pipes, or the water table has been lowered to the screen in some wellpoints so that air is entering the system through one or more wellpoint screens. One method of eliminating air entering the system through the wellpoints is to use a riser pipe 25 feet or more in length. If the soil formation requires the use of a shorter riser pipe, entry of air into the system can be prevented by partially closing the main valve between the pump and the header or by adjusting the valves in the swing connections until air entering the system is stopped. This method is commonly used for controlling air entry and is known as "tuning."

7.2.1.2 A wellpoint leaking air will frequently cause an audible throbbing or bumping in the swing-joint connection, which may be felt by placing the hand on the swing joint. The throbbing or bumping is caused by intermittent charges of water hitting the elbow at the top of the riser pipe. In warm weather, wellpoints that are functioning properly feel cool and will sweat due to condensation in a humid atmosphere. A wellpoint that is not sweating or that feels warm may be drawing air through the ground, or it may be clogged and not functioning. In very cold weather, properly functioning wellpoints will feel warm to the touch of the hand compared with the temperature of the atmosphere. Wellpoints that are disconnected from the header pipe can admit air to the aquifer and may affect adjacent wellpoints. Disconnected wellpoints with riser pipes shorter than 25 feet should be capped.

7.2.1.3 Wellpoint headers, swing connections, and riser pipes should be protected from damage by construction equipment. Access roads should cross header lines with bridges over the header to prevent damage to the headers or riser connections and to provide access for tuning and operating the system.

7.2.2 Deep Wells and Vacuum Wells. Optimum performance of a deep-well system requires continuous uninterrupted operation of all wells. If the pumps produce excessive drawdowns in the wells, it is usually preferable to regulate the flow from all of the wells to match the flow to the system, rather than reduce the number of pumps operating and thus create uneven drawdown in the dewatered area. The discharge of the wells may be regulated by varying the pump speed (if other than electric power is used) or by varying the discharge pressure head by means of a throttling valve installed in the discharge lines. Uncontrolled discharge of the wells may cause the pumps to break suction, with undesirable surging and uneven performance of the pumps.

7.2.3 Pumps. Pumps, motors, and engines should always be operated and maintained according to the manufacturer’s directions. All equipment should be maintained in first-class operating condition at all times. Standby pumps and power units in good operating condition should be provided for the system. Standby equipment may be required to operate during breakdown of a pumping unit or during periods of routine maintenance and oil changes of the regular dewatering equipment. All standby equipment should be periodically operated to ensure that it is ready to function in event of a breakdown of the regular equipment. Automatic starters, clutches, and valves may be included in the standby system if the dewatering requirements so dictate. Signal lights or warning buzzers may be desirable to indicate, respectively, the operation or breakdown of a pumping unit. If control of the groundwater is critical to safety of the excavation and the safety of the general public or the integrity of the foundation, sufficient operating and maintenance personnel should be on duty at all times. Where gravity flow conditions exist that allow the water table to be lowered an appreciable amount below the bottom of the excavation and the recovery of the water table is slow, the system may be pumped only part time, but this procedure is rarely possible or desirable. Such an operating procedure should not be attempted without first carefully observing the rate of rise of the groundwater table at critical locations in the excavations and analyzing the data with regard to existing soil formations and the status of the excavation. The capability (labor, supervision, spare system components, and equipment) to be maintained onsite during non-working hours depends on the criticality of the dewatering system and should be carefully considered when developing the specifications.

7.2.4 Maintenance and Rehabilitation of Wells and Wellpoints Pumping Encrusting Groundwater.

7.2.4.1 The efficiency of dewatering wells and wellpoints will deteriorate with increasing time of pumping because of blockage of well screens and filters due to mineral precipitation and/or bacterial action. Well efficiency should be monitored during operation so that timely corrective action can be taken. Falling head tests are useful to evaluate deterioration of efficiency in wellpoints. It is useful to suspend screen coupons similar to that shown in Figure 72 in selected wells or wellpoints and retrieve the coupons from time to time during system operation for visual and possibly microscopic examination, especially on projects where operation of a dewatering system is expected to be more than a few months or where the groundwater chemistry and microbiology has been tested and is expected to be problematic for encrustation. The advantage of deploying coupons is that they are surrogates for the well screens and can be easily examined at the surface and even sent to a laboratory for microscopic examination, whereas the only way to visually inspect a well screen is to remove the pump and insert a special downhole camera. If well screens are more than a few feet long, multiple coupons should be deployed at various depths below the pumping water level.

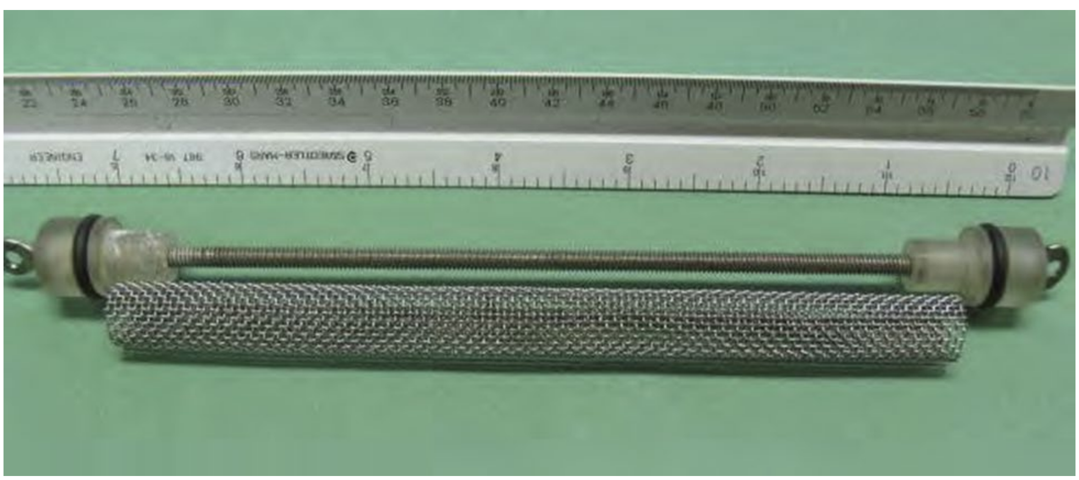


Figure 72. Stainless steel coupon (Courtesy of AECOM)

7.2.4.2 Following are discussion and recommendations developed by Water Systems Engineering (Schnieders and Wiseman 2008) for evaluating monitoring coupons installed in several permanent drainage wells installed at Waterbury Dam in Vermont:

7.2.4.2.1 “As part of the monitoring program, ongoing analysis of the existing coupons to monitor for deposit control, corrosion rates, and biofouling tendencies is recommended. Corrosion coupons are utilized to both monitor and interpret events within the system. While use of these coupons is subjective, they can be very useful in the interpretation of encrustation and corrosion processes, serve as excellent monitors of system operation, and are relatively easy to use and interpret. For these wells, the potential of plugging in the filter packs is of great concern, so the coupons will not only be representative of the activity in the well, but also of the filter pack surrounding the wells. They will work in tandem with the periodic slug testing proposed to monitor the fouling condition of the combined filter packs.

7.2.4.2.2 The string of coupons should be removed regularly, quarterly at the minimum, and examined using a basic microscope or good magnifying glass. The suspension wire and coupon attachments should be carefully inspected for corrosion, and any weak areas replaced so that the coupons are not lost down the well. The coupons themselves should be observed for deposit formation, corrosion activity, and slime production, utilizing the provided Monitoring Coupon Comparison Chart (Figure 73).

7.2.4.2.3 After examination, the coupon strand should be lowered back into the well from which it was pulled so the coupons may continue to develop and represent the fouling of the well screens and gravel pack. While the coupon strand is out of the well, care should be taken to not contaminate the coupons so that impurities are not introduced into the wells.

7.2.4.2.4 When the wells are cleaned, the coupons should be removed and sent to a laboratory for evaluation. After the wells have been cleaned and disinfected, new coupons should be placed in the wells in the same positions as the previous coupon sets for continued monitoring purposes.”

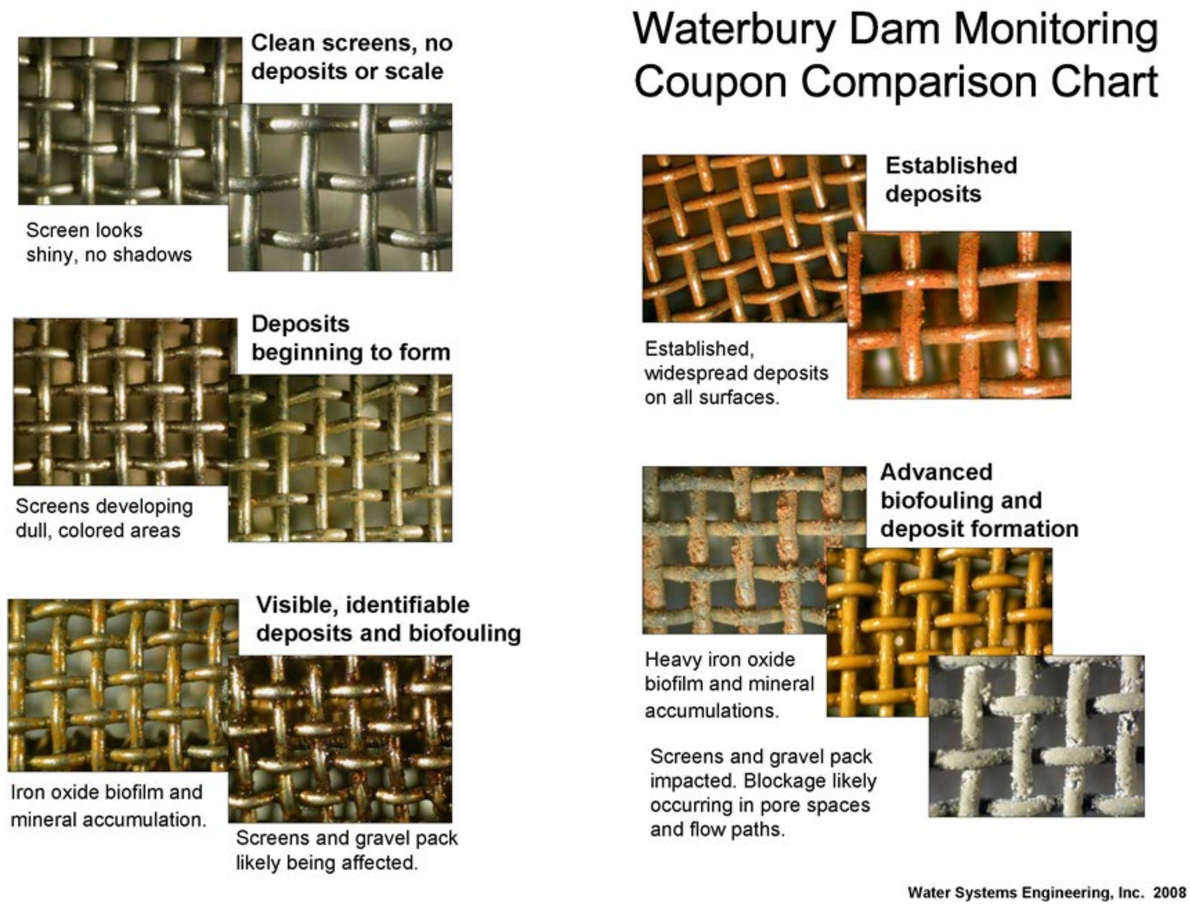


Figure 73. Well coupon comparison chart (Water Systems Engineering from URS 2010)

7.2.4.3 Good references on well rehabilitation include Powers et al. (2007), Schnieders (2003), Roscoe Moss Company (1990), and Sterrett and Schnieders (2007). Also see EM 1110-2-1914 (Relief Wells).

7.2.5 Surface Water Control. Ditches, dikes, sumps, and pumps for the control of surface water and the protection of dewatering pumps and controls should be maintained throughout construction of the project. Maintenance of ditches and sumps is of particular importance. Silting of ditches may cause overtopping of dikes and serious erosion of slopes that may clog the sumps and sump pumps. Failure of sump pumps may result in flooding of the dewatering equipment and complete breakdown of the system. Dikes around the top of an excavation to prevent the entry of surface water should be maintained to their design section and grade at all times. Any breaks in slope protection should be promptly repaired.

7.3 Control and Evaluation of Performance.

7.3.1 Pump Test. After a dewatering or groundwater control system is installed, it should be pump-tested to check its performance and adequacy. This test should include measurement of initial groundwater table or piezometric level, drawdown at critical locations in the excavation, flow from the system, elevation of the water level in the wells or vacuum at various points in the header, and distance to the “effective” source of seepage, if possible. These data should be analyzed, and if conditions at the time of test are different than those for which the system was designed, revisions and adjustments should be made as appropriate. It is important to evaluate the system as early as possible to determine its adequacy to meet full design requirements. Testing a dewatering system and monitoring its performance require the installation of piezometers and measuring the flow from the system or individual wells. Pressure and vacuum gages should also be installed at the pumps and in the header lines. For multistage wellpoint systems, the installation and operation of the first stage of wellpoints may offer an opportunity to check the hydraulic conductivity of the pervious strata, radius of influence or distance to the source of seepage, and the head losses in the wellpoint system. Thus, from observations of the drawdown and discharge of the first stage of wellpoints, the adequacy of the design for lower stages may be checked to some degree.

7.3.2 Critical Installations. For critical (high risk) installations the system should be tested and approved by registered professional engineers (the designer and the Project Construction Engineer) prior to excavation. Testing and approval of the backup system should also be made by the engineer/owner prior to excavation.

7.3.3 Piezometers.[7]

7.3.3.1 The locations of piezometers should be selected to produce a reasonably complete and reliable picture of the drawdown produced by the dewatering system. Examples of types of piezometers and methods of installation are given in EM 1110-2-1908. Piezometers should be located so they will clearly indicate whether water levels required by specifications are attained at significant locations. The number of piezometers depends on the size and configuration of the excavation and the dewatering system. When there is one pervious formation to be dewatered, piezometers should be installed at a minimum in the four corners and at the center of the excavation. If the pervious strata are stratified and artesian pressure exists beneath the excavation, piezometers should be located in each significant stratum. Piezometers should be

[7] Automation of instrumentation and alarming (including autodialing and SMS texting) should be considered for critical installations (i.e., when there is limited time to restore system performance before failure will occur).See EM 1110-2-1908 (Instrumentation of Embankment Dams and Levees) for design of automated instrumentation.

installed at the edge of and outside of the excavation area to determine the shape of the drawdown curve to the dewatering system and the effective source of seepage to be used in evaluating the adequacy of the system. Piezometers should be installed as close as practical to the point where drawdown is considered critical and as far as practical from the suction well. If recharge of the aquifer near the dewatering system or a cutoff is required to prevent settlement of adjacent structures, control piezometers should be installed in these areas. Where the groundwater is likely to cause incrustation of well screens, piezometers may be installed close to each well (at a radius of 5 feet, for example) and inside the well screen to monitor the head loss across this zone as time progresses. In this way, if a significant increase in head loss is noted, cleaning and reconditioning of the screens and chemical treatment of the filter should be undertaken to improve the efficiency of the system. Provisions for measuring the drawdown in the wells or at the line of wellpoints are desirable for operations and monitoring. As construction progresses, piezometers should be considered disposable and replaceable and should be abandoned and replaced as necessary.

7.3.3.2 Piezometer data should be collected and evaluated to compare the drawn down groundwater surface to the excavation bottom elevation to ensure the groundwater surface is meeting the specified requirements. This comparison should be performed on a frequency based on the risk of failure of the system which may even be a continuous comparison if an automated monitoring system is installed for a critical structure. This should be done when additional capacity is added to the dewatering system, or when operational adjustments are made to the dewatering system, or when high water rises against the dewatering system. This comparison could include developing contour maps of the groundwater surface and comparing to contours of the excavation.

7.3.4 Flow Measurements.[8] Measurement of flow from a dewatering system is necessary to evaluate the performance of the system relative to design predictions. Flow measurements are also useful in recognizing any loss in efficiency of the system due to incrustation or clogging of the wellpoints, well screens and filters. Appendix D describes several methods by which flow measurements can be made. Data loggers or chart recorders are useful for automatic recording of individual well and total system flows during operation of a dewatering system.

7.3.5 Operational Records, Automation of Instruments and Alarms/Notifications.

7.3.5.1 Operational Records and Minimum Reading Frequency. Piezometers located within the excavated area should be observed at least once a day, or more frequently, if the situation demands (such as unexpected condition or performance or poor contractor performance), to

[8] See previous footnote regarding automation and alarming of critical installations.

ensure that the required drawdown is being maintained. Vacuum gages and tachometers on pumps and engines should be checked at least every few hours by the operators as they make their rounds. Piezometers located outside the excavated area, and discharge of the system, may be observed less frequently after the initial pumping test of the completed system. Piezometer readings, flow measurements, stages of nearby streams or the elevation of the surrounding groundwater, and the number of wells or wellpoints operating should be recorded and plotted throughout the operation of the dewatering system. The data on the performance of the dewatering system should be continually evaluated to detect any irregular functioning or loss of efficiency of the dewatering system before the construction operations are impeded, or the excavation or foundation is damaged.

7.3.5.2 Automation of Instruments. Automation of instruments during operation of a dewatering system should be considered (1) when instruments will be repeatedly read over a long period of time, or (2) when the project requires monitoring at a relatively high frequency for the safety of critical excavations. In the first case automation could reduce the cost of monitoring and data reduction, and in the second case automation can reduce risk of missing developing problems that could compromise safety. Refer to EM 1110-2-1908 for details on instrumentation automation.

7.3.5.3 Alarms and Auto-Messaging. Technological advances have been made on hardware and software for alerting and notifying appropriate personnel when there are problems with the operation of a dewatering system on a project. The simplest alarms are status lights on pump controls, which enable the system operators to ascertain visually whether a pump is operational or not. Alarm systems can be designed and installed to detect high or low piezometric levels, pump outages, and power outages, and to send emails, prerecorded voice messages, Short Message Service (SMS) text (a text messaging service component of phone, web, or mobile communication systems), flash SMS text, and Multimedia Messaging Service (MMS) text (messages including image, video and sound content) to the appropriate project personnel when triggered by the programming. An example of this type of alarm system is included in the Isabella Dam specification in Appendix F. Simpler, less technical alerts can also be devised, such as installing a float in a standpipe piezometer within an excavation and cutting off the riser pipe near the bottom of an excavation. When the piezometric level rises, the float will rise above the top of the riser pipe, signaling workmen and supervisors visually that the piezometric level is rising and action may need to be taken to correct a problem or to evacuate people and equipment from the excavation.

7.4 Removal of System and Abandonment of Wells, Wellpoints and Piezometers. Following the completion of dewatering activities, all dewatering equipment should be removed. Wells can be removed, but in thick aquifers extending to the ground surface with no aquitard's wells may be backfilled with sand and cut off a few feet below the ground surface. If positive seals are required for abandonment of wells, the criteria for such seals need to be completely specified. If there are aquitards within or between aquifers penetrated by wells, the wells should be designed and installed with annular seals through such aquitards and the wells abandoned by filling them with cement grout or bentonite pellets. Wellpoints are normally removed, and the soil is allowed to collapse naturally. If abandonment of wellpoints by this method is not acceptable, the specification should define exactly how wellpoints are to be abandoned. Piezometers should be left in-place long enough after the dewatering system has been removed to understand the groundwater conditions to evaluate potential impacts on removal of the dewatering system (if the groundwater rebounded as expected, any negative impacts on the project from the rebound, etc.). Once the purpose of the piezometers is met, the piezometers should be abandoned unless they are required as part of monitoring a potential failure mode in a long term (post construction) capacity.

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Chapter 8
Dewatering System Designed By Owner/Engineer or Contractor

8.1 General. Dewatering systems designed by either the contractor or the owner/engineer have been successful. The engineer has typically studied the site for a significant period of time prior to bid and likely understands the site conditions and their relationship to dewatering better than the contractor can be expected to understand in the short time allocated for bid preparation. Therefore, it is incumbent on the engineer to impart his or her knowledge to the greatest degree practical on the design and requirements of the dewatering system regardless of whether an owner/engineer or contractor-designed system is used.

8.2 Circumstances for Owner/Engineer Design. There are specific circumstances when it may be prudent for the owner/engineer to design the dewatering system. Factors and possible scenarios that could lead an owner/engineer to take on the responsibility of designing the dewatering system include:

a. The performance of the dewatering system is critical to public safety.

b. The performance of the dewatering system is critical to the project schedule, assuming there is no time for extensive trial and error of the dewatering system during construction.

c. Subsurface conditions are complex, and bidders may be tempted to omit dewatering of a deeper pervious layer that requires pressure relief for bottom stability of the excavation.

d. The successful prime contractor bids the project using an inexperienced dewatering subcontractor that provides an inadequate design, and the prime contractor is reluctant to increase the scope of dewatering beyond that included in the subcontractor's bid, or the prime selects an overly conservative and expensive system to reduce risk.

e. The successful prime contractor does not receive any bids from experienced dewatering subcontractors and the prime contractor estimates dewatering costs using their own opinion of the required dewatering scope.

f. An engineer/owner designed system provides a basis to bid for prime contractors and, therefore, removes most of the uncertainty from this bid item, of which the costs are typically highly variable, thus potentially reducing claims.

8.3 Owner/Engineer Design. If the owner/engineer assumes the responsibility of designing the dewatering system, the design should be performed by a registered professional engineer with extensive experience in the design, installation, and performance of dewatering sites of similar site conditions. An owner/engineer designed system must include all the critical details of the system and not leave these details to the contractor. In addition, the owner must be willing to authorize additional dewatering efforts quickly due to potential risks associated with the safety of the excavation, and to maintain project schedules.

8.4 Contractor Design. If the dewatering system will be contractor-designed, the specifications should be as explicit as possible regarding dewatering requirements. The specifications should also include:

a. The experience requirements for the engineer designing the dewatering system.

b. The requirement that no seepage flow, either vertical, horizontal, or from any angle enters the excavation.

c. The maximum flood level for which the dewatering system will be designed.

d. A requirement for contractor developed emergency action plans should a larger flood occur.

e. A requirement to collect and control precipitation falling into the excavation.

f. Recommendations for contractor-designed specifications, as well as owner/engineer designed specifications are discussed in Chapter 9.

Chapter 9
Dewatering Specifications

9.1 General. Good specifications are essential to ensure adequate dewatering and groundwater control. Specifications must be clear, concise, and complete with respect to the desired results, special conditions, inspection and control, payment, and responsibility. The extent to which specifications should specify procedures and methods is largely dependent on public safety, the complexity and magnitude of the dewatering problem, criticality of the dewatering with respect to schedule and damage to the work, and the experience of the probable bidders. Regardless of the type of specification selected, the dewatering system(s) should be designed, installed, operated, and monitored according to the principles and criteria set forth in this manual.

9.2 Types of Specifications. Dewatering specifications can be divided by the type of system required: non-critical and critical. A non-critical system is one that does not involve unusual or complex features and failure or inadequacy of the system would not adversely affect worker safety, safety of the general public, the schedule, performance of the work, foundation for the structure, or the completed work. A critical system is complex and one whose failure could have significant impacts on worker safety, safety of the general public, the schedule, and could significantly damage the foundation, adjacent structures, or the completed work.

9.2.1 Contractor-Designed.

9.2.1.1 This type of specification requires the contractor to assume full responsibility for the design, installation, operation, and maintenance.

9.2.1.2 A non-critical system specification is used to specify basic dewatering methods that a general contractor could install and maintain (sumps/pumps and ditches). This specification requires a “minimum” system that will ensure an adequate degree of dewatering to construct the work in the dry but not specify the dewatering methods or contractor qualifications.

9.2.1.3 A critical system specification is used to specify dewatering methods that require a specialty contractor to install and maintain complex systems (wells, wellpoints, etc.). These systems must be designed by a registered professional engineer with significant dewatering experience. Qualification requirements of the specialty dewatering contractor must also be included. This type of specification should not be used unless the owner (or owner’s engineer) employs or contracts with a registered professional engineer recognized as an expert in dewatering with at least 10 years of responsible experience in the design and installation of critical dewatering systems. This engineer should prepare an independent check of the specialty contractor’s design, review the dewatering submittal, and be involved in reviewing proposed modifications and performance data during construction.

9.2.2 Owner/Engineer Designed.

9.2.2.1 This type of specification requires the owner to assume full responsibility for the design, and for the requirements of the system but specifies that the contractor is responsible for the installation, operation, and maintenance. Typically, owner/engineer designed systems only apply to critical systems since non-critical systems require little to no design.

9.2.2.2 A specification for a critical system sets forth in detail the design and installation of a system that has been designed to achieve the desired control of groundwater wherein the Owner/Engineer assumes full responsibility for the design and performance but requires the contractor to be responsible for routine maintenance and operations. Modifications to the design (adding wells, wellpoints, pumps, etc.) could be handled by including additional pay items as discussed below.

9.3 Data to be Included in the Specifications. All data obtained from field investigations relating to dewatering or control of groundwater made at the site of the project should be included with the specifications and drawings or appended thereto. This data should include logs of borings; soil profiles; results of laboratory tests including mechanical analyses, water content of silts and clays, and any chemical analyses of the groundwater; pumping tests; groundwater levels in each aquifer, if more than one, as measured by properly installed and tested piezometers, and its variation with the season or with river stages; and river stages and tides for previous years, if available. It is essential that all field or laboratory test data be included with the specifications, or referenced, and that the data be accurate. The availability, adequacy, and reliability of electric power, if known, should be included in the contract documents. The same is true for the disposal of water to be pumped from the dewatering systems. The location and ownership of water wells off the project site that might be affected by lowering the groundwater level should be shown on one of the contract drawings.

9.4 Dewatering Specification Requirements.

9.4.1 Contractor-Designed.

9.4.1.1 For non-critical system specifications, the desired results should require that all permanent work be accomplished in the dry and on a stable subgrade; and require the contractor to be responsible for designing, providing, installing, operating, monitoring, and removing the dewatering system by a plan approved by the owner/engineer. This type of specification should note the limitations of groundwater information furnished since seepage conditions may exist that were not discovered during the field exploration program. It should be made clear that the contractor is not relieved of responsibility of controlling and disposing of all water, even though the discharge of the dewatering system required to maintain satisfactory conditions in the excavation may be in excess of that indicated by tests or analyses performed by the owner/engineer. The method of payment should also be clearly specified.

9.4.1.2 Prior to the start of excavation, the contractor should be required to submit for review a dewatering plan that includes proposed method(s) for dewatering the excavation, disposing of the water, and removing the system, as well as a list of the equipment to be used.

9.4.1.3 Perimeter and diversion ditches and dikes should be required and maintained as necessary to prevent surface water from entering any excavation. The specifications should also provide for controlling the surface water that falls or flows into the excavation by adequate pumps and sumps. Seepage of any water from excavated slopes should be controlled to prevent sloughing, and ponding of water in the excavation should be prevented during construction operations. If the flow of water into an excavation becomes excessive and cannot be controlled by the dewatering system that the contractor has installed, excavation should be halted until satisfactory remedial measures have been taken.

9.4.1.4 For critical system specifications, all of the requirements of the non-critical system should be included with the following additional requirements. If wells, wellpoints, or other special measures are believed to be necessary, then these requirements should be clearly stated. Performance requirements should be clearly identified – e.g., piezometric heads must be lowered to at least 2 feet below the bottom of the excavation, as demonstrated with piezometers installed according to the approved dewatering plan. Prior to the start of excavation, the contractor should be required to submit a dewatering plan that is prepared and sealed by a dewatering specialist that includes:

a. Design calculations.

b. Detailed descriptions and characterization of the formations to be dewatered and groundwater conditions and characteristics at the site.

c. Drawings of the proposed dewatering system(s) including a plan drawing, appropriate sections, pump and pipe capacities and sizes, power system(s), standby power and pumps, grades, filter gradation, surface water control, and valving.

d. Types of proposed dewatering systems, including a list of all equipment and standby equipment for emergency use.

e. Proposed locations and elevations of piezometers and flow measuring devices and other monitoring devices to measure the performance of the system.

f. A plan and schedule for monitoring performance of the system(s).

g. Corrective measures to ensure performance.

h. Operations and monitoring schedules and a description of installation and operational procedures.

i. Methods of disposing of the water.

j. Proposed methods to abandon the dewatering system.

9.4.1.5 This plan should be detailed and adapted to site conditions and should provide for 24/7 dewatering operations.

9.4.1.6 These specifications should also require that the contractor employ or subcontract the dewatering and groundwater control to a recognized specialty contractor with at least 10 years of experience in the management, design, installation, and operation of critical dewatering systems. The specification should also state that the system(s) must be designed by a registered professional engineer recognized as an expert in dewatering with a minimum of 10 years of responsible experience in the design and installation of critical dewatering systems.

9.4.1.7 Any water encountered in an excavation for a shaft or tunnel must be controlled, before advancing the excavation, to prevent sloughing of the walls or “boils” in the bottom of the excavation or blow-in of the tunnel face. Dewatering of excavations for shafts, tunnels, and lagged open excavations should continue for the duration of the work to be performed in the excavations unless the tunnel or shaft has been securely lined and is safe from hydrostatic pressure and seepage.

9.4.2 Owner/Engineer Designed. These specifications should set forth not only the required results for dewatering, pressure relief, and surface water control, but also a detailed list of the materials, equipment, and procedures that are to be used in achieving the desired system(s). The Contractor will be responsible for operating and maintaining the system(s). The Contractor should also be advised that he or she is responsible for correcting any unanticipated seepage or pressure conditions and taking appropriate measures to control such, payment for which would depend upon the type of specification and terms of payment.

9.5 Measurement and Payment. Payment when using specifications for contractor-designed non-critical and critical systems is generally handled by a “Lump Sum” or “Job” payment. A unit price item could be included to separate operational and maintenance costs on a time basis (say monthly) in the event that a design change impacts the duration of dewatering. Payment when using owner/engineer designed system specifications is generally on the basis of various unit prices of such items as wells, pumps, and piping, in keeping with normal payment practices for specified work. Operations and maintenance generally should be set up as a unit price payment on a time basis (monthly). Major repairs most likely would need to be negotiated if at no fault of the contractor. Payment for monitoring piezometers and flow measuring devices is generally made according to the method of payment for the various types of dewatering specifications described above.

Examples of Dewatering Specifications. Dewatering specifications from three USACE projects that have either been constructed or are in the process of being constructed, are included in Appendix E. Very minor (editorial) changes were made to these specifications for publication purposes. The first two specifications are for contractor designed systems, and the last specification is for a USACE (owner) designed system. These specifications reference other specification sections that were not included in this ETL since the other specifications were considered to be beyond the scope of this ETL. These specifications were developed based on site-specific conditions and will need to be modified based on the conditions at other sites.

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Appendix A
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Appendix B
Field Pumping Test

B.1. General. There are two basic types of pumping tests: equilibrium (steady-state flow) and nonequilibrium (transient flow).

B.1.1 Equilibrium-type Test. When a well is pumped, the water discharged initially comes from aquifer storage adjacent to the well. As pumping continues, water is drawn from an expanding zone until a state of equilibrium has been established between well discharge and aquifer recharge. A state of equilibrium is reached when the zone of influence has become sufficiently enlarged such that: natural flow into the aquifer equals the pumping rate; a stream or lake is intercepted that will supply the well (Figure B.1); or vertical recharge from precipitation on the area above the zone of influence equals the pumping rate. If a well is pumped at a constant rate until the zone of drawdown has become stabilized, the hydraulic conductivity k of the aquifer can be computed from equilibrium formulas presented in Section B.3.

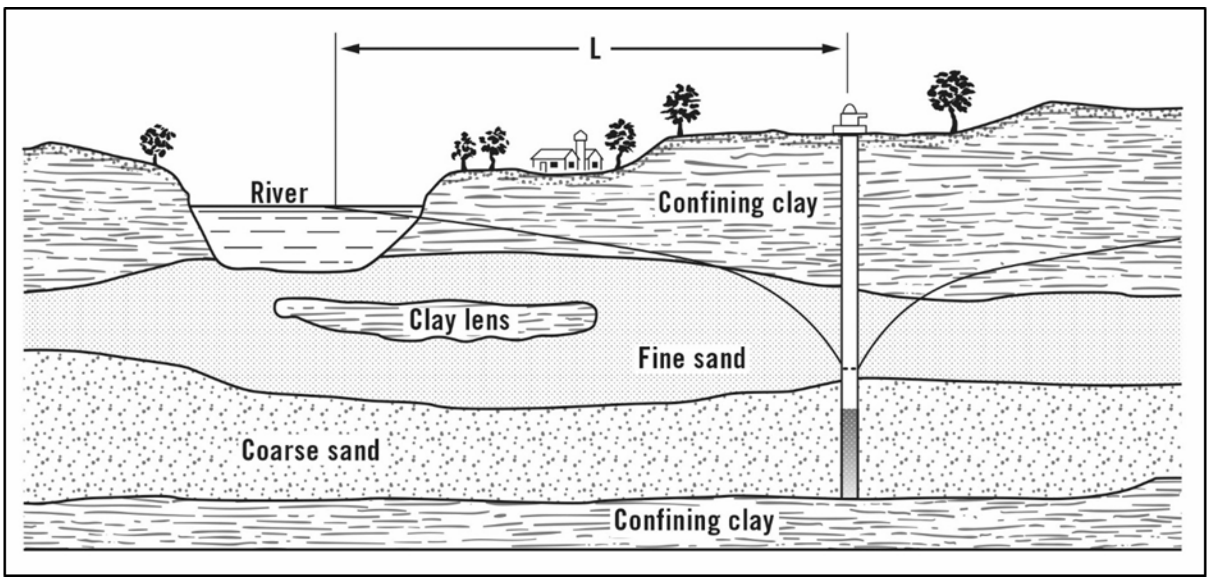


Figure B.1. Seepage into an aquifer from an adjacent river (Courtesy of the EPA)

B.1.2 Nonequilibrium-type Test.

B.1.2.1 In this type of test, the value of k is computed from a relationship between the rate of pumping Q, drawdown H' at a point P near the well, distance from the well to the point of drawdown measurement r, coefficient of storage of the aquifer S, and elapsed pumping time t. This relationship permits the evaluation of k from aquifer performance, while water is being drawn from storage and before stabilization occurs.

B.1.2.2 Nonequilibrium equations are directly applicable to confined aquifers (artesian flow conditions) and may also be used with limitations to unconfined aquifers (gravity flow conditions). These limitations are related to the percentage of drawdown in observation wells relative to the total aquifer thickness. Nonequilibrium equations should not be used if the drawdown exceeds 25 percent of the aquifer thickness at the wall. Little error is introduced if the percentage is less than 10.

B.1.3 Basic Assumptions.

B.1.3.1 Both equilibrium and nonequilibrium methods for analyzing aquifer performance are generally based on the assumptions that:

a. The aquifer is homogeneous and isotropic.

b. The aquifer is infinite in areal extent and has a uniform thickness.

c. The well screen fully penetrates the pervious formation.

d. The flow is laminar.

e. The initial static water level is horizontal.

B.1.3.2 Although the assumptions listed above would seem to limit the analysis of pumping test data, in reality they do not. For example, most pervious formations do not have a constant k or transmissivity T (T = k x aquifer thickness D), but the average T can readily be obtained from a pumping test. Where the flow is artesian, stratification has relatively little importance if the well screen fully penetrates the aquifer; of course, the derived hydraulic conductivity for this case is actually k_h. If the formation is stratified and $k_h > k_v$, and the flow to the well is gravity in nature, the computed hydraulic conductivity k would be less than k_h and greater than k_v.

B.1.3.3 Marked changes of well or aquifer performance during a nonequilibrium test indicate that the physical conditions of the aquifer do not conform to the assumptions made in the development of the formula for nonsteady flow to a well. However, such a departure does not necessarily invalidate the test data; in fact, analysis of the change can be used as a tool to better determine the flow characteristics of the aquifer.

B.2. Pumping Test Equipment and Procedures. Estimation of k from a pumping test requires: (a) installation of a test well, (b) two or preferably more observation wells or piezometers, (c) a suitable pump, (d) equipment for sounding the well and adjacent piezometers, and (e) a means for accurately measuring the flow from the well.

B.2.1 Test and Observation Wells. The test well should fully penetrate the aquifer to avoid uncertainties involved in the analysis of partially penetrating wells, and the piezometers should be installed at depths below any anticipated drawdown during the pumping test. The number, spacing, and arrangement of the observation wells or piezometers will depend on the characteristics of the aquifer and the geology of the area (Figures B.2 and B.3). Where the test well is located adjacent to a river or open water, the piezometers should be installed on one line perpendicular to the river, one line parallel to the river, and, if possible, one line away from the river. At least one line of piezometers should extend 500 feet or more out from the test well. The holes made for installing piezometers should be logged for use in the analysis of the test. The distance from the test well to each piezometer should be measured, and the elevation of the top of each hole should be accurately assessed. Each piezometer should be capped with a vented cap to keep out dirt or trash and to permit changes in water level in the piezometer without creating a partial vacuum or pressure change. The test well and piezometers should be carefully installed and developed as discussed in EM 1110-2-1908, and their performance checked by individual pumping or falling head tests according to the procedures discussed in EM 1110-2-1908.

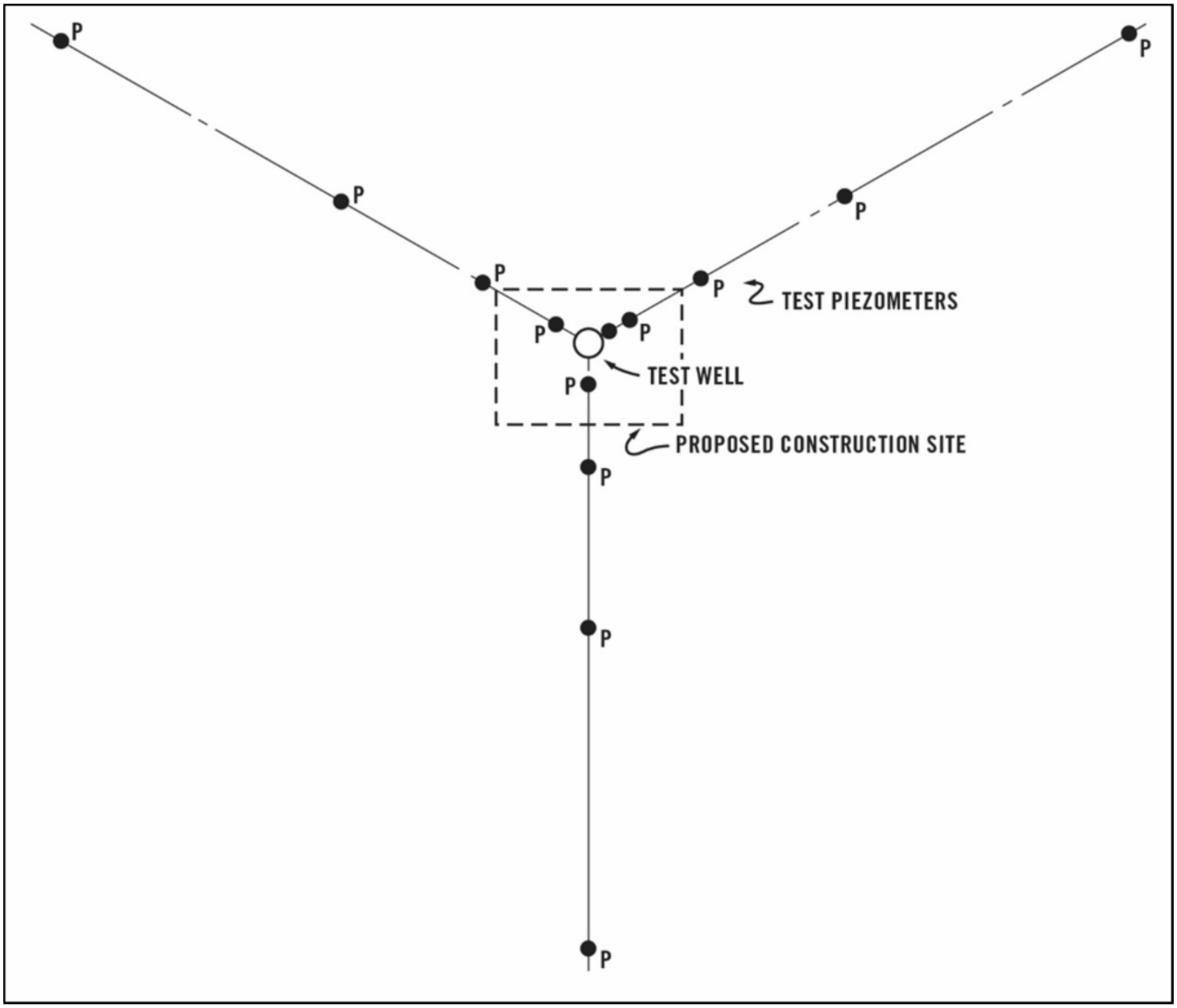


Figure B.2. Layout of piezometers for a pumping test

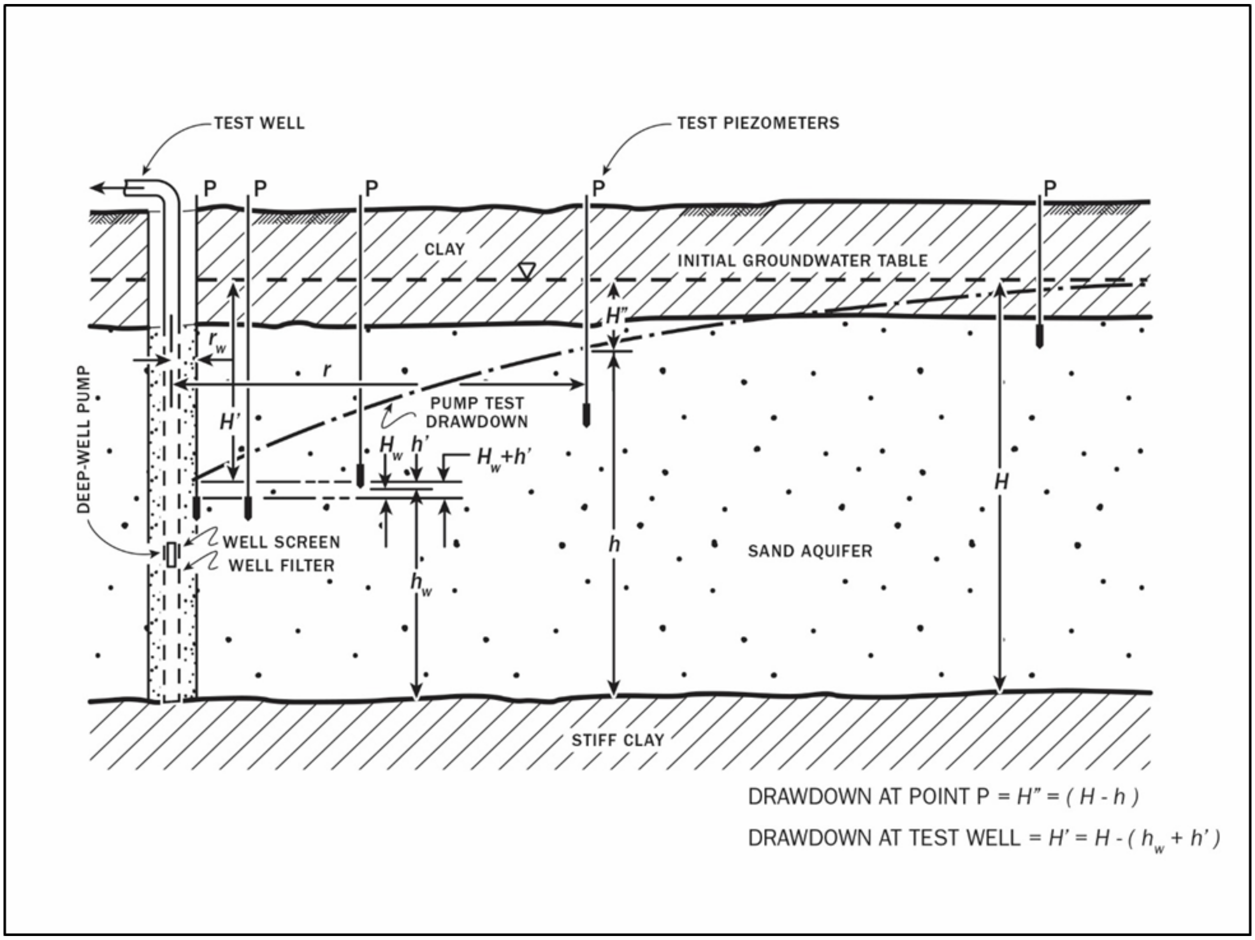


Figure B.3. Section of well and piezometers for a pumping test with gravity flow near well

B.2.2 Pumps.

B.2.2.1 The test pump should be a centrifugal, or more preferably, a lineshaft turbine or submersible pump installed in the well. It should be capable of lowering the water level in the well at least 10 feet or more depending upon the characteristics of the formation being tested. The pump should preferably be driven with an electric motor or with an engine capable of operating continuously for the duration of the test. The pump discharge line should be equipped with a valve so that the rate of discharge can be accurately controlled. At the beginning of the test, the valve should be partially closed so that the back pressure on the pump can be varied as the test progresses to keep the rate of flow constant.

B.2.2.2 During a pumping test, it is imperative that the rate of pumping be maintained constant. Lowering of the water level in the well will usually cause the pumping rate to decrease unless the valve in the discharge line is opened to compensate for the additional head or lift created on the pump. If the pump is driven with a gas or diesel engine, changes in temperature and humidity of the air may appreciably affect the operation of the engine and thus cause variations in the pumping rate. Variations in line voltage may similarly affect the speed of electric motors and thus the pumping rate. Any appreciable variation in pumping rate should be recorded, and the cause of the variation noted. The flow from the test well must be conveyed from the test site so that recharge of the aquifer from water being pumped does not occur within the zone of influence of the test well.

B.2.3 Flow and Drawdown Measurements.

B.2.3.1 Flow Measurements and Regulation of Flow.

B.2.3.1.1. The discharge from the well can be measured by means of an orifice, pitometer, venturi, or flow meter installed in the discharge pipe, or an orifice installed at the end of the discharge pipe, as described in Appendix D. The flow can also be estimated from the drop of a jet emerging from a smooth discharge pipe or measured by means of a weir or flume installed in the discharge channel. For such flow measurements, appropriate consideration must be given to the pipe or channel hydraulics in the vicinity of the flow-measuring device.

B.2.3.1.2. Detailed methods for measuring flow are discussed in Appendix D. Flow measurements should generally be accurate to within 2 percent of the measured flow. If the flow meter used has totalizing indication, totalizer readings should be recorded frequently during the test and flows calculated using pairs of totalizer measurements. A totalizer or flow meter with totalizing indication provides a running total of the flow through the flow meter in a given amount of time. Instantaneous flow measurements or measurements at a single point in time may also be recorded and are useful for adjustments to maintain constant flow during the test. A throttling valve should be used to adjust the head on the pump to maintain constant flow; the valve should be located several pipe diameters downstream of the flow meter if a meter is used. For pumping tests at low flows (less than 5 gpm), it may be impractical to maintain constant flow using a throttling valve because of the pump characteristics combined with the imprecision of the valve. In such cases, it may be more practical to regulate average drawdown in the well by installing level controls and allowing the pump to cycle on and off between two set points a few feet apart vertically. In such cases, totalizing indication of flow or measuring the total flow over a certain length of time rather than taking flow measurements at a single point in time is required for accurate measurement of the average flow.

B.2.3.2 Water Level (drawdown) Measurements.

B.2.3.2.1. Electric water level indicator tapes on reels are a convenient method for measuring water levels accurately. Another method for measuring water levels accurately in standpipe piezometers or observation wells is by sounding with the use of chalked tape. When many piezometers and observation wells are to be monitored or when the anticipated duration of the pumping test will be several days, it may be worthwhile to install transducers in the wells/piezometers and use a multi-channel data logger to collect data. Refer to EM 1110-2-1908 for details on measuring water levels in standpipe piezometers and observation wells, and on installing transducers.

B.2.3.2.2. It is of utmost importance to make accurate manual water level measurements in each well or piezometer in which a transducer is installed: (a) when the instrument is first installed, and (b) from time to time during the pumping test. Water levels from these manual measurements are used as a quality check on the water levels calculated from the reduction of the pressure data from the transducers. The first quality check should be made and reviewed soon after the initial installation of a transducer and before the pumping test is started. The number and frequency of manual water level measurements should be sufficient to permit evaluation of the pumping test data if the data collected from the transducers is lost or subsequently found to be inaccurate.

B.2.3.3 Data Loggers. Installing instruments and a data logger is often justified by a reduction in monitoring labor costs when it is necessary to record a large amount of data from a number of instruments during a pumping test or when it is desirable to monitor instruments for several days or weeks before and after the actual test. Refer to EM 1110-2-1908 for details on data loggers.

B.2.3.4 General Test Procedures.

B.2.3.4.1. Before a pump test is started, the test well should be pumped for a brief period to ensure that the pumping equipment and measuring devices are functioning properly and to determine the approximate valve and power settings for the test. The water level in the test well and all observation wells and piezometers should be observed for at least 24 hours (and preferably for one to two weeks) prior to the test to determine the initial groundwater table. If the groundwater prior to the test is not stable, observations should be continued until the rate of change is clearly established; these data should be used to adjust the actual test drawdown data to an approximate equilibrium condition for analysis. Pumping of any wells in the vicinity of the test well, which may influence the test results, should be regulated to discharge at a constant, uninterrupted rate prior to and during the complete test, including the recovery period. If the test well is close to a body of water (such as an estuary, river, lake, or reservoir), water elevations for the closest gage should be obtained for comparison with groundwater level measurements before, during, and after the pumping test. If the body of water is affected by tides, tide tables should be obtained for the nearest tide station and also actual gage height readings before, during, and after the pumping test.

B.2.3.4.2. Drawdown observations in the test well itself are generally less reliable than those in the piezometers because of well inefficiency, pump vibrations and momentary variations in the pumping rate that cause fluctuations in the water surface within the well. A sounding tube with small slots installed inside the well screen can be used to dampen the fluctuation in the water level and improve the accuracy of well soundings. All observations of the groundwater level and pump discharge should include the exact time that the observation was made.

B.2.3.4.3. As changes in barometric pressure may cause the water level in test wells and piezometers to fluctuate, the barometric pressure should be recorded frequently during the test and compared with barometric pressure records for the nearest weather station.

B.2.3.4.4. When a pumping test is started, changes in water levels occur rapidly, and readings should be taken as often as practicable for certain selected piezometers (e.g., time t = 2, 5, 8, 10, 15, 20, 30, 45, and 60 minutes) after which the period between observations may be increased. Sufficient readings should be taken to accurately define a curve of water level or drawdown versus elapsed pumping time plotted as a semi-logarithmic chart. After pumping has stopped, the rate of groundwater-level recovery should be observed. Frequently, such data are important in evaluating the performance and characteristics of an aquifer.

B.3. Equilibrium Pumping Test. In an equilibrium-type pumping test, the well is pumped at a constant rate until drawdowns in the well and piezometers become stable.

B.3.1 A typical time versus drawdown curve (or simply time-drawdown curve) for a piezometer near a test well is plotted on an arithmetical scale in Figure B.4 and on a semilog scale in Figure B.5. (The computations in Figure B.5 are discussed subsequently). Generally, a time-drawdown curve plotted on a semilog scale becomes straight after the first few minutes of pumping. If true equilibrium conditions are established, the drawdown curve will become horizontal. The drawdown measured in the test well and adjacent observation wells or piezometers should always be plotted on a semilog chart during the test to check the performance of the well and aquifer. Although the example presented in Figure B.5 shows stabilization to have essentially occurred after 500 minutes, the usual rules of thumb are to pump artesian wells for 24 hours and to pump test wells where gravity flow conditions exist for 2 or 3 days.

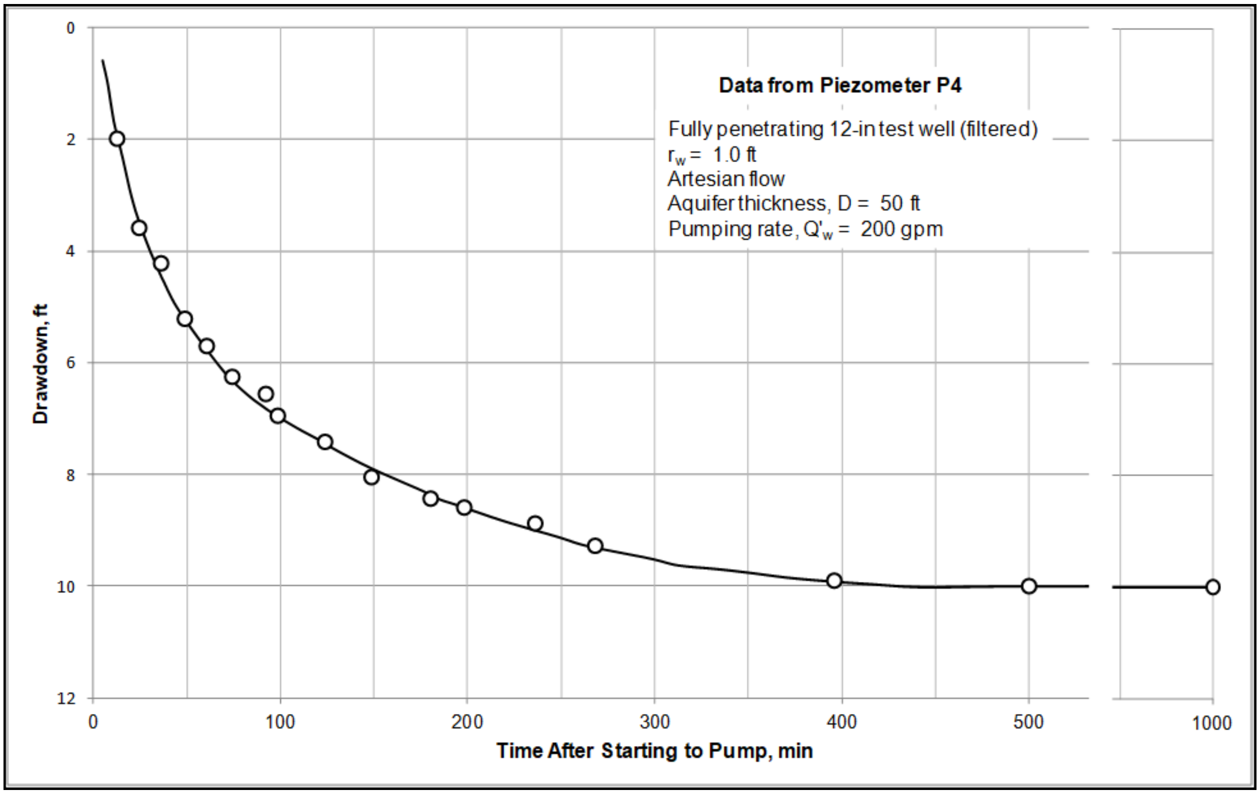


Figure B.4. Drawdown in an observation well versus pumping time (arithmetical scale) (Courtesy of the EPA)

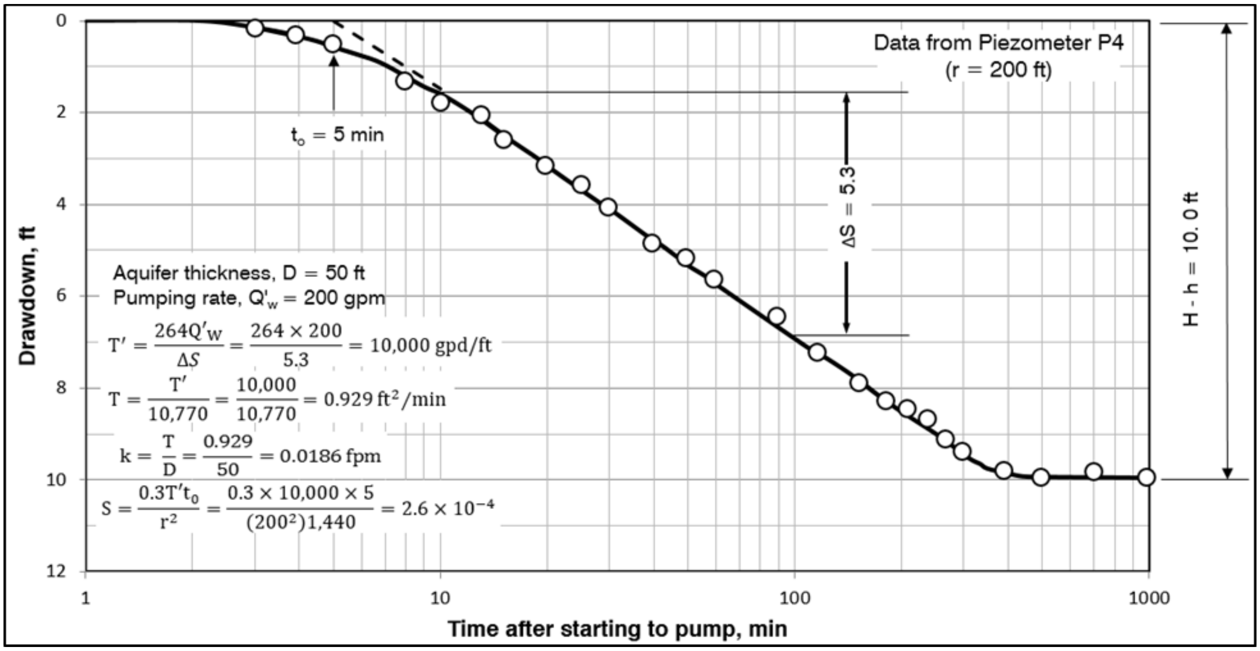
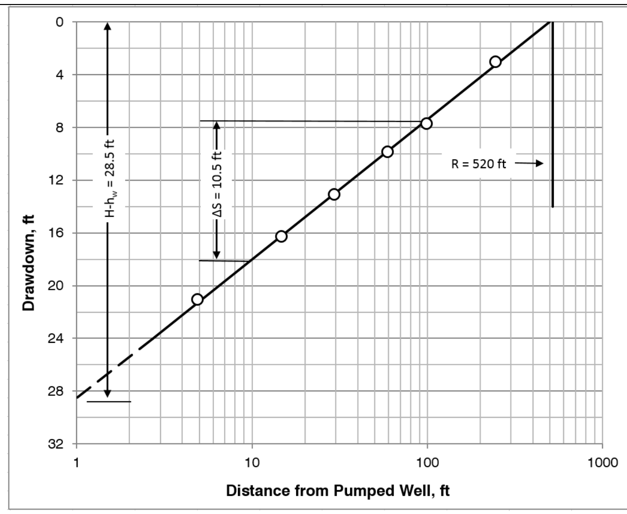


Figure B.5. Drawdown in an observation well versus pumping time (semilog scale) (Courtesy of the EPA)

B.3.2 The drawdown in an artesian aquifer as measured by piezometers on a radial line from a test well is plotted versus distance from the test well on a semilog chart in Figure B.6. In a homogeneous, isotropic aquifer with artesian flow, the semilog plot of drawdown (H-h) versus distance from the test well will form a straight line when the flow in the aquifer has stabilized. The semilog plot of drawdown H^2-h^2 versus (log) distance will also form a straight line for gravity flow. However, the drawdown in the well may be somewhat greater than would be indicated by a projection of this straight line to the well because of well entrance losses and the effect of a "free" discharge surface at gravity wells. Extension of the semilog drawdown versus distance line to zero drawdown indicates the effective source of seepage or radius of influence R, beyond which no drawdown would be produced by pumping the test well (Figure B.6).



Note: Drawdowns plotted were measured after groundwater table had stabilized.

Example: Fully penetrating 12-in test well (filtered), r_w = 1 ft
Artesian flow
Aquifer thickness = D = 50 ft
Pumping rate, Q_w' = 200 gpm
Pumping period = 1,000 min

$$k = \frac{Q'_w \ln(R/r_w)}{2\pi D(H - h_w)} = \frac{200 \ln(520/1)}{7.5(2\pi)(50)(28.5)} = 0.0189 \text{ fpm}$$

Figure B.6. Drawdown versus distance from test well (Courtesy of the EPA)

B.3.3 For flow from a circular source of seepage, the hydraulic conductivity k can be computed from the following formulas for fully penetrating wells.

B.3.3.1 Artesian Flow.

$$Q_w = \frac{2\pi kD(H - h)}{\ln\left(\frac{R}{r}\right)} \tag{B.1}$$

B.3.3.2 Gravity Flow.

$$Q_w = \frac{\pi k(H^2 - h^2)}{\ln\left(\frac{R}{r}\right)} \tag{B.2}$$

Where:

Q_w = flow from the well

D = aquifer thickness

H = initial height of groundwater table (GWT)

h = height of GWT at r

$(H\text{-}h)$ or $(H^2\text{-}h^2)$ = drawdown at distance r from well or difference between square of H and square of height of phreatic surface at distance r

R = radius of influence

B.3.4 An example calculation of R and k from an equilibrium pumping test is shown in Figure B.6.

B.3.5 For combined artesian-gravity flow, seepage from a line source and a partially penetrating well, the hydraulic conductivity k can be computed from well-flow formulas presented in Chapter 5 of this manual.

B.4. Nonequilibrium Pumping Test.

B.4.1 Constant Discharge Tests. The transmissivity T, hydraulic conductivity k, and storativity S of a homogeneous, isotropic aquifer of infinite extent with no recharge can be estimated from a nonequilibrium-type pumping test. Average values of S and T in the vicinity of a well can be obtained by measuring the drawdown with time in one or more piezometers while pumping the well at a known constant rate and analyzing the data according to methods described in (1), (2), and (3) below.

B.4.1.1 Method 1 (Theis formula). The formula for nonequilibrium flow can be expressed as

$$H - h = \frac{115{Q'}_w W(u)}{T'} \tag{B.3}$$

Where:

$H\text{-}h$ = drawdown at observation piezometer (feet)

Q'_w = well discharge (gpm)

$W(u)$ = exponential integral termed "well function" (see Table B.2)

T' = transmissivity (gallons per day per width) and

$$u = \frac{1.87r^2S}{T't'} \qquad (B.4)$$

Where:

r = distance from test well to observation well or piezometer (feet)

S= storativity

t'= elapsed pumping time (days)

The formation constants can be approximated from pumping test data using a graphical method of superposition, which is outlined below:

a. *Step 1*. Plot W(u) versus u on a log-log chart, known as a “type-curve,” using Table B.2 and plotted as shown in Figure B.7.

b. *Step 2*. Plot drawdown (H-h) versus r^2/t' on a log-log chart of same size as the type-curve shown in Figure B.7.

c. *Step 3*. Superimpose observed data curve on type-curve, keeping coordinates axes of the two curves parallel, and adjusting until a position is found by trial whereby most of the plotted data fall on a segment of the type-curve as shown in Figure B.7.

d. *Step 4*. Select an arbitrary point on a coincident segment, and record coordinates of matching point (Figure B.7).

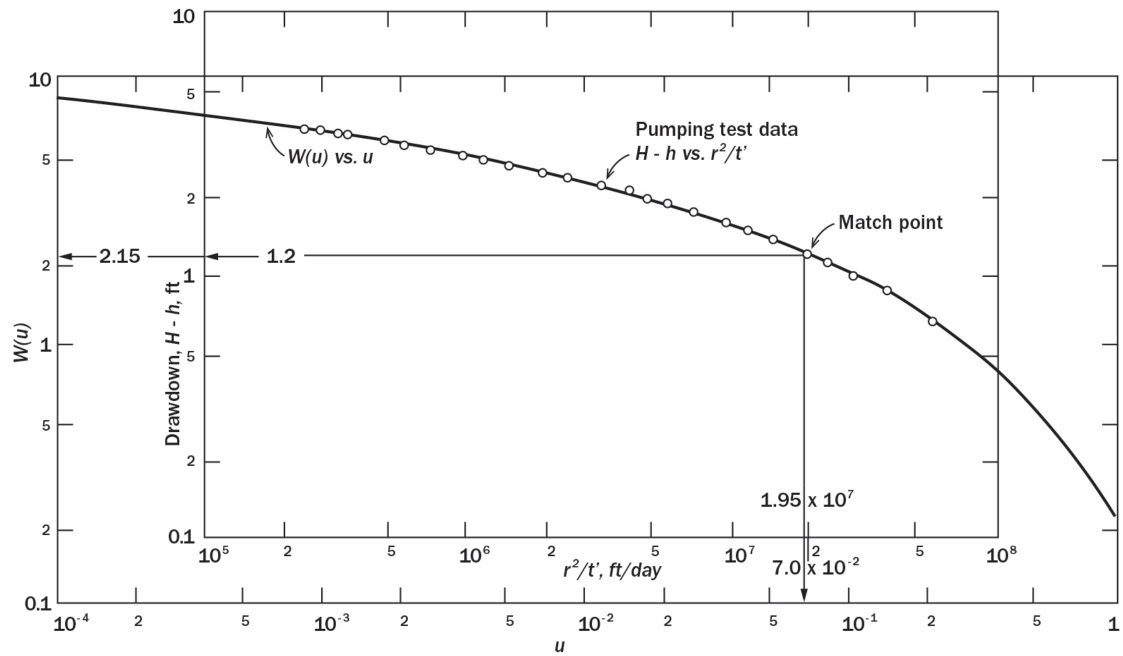
e. *Step 5*. With values of W(u), u, H-h, and r^2/t' determined, compute S and T' from Equations (B.3) and (B.4).

f. *Step 6*. Compute T and k from the following equations:

$$T = \frac{T'}{10{,}770} \text{ (square feet per minute)} \qquad (B.5)$$

And

$$k = \frac{T'}{10{,}770D} \text{ (feet per minute)} \tag{B.6}$$



EXAMPLE: $Q_w' = 500$ GPM

r = 200 FT

$$T' = \frac{115\, Q_w'\, W(u)}{H - h} = \frac{115(500)(2.15)}{1.2} = 103{,}000 \text{ GPD/FT}$$

$$S = \frac{uT'}{1.87\, r^2/t'} = \frac{(7.0 \times 10^{-2})(103{,}000)}{1.87\,(1.95 \times 10^{7})} = 1.98 \times 10^{-4}$$

Figure B.7. Method 1 (Theis) for solution of the nonequilibrium equation (Todd, 1980)

B.4.1.2 <u>Method 2</u>. This method can be used as an approximate solution for nonequilibrium flow to a well to avoid the curve-fitting techniques of method 1 by using the techniques outlined below

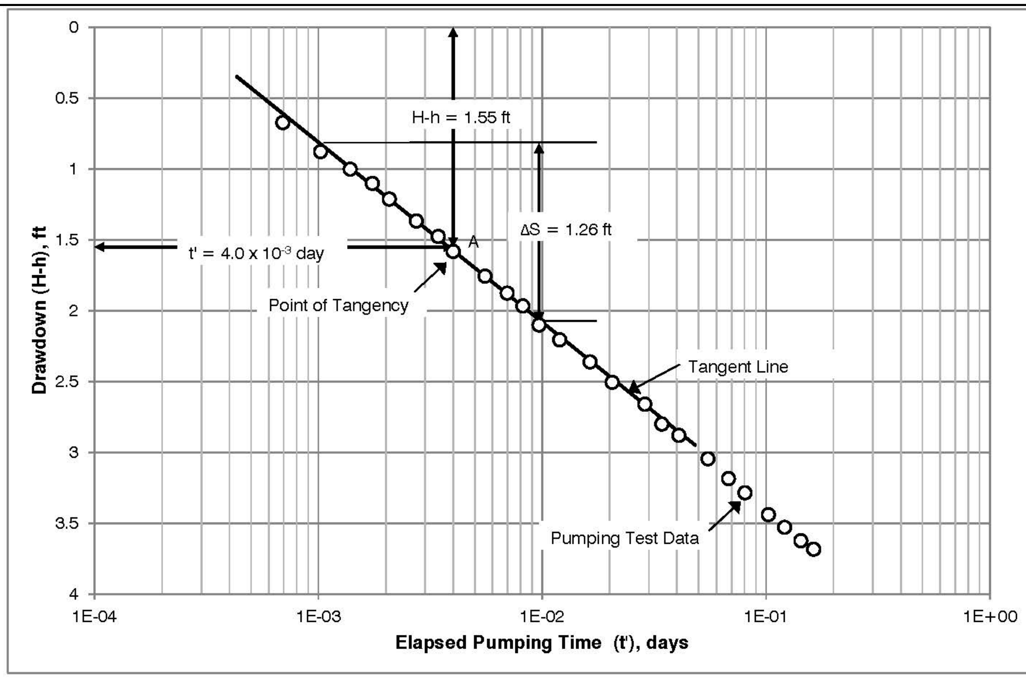
a. *Step 1*. Plot time versus drawdown on semilog chart as shown in Figure B.8.

b. *Step 2*. Choose an arbitrary point on time-drawdown curve (see point A on Figure B.8), and note coordinates t and H-h.

c. *Step 3*. Draw a tangent to the time-drawdown curve through the selected point, and determine Δs, the drawdown in feet per log cycle of time.

d. *Step 4*. Compute $F(u) = (H - h)/\Delta s$ and determine corresponding W(u) and u from Figure B.9.

e. *Step 5*. Determine the formation constants by Equations (B.3) and (B.4)



$Q'_w = 500$ gpm
Distance to observation well, r = 200 ft
At Point A: t' = 4.0 x 10^{-3} day
H - h = 1.55 ft
Tangent through A: ΔS = 1.26 ft / log cycle of pumping time in days
Then,

$$F(u) = \frac{H - h}{\Delta S} = \frac{1.55}{1.26} = 1.23$$

See Figure C-10 for F(u)

$$T' = \frac{115Q'_w W(u)}{H - h} = \frac{115(500)(2.72)}{1.55} = 101{,}000\ gpd/ft$$

$$S = \frac{T't'u}{1.87r^2} = \frac{101{,}000(4.0x10^{-3})(0.038)}{1.87(200)^2} = 2.05x10^{-4}$$

Figure B.8. Method 2 for solution of the nonequilibrium equation (Todd, 1980)

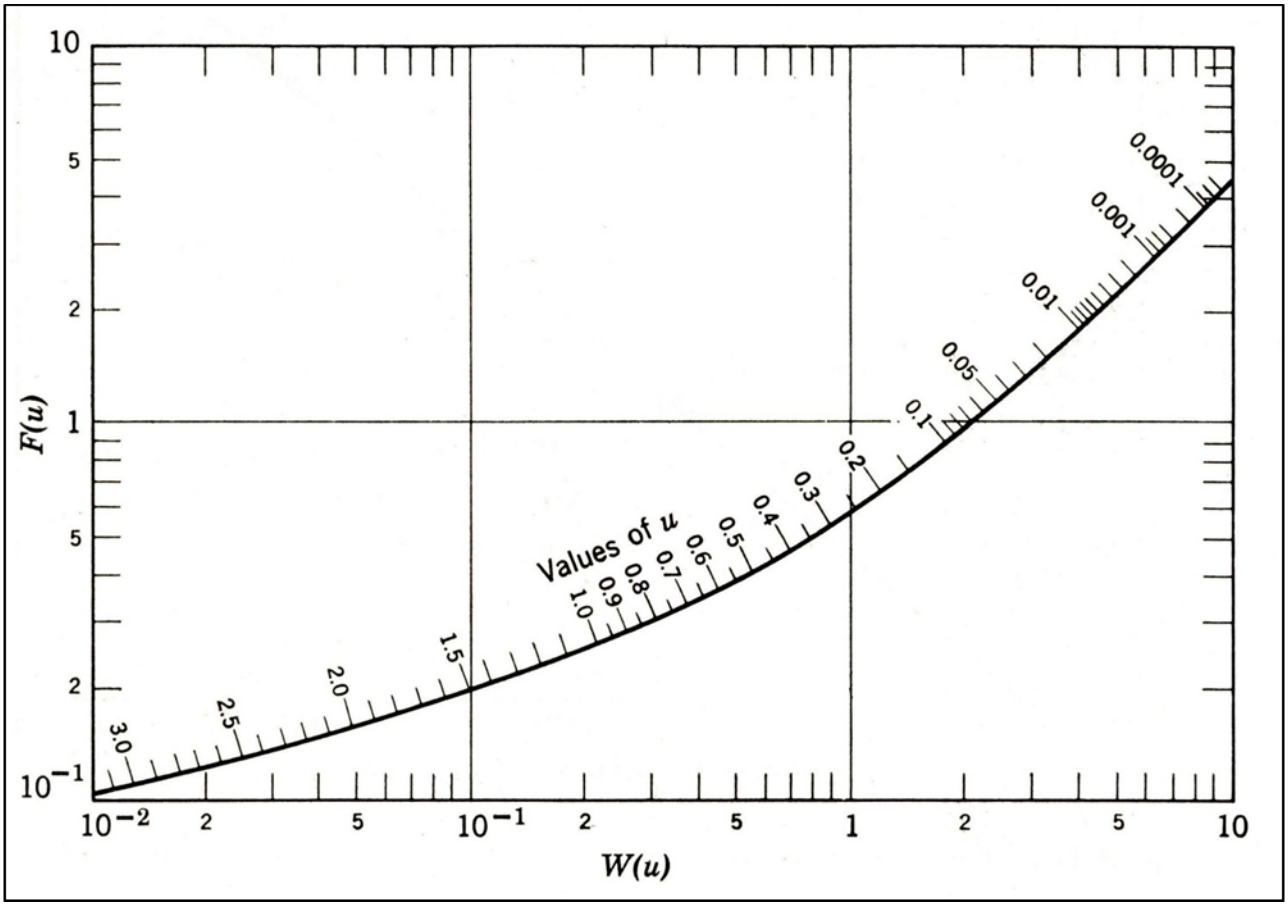


Figure B.9. Relation among F(u), W(u), and u (Todd, 1980)

B.4.1.3 Method 3. This method can be used as an approximate solution for nonequilibrium flow to a well if the time-drawdown curve plotted as a semilog chart becomes a straight line (Figure B.5). An example of this method of analysis used to determine values of T, S, and k is given in Figure B.5, using the nonequilibrium portion of the time-drawdown curve. The formation constants (T' and S) can be computed from

$$T' = \frac{264Q'_w}{\Delta s} \tag{B.7}$$

and

$$S = \frac{0.3T't_0}{r^2} \tag{B.8}$$

Where:

Δs = drawdown in feet per cycle of (log) time-drawdown curve

t_0 = time at zero drawdown (days)

B.4.2 Gravity Flow. Although the equations for nonequilibrium pumping tests are derived for artesian flow, they may be applied to gravity flow if the drawdown is small with respect to the saturated thickness of the aquifer and the storativity is equal to the specific yield of the dewatered portion of the aquifer plus the yield caused by compression of the saturated portion of the aquifer as a result of lowering the groundwater. The procedure for computing T' and S for nonequilibrium gravity flow conditions is outlined below.

a. *Step 1*. Compute T' from Equation (B.3).

b. *Step 2*. Compute S from Equation (B.4) for various elapsed pumping times during the test period, and plot S versus (log) t'.

c. *Step 3*. Extrapolate the S versus (log) t' curve to an ultimate value for S'.

d. *Step 4*. Compute u from Equation (B.4), using the extrapolated S’, the originally computed T', and the original value of r^2/t'.

e. *Step 5*. Recompute T' from Equation (B.3) using a W(u) corresponding to the computed value of u.

B.4.3 Recharge. Time-drawdown curves for a test well are significantly affected by recharge or depletion of the aquifer, as shown in Figure B.10. Where recharge does not occur, and all water is pumped from storage, the semilog H' versus t curve would resemble curve “a”. Where the zone of influence intercepts a source of seepage, the H' versus t curve would resemble curve “b”. There may be geological and recharge conditions where there is some recharge, but not enough to equal the rate of well flow (e.g., curve “c”). In many areas, formation boundary conditions exist that limit the areal extent of aquifers. The effect of such a boundary on a semilog H' versus t graph is opposite of the effect of recharge. Thus, when an impermeable boundary is encountered, the slope of the semilog H' versus t curve steepens as illustrated by curve “d”. It should be noted that a nonequilibrium analysis of a pumping test is valid only for the first segment of a time-drawdown curve.

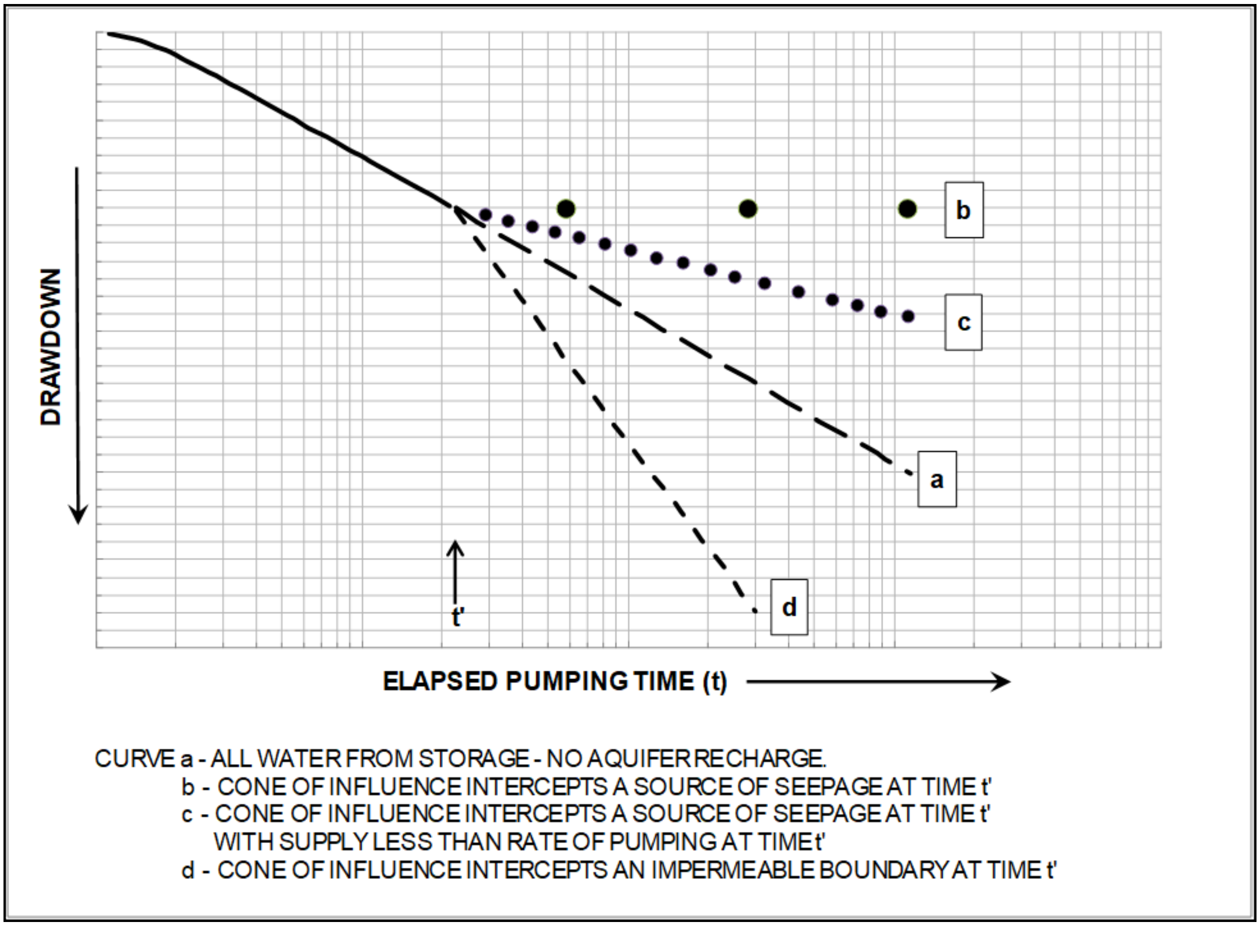


Figure B.10. Time-drawdown curves for various conditions of recharge

B.4.4 Step-drawdown Pump Test.

B.4.4.1 The efficiency of a well with respect to entrance losses and friction losses can be determined from a step-drawdown pumping test, in which the well is pumped at a constant rate of flow until either the drawdown becomes stabilized or a straight-line relation of the time-drawdown curve plotted on a semilog chart is established. Then, the rate of pumping is increased, and the above-described procedure is repeated until the well has been pumped at three to five rates. The drawdown from each step should be plotted as a continuous time-drawdown curve as illustrated in Figure B.11. The straight-line portion of the time-drawdown curves is extended as shown by the dashed lines in Figure B.11, and the incremental drawdown $\Delta H'$ for each step is determined as the difference between the plotted and extended curves at an equal time after each step in pumping. The drawdown H' for each step is the sum of the preceding incremental drawdowns and can be plotted versus the pumping rate as shown in Figure B.12. If the flow is entirely laminar, the drawdown (H-h for artesian flow and H^2-h^2 for gravity flow) versus pumping rate will plot as a straight line; if any of the flow is turbulent, the plot will be curved.

B.4.4.2 Efficiency of a well is defined as the drawdown in a theoretical well subject only to aquifer losses divided by the drawdown in a well subject to well and aquifer losses. Well and aquifer losses are described by well loss coefficients as proposed by Jacob (1947).

B.4.4.2.1. These coefficients include:

a. B_1 – a linear aquifer-loss coefficient caused by head losses in the aquifer.

b. B_2 – a linear well-loss coefficient caused by drilling damage to aquifer, drilling mud plugging the aquifer, and losses in the gravel pack and well screen.

c. B_3 – a partially penetrating-loss coefficient.

d. C – a nonlinear well-loss coefficient caused by friction losses inside the well housing and losses due to turbulent flow in the vicinity of the pump intake.

B.4.4.2.2. Aquifer losses are always present and will negatively impact the performance of the dewatering well. The B_1 coefficient can be estimated from the transmissivity and storativity components obtained from a constant rate pumping test completed with time/drawdown data obtained from observation wells. B_2, also called H_e as described below, can be determined from a step-drawdown test with the additional drawdown in excess of the aquifer losses from B_1 coefficient. The B_3 coefficient can estimated by evaluating results of a step-drawdown pump test on a partially penetrating well using methods described in the technical literature. The C coefficient can also be estimated from the results of a step-drawdown test. The estimate of head loss using a step-drawdown test is described below.

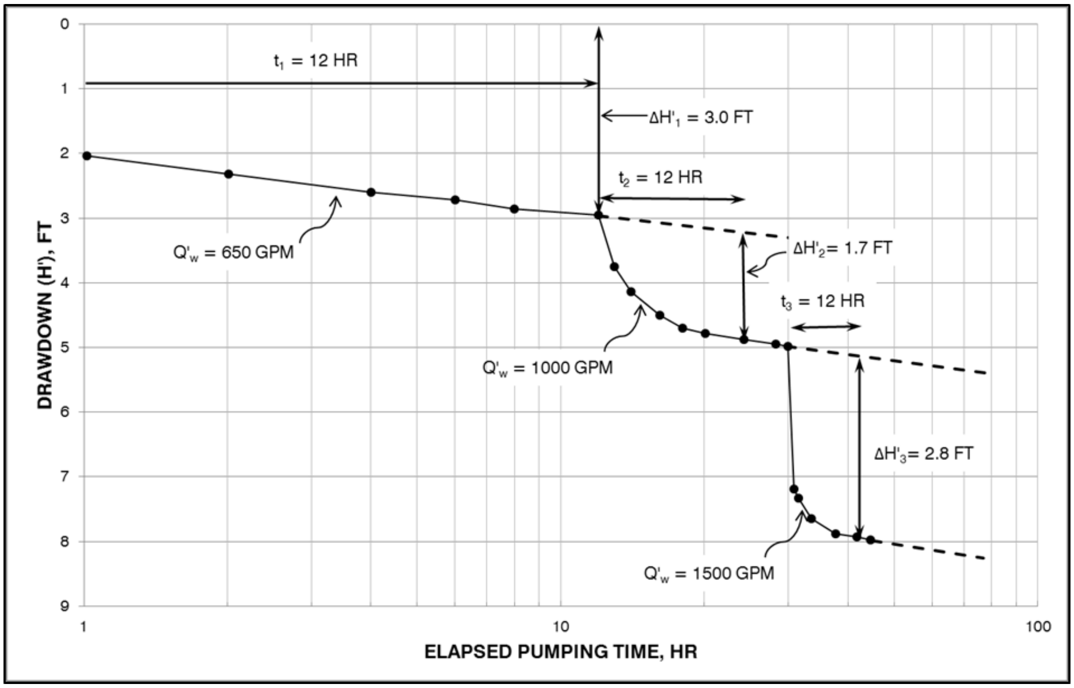


Figure B.11. Drawdown versus elapsed pumping time for a step-drawdown test

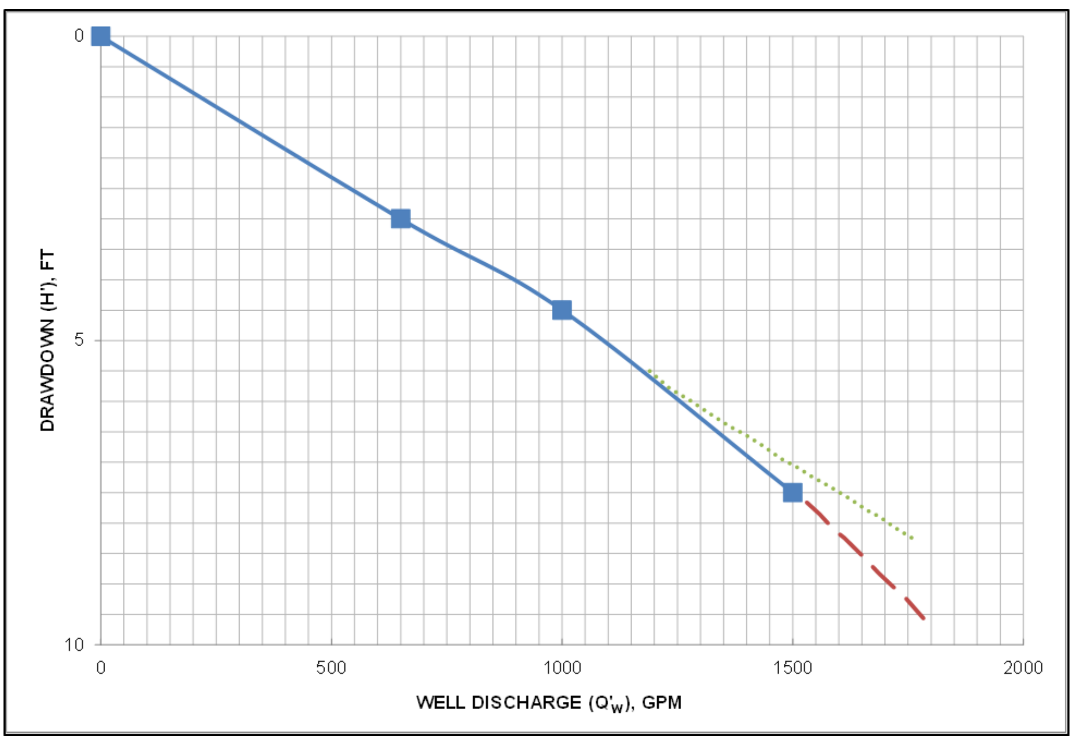


Figure B.12. Drawdown versus pumping rate for a step-drawdown test

B.4.4.2.3. The well-entrance loss H_e, consisting of frictional losses at the aquifer and filter interface through the filter and through the well screen, can be estimated from the semilog drawdown versus distance plots for a step-drawdown pump test as illustrated in Figure B.13. The difference in drawdown between the extended drawdown-distance curve and the water elevation measured in the well represents the well-entrance loss and can be plotted versus the pumping rate as shown in Figure B.14. Curvature of the H_w versus Q_w line indicates that some of the entrance head loss is the result of turbulent flow into or in the well.

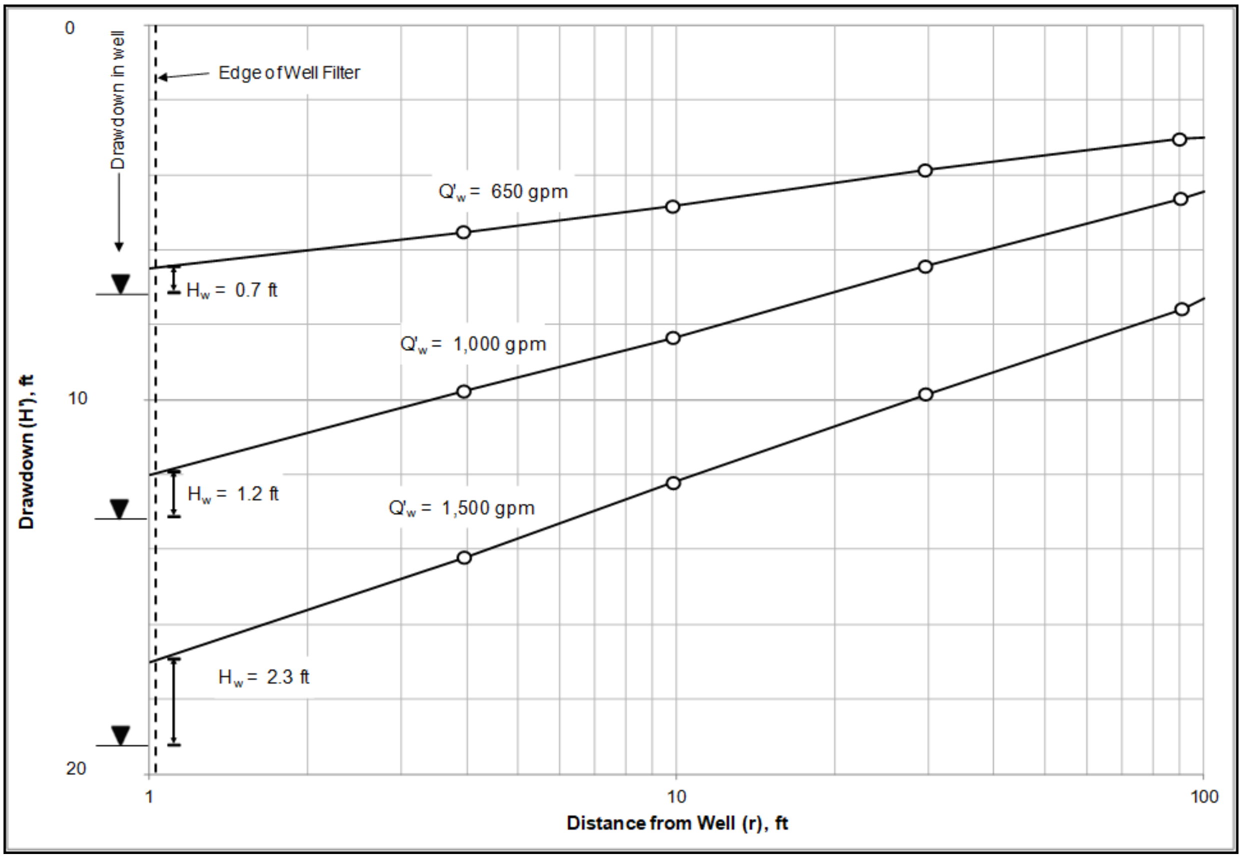


Figure B.13. Drawdown versus distance for a step-drawdown test for determining well-entrance loss

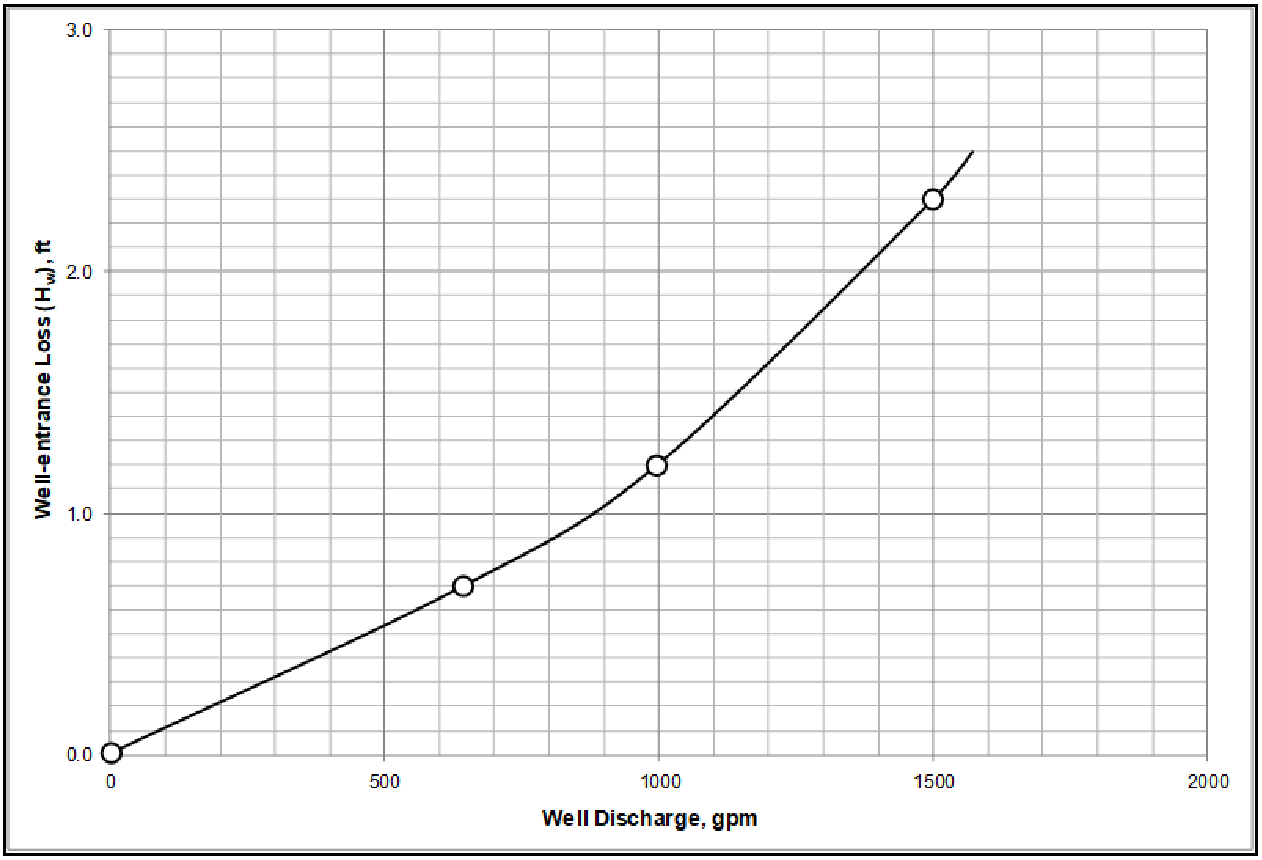


Figure B.14. Well-entrance loss versus pumping rate for a step-drawdown test

B.4.5 <u>Specific Capacity Tests</u>. Specific capacity is defined as the flow rate per unit of drawdown in the well being tested:

$$q_{sc} = Q_w / (h_{swl} - h_{pwl}) \tag{B.9}$$

Where:

q_{sc}	=	Specific capacity of the well being tested
Q_w	=	Test flow rate
h_{swl}	=	Head in well before pumping (static water level)
h_{pwl}	=	Head in well during pumping (pumping water level)

B.4.5.1 This test should be performed on all wells installed in a dewatering system as an index of well flow capacity. The flow rate selected for testing should be either the average flow rate expected for all of the wells in a dewatering system or (if the expected average flow is unknown) the flow required to achieve a drawdown between 5 and 10 feet in the well tested. The specific capacity is an index of the transmissivity (k_D) of the aquifer in the immediate vicinity of the well. The duration of the test should be about 1 or 2 hours.

B.4.5.2 If there are other nearby wells or piezometers that will be affected by pumping the subject well, they should be at equilibrium levels and should not be pumped during the specific capacity test on the "subject" well. The water levels in nearby wells or piezometers (ideally at least 4 wells or piezometers at varying distances from the subject well) should be read before testing the subject well and again just before the end of the test on the subject well. If the aquifer is confined and the test duration is sufficient to achieve transient steady-state flow[9], the drawdown data from nearby wells can be analyzed to estimate the approximate transmissivity and storativity of the aquifer using the Cooper and Jacob (1946) non-equilibrium formulas discussed in Kruseman and de Ridder (1990, page 68, Procedure 3.5.)

B.4.5.3 The specific capacity of a well may also be calculated from the data obtained from a step-drawdown test described in Section B.4.3.

B.4.5.4 The volumetric sand content in the well discharge should always be measured soon after a well is installed using a centrifugal sand tester similar to the Rossum sand tester manufactured by the Roscoe Moss Company (or equivalent). Figure B.15 shows a Rossum sand tester sampling the discharge from a well being tested for specific capacity. Ideally, the sand content measured after a few minutes of pumping will be zero or negligible.[10]

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[9] Transient steady-state flow in a confined aquifer is a condition when the drawdown difference (and therefore the hydraulic gradient) between piezometers remains constant, even though the drawdown continues to increase. See Kruseman and de Ridder (1990), page 58 for a discussion of what constitutes transient steady-state flow in a confined aquifer.

[10] A typically specified maximum limit for sand content in the discharge from a well-designed and constructed well is 5 parts per million (ppm). For high-yielding wells, it may be advisable to limit sand production to not more than 2 ppm.

Figure B.15. Rossum sand tester (on left side) equipped with tee valve and Dole orifice to regulate flow through tester (Courtesy of AECOM)

B.4.5.5 Other measurements and tests may be performed for evaluating subsequent well performance, including water temperature, oxidation-reduction potential (ORP), pH, and inorganic water chemistry, and microbiology (see discussion of incrustation and corrosion in Section 4.3 of this manual.

B.4.6 Recovery Test.

B.4.6.1 A recovery test may be made at the conclusion of a pumping test to provide a check of the pumping test results and to verify recharge and aquifer boundary conditions assumed in the analysis of the pumping test data. A recovery test is valid only if the pumping test has been conducted at a constant rate of discharge. A recovery test made after a step-drawdown test cannot be analyzed.

B.4.6.2 When the pump is turned off, the recovery of the groundwater levels is observed in the same manner as when the pump was turned on, as shown in Figure B.16. The residual drawdown *H'* is plotted versus the ratio of log t'/t″, where t' is the total elapsed time since the start of pumping and t″ is the elapsed time since the pump was stopped (Figure B.17). This plot should be a straight line and should intersect the zero residual drawdown at a ratio of t'/t″ = 1 if there is normal recovery, as well as no recharge and no discontinuities in the aquifer within the zone of drawdown. The ratio t'/t" approaches unity as the length of the recovery period is extended, assuming that the static GWT or piezometric level has not changed since the pumping test was started.

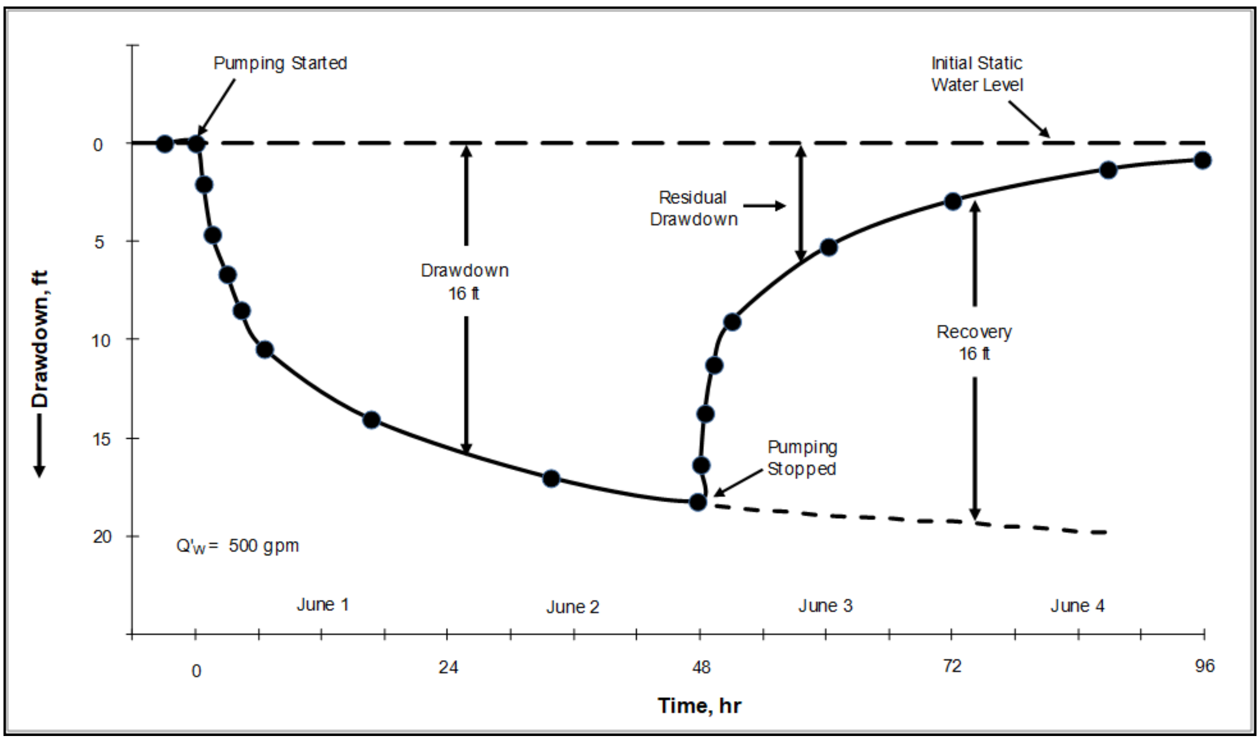

Figure B.16. Typical drawdown and recover curves for a well pumped and then allowed to rebound (Courtesy of the EPA)

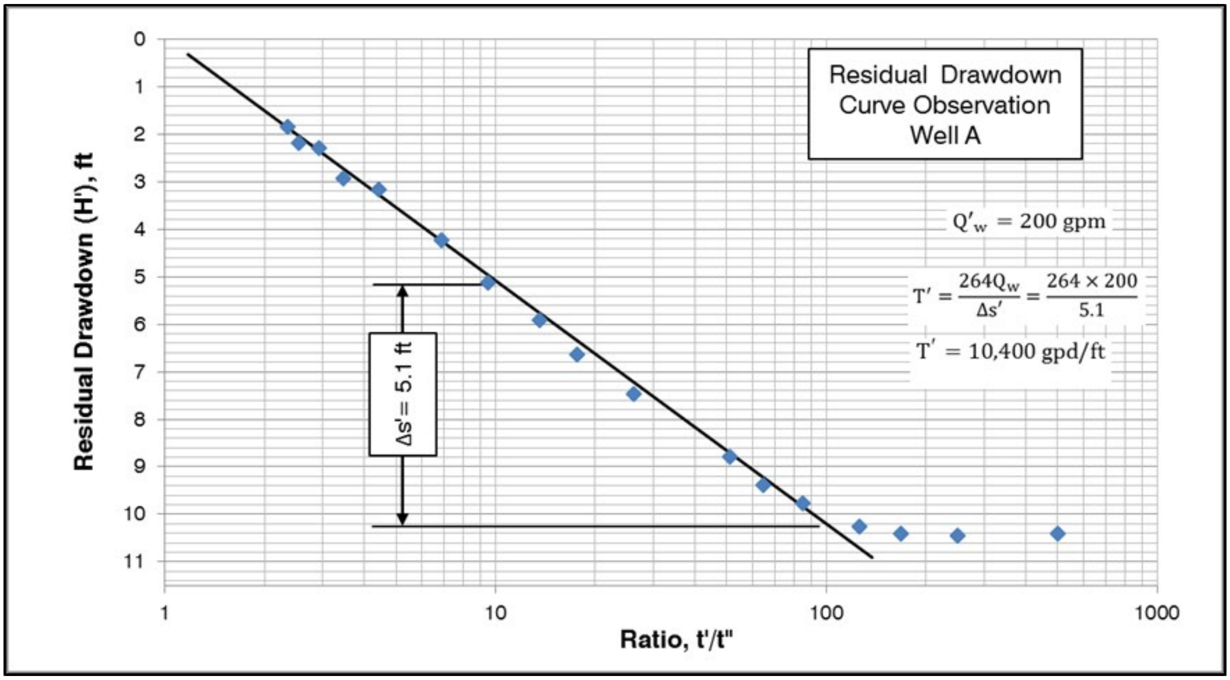

Figure B.17. Residual drawdown versus t'/t" (time during recovery period increased toward the left) (Courtesy of the EPA)

B.4.6.3 The transmissivity of the aquifer can be calculated from the equation

$$T' = \frac{264Q'_w}{\Delta s'} \tag{B.10}$$

where Δs' = residual drawdown in feet per log cycle of the semilog t'/t" versus residual drawdown curve. Displacement of the semilog residual drawdown versus t'/t" curve, as shown in Figure B.18, indicates a variance from the assumed conditions.

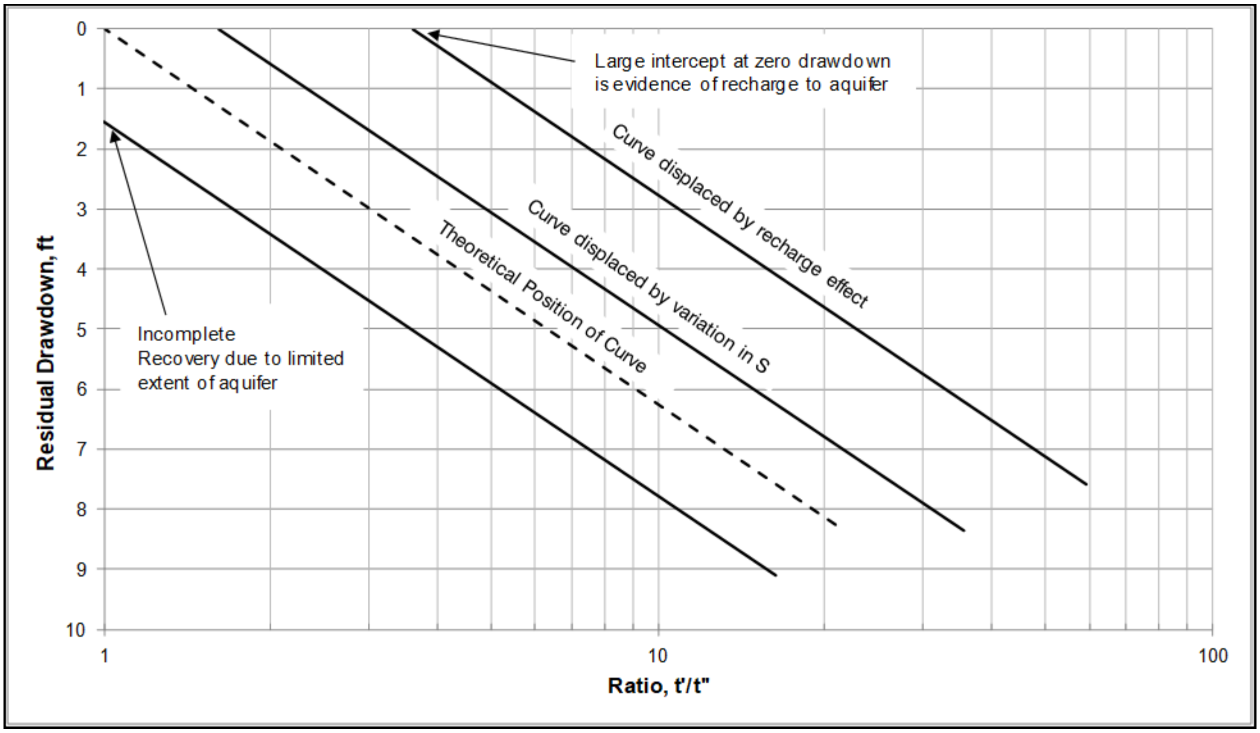

Figure B.18. Displacement of residual drawdown curve when aquifer conditions vary from theoretical (Courtesy of the EPA)

B.5. <u>Software for Analysis of Pumping Tests.</u>

B.5.1 Commercially available software packages are widely used for the analysis of pumping and borehole tests. Professional versions include a variety of analytical methods (including type curve matching) for differing aquifer conditions (confined, unconfined, confined to unconfined conversion, leaky, two aquifers, non-uniform aquifer), boundary conditions (using image wells), well penetration, horizontal well, interceptor trench, equilibrium and nonequilibrium flow, recovery, as well as different pumping protocols (step testing, constant rate, constant head, variable rate, multiple pumping wells). The software packages provide analysis for:

a. Single-Well solutions.

b. Slug Test Analysis.

c. Step-Test Analysis.

d. Variable-Rate Analysis.

e. Recovery Test Analysis.

f. Constant-Head Tests.

g. Delayed Response.

h. Aquifer-Test Design.

i. Two Aquifer Systems.

j. Single Fractures.

k. Partial Penetration Analysis.

l. Large-Diameter Well Solutions.

m. Interceptor Trench.

n. Horizontal Well.

o. Oscillatory Slug Tests.

B.5.2 Such software packages greatly simplify data plotting and type curve matching, thereby significantly reducing analytical time, especially when boundary conditions are complex. They also facilitate sensitivity testing of assumptions made for boundary conditions.

B.6. Additional References on Pumping Tests. The following references in Table B.1 are recommended for the interpretation of the results of pumping tests:

Table B.1

Additional References on Pumping Tests

Reference	Title
1.	Batu, V. (1998) *Aquifer Hydraulics – A Comprehensive Guide to Hydrogeologic Data Analysis*, John Wiley & Sons, Inc. New York, 727 pages
2.	Kruseman, G.P. and de Ridder, N.A.(1990) *Analysis and Evaluation of Pumping Test Data*, Second Edition, ILRI Publication 47, International Institution for Land Reclamation and Improvement, Wageningen, The Netherlands, 375 pages
3.	Powers, J.P, A.B. Corwin, P.C. Schmall, and W.E. Kaeck (2007) *Construction Dewatering and Groundwater Control – New Methods and Applications*, Third Edition, John Wiley & Sons, Inc., Hoboken, New Jersey, pages 121-140
4.	Roscoe Moss Company (1990) *Handbook of Ground Water Development*, John Wiley & Sons, Inc., pages 84-107
5.	Sterrett, Robert J., Editor (2007) Groundwater and Wells, Third Edition, Johnson Screens, a Weatherford Company, New Brighton, MN, pages 179-251

Table B.2

Values of W(u) for values of u

u	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0
$\times 1$	0.219	0.049	0.013	0.0038	0.0011	0.00036	0.00012	0.000038	0.000012
$\times 10^{-1}$	1.82	1.22	0.91	0.70	0.56	0.45	0.37	0.31	0.26
$\times 10^{-2}$	4.04	3.35	2.96	2.68	2.47	2.30	2.15	2.03	1.92
$\times 10^{-3}$	6.33	5.64	5.23	4.95	4.73	4.54	4.39	4.26	4.14
$\times 10^{-4}$	8.63	7.94	7.53	7.25	7.02	6.84	6.69	6.55	6.44
$\times 10^{-5}$	10.94	10.24	9.84	9.55	9.33	9.14	8.99	8.86	8.74
$\times 10^{-6}$	13.24	12.55	12.14	11.85	11.63	11.45	11.29	11.16	11.04
$\times 10^{-7}$	15.54	14.85	14.44	14.15	13.93	13.75	13.60	13.46	13.34
$\times 10^{-8}$	17.84	17.15	16.74	16.46	16.23	16.05	15.90	15.76	15.65
$\times 10^{-9}$	20.15	19.45	19.05	18.76	18.54	18.35	18.20	18.07	17.95
$\times 10^{-10}$	22.45	21.76	21.35	21.06	20.84	20.66	20.50	20.37	20.25
$\times 10^{-11}$	24.75	24.06	23.65	23.36	23.14	22.96	22.81	22.67	22.55
$\times 10^{-12}$	27.05	26.36	25.96	25.67	25.44	25.26	25.11	24.97	24.86
$\times 10^{-13}$	29.36	28.66	28.26	27.97	27.75	27.56	27.41	27.28	27.16
$\times 10^{-14}$	31.66	30.97	30.56	30.27	30.05	29.87	29.71	29.58	29.46
$\times 10^{-15}$	33.96	33.27	32.86	32.58	32.35	32.17	32.02	31.88	31.76

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Appendix C
Examples of Design of Dewatering and Pressure Relief Systems

C.1. Examples. Examples are included in this section for the following cases:

C.1.1 Open excavation with artesian flow. The pressure relief design is done using flow nets with deep wells and is compared to a design using a 2D plan view finite element model. (Figures C.1 and C.2)

C.1.2 Open excavation with gravity flow. Design of a pressure relief system with deep wells. (Figure C.3)

C.1.3 Open excavation with gravity flow. Design of a pressure relief system that includes a combination of deep wells and a wellpoint system. (Figure C.4)

C.1.4 Trench excavation with artesian flow. Design of pressure relief system with wellpoints. (Figure C.5)

C.1.5 Rectangular excavation with artesian flow. Design of a pressure relief system with deep wells. Hand calculations compared to 2D plan view finite element model. (Figures C.6 and C.7)

C.1.6 Shaft excavation with artesian and gravity flows through a stratified foundation. Design of a deep well vacuum system. (Figure C.8)

C.1.7 Dewatering of a tunnel with gravity flows. Design of a pressure relief system with deep wells. (Figure C.9)

C.1.8 Runoff. Evaluate the sump and pump capacity needed to manage surface runoff into an excavation. (Figure C.10)

PROBLEM: Given the flow net, the data in the figure, and the plan of wells as shown, compute the well flow required to reduce the head in the sand stratum at El. 40 ft at point D, the corresponding head h_w at the wells, h_m midway between wells, and h_D at the center of the excavation. Assume that wells fully penetrate the pervious stratum and that D = 40 ft, k = 500 x 10^{-4} cm/sec = 0.1 fpm, and r_w = 1.0 ft. Flow rates should be reported in gallons per minute (gpm).

SOLUTION: Flow to slot (or wells) from flow net, Eq 5 (Fig. 61)

$$Q_T = k(H - h_e)\frac{N_f}{N_e}D = 0.1(90 - 60)\frac{10.0}{4.0} \times 40$$
$$= 300 \text{ cfm} = 2{,}250 \text{ gpm}$$

Assume 10 wells located as shown in “Well Plan”. Since a well has been spaced at the center of each flow channel, the flow per well is the same for all wells. Thus Q_w = 225 gpm or 30 cfm per well.

From Eq 2 (Fig. 62) with an average well spacing, a, of approximately 80 ft.

$$H - h_w = \frac{30}{0.1(40)}\left[10\left(\frac{4}{10}\right) + \frac{1}{2\pi}\ln\frac{80}{2\pi(1)}\right] = 33.0 \text{ ft}$$

Compute Δh_m from Eq 3 (Fig. 54) for a = 80 ft.

$$\Delta h_m = \frac{30}{2\pi(0.1)40}\ln\frac{80}{\pi(1)} = 3.9 \text{ ft}$$

Thus

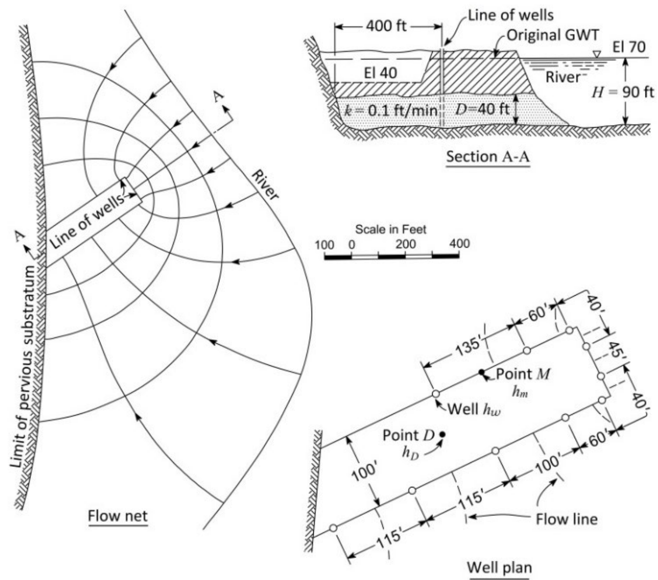
$$H - h_m = H - h_w - \Delta h_m = 33.0 - 3.9 = 29.1 \text{ ft}$$

From Eq 1 (Fig. 54) for a = 80 ft,

$$\Delta h_D = \Delta h_w = \frac{30}{2\pi(0.1)40}\ln\frac{80}{2\pi(1)} = 3.0 \text{ ft}$$

Thus

$$H - h_D = H - h_w - \Delta h_D = 33.0 - 3.0 = 30.0 \text{ ft}$$



The heads h_w, h_m, and h_D in terms of elevation are as follows:

$$h_w = 70 - 33.0 = 37.0 \text{ ft MSL}$$
$$h_m = 70 - 29.1 = 40.9 \text{ ft MSL}$$
$$h_D = 70 - 30.0 = 40.0 \text{ ft MSL}$$

Since GWT is to be lowered to El 40 at point D and since the computed head at this point is at El 40.0, Q_w = 30 cfm, or 225 gpm per well will produce the required head reduction. The values of Δh_D, Δh_m, and ($H - h_w$) also can be computed from Eq 1 and 3 (Fig. 55) and 3 (Fig. 62) respectively, as shown below. Note that the values so obtained are similar to those computed above.

From Fig. 55, θ_a = 0.4 and θ_m = 0.52 for a/r_w = 80 and W/D = 100 percent.

From Eq 3 (Fig. 62)

$$H - h_w = \frac{30}{0.1(40)}\left[10\left(\frac{4}{10}\right) + 0.4\right] = 33.0 \text{ ft}$$

From Eq 3 (Fig. 62)

$$\Delta h_m = \frac{30(0.52)}{(0.1)40} = 3.9 \text{ ft}$$

From Eq 1 (Fig. 55)

$$\Delta h_D = \Delta h_w = \frac{30(0.4)}{(0.1)40} = 3.0 \text{ ft}$$

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Figure C.1. Open excavation; artesian flow; pressure relief design by flow net

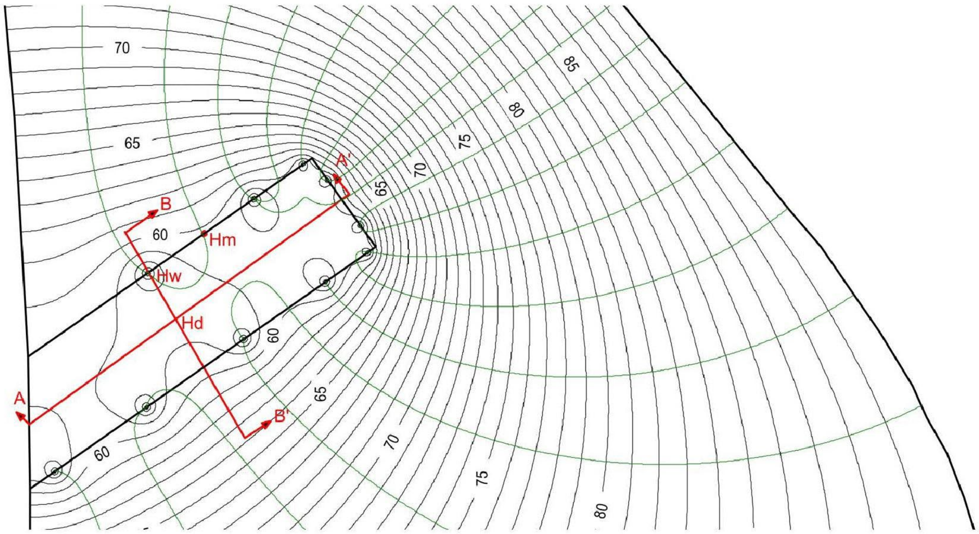
PROBLEM: Given the information shown in Figure C.1, compute the well flow required to reduce the head in the sand stratum at El. 40 ft at point D using a 2-D plan view finite element model and compare to the results obtained in Figure C.1.

SOLUTION: The problem was modeled using SEEP/W v8.11.1.7283 (GeoStudio) to calculate drawdown, flow rates, and the minimum number of wells required to relieve pressure in the aquifer underlying the excavation, as shown in Figure C.2a below. Boundary conditions were modeled using the full-size plan view flow net shown in Figure C.1. Boundary conditions included a no flow boundary at the "Limit of pervious substratum" line shown in Figure C.1 and a constant total head boundary of 90 feet at the "River" line shown in Figure C.1. The mesh was auto-generated by SEEP/W with a mesh density varied across the model through the use of a 1-ft mesh specified at the ring of wells to a 50-ft mesh specified at the river.

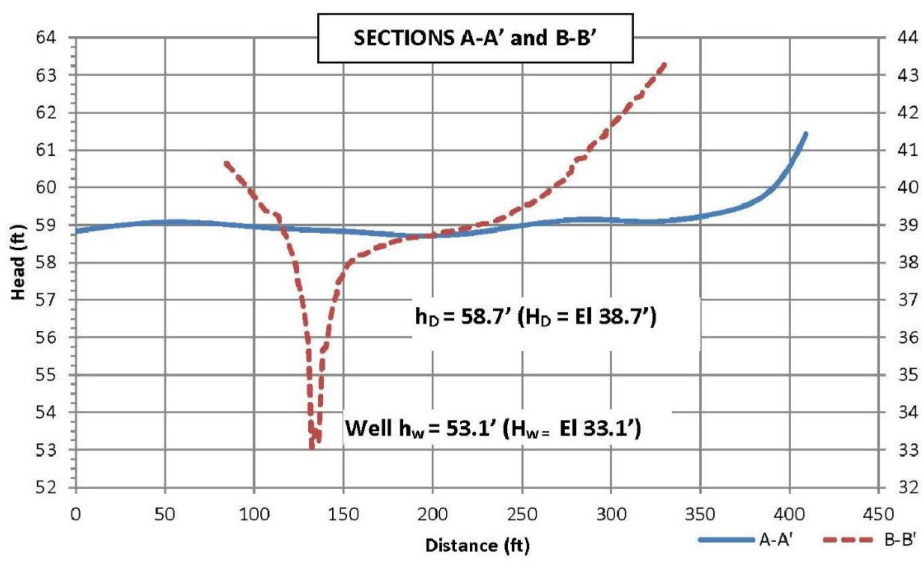
Step 1: Model a head condition of 60 feet (El. 40 ft) as a continuous seepage face along the line of wells. A total flow (Q_T) of 2,255 gpm was calculated along the line of wells.

Step 2: Model 10 wells at single nodes along the line of wells with a nodal flow of 225 gpm to evaluate drawdown across the excavation.

The variations in model head/elevation along Sections A-A' and B-B' in Figure C.2b below. Midway between the wells, the calculated head, h_m, is 59.8 ft (H_m = El 39.8 ft). The model heads/elevations in the plan and along Section A-A' and B-B' are slightly lower than those calculated in Figure C.1. This difference is attributable to the approximation of the rectangular well array as a single infinite line of equally spaced wells in the calculations in Figure C.1.

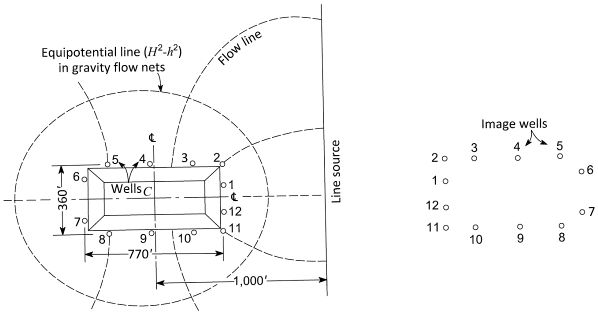


a) PARTIAL PLAN (HEAD CONTOUR LABELS ARE IN FEET ABOVE BOTTOM OF AQUIFER)

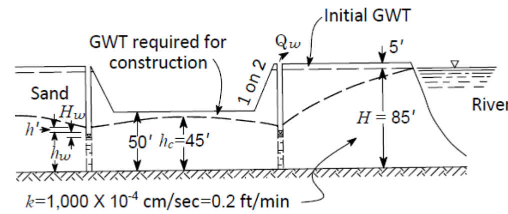


b) HEAD/ELEVATION ALONG SECTION A-A' AND B-B'

Figure C.2. Open excavation; artesian flow; pressure relief design using 2D plan view numerical model



PLAN



SECTION

PROBLEM: Design a system of 10-inch slotted screen wells, pumped by deep-well turbine pumps, for lowering the groundwater level 5 ft below the bottom of the excavation. Assume maximum allowable Q_w = 1,200 gpm, wells located 5 ft from top of slope, well radius r_w = 1 ft, and D_{10} of gravel filter = 0.25 mm.

SOLUTION: Estimate total flow required from Eq 3 (Fig. 51) using radius A_e of an equivalent large diameter well computed from Eq 6 (Fig. 48).

$$A_e = \frac{4}{\pi}\sqrt{770/2 \times 370/2} = 340 \text{ ft}$$

$$Q_T = \frac{\pi(0.2)(85^2 - 45^2)}{\ln[(2 \times 1000)/340]} = 1{,}840 \text{ cfm} = 13{,}800 \text{ gpm}$$

Use 12 wells with Q_w = 1,150 gpm. Locate wells as shown in plan so as to intercept equal quantity of flow as indicated by flow net and to obtain approximate level drawdown beneath excavation. Compute head h_e at center of excavation and head h_w at a well from Eq 3 and Eq 4 (Fig. 52) to check adequacy of system. Values of S_i and r_i were rounded to the nearest 10 feet.

Head at Point C and Well 4 Computed by Method of Images for Qw = 1,150 gpm = 154 cfm

Well	Head at Point C S_i ft	r_i ft	$\ln\frac{S_i}{r_i}$	Head at Well 4 $S_{i,4}$ ft	$r_{i,4}$ ft	$\ln\frac{S_{i,4}}{r_{i,4}}$
1	1,620	390	1.42	1,650	410	1.39
2	1,630	420	1.36	1,640	400	1.41
3	1,800	290	1.82	1,800	240	2.02
4	2,040	180	2.42	2,050	1	7.63
5	2,280	330	1.93	2,300	250	2.22
6	2,400	390	1.82	2,420	370	1.88
7	2,400	390	1.82	2,435	460	1.67
8	2,280	330	1.93	2,330	440	1.67
9	2,040	180	2.42	2,090	370	1.73
10	1,800	290	1.82	1,840	435	1.44
11	1,630	420	1.36	1,675	540	1.13
12	1,620	390	1.42	1,650	480	1.24

$F'_c = 21.54 \times 154 = 3{,}320$ $\quad F'_w = 25.44 \times 154 = 3{,}920$

From Eq 2 and 3 (Fig. 52), $H^2 - h_c^2 = \frac{3{,}320}{\pi(0.2)} = 5{,}280$. From Eq 3 and 4 (Fig. 52), $H^2 - h_w^2 = \frac{3{,}920}{\pi(0.2)} = 6{,}240$.

$$h_c = \sqrt{85^2 - 5{,}280} = 44.1 \text{ ft} \qquad h_w = \sqrt{85^2 - 6{,}240} = 31.4 \text{ ft}$$

The corresponding flow per foot of well screen is 1,150 / 32, or 36 gpm per ft. Compute head loss in well H_w from Fig. 58.

H_e = 0.70 ft (from Fig. 58a) $\qquad H_v$ = 0.35 ft (from Fig. 58c)

$$H_r + H_s = 1.25\left(\frac{32}{100} \times \frac{1}{2}\right) = 0.2$$

(from Fig. 58b and using the flow through one-half the length of the screen)

H_w = 1.25 ft, say 1.3 ft

Thus $h_w - H_w$ = 32.0 – 1.3 = 30.7 ft. Bowls of pump should be set about 2 ft below this level, and the pump provided with a 10-ft suction pipe. With such a suction pipe, $H_r + H_s$ will be slightly less than the value computed above. Had the appropriate method in Fig. 53 (array 4) been used, the following values of F'_c and F'_w would have been obtained.

$$F'_c = 154 \times 12 \ln\frac{2 \times 1{,}000}{340} = 3270$$

$$F'_w = 154 \times \left[12 \ln\left(\frac{2 \times 1{,}025}{340}\right) + \ln\frac{340}{12 \times 1}\right] = 3840$$

These values agree closely with those computed by the exact method.

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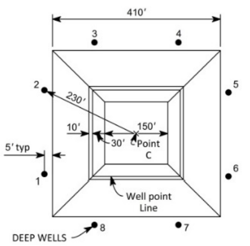
Figure C.3. Open excavation; deep wells; gravity flow

PROBLEM: Design a combined deep-well and wellpoint system to lower the groundwater to 2 ft below the bottom of the excavation. Use deep wells located 5 ft back of the edge of the excavation to lower the groundwater to permit the installation of a single stage of wellpoints for lowering the groundwater below the bottom of the excavation.



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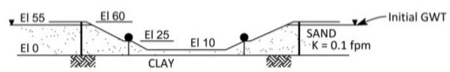
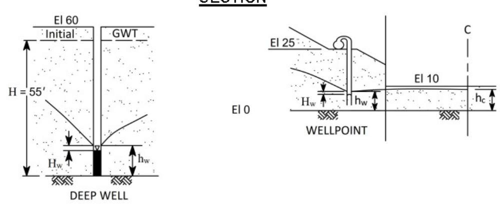


SECTION



Design of a combined deep-well and well and wellpoint system for dewatering a slope

SOLUTION:

Deep wells. The deep-well system must be designed such that the groundwater level is lowered 2 ft below the elevation at which the header pipes for the wellpoint system will be set. Set header pipes for wellpoint system at El 25. Required drawdown:

$$H - h_c = 55 - 23 = 32 \text{ ft}$$

Locate fully penetrating wells in a circular array around the perimeter of the excavation, A_e = 230 ft. Estimate radius of influence, R, from Fig. 58.

For k = 0.1 fpm and final drawdown, H- h_D = 55 – (10 – 2) = 47 ft, R = 3180 ft

Calculate flow to well system from a combination of Eq 3 (Fig. 47) and 2 (Fig. 48).

$$H^2 - h_c^2 = \frac{nQ_w \ln R/A_e}{\pi k}$$

$$(55)^2 - (23)^2 = \frac{nQ_w \ln 3180/230}{\pi 0.1}$$

$$nQ_w = 299 \text{ cfm} = 2233 \text{ gpm}$$

Try eight wells with radius, r_w = 1.0 ft (12-inch screen with 6-inch filter).

$$Q_w = \frac{299}{n} = \frac{299}{8} = 37.4 \text{ cfm} = 280 \text{ gpm}$$

Calculate drawdown at well from a combination of Eq 3 (Fig. 47) and 1 (Fig. 48).

$$H^2 - h_w^2 = \frac{Q_w}{\pi k} \ln\left(\frac{R^n}{n r_w A_e^{(n-1)}}\right) = \frac{Q_w}{\pi k}[n \ln R - \ln n\, r_w - (n-1)\ln A_e]$$

$$(55)^2 - h_w^2 = \frac{37.4}{\pi(0.1)}[8 \ln 3180 - \ln 8(1.0) - (8-1)\ln 230]$$

$$h_w = 11.1 \text{ ft}$$

Wells should have about 15 ft of 12-inch well screen. From Fig. 59, estimate H_w = 0.9 ft; $h_w - H_w$ = 11.1 – 0.9 = 10.2 ft.

Wellpoints. Use 3-ft slotted wellpoints with filter, r_w = 0.5 ft, and 2-inch riser pipes 21 ft long; set header pipe at El 25. Assume wellpoint pump vacuum equals 24 ft with 2-ft friction loss in header and pump suction set 2 ft above header pipe. Net vacuum in header pipe equals 20 ft. Install wellpoints 110 feet from centerline; from Eq 6 (Fig. 48). A_e = 140 ft.

Assume some head, h, at the line of wells; the flow to the combined system can be expressed as follows (Eq 3 (Fig. 47) and 2 (Fig. 48)).

$$H^2 - h^2 = \frac{nQ_w + Q_{p(T)}}{\pi k} \ln \frac{R}{A_e} \quad \text{(flow to line of wells)}$$

$$H^2 - h_c^2 = \frac{Q_{p(T)}}{\pi k} \ln \frac{R}{A_e} \quad \text{(flow from line of wells to wellpoints, } R = A_e \text{ for well, i.e., } R = 230 \text{ ft)}$$

Equate h^2 and solve for $Q_{p(T)}$

$$H^2 - h_c^2 = \frac{nQ_w + Q_{p(T)}}{\pi k} \ln \frac{R}{A_e} + \frac{Q_{p(T)}}{\pi k} \ln \frac{R}{A_e}$$

In order to prevent excessive drawdown at the wells, with both wells and wellpoints operating, reduce Q_w by 50 percent. Then Q_w = 0.50(37.4) = 18.7 cfm.

$$(55)^2 - (8)^2 = \frac{8(18.7) + Q_{p(T)}}{0.1\pi} \ln \frac{3180}{230} + \frac{Q_{p(T)}}{0.1\pi} \ln \frac{230}{140}$$

$$Q_{p(T)} = 172 \text{ cfm} = 1287 \text{ gpm}$$

The flow per foot of header is

$$Q_p = \frac{Q_{p(T)}}{\text{length}} = \frac{1287}{4(220)} = 1.46 \text{ gpm/ft}$$

Assume a wellpoint spacing (a) of 8 ft. Thus the flow per wellpoint, Q_w, is: Q_w = 8(1.46) = 11.7 gpm = 1.56 cfm

Compute head at wellpoint, h_w, from Eq 1 (Fig. 56) ($h_e = h_p$)

$$h_c^2 - h_w^2 = \frac{Q_w}{\pi k} \ln \frac{a}{2\pi r_w}$$

$$(8)^2 - h_w^2 = \frac{1.56}{0.1\pi} \ln \frac{8}{2\pi(0.5)}$$

$$h_w = 7.7 \text{ ft}$$

For Q_w = 11.7 gpm, the hydraulic head losses are as follows:

$H_e = 0.1$ ft,	from Fig. 60a, curve 5
$H_s = 1.0$ ft,	from Fig.60b
$H_v + H_r = 0.4$ ft,	from Fig. 60c
$H_w = 1.5$ ft	

Thus, $h_w - H_w = 7.7 - 1.4 = 6.2$ ft

Therefore the required effective vacuum at the header = El 25 – 6.2 = 18.8 ft. Since this is less than the available 20 ft, a wellpoint spacing of 8 ft with the header at El 25 and the top of the wellpoint screen at El 4 would be satisfactory. Calculated drawdown at well from Eq 3 (Fig. 47) and 1 (Fig. 48).

$$H^2 - h_w^2 = \frac{Q_w}{\pi k}[n \ln R - \ln n\, r_w - (n-1)\ln A_e] + \frac{Q_{p(T)}}{\pi k} \ln \frac{R}{A_e}$$

$$(55)^2 - h_w^2 = \frac{18.8}{0.1\pi}[8 \ln 3180 - \ln 8\,(1.0) - (8-1)\ln 230] + \frac{172}{0.1\pi} \ln \frac{3180}{230}$$

$$h_w = 11.3 \text{ ft}$$

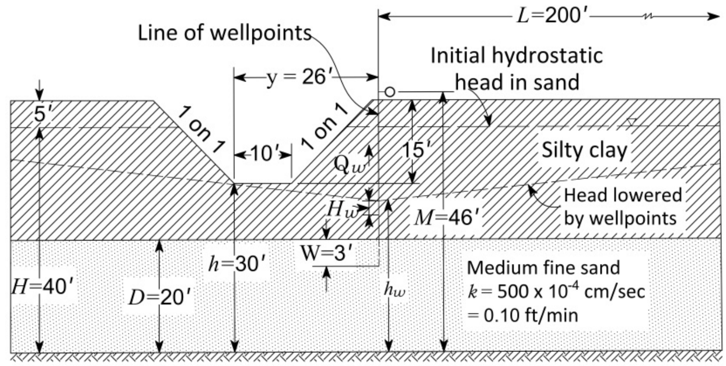
From Fig. 59, estimate H_w = 0.7 ft: $h_w - H_w = 11.3 - 0.7 = 10.6$ ft

In order to provide adequate pump submergence, set deep-well pump at El 3. (Since the actual drawdown in a well may be greater than the computed drawdown, it is generally advisable to set the pump intake not less than 7 to 10 ft below the computed drawdown in the well.)

Figure C.4. Open excavation; combined deep-well and wellpoint system; gravity flow

PROBLEM: Determine required spacing of 2-1/2-inch-ID 0.35-inch long, style CB self-jetting wellpoints with 2-inch-ID riser pipes to lower hydrostatic head to bottom of trench. The wellpoint has a wire mesh wellscreen. Assume effective vacuum at top of riser pipe = 20 ft, L = 200 ft, and r_w = 0.104 ft.

SOLUTION: Use a single line of wellpoints at top of excavation, one stage being required. For W/D = 3/20 = 0.15, λ = 0.82 from Fig. 38b; therefore λD = 0.82 × 20 = 16.4 ft.



Maximum h at trench = 30 ft. Assume this value of h at the far edge of the trench, a distance y of 26 ft from the line of wellpoints. Compute the required h_e from Eq 2 (Fig. 38) as follows:

$$30 = h_e + (40 - h_e)\frac{26 + 16.4}{200 + 16.4} \quad \text{or} \quad h_e = 27.7 \text{ ft}$$

The flow Qp per unit length of system as computed from Eq 1 (Fig. 38) is

$$Q_p = \frac{2 \times 0.1 \times 20 \times 1 \times (40 - 27.7)}{200 + 16.4} = 0.23 \text{ cfm} = 1.7 \text{ gpm per ft of trench}$$

Compute Δh_w from Eq 1 (Fig. 54), h_w from Eq 2 (Fig. 55), and H_w from Fig. 60, and select a so that $h_w - H_w \geq 26$ ft (M minus the vacuum at the top of the riser pipe).

a	Q_w	Δh_w	h_w	Head Loss in Wellpoint, ft			H_w	$h_w - H_w$
ft	cfm	ft	ft	H_s†	H_e‡	$H_r + H_v$§		ft
10	2.3	0.50	27.2	1.75	0.22	0.87	2.84	24.4
8	1.8	0.36	27.3	1.16	0.17	0.54	1.87	25.4
6	1.4	0.25	27.5	0.74	0.13	0.34	1.21	26.3

† From Fig. 60b.
‡ From Fig. 60a, assuming H_e same as that give by curve 7.
§ From Fig. 60c, assuming C = 110.

Thus, a spacing of 6 ft would be required, since $h_w - H_w$ should not be less than 26 ft. The tops of the wellpoint screens would be set slightly below the top of the sand stratum.

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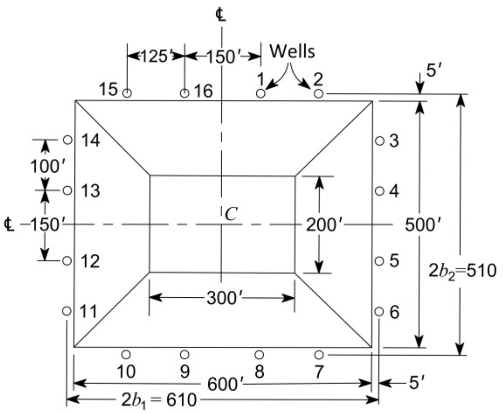
Figure C.5. Trench excavation; pressure relief by wellpoints; artesian flow

PROBLEM: Determine the number of 10-inch diameter wells with 6-inch gravel filter required to lower the head in the sand stratum 5 ft below bottom of excavation, for wells located at the top of slope and pumped by deep-well turbine pump (assume r_w = 1.0 ft). Use a fully penetrating system of wood-stave wells with 3/16-inch slots and a gravel filter with D_{10} size = 0.25 mm. Area of slot ≈ 10 percent of circumferential area of well screen. Geologic and soil conditions indicate a circular source of seepage k = 1,300 × 10^{-4} cm/sec or 2,560 × 10^{-4} fpm.



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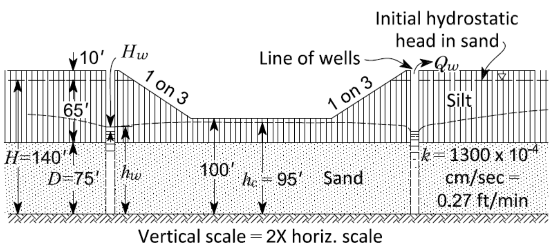

SECTION

SOLUTION: Determine equivalent radius A_e of well system from Eq 6 (Fig. 48) with wells located 5 ft from crown of slope

$$A_e = \frac{4}{\pi}\sqrt{b_1 b_2} = \frac{4}{\pi}\sqrt{\left(\frac{610}{2}\times\frac{510}{2}\right)} = 355 \text{ ft}$$

From Fig. 58, R ≈ 4,870 ft for k = 1,300 × 10^{-4} cm/sec and H – h_w = 45 ft. Compute total required flow, Q_T, from Eq. 2 (Fig. 44) for h_w = 95 ft and r_w = A_e = 355 ft.

$$Q_T = \frac{2\pi kD(H-h_w)}{\ln (R/r_w)} = \frac{2\pi(0.256)(75)(140-95)}{\ln (4{,}870/355)} = 2{,}073 \text{ cfm}$$
$$= 15510 \text{ gpm} \approx 16{,}000 \text{ gpm}$$

As 1,000 gpm is about the maximum that can be pumped in a normal 10-in. deep well pump, 16 wells would be required. Try spacing shown on plan. Make computations for the 4 wells in one quadrant and multiply the results by 4.

For 4 wells: drawdown at center of excavation H – h_c is determined from Eq 1 and 2 (Fig. 44) (R_i = R): $Q_{wi} = \frac{15{,}510}{16} = 970 \approx 1{,}000 \text{ gpm} = 134 \text{ cfm}$

Well	R, ft	r_i, ft	$\ln\frac{R}{r_i}$
1	4,870	266	2.91
2	4,870	324	2.71
3	4,870	352	2.63
4	4,870	314	2.74
			∑ = 10.99

$$H - h_c = \frac{\sum_{m=1}^{m=4} Q_{wi}\ln\frac{R_i}{r_i}}{2\pi kD} = \frac{134(10.99)}{2\pi(0.256)(75)} = 12.2 \text{ ft}$$

For 16 wells:

$$H - h_c = 4(12.2) = 48.8 \text{ ft}$$

or

$$h_c = 140 - 48.8 = 91.2 \text{ ft}$$

Since the maximum allowable h_c is 95 ft, the system shown in plan is adequate. The approximate head h_w at a well is computed from Eq 1 (Fig. 54) using an average well spacing, a, of 2 × (510 + 610) / 16 = 140 ft.

$$\Delta h_w = \frac{Q_w}{2\pi kD}\ln\frac{a}{2\pi r_w} = \frac{134}{2\pi(0.256)75}\ln\frac{140}{2\pi(1.0)} = 3.4 \text{ ft;}$$

or $h_w \cong 91.2 - 3.4 = 87.8$ ft

Hydraulic head losses in wells are obtained from Fig. 59, assuming intake of pump is about 85 ft above the bottom of the sand.

$H_e = 0.26$ ft (from Fig. 59a, Q_w=1,000 gpm/75 ft = 13.3 gpm/ft)
$H_s = 0.37$ ft (from Fig. 59b, screen length of 0.5(75) = 37.5 ft)
$H_r = 0.07$ ft (from Fig. 59b, for 10 ft of riser pipe and C = 130)
$H_v = 0.26$ ft (from Fig. 59c)
$H_w = 0.96$ ft

Thus, the water surface in the wells would be about 87.8 – 1.0 = 86.8 ft above the bottom of sand. Set pump bowl about 85 ft above bottom of sand and provide with 10-ft suction pipe.

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Figure C.6. Rectangular excavation; pressure relief by deep wells; artesian flow

PROBLEM: Determine the number of 10-in. diameter wells with 6-in. gravel filter required to lower the head in the sand stratum 5 ft below bottom of excavation, for wells located at the top of slope and pumped by deep-well turbine pump using a 2-D plan view finite element methods model and compare to results in Figure C.6.

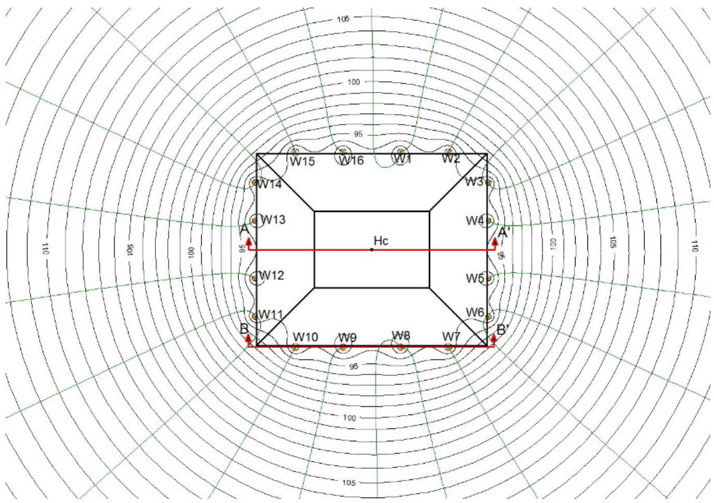
SOLUTION: The problem was modeled using SEEP/W v8.11.1.7283 (GeoStudio) to calculate the heads produced by pumping 16 wells at 1,000 gm each. A circular source of seepage with a constant head boundary of 140 feet was modeled with a radius of 5,000 ft from the center of the excavation. The mesh was auto generated by SEEP/W. The mesh density was varied across the model with a 1-ft mesh at the edge of the excavation increasing to a 100-ft mesh at the circular source.

Step 1: Model a head condition of 95 feet at the edge of the excavation. A total flow (Q_T) of 15,650 gpm was calculated. The difference in the Q_T calculated from the finite element methods model and the Q_T for an equivalent well (16,000 gpm) is due principally to the approximation made in Figure C.6 for the radius of the equivalent well. If well capacity is 1,000 gpm (Q_w), then the minimum number of wells is Q_T/Q_W => 15.4 wells ≈ 16 wells.

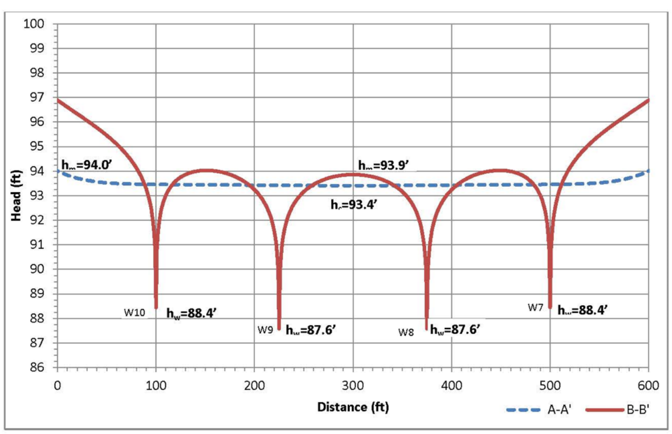
Step 2: Model 16 wells as single nodes with a nodal flow rate of 1,000 gpm to evaluate drawdown across the excavation. Head contours are shown in a) PLAN OF WELLS AND PIEZOMETRIC HEAD CONTOURS (PARTIAL).

The head values along Sections A-A' and B-B' in the plan are shown in b) HEAD PROFILE ALONG SECTIONS. In the section A-A' profile, note that the head calculated by the model at the center of the excavation (h_c) is 93.4 ft, only slightly lower than h_c calculated using the principle of superposition (93.6 ft) in Figure C.6.

In the original problem, the average head h_w at a wall during pumping (before accounting for hydraulic head losses in the wells) was calculated based on an average well spacing of 140 ft to be 90.3 ft, whereas the finite element methods model calculates this head to range from 87.6 ft to 88.4 ft for wells W7 through W10. This is very close agreement considering that the calculated h_w in Figure C.6 is based on a formula for h_w developed for an infinite line of equally spaced wells.



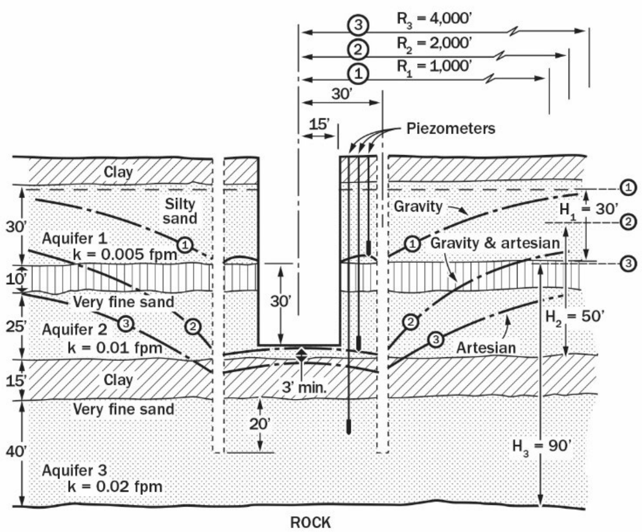
a) PLAN OF WELLS AND PIEZOMETRIC HEAD CONTOURS (PARTIAL)



b) HEAD PROFILE ALONG SECTIONS

Figure C.7. Rectangular excavation; pressure relief using deep wells; artesian flow; 2D plan view numerical model

PROBLEM: Design a deep well and vacuum system to dewater a 70-ft deep shaft to be sunk into stratified clays and sand below the groundwater table. Assume a ring of wells installed 15 ft out from perimeter of shaft with an equivalent radius of influence, A = 30 ft. Wells to fully penetrate the sand strata penetrated by the shaft and pumps to have a capacity in excess of the flow to each well. Vacuum to be maintained in wells equals 15 ft. Assume radius of influence of vacuum (R) to be the same for seepage (i.e. vacuum varies from that at well or wells to zero at R). Maximum height of shaft exposed at any one time equals 30 ft.



SOLUTION:

Aquifer 1. Compute flow of water to wells assuming gravity flow for hydrostatic head, and "equivalent" artesian flow for the additional head produced by the vacuum in the wells. Assume h_w = 2 ft.

Hydrostatic water flow, from Eq 2 (Fig. 45) (r_w = A)

$$Q_{T-H-1} = \frac{\pi k(h^3 - h_w^3)}{\ln R/A} = \frac{0.005\pi(30^2 - 2^2)}{\ln 1000/30} = 4.01 \text{ cfm}$$

Vacuum water flow, from Eq 2 (Fig 44) (r_w = A, effective aquifer thickness, $D = \frac{H + h_w}{2}$, and drawdown, $H - h_w = V$)

$$Q_{T-V-1} = \frac{2\pi k\left(\frac{H + h_w}{2}\right)V}{\ln R/A} = \frac{2(0.005)\pi\left(\frac{30+2}{2}\right)(15)}{\ln 1000/30} = 2.15\text{cfm}$$

Total water flow, aquifer 1 $Q_{T-1} = Q_{T-H-1} + Q_{T-V-1}$ = 4.01 + 2.15 = 6.16 cfm = 46.1 gpm

Aquifer 2. Compute the flow of water assuming combined artesian-gravity flow conditions for "Hydrostatic" water flow. Compute the additional flow caused by vacuum in wells assuming an equivalent artesian flow condition under the net vacuum head existing in the gravity flow region. Assume h_w = 2 ft.

Hydrostatic water flow, from Eq 1 (Fig 46):

$$Q_{T-H-2} = \frac{\pi k(2DH - D^2 - h_w^2)}{\ln R/A} = \frac{0.01\pi(2(25)(50) - (25)^2 - (2)^2}{\ln 2000/30} = 14.0 \text{ cfm}$$

Vacuum water flow; compute $\bar{R}$ from Eq 3 (Fig 46):

$$\ln \bar{R} = \frac{(D^2 - h_w^2)\ln R + 2D(H - D)\ln A}{2DH - D^2 - h_w^2} = \frac{(25^2 - 2^2)\ln 2000 + 2(25)(50-25)\ln 30}{2(25)(50) - (25)^2 - (2)^2} = 4.80$$

Then $\bar{R}$ = 121 ft.

Estimate vacuum at artesian-gravity flow boundary by plotting the vacuum versus log r.

(V = 15 ft at A = 30 ft; V = 0 at R = 2,000 ft), the vacuum is 10 ft at R = 121 ft. Thus, the net vacuum in the gravity flow region = 15 ft – 10 ft = 5 ft. Vacuum water flow from Eq 2 (Fig. 44).

$$Q_{T-V-2} = \frac{2\pi k\left(\frac{D + h_w}{2}\right)V}{\ln \bar{R}/A} = \frac{2(0.01)\pi\left(\frac{25+2}{2}\right)(5)}{\ln 121/30} = 3.04 \text{ cfm}$$

Total water flow, aquifer 2, $Q_{T-2} = Q_{T-V-2} + Q_{T-H-2}$ = 14.0 + 3.04 = 17.04 cfm = 127.5 gpm

Aquifer 3. For artesian flow, the head producing flow for the combined hydrostatic-vacuum system is $H + V - h_e$. Assuming the circular array of wells to be a continuous drainage slot, for which W/D = 50 percent and R/A = 133, it can be seen from Fig. 41 that the head in the center of the circular drainage slot approaches the head in the slot as R/A increases. Therefore, the flow to the wells for this situation can be computed from Eq. 2 (Fig. 44, in which $(H - h_w) = (H + V - h_e)$, G =1, and r_w = A.

$$Q_T = \frac{2\pi kD(H + V - h_e)}{\ln R/A} = \frac{2(0.02)\pi(40)(90 + 15 - 27)}{\ln 4000/30} = 80.1 \text{ cfm} = 599.4 \text{ gpm}$$

Total flow to well system = 46.1 + 127.5 + 599.4 = 773 gpm

Use 12 wells located 30 ft from the center of the shaft, with a spacing (a) between wells of 15.5 ft.

Flow per well, $Q_W = \frac{773}{12} = 64.4 \text{ gpm} = 8.61 \text{ cfm}$

Compute Δh_w for the artesian flow in Aquifer 3 to determine the required draw-down in the well. From Eq. 1 (Fig. 55 for values of θ_a).

$$\Delta h_w = \frac{Q_w \theta_a}{kD} = \frac{64 \, x \, 0.5}{7.5 \text{ x } 0.02 \text{ x } 40} = 5.33 \text{ ft}$$

Thus h_w = 57 - 5.3 = 51.7

Use 8-in well screens with 6-in thick filter surrounding the screen. Check screen hydraulics.

	Aquifer 1	Aquifer 2	Aquifer 3
Total flow	46.1 gpm	127.5 gpm	599.4 gpm
Well flow, Q_w	3.8 gpm	10.6 gpm	50.0 gpm
Wetted screen length	2.0 ft	2.0 ft	20.0 ft
Flow per ft of screen	1.9 gpm	5.3 gpm	2.5 gpm

From Fig. 59a, it can be seen that the well entrance losses (He) should be negligible.

Vacuum system. It is assumed that the overlying clay is a continuous impermeable formation, and that the quantity of air that may enter the aquifers through the clay is negligible compared to that which will enter through the exposed excavation surface. It is also assumed that only one aquifer will be exposed at a time, and that the permeability of the aquifers for the flow of air is effectively reduced by half due to the capillary water retained in the voids of soil following the lowering of the water table.

Compute the airflow to the wells from Eq 1 (Fig. 37). To obtain the shape factor construct a plan flow net of air from the shaft excavation to the wells, for which:

$$S = \frac{N_f}{N_e} = 0.6\ per\ well$$

$$Q_a = \Delta p(D - h_w)\frac{\mu_w}{\mu_a}$$

Aquifer 1

$$Q_a = 15ft(30ft - 2ft)\left(\frac{0.005}{2}\right)\left(\frac{2.359x10^{-5}lbsec/ft^2}{3.744x10^{-7}lbsec/ft^2}\right)0.6 = 39.7\ cfm/well$$

Total airflow = 12(39.7) = 47.6 cfm at the mean absolute pressure,

$\bar{p}$ of $\frac{34+(34-15)}{2}$ = 26.5 ft of water.

Compute the required vacuum pump capacity

$$Q_{a-vp} = Q_a\frac{\bar{p}}{34} = 476\left(\frac{26.5}{34}\right) = 371\ cfm$$

Aquifer 2

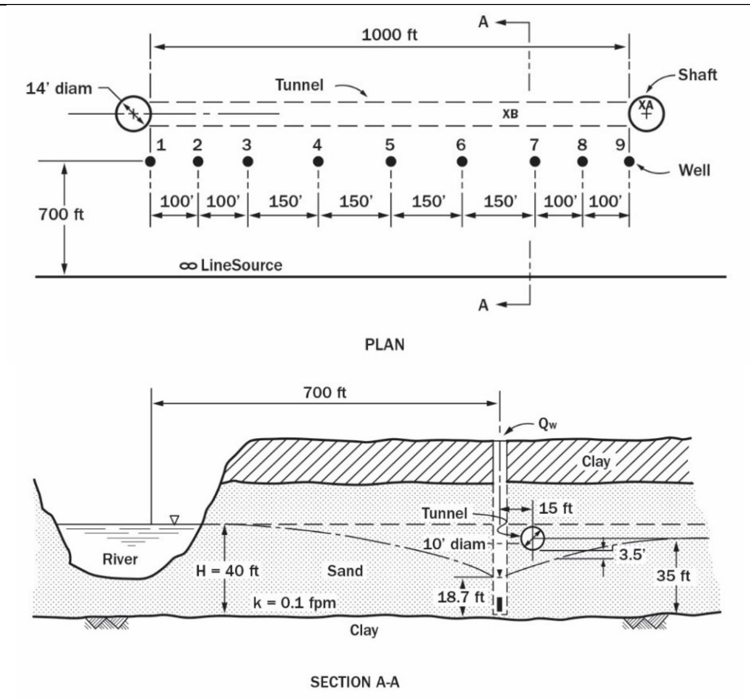
$$Q_a = 15ft(25ft - 2ft)\left(\frac{0.001}{2}\right)\left(\frac{2.359x10^{-5}lbsec/ft^2}{3.744x10^{-7}lbsec/ft^2}\right)0.6 = 65.2\ cfm/well$$

Total airflow = 12(65.2) = 783 cfm at $\bar{p}$ = 26.5 ft of water

$$Q_{a-vp} = 783\left(\frac{26.5}{34}\right) = 610\ cfm$$

Provide vacuum pumps with a total capacity of 610 cfm at 15 ft (of water) vacuum.

Figure C.8. Shaft excavation; artesian and gravity flows through stratified foundation; deep-well vacuum system



PROBLEM: Design a deep-well system to dewater an excavation for a 10 ft. diameter tunnel with 14 ft. diameter entrance shafts for the conditions shown. The deep-well system should lower the groundwater table 3.5 feet below the bottom of the tunnel. For a single-line source, use the method of image analysis. The layout, as shown, was determined from an approximate flow net sketched for preliminary design purposes. Point A is located at the center of one of the shafts and Point B is located in the center of the tunnel equal distances from wells 6 and 7.

SOLUTION: Assume 12-in fully penetrating wells with surrounding filter, r_w = 1.0 ft. For an assumed Q_w = 150 gpm, the drawdown at points A and B and at well 5 are computed from Eq. 2 and 3 (Fig. 52). Values of r_i and S_i in the table below are rounded to the nearest 10 feet.

$$\mathrm{H}^2 - h_p^2 = \frac{1}{\pi k} \sum_{\mathrm{i=1}}^{\mathrm{1=n}} \mathrm{Q_{wi}} \ln \frac{\mathrm{S_i}}{\mathrm{r_i}}$$

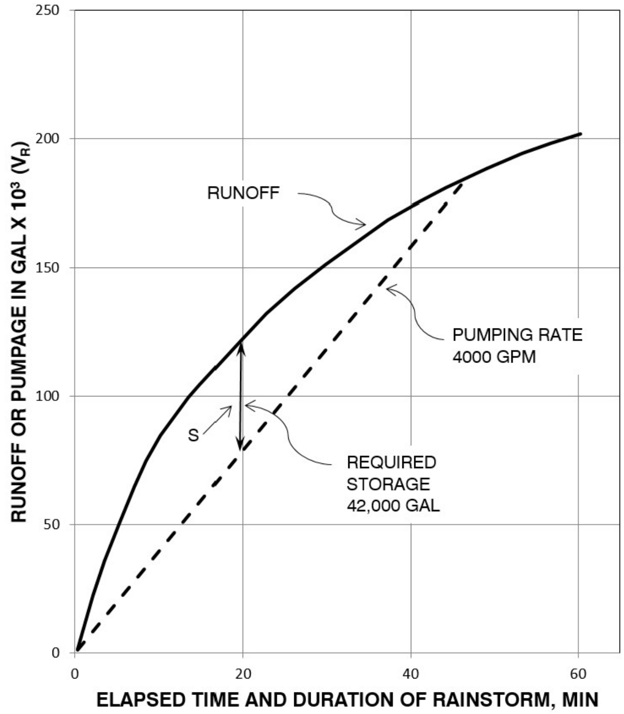
	At Point A			At Point B			At Well 5		
Well	r_i (ft)	S_i (ft)	ln S_i/r_i	r_i (ft)	S_i (ft)	ln S_i/r_i	r_i (ft)	S_i (ft)	ln S_i/r_i
1	1010	1740	0.54	730	1590	0.78	500	1490	1.09
2	910	1680	0.61	630	1540	0.89	400	1455	1.29
3	810	1630	0.70	530	1510	1.05	300	1430	1.56
4	660	1560	0.86	380	1465	1.35	150	1410	2.24
5	510	1500	1.08	230	1430	1.83	1	1400	7.24
6	360	1460	1.40	80	1415	2.87	150	1410	2.24
7	210	1430	1.92	80	1415	2.87	300	1430	1.56
8	110	1420	2.56	175	1425	2.10	400	1455	1.29
9	17	1415	4.42	27d5	1440	1.66	500	1490	1.09
Total			14.09			15.40			19.60

$\mathrm{h_p^2} = 1600 - \dfrac{150\ (14.09)}{0.1\pi\ (7.5)}$ $\mathrm{h_p^2} = 1600 - \dfrac{150\ (15.40)}{0.1\pi\ (7.5)}$ $\mathrm{h_p^2} = \mathrm{h_w^2} = 1600 = \dfrac{150\ (19.60)}{0.1\pi\ (7.5)}$

$\mathrm{h_p} = 26.5$ ft $\mathrm{h_p} = 24.9$ ft $\mathrm{h_w} = 18.8$ ft

For h_w = 18.8 ft, the flow per foot of well screen would be $\frac{150\ \mathrm{gpm}}{18.8} = 8.0$ gpm/ft, which is a satisfactory rate of inflow. The maximum allowable head at points A and B is 35 – 5 – 3.5 = 26.5. Thus, the system is adequate.

Figure C.9. Tunnel dewatering; gravity flows; deep-well system



PROBLEM: Determine sump and pump capacity to control surface water in an excavation, 4 acres in area, located in North Little Rock, AR., for a rainstorm frequency of 1 in 5 years and assuming c= 0.9. Assume all runoff to one sump in bottom of excavation.

SOLUTION:

$$V_R = cRA$$

Where:
V_R = runoff volume. ft^3
c = coefficient of runoff
R = rainfall depth, ft
A = drainage area, ft^2

From figure above and NOAA Atlas 14 Precipitation Frequency Data Server (Figure 34):

Rainstorm, min	R, in	V_R - (x 10^3 gal)
10	0.86	84
30	1.56	152
60	2.08	203

Assume sump pump capacity = 4000 gpm. From plot, required storage = 42,000 gal.

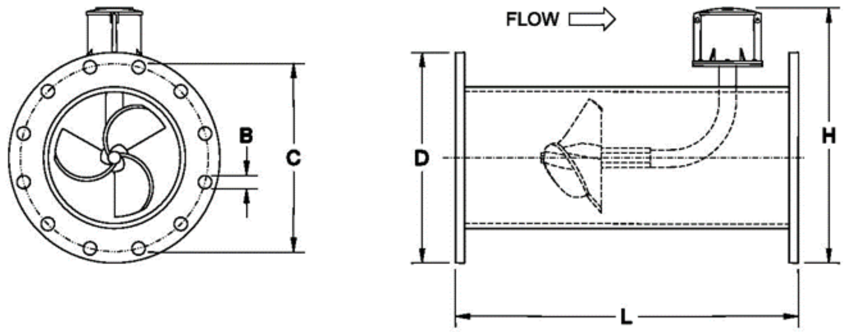
Figure C.10. Sump and Pump Capacity for Surface Runoff to an Excavation

Appendix D
Well and Total Discharge Measurements

D.1. General. This appendix contains discussions of typical ways that flows are measured for pumping tests and dewatering systems. The Bureau of Reclamation's *Water Measurement Manual* (1997) is a comprehensive textbook on water flow measurement that includes all of the flow measurement techniques presented in this appendix. The simplest method for determining the flow from a pump is to measure the volume of the discharge during a known period of time by collecting the water in a container of known size. However, this method is practical only for pumps of small capacity; other techniques must be used to measure larger flows.

D.2. Pipe-flow Measurements.

D.2.1 Propeller Flow Meters. Propeller flow meters require no external power source and are widely used for dewatering applications in pipes flowing full under pressure. They are relatively inexpensive and are manufactured in sizes ranging from 2 to 96 inches in diameter for flows between 40 and 75,000 gpm. They are normally designed for water flow velocities up to 17 ft/sec. All propeller meters have both instantaneous and totalizing flow indication. Their typical accuracy is ±2% of the actual flow with a typical turndown ranging from 10:1 to 15:1, as shown in Figure D.1. Turndown is the ratio of the maximum rated flow to the minimum rated flow of the meter. For example, a meter with a 15:1 turndown and a maximum rated flow of 1500 gpm is capable of accurately measuring flows between 100 and 1,500 gpm. Repeatability is typically ±0.25%. Repeatability is the ability of the meter to reproduce flow measurements under similar conditions and is a component of its accuracy. The pressure rating of most propeller meters is 150 psi and the maximum allowable fluid temperature is 160° F. As with all flow meters, the meter has to be installed according to the manufacturer's instructions to function properly at the rated accuracy. Usually these requirements specify that the pipe flow full and that there be at least five diameters of straight pipe upstream of the meter and at least one diameter of straight pipe downstream from the meter. They can be mounted either horizontally or vertically. Small changes in frictional resistance of the bearings can cause large decreases in accuracy, especially at lower flows. It is advisable to require that propeller meters be new or recently calibrated by the factory. Propeller meters should be selected to operate near the middle of their design discharge range, where accuracies are better than the rated accuracy for the full range of the meter.

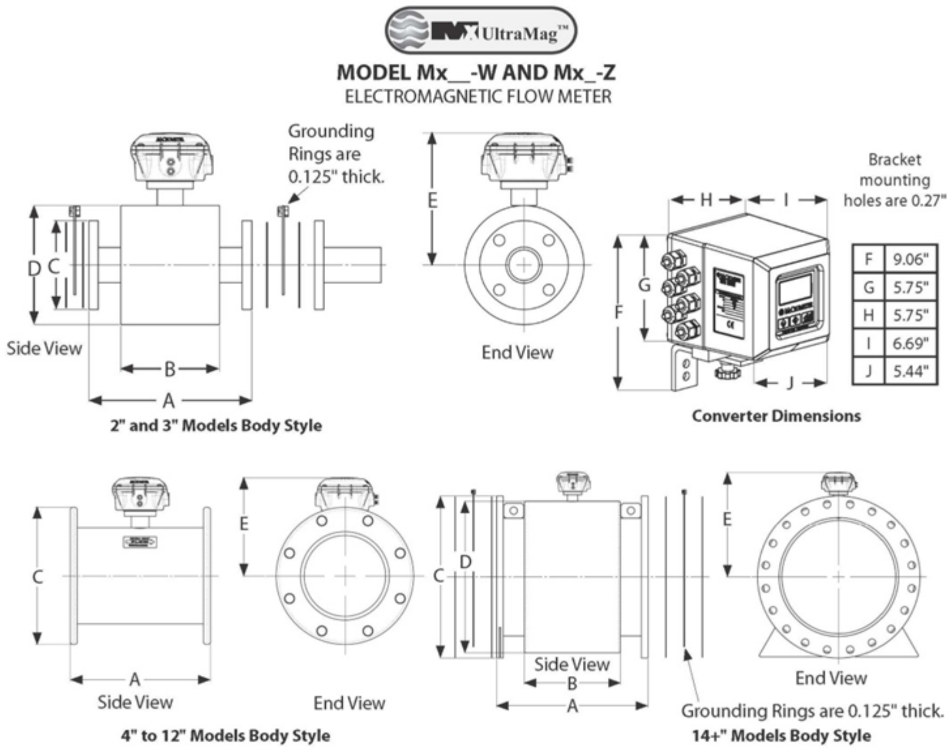


MF100	DIMENSIONS							
Meter and Nominal Pipe Size (inches)	2	2 1/2	3	4	6	8	10	12
Maximum Flow U.S. GPM	250	250	250	600	1200	1500	1800	2500
Minimum Flow U.S. GPM	40	40	40	50	90	100	125	150
Standard Dial Face (GPM/GAL)	250/ 10	250/ 10	250/ 10	800/ 100	1300/ 100	2500/ 100	3000/ 1000	4000/ 1000
Head Loss in Inches at Max. Flow	29.50	29.50	29.50	23.00	17.00	6.75	3.75	2.75
Shipping Weight, lbs.	40	40	40	50	60	102	157	176
B (inches)	3/4	3/4	3/4	3/4	7/8	7/8	1	1
C (inches)	4 3/4	5 1/2	6	7 1/2	9 1/2	11 3/4	14 1/4	17
D (inches)	6	7	7 1/2	9	11	13 1/2	16	19
H (inches)	12.16	12.66	12.91	13.66	16.03	17.28	22.53	24.03
L (inches)	13	13	13	20	20	20	20	20
No. of Bolts Per Flange	4	4	4	8	8	8	12	12

Larger flowmeters on special order. *McCrometer reserves the right to change design or specification without notice.*

Figure D.1. Propeller Flow Meter Ranges and Dimensions (courtesy of McCrometer Inc., Hemet, CA)

D.2.2 Electromagnetic Flow Meters. Electromagnetic flow meters require power to operate and a transmitter to record or send signals to stations or a data logger. They are available in a wide variety of sizes and are generally accurate to ±0.5% of the measured flow over a very wide range of velocities. The principle of operation is that a voltage is induced in an electrical conductor moving through a magnetic field. The conductor is the water in the case of magnetic flow meters. For a given field strength, the magnitude of the voltage is proportional to the velocity of the conductor. Most magnetic meters consist of a nonmagnetic and nonelectrical tube or pipe through which the water flows. Two magnetic coils are used, one on each side of the pipe. Two electrodes in each side of the insulated pipe wall sense the voltage induced by the flow of the water through the pipe. Some manufacturers sell agricultural saddle-mounted magnetic meters with a claimed accuracy of ±1% with 5-year batteries. All of the magnetic meters have telemetry capability and can transmit rate-of-flow signals in a variety of ways. The water needs to have sufficient conductivity, but other properties do not change the calibration. The principal advantages of magnetic meters are their reliability, ruggedness (because there are no moving parts), and accuracy. Disadvantages include the necessity for a power source and the need to mount the signal converter above potential flood levels to prevent damage of the signal converter (the most expensive meter component) due to immersion. See Figure D.2 for dimensions and flow ranges of various sizes of electromagnetic flow meters.



Pipe Size (Nominal)	Meter Pipe ID	Flow Ranges GPM Standard .2 to 32 FPS Min - Max	DIMENSIONS (Lay Lengths)							Estimated Shipping Weight (lbs.)	
			A*		B	C		D	E		
			Mx_-W	Mx_-Z		Mx_-W	Mx_-Z			Mx_-W	Mx_-Z
2"	2.117	2 - 340	11.00	11.00	6.70	6.00	6.50	7.90	9.26	93	107
3"	3.220	5 - 730	13.40	13.40	6.70	7.50	8.25	9.40	10.01	97	111
4"	3.720	8 - 1,140	13.40	13.40	n/a	9.00	10.00	n/a	8.06	78	108
6"	5.692	19 - 2,660	14.60	14.60	n/a	11.00	12.50	n/a	9.06	82	138
8"	7.692	33 - 4,870	16.10	17.25	n/a	13.50	15.00	n/a	10.06	115	195
10"	9.682	52 - 7,670	18.50	18.50	n/a	16.00	17.50	n/a	10.46	144	247
12"	11.682	74 - 11,180	19.70	19.70	n/a	19.00	20.50	n/a	12.31	193	342
14"	13.440	90 - 16,070	21.70	22.75	12.00	21.00	23.00	20.30	15.46	321	476
16"	15.440	118 - 20,900	23.60	25.25	14.20	23.50	25.50	21.10	16.21	390	645
18"	17.440	150 - 26,480	23.60	25.25	14.20	25.00	28.00	21.10	17.21	446	750
20"	19.440	185 - 32,720	25.60	28.25	16.20	27.50	30.50	24.80	18.26	588	874
24"	23.440	270 - 47,180	30.70	35.75	21.70	32.00	36.00	29.60	20.11	769	1,568
30"	29.190	420 - 73,620	35.80	41.75	26.50	38.75	43.00	35.90	23.26	1,261	2,317
36"	35.190	610 - 105,930	46.10	46.10	28.20	46.00	50.00	42.70	26.66	1,696	2,915
42"	41.190	830 - 144,370	48.05	**	32.10	52.75	**	48.35	29.99	**	**
48"	47.190	1,080 - 188,430	50.00	**	36.00	59.50	**	54.00	33.31	**	**

* Laying lengths for meters with ANSI Class 150 Flanges are equal to Mx__-Z laying lengths
** Consult factory

Figure D.2 Electromagnetic Flow Meter Ranges and Dimensions (courtesy of McCrometer Inc., Hemet, CA)

D.2.3 Portable Ultrasonic ("Doppler") Flow Meters. Portable ultrasonic or Doppler flow meters are useful where there are a variety of pipe sizes and flows to be measured and it is not necessary to have a continuous record of flow at a particular well or location. This type of flow meter measures the velocity of particles moving with the flowing water. Acoustic signals are transmitted, reflected from the particles, and are picked up by a receiver. The signals are analyzed for frequency shifts and the resulting mean value of the frequency shifts is related to the velocity of the particles moving with the water. Figure D.3 shows a portable ultrasonic flow meter being used to measure flow in a 12-inch diameter pipe on a project where flow measurements in pipe as small as 2-inch diameter were also being made using the same instrument. The typical accuracy of an ultrasonic meter installed according to the manufacturer's recommendations is ±1% of measured flow for water velocities greater than 1.5 ft/sec. The meter can be used for measuring flow in pipes that are made of almost any material and with a wide range of wall thicknesses, including linings. The length of straight pipe upstream of ultrasonic flow meters should be at least 10 pipe diameters.

Figure D.3. Portable Krohne Optisonic 6300 medium ultrasonic flow meter strapped to 12-inch diameter discharge pipe; this model is rated for measuring flow in 2- through 16-inch diameter pipes (Courtesy of AECOM)

D.2.4 Paddle-wheel Flow Meters. Paddle-wheel flow meters are typically available in tee-mounted configurations for pipe diameters between ½ and 8 inches (although they are available for pipe diameters up to 36 inches) and measure the velocity of flow using a paddle wheel that rotates due to the water moving past it similar to the rotation of the paddle wheel of a river boat. These meters are rugged and reliable, with a typical accuracy of ±1% of the maximum flow, a flow velocity range between 1 and 20 ft/sec (a turndown of 20:1). Abrasives such as sand in the water will cause bearing wear that will reduce the accuracy of the meter. They are self-powered, so no external source of power is necessary.

D.2.5 Venturi Meter. The flow from a dewatering system can be accurately measured by means of a venturi meter installed in the discharge line. In order to obtain accurate measurements, the meter should be located about 10 pipe diameters from any elbow or fitting, and the pipe must be flowing full of water. The flow through a venturi meter can be computed from the equation below. The pressures h_1 and h_2 may be taken using a venturi meter for low pressures, or by a differential mercury manometer for high pressures. Gages may be used but will be less accurate.

$$Q = 3.12CA = \frac{\sqrt{2g(h_1 - h_2)}}{\sqrt{1 - R^4}} \tag{D.1}$$

Where:

$$3.12 = \text{conversion factor} = \frac{7.48\ \text{gal/ft}^3 \ \times 60\ \text{sec/min}}{144\ \text{in.}^2/\text{ft}^2}$$

Q	=	flow, gallons per minute
C	=	calibrated coefficient of discharge (usually about 0.98)
A	=	area of entrance section where upstream manometer connection is made (square inches)
g	=	acceleration of gravity (32.2 feet per second squared)
h_1-h_2	=	difference in pressure between entrance section and throat, as indicated by manometer (feet)
R	=	ratio of entrance to throat diameter = D_1/D_2

D.2.6 Orifices.

D.2.6.1 The flow from a pipe under pressure can be conveniently measured by installation of an orifice on the end of the pipe or by insertion of an orifice plate between two flanges in the pipe. The pressure tap back of the orifice should be drilled at right angles to the inside of the pipe and should be perfectly smooth. A rubber tube and glass or plastic pipe may be used to measure the pressure head. The diameter of the orifice plate should be accurate to 0.01 inch; the edge of the plate should be square and sharp, should have a thickness of 1/8 inch, and should be chamfered at 45 degrees. The approach pipe must be smooth, straight, and horizontal; it must flow full, and the orifice should be located at least eight pipe diameters from any valves or fittings. The flow for various sized cap orifice-pipe combinations can be obtained from Figure D.4.

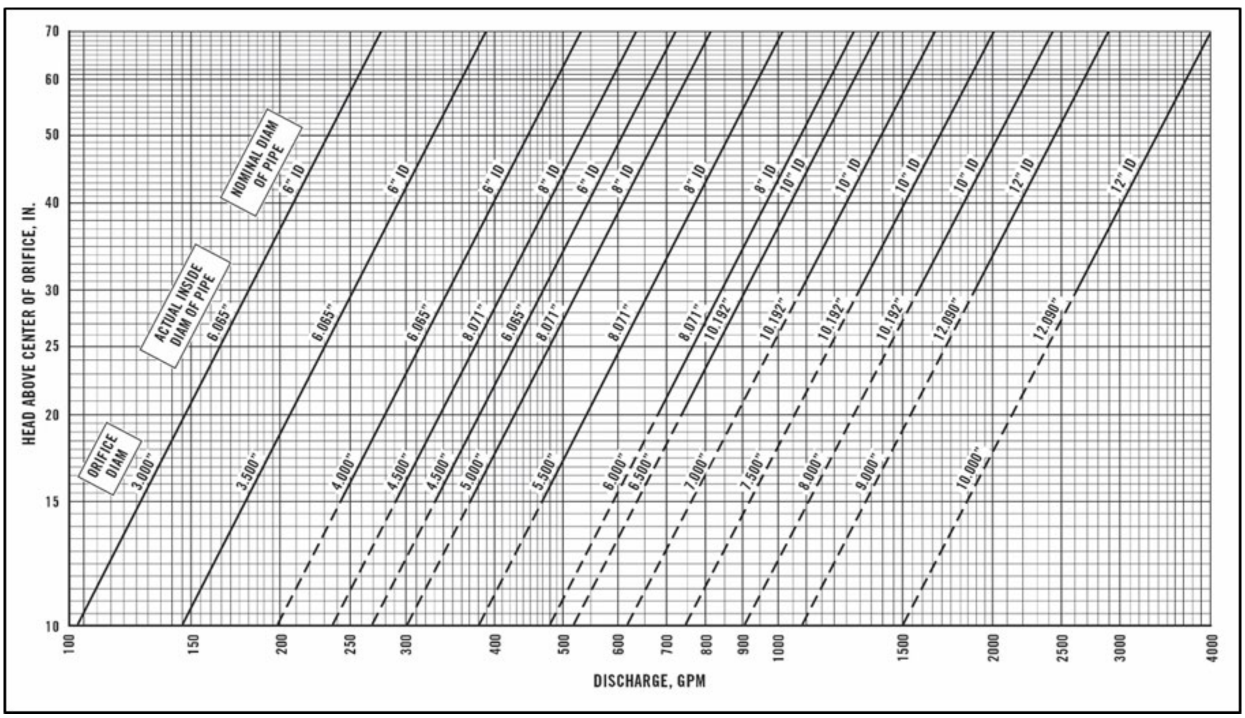


Figure D.4. Pipe cap orifice chart (Used with permission of Pentair Flow Technologies, LLC. All rights reserved. Originally published in 16th edition of the Fairbanks Morse Hydraulic Handbook)

D.2.6.2 The flow through an orifice in a pipe can be computed from

$$Q = \frac{CA_2\sqrt{2gh}}{\sqrt{1 - \left(\frac{d_2}{d_1}\right)^4}} \tag{D.2}$$

Where:

Q	=	capacity (cubic feet per second)
C	=	orifice discharge coefficient
A_2	=	area of orifice (square feet)
d_2	=	orifice diameter (inches)
d_1	=	pipe diameter (inches)
g	=	32.2 feet per second squared
h	=	pressure drop across the orifice in feet of head

The expression $\sqrt{1 - \left(\frac{d_2}{d_1}\right)^4}$ corrects for the velocity of approach. The reciprocal of this expression and the coefficient C are listed in Table D.1 for various values of d_2/d_1.

Table D.1

Velocity Approach Expression

d_2/d_1	C	$\dfrac{1}{\sqrt{1-\left(\frac{d_2}{d_1}\right)^4}}$
0.25	0.604	1.002
0.30	0.605	1.004
0.35	0.606	1.006
0.40	0.606	1.013
0.50	0.607	1.033
0.60	0.608	1.072
0.70	0.611	1.146
0.80	0.643	1.301
0.90	0.710	1.706

Note: The diameter of the orifice should never be larger than 80 percent of the pipe diameter in order to obtain a satisfactory pressure reading.

D.2.7 Pitot Tube. The flow in a pipe flowing full can also be determined by measuring the velocity at different locations in the pipe with a pitot tube and differential manometer and computing the flow. The velocity at any given point can be computed from

$$V = C\sqrt{2gh_v} \tag{D.3}$$

Where:

V = velocity

C = meter coefficient

g = acceleration of gravity

h_v = velocity head

The flow is equal to the area of the pipe A times the average velocity V, or

$$Q = AV \tag{D.4}$$

Where

$$V = \frac{\sum_o^n v}{n}$$

and

v= velocity at center of concentric rings of equal area

n= number of concentric rings

D.3. Approximate Measurement Methods.

D.3.1 Jet Flow.

D.3.1.1 Flow from a pipe can be determined approximately by measuring a point on the arc of the stream of water emerging from the pipe (Figure D.5), using the following equation:

$$Q = \frac{3.61Ax}{\sqrt{y}} \tag{D.5}$$

Where:

Q = flow (gallons per minute)

A = area of stream of water at end of pipe (square inches). If the pipe is not flowing full, the value of A is the cross-sectional area of the water jet where it emerges from the pipe. The area of the stream can be obtained by multiplying the area of the pipe times the Effective Area Factor (EAF) in Figure D.6 using the ratio of the freeboard F to the inside diameter of the pipe D.

x = distance along axis of the discharge pipe through which the stream of water moves from the end of the pipe to a point (S) (inches)

y = distance perpendicular to the axis of the discharge pipe through which the stream of water drops, measured from the top or surface of the stream of water to point (S) (inches)

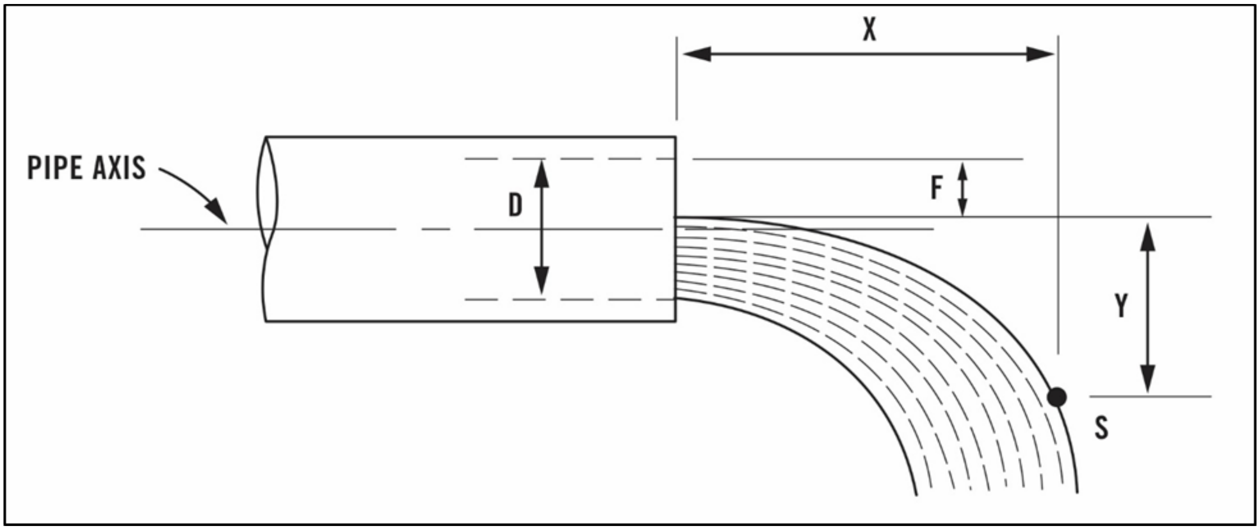


Figure D.5.Flow from pipe

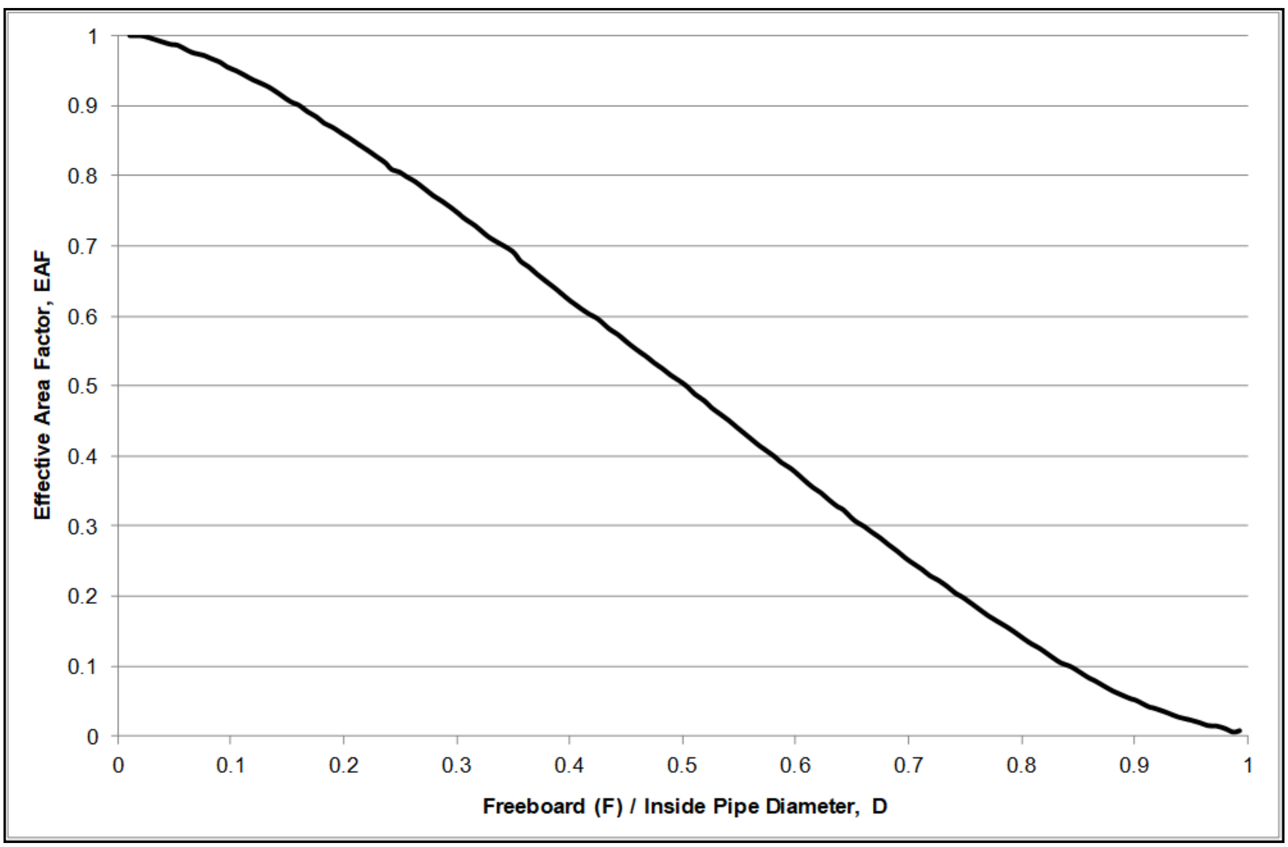


Figure D.6. Effective area factor for partially filled pipe

D.3.1.2 It should be noted that the x and y distances are measured from the top of the stream of water; if y is measured in the field from the top of the pipe, the pipe thickness and freeboard must be subtracted from the measured y to obtain the correct value of y.

D.3.2 Fountain Flow. The flow from a vertical pipe can be approximated by measuring the height of the stream of water above the top of the pipe (Figure D.7). Two types of flow must be recognized when dealing with fountain flow. At low crest heights, the discharge has the character of weir flow, while at high crest heights the discharge has the character of jet flow. Intermediate values result in erratic flow with respect to the height of the crest H.

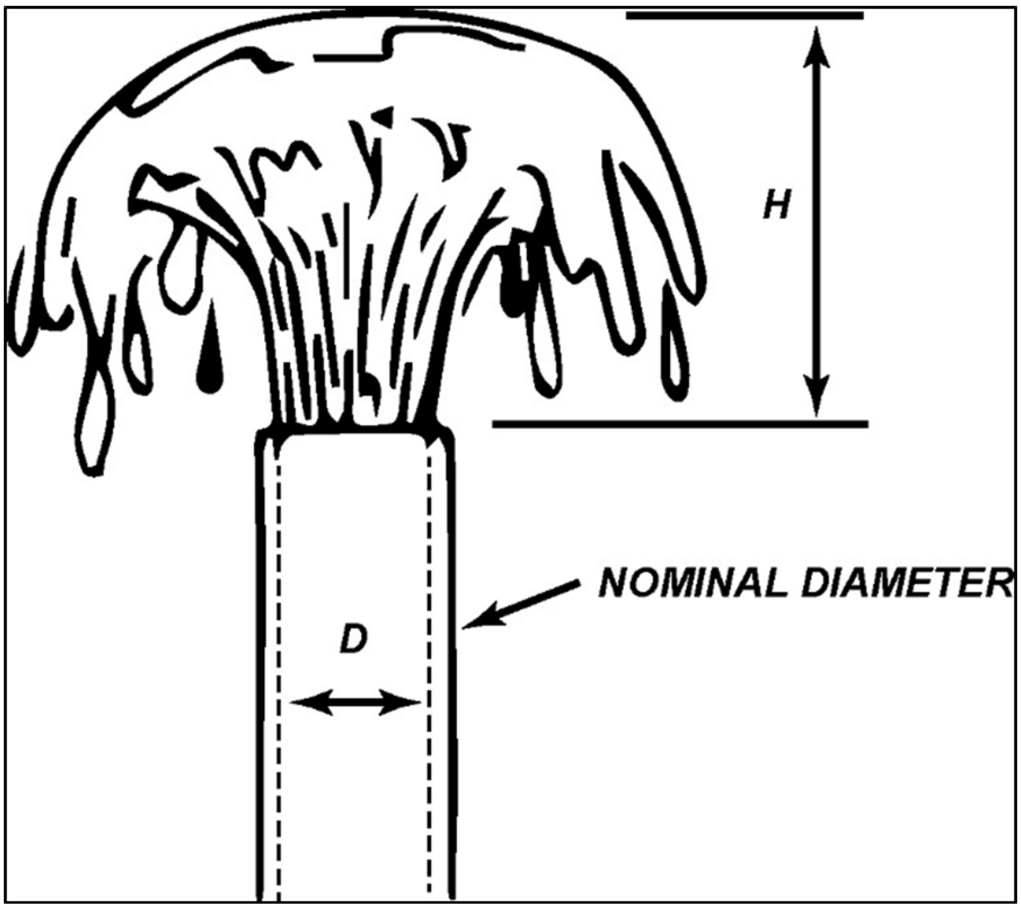


Figure D.7. Fountain flow measurement

D.3.2.1 Where the flow exhibits jet character

$$Q = 5.68KD^2\sqrt{H} \tag{D.6}$$

Where:

Q = flow (gpm)

K = constant varying from 0.87 to 0.97 for pipes 2 to 6 inches in diameter and H = 6 to 24 inches

D = inside pipe diameter (inches)

H = vertical height of water jet (inches)

Where the flow exhibits weir characteristics, it can be approximated by using the Francis Formula,

$$Q = 3.33BH^{3/2} \tag{D.7}$$

Where:

Q = flow (cfs)

B = pipe circumference (feet)

H = vertical height of water jet (feet)

D.3.2.2 Some values of fountain flow for various nominal pipe sizes and heights of crest are given in Table D.2.

Table D.2

Flow (gpm) from Vertical Pipes

Height of Crest, H	Nominal Diameter of Pipe (inches)					
Inches	2	3	4	5	6	8
1-1/2	22	43	68	85	110	160
2	26	55	93	120	160	230
3	33	74	130	185	250	385
4	38	88	155	230	320	520
5	44	99	175	270	380	630
6	48	110	190	300	430	730
8	56	125	225	360	510	900
10	62	140	255	400	580	1050
12	69	160	280	440	640	1150
15	78	175	315	500	700	1300
18	85	195	350	540	780	1400
21	93	210	380	595	850	1550
24	100	230	400	640	920	1650

Reference: U.S. Army Corps of Engineers (TM 5-818-5)

D.4. Open Channel and Partially full Pipe Flows.

D.4.1 Weirs. Flow in open channels can be measured using weirs constructed in the channel. Typical weir shapes are trapezoidal, square and V-notch. Weirs typically are more accurate than flumes for measuring low flow rates. Refer to EM 1110-2-1908 for details on weirs.

D.4.2 Parshall Flumes. Flow in an open channel may also be measured using a Parshall flume. A Parshall flume includes a restricted shape in the channel which is used to measure flow rates. Refer to EM 1110-2-1908 for details on Parshall flumes.

D.4.3 Measuring Flow in Partially Filled Pipe. In partially filled pipes, open channel flow measurement methods must be used. Certain manufacturers, including Krohne, have developed electromagnetic meters that will accurately measure the flow rate in partially filled pipe (between 10% and 100% of the pipe area) using sensors in the wall of the meter to measure the level in the pipe. The claimed accuracy is ±1% of maximum flow for partially filled pipe and ±1% of the measured flow for 100% full pipe. Several other manufacturers have developed flow measuring systems that involve insertion of an instrument into the pipe that measures flow velocity and water level to calculate the flow in partially full pipe. The accuracy and repeatability of flow measurements made using these latter devices has been generally marginal to poor, even when installed as recommended by the manufacturer in the presence of a qualified manufacturer's representative. A possible solution to the accuracy/repeatability problem of such instruments is to install a prefabricated flume in a pipeline. PlastiFab, Inc, Tualatin, OR is a resource for the design and manufacture of a variety of prefabricated flumes, including Parshall, Palmer-Bowlus, Trapezoidal, Cutthroat and H-flumes, each available with a wide range of end fittings and accessories to satisfy varying installation requirements.

Appendix E

Example Dewatering Specifications

SECTION TABLE OF CONTENTS

DIVISION 31 - EARTHWORK

SECTION 31 23 19.13

DEWATERING

-- END OF SECTION TABLE OF CONTENTS --

SECTION 31 23 19.13

DEWATERING

PART 1 GENERAL

1.1 REFERENCES

THE PUBLICATIONS LISTED BELOW FORM A PART OF THIS SECTION TO THE EXTENT REFERENCED. THE PUBLICATIONS ARE REFERRED TO WITHIN THE TEXT BY THE BASIC DESIGNATION ONLY.

ASTM INTERNATIONAL (ASTM)

ASTM C150/C150M (2016; E 2016) STANDARD SPECIFICATION FOR PORTLAND CEMENT

1.2 SCOPE OF WORK

WORK UNDER THIS SECTION INCLUDES ALL DEWATERING EFFORTS ASSOCIATED WITH EXCAVATIONS AT THE AUXILIARY DAM AND MAIN DAM. THIS INCLUDES ALL DEWATERING FOR REMOVAL AND BACKFILLING THE BOREL CONDUIT OUTLET WORKS AND PORTIONS OF THE BOREL CANAL DOWNSTREAM AND UPSTREAM OF THE ISABELLA AUXILIARY DAM, EMBANKMENT AND FOUNDATION EXCAVATIONS AND BACKFILLING OF THEAUXILIARY DAM DOWNSTREAM TOE, AND EMBANKMENT AND FOUNDATION EXCAVATIONS ANDBACKFILLING OF THE MAIN DAM DOWNSTREAM TOE.

SUBMIT SEPARATE DEWATERING PLANS FOR THE AUXILIARY DAM EMBANKMENT AND FOUNDATION EXCAVATIONS AND BUTTRESS CONSTRUCTION AND THE MAIN DAM EMBANKMENT AND FOUNDATION EXCAVATIONS AND BUTTRESS CONSTRUCTION. SUBMIT ASEPARATE DEWATERING PLAN FOR THE DOWNSTREAM BOREL CONDUIT MONOLITH AND OUTLET WORKS EXCAVATION IF THIS EXCAVATION IS STAGED SEPARATELY FROM THE AUXILIARY DAM FOUNDATION EXCAVATION.

1.3 SUBMITTALS

GOVERNMENT APPROVAL IS REQUIRED FOR SUBMITTALS WITH A "G" DESIGNATION; SUBMITTALS NOT HAVING A "G" DESIGNATION ARE FOR INFORMATION ONLY. ALL CALCULATIONS CONTAINED WITHIN THE DEWATERING PLANS MUST BE STAMPED AND SIGNED BY A PROFESSIONAL CIVIL ENGINEER LICENSED IN THE STATE OF CALIFORNIA. WHEN USED, A DESIGNATION FOLLOWING THE "G" DESIGNATION IDENTIFIES THE OFFICE THAT WILL REVIEW THE SUBMITTAL FOR THE GOVERNMENT.SUBMIT THE FOLLOWING ACCORDING TO SECTION 01 33 00 SUBMITTAL PROCEDURES:

SD-01 PRECONSTRUCTION SUBMITTALS

DEWATERING PLANS; G DEWATERING ACTION

PLAN; G

DEWATERING SYSTEM DESIGN ENGINEER; GDEWATERING

SPECIALIST; G

PERMITS

SD-06 TEST REPORTS

Daily Dewatering Performance Reports; G

1.4 QUALIFYING EXPERIENCE

1.4.1 Dewatering System Design Engineer

The dewatering system must be designed by a Professional Civil Engineer licensed in the State of California and has a minimum of 10 years of verifiable experience in the design, construction and operation of dewatering systems including but not limited to pumping, deep wells, well points, sheet piles, and earthen cofferdam or settlement basins, under similar subsurface and site conditions. Provide a resume that demonstratesthe required experience and a list and description of past dewatering projects of similar complexity for which the individual was the dewatering system designer. Submit the resume for approval 60 days before commencement of excavation and dewatering.

1.4.2 Dewatering Specialist

The Contractor must have on staff and at the job site a Dewatering Specialist with a minimum of 10 years of verifiable experience in the installation, initial startup, testing, operation, and maintenance of dewatering facilities of a complexity comparable to this project. The Dewatering Specialist must oversee the installation and operation of the dewatering system. The Dewatering Specialist must be on the job site eachday when the dewatering system is in operation. The Dewatering Specialistalso must be on call in event of an emergency, and must be able to arrive on site within 90 minutes, 24 hours per day including weekend and holidays. Submit a resume of the Contractor's dewatering specialist that demonstrates the required experience and a list and description of past dewatering projects in which the individual was responsible for the installation and operation of dewatering systems of similar complexity.
Submit the resume for approval 60 days before commencement of excavationand dewatering.

1.5 DEWATERING PLANS

Submit Dewatering Plans that include drawings and data showing the methodto be employed in dewatering excavated areas for approval 60 days before commencement of excavation. Excavation operations will not be allowed until the Contracting Officer has approved the Dewatering Plan for each feature. Submit plans in a format acceptable to the permitting agency forinclusion in required permit applications to any and all regulatory agencies for which permits for discharge water from the dewatering systemare required due to the discharge reaching regulated bodies of water.

The Dewatering Plans must detail how surface and groundwater will be controlled throughout construction and for each dewatering phase required to be constructed in the dry or dewatered state. The Dewatering Plans mustshow the proposed methods to dewater each working area and control the water from rain, sheet flow and other surface water. Provide the engineering design and adequacy of each dewatering system. All calculations in the dewatering plan must be stamped and sealed by a Professional Civil Engineer licensed in the State of California. Describe in detail facilities and procedures for insuring discharge water quality according to the applicable provisions of the Erosion Control Plan and SWPPP and NPDES requirements. The information in the Dewatering Plans mustbe according to Federal, State, and local laws and regulations, and the permits indicated in SECTION 01 57 20 ENVIRONMENTAL PROTECTION. As a minimum, include the following information in the Dewatering

Plans:

a. Detailed description on the methods, installation and details of the dewatering systems proposed to be employed to dewater the site and keep it dry during the construction cycle. Submit details of the dewatering facilities, including equipment and erosion protection facilities. Include location, depth and size of well points, headers, sumps, ditches, size and location of discharge lines, type and capacities of pumps and standby/backup units, and detailed description of dewatering methods to be employed to convey the water from site to adequate disposal.

b. Site plan of the project component with a description of the dewatering system and equipment, layout including the location of sumps, wells, well points, backup pumps, temporary containment berms, cofferdams, or diversion ditches as necessary; installation methods; description and layout of the on site water detention systems (settling basins); location of the proposed discharge point(s), discharge rates, and the associated water quality monitoring locations; and re-watering procedures.

c. Information related to backup pumping systems, backup power systems, and warning systems to protect against power failure, system failure, and high groundwater.

d. Information related to operation, maintenance, monitoring, removal, decommissioning wells, and system abandonment procedures. Describe the removal of all provisions for dewatering at the end of construction and the restoration of the site and disposal of all water treatment byproducts.

e. A detailed description of the sequence of construction and dewatering, including a description of control elevations during cofferdam/stability berm construction.

f. Supporting design information including design calculations prepared by a Professional Civil Engineer licensed in the State of California and seepage and slope stability analysis required in paragraph ANALYSES.

g. Detailed description and drawings of the outflow settling basin system for each phase of dewatering. Include at a minimum:

   (1) Types and capacity of the initial electric pump and the standby electric pump;

   (2) Sources of initial pump system and standby pump system;

   (3) Piping/hoses diameter;

   (4) Suction and discharger details including energy dissipation splash blocks and outfall weirs;

   (5) Suction sump (if any) detail - size, construction, etc;

   (6) Operating and maintenance;

   (7) Drawings showing the layout of the initial system and subsequent standby system including location of the pump(s), sump, suction pipe, discharge pipe, etc;

(8) Impermeable geomembrane strength data, seam welding and testing, geomembrane anchoring.

h. The location and type of turbidity control devices and methods necessary to ensure State Water Quality will be met.

i. Calculations estimating the area of influence of dewatering, depth of dewatering, pumping rates, duration and volumes, and stability of system, consistent with planned construction activities.

j. When it is not feasible to retain dewatering effluent onsite, include all of the following:

(1) Operational plan, which demonstrates that the discharge to the receiving water body meets all applicable State Water Quality standards and the requirements of SECTION 01 57 20 ENVIRONMENTAL PROTECTION prior to discharge, and also contains the proposed sampling locations and daily turbidity measurements.

(2) Contingency plan, which includes procedures for ceasing dewatering operations and corrective actions until water quality standards are met. Do not commence earthwork operations until the Dewatering Plan is approved. Allow 45 calendar days in the schedule for the Government's review. No adjustment for time or money will be made if resubmittal of the Dewatering Plan is required due to deficiencies in the plan.

1.6 DAILY DEWATERING PERFORMANCE REPORTS

Submit Daily Dewatering Performance Reports to the Contracting Officer daily. The daily report, covering a 24 hour period (from midnight to midnight), must include records, results, and data obtained from required testing, inspection, maintenance, and daily monitoring of systems to control surface water and groundwater. Include measurements of water levels in sumps and observation wells or piezometers, the quantity of waterdischarged from the settling tanks, and a description of the dewatering system's performance. The data must be provided as digital files (EXCEL), as approved. Submit the report no later than one day after the Record day.

1.7 DEWATERING ACTION PLAN

Submit a Dewatering Action Plan for approval 60 days before commencement of excavation. This plan must be prepared to describe the detailed actions that will be taken in the event that signs of soil transportation (i.e., cloudy water, etc.), softening of the bottom of excavation, or formation of"quick" conditions or "boils", are noticed. Post the plan in visible areassuch as the Contractor's Construction Trailer. The names, addresses, and phone numbers of the personnel required to be on call must be submitted to the Contracting Officer prior to commencement of dewatering operations.

1.8 GROUNDWATER CONDITIONS

Assess the foundation and groundwater conditions, topography, and historical rainfall and lake levels presented by the plans and specifications and the Geotechnical Data Report. Groundwater levels along the downstream toe of Auxiliary Dam are influenced by the reservoir pool elevation, precipitation, water surface elevation in the Borel Canal, the buried toe drain between Station 58+30 and 61+30, and operation of the sumppump at Station 58+30.

Groundwater levels along the downstream toe of MainDam are influenced by the reservoir pool, precipitation, tailwater elevation in the outlet works stilling basin, and releases from the Main Dam outlet works and powerhouse. Groundwater levels along the downstream toe of Auxiliary and Main Dams measured from piezometers during various reservoir pool elevations are provided in the Geotechnical Data Report.

PART 2 PRODUCTS

Not Used

PART 3 EXECUTION

3.1 DEWATERING THE SITE

3.1.1 General

This work consists of performing all operations necessary in dewatering the surface water and subsurface water at each site of the work and maintaining groundwater elevations at the sites at a level which facilitates the excavation and backfilling work. Design, furnish, install, maintain, and operate a dewatering system that prevents loss of fines, sand boils, quick conditions, or softening of foundation strata and maintain stability of bottom and slopes of excavations so that every phase of work is performed in the dry and under conditions where free, running, flowing, or ponding water are not present.

When the dewatering system does not meet the specified requirements, and asa consequence, loosening or disturbance of the foundation strata, instability of the slopes, or damage to the foundations, embankments, or structures occurs, the Contractor is responsible for supplying all materials and labor and performing all work for restoring foundation soils,slopes, foundations, and structures, to the satisfaction of the ContractingOfficer, at no additional cost to the Government.

When failure to provide adequate dewatering and drainage causes disturbanceof the soils below design foundation or excavation grade, provide adequate dewatering and excavate and re-fill the disturbed areas with approved, properly compacted fill material. Such work must be at the Contractor's expense and at no additional cost to the Government.

3.1.2 Dewatering Requirements

Perform all construction in areas free from water. Water in varying quantities may be encountered at any location but especially at the downstream toe of the dam embankments and low lying staging and access roads. Provide suitable well points, deep wells, ejector wells, sumps, pumps, cofferdams, dikes, diversions ditches, and containment berms constructed at all locations where construction work is at lower elevationthan the elevation of the ground water at the time of doing the work.
Dewater the construction area prior to commencement of the work, and keep all subgrades and excavation slopes whether for earth fill, rock fill, filter sand, or concrete, drained and free of water throughout the working period. During dewatering and rewatering operations, keep the groundwater level a minimum of four (4) feet below the bottom of the working excavationlevel which includes excavated slopes in areas of soil and decomposed rock or rippable rock. In areas of highly weathered to unweathered rock or non-rippable rock, keep the groundwater level a minimum of one (1) foot below the bottom of the working excavation level which includes excavated

slopes during dewatering and rewatering operations. The groundwater level must be maintained at the required minimum depth under all excavated surfaces to be dewatered which includes final bottom grades and excavated slopes. If an unstable subgrade condition ("pumping") occurs of if the ground becomes visibly saturated, the depth of the groundwater level must be increased to a point where foundation soils are no longer disturbed by excavation or backfill operations.

3.1.3 Methods

Submit in the Dewatering Plan the proposed dewatering method at least 60 days before dewatering operations are commenced. Excavation operations inareas to be dewatered will not be allowed until the Dewatering Plan for that feature has been approved by the Contracting Officer. Describe the methods to be employed to dewater the site and keep it dry during the construction cycle. The dewatering system must be designed by a Professional Civil Engineer licensed in the State of California and has a minimum of 10 years of demonstrated experience in the design and construction of dewatering systems. Use a method of dewatering which willaccomplish the desired results. If pumps are utilized, provide sufficientpumps and other equipment to dewater the area. The Contractor must assumefull responsibility for the adequacy of the dewatering method.

3.1.4 Analyses

Perform and submit seepage and slope stability analyses of the dewatered areas. As a minimum, provide a two dimensional (2D) or three dimensional (3D) finite element seepage and stability model using commercial software such as SEEP/W (with SLOPE/W), SEEP2D (with UTEXAS4) or SLIDE. Provide seepage models for all stages of excavation, rewatering and backfilling forboth the Auxiliary and Main Dams. Use the model to verify estimates of outflow and the effectiveness of selected dewatering methods for both dams at the Restricted Pool Elevation (elevation 2589.26 feet, NAVD88). Conductstability analyses of the dam embankments and toe excavation cut slopes, and cofferdam excavation for each phase of dewatering coupled with the seepage analysis for the critical excavation cases in each phase.
Calculations must include site characterization of each earthwork phase to establish the design hydraulic parameters used in the seepage analyses for each dewatering model. As a minimum, hydraulic conductivity, porosity and unit weight values must be established for each subsurface or surface fillmaterial contained within the confines of the seepage model. The seepage and slope stability analyses must be performed under the direction of a Professional Civil Engineer licensed in the State of California. The Contractor must provide as a minimum, two dimensional (2D) or three dimensional (3D) seepage analyses coupled with limit equilibrium slope stability analyses of dewatered areas for each phase of earthwork as follows:

a. A minimum of two orthogonal cross section models must be established for full depth of excavation for the Borel Conduit outlet works dewatering using the 2D or 3D seepage analysis software. The seepage analyses must be phased with excavation to show discharges for the dewatering system at each level of dewatering bench and for the steady state case at the bottom of excavation grade. Accompanying slope stability analyses must be performed prior to dewatering and at final excavation depth. The cross section models must also be used in the seepage analyses to determine dewatering and rewatering discharge quantities.

b. A minimum of eight orthogonal cross section models must be established for full depth of excavation for the Auxiliary Dam dewatering using the 2D or 3D seepage analysis software. The seepage analyses must be phased with excavation to show discharges for the dewatering system at each level of dewatering collection and outflow distribution bench and for the steady state case at the bottom of excavation grade. Accompanying slope stability analyses must be performed prior to dewatering and at final excavation depth. The cross section models must also be used in the seepage analyses to determine dewatering and rewatering discharge quantities at each stage of the excavation and filling.

c. A minimum of four orthogonal cross section models must be established for full depth of excavation for the Main Dam dewatering using the 2D or 3D seepage analysis software. The seepage analyses must be phased with excavation to show discharges for the dewatering system at each level of dewatering collection and outflow distribution bench and for the steady state case at the bottom of excavation grade. Accompanying slope stability analyses must be performed prior to dewatering and at final excavation depth. The cross section models must also be used in the seepage analyses to determine dewatering and rewatering discharge quantities at each stage of the excavation and filling.

d. Seepage analyses for dewatering can be performed with the following Corps of Engineers community of practice approved software:

(1). SLIDE, latest version available at time of bid. (2).

SEEP2D (GMS v. 9.0.2) or later at time of bid. (3). MODFLOW

(GMS v. 9.0.2) or later at time of bid. (4). SEEP/W, latest

version available at time of bid.

e. Slope stability analyses for dewatering cut slopes can be performed with the following Corps of Engineers community of practice approved software:

(1). SLIDE, latest version available at time of bid.(2).

UTEXASIV or later version at time of bid.

(3). SLOPE/W, latest version available at time of bid. (4).

PCSTABL, latest version available at time of bid.

f. Slope stability analyses for every earthwork phase must be conducted to establish the slope angles and dewatering system layout to meet the minimum factor of safety of 1.4 at the Restricted Pool elevation of 2589.26 feet, NAVD88. Analyses must be conducted as a minimum for the steady state case with continuous pumping down to the bottom of excavation final grade as well as any condition in the life cycle of the dewatering operations that may lead to a more unsafe condition than that of the steady state. All stability analyses must be performed using the Spencer's limit equilibrium method.

g. All seepage analyses and slope stability analyses must be conducted for the reservoir at the Restricted Pool elevation (2589.26 feet, NAVD88). In addition, in case of a flood condition in which the reservoir levels cannot be maintained below the Restricted Pool elevation, the dewatering system for each earthwork phase must be able to re-water the total excavation template so as not to introduce hydraulic exit gradients into the bottom of the excavation or slopes that exceed 0.5 at the Restricted Pool elevation (2589.26 feet, NAVD88) or exceed 0.7 at a reservoir pool elevation of 2614.26 feet, NAVD88 (5 feet above the Gross Pool elevation of 2609.26 feet, NAVD88). This must be validated by seepage and slope stability analyses to establish an acceptable rate of re-watering within the excavation pit. The flood re-watering analyses must be included in the Dewatering Plan.

3.2 IMPLEMENTATION

3.2.1 General

Protect the work from damage by water originating from any source. Design, install, power, operate, maintain, and remove the dewatering system. Furnish all tools, equipment, labor, and materials required. The dewatering system must be of sufficient size and capacity to control and remove all surface water and groundwater from work excavations to below therequired elevations along the full width and length of excavation. The dewatering system must remain in continuous operation to permit excavation,placement, and compaction of foundation and embankment materials to meet the specification requirements.

3.2.2 Continuous Operation

Groundwater pumping systems must be operated continuously during dewatering operations and must not be shut down between shifts, on holidays, on weekends, or during work stoppages without approval of the Contracting Officer.

3.2.3 Backup Pumping

Provide a backup pump for all primary dewatering pumps of the same capacityand horsepower to ensure continuous pumping in case of power disruption or equipment malfunction. Backup pumps must be available on site at all times during pumping at every dewatering location.

3.2.4 Power System

Power the dewatering system by direct line power from the Southern California Edison for temporary site power according to the requirements of Section 01 50 00 TEMPORARY CONSTRUCTION FACILITIES AND CONTROLS and approval of the Contracting Officer. Provide backup power in case of power disruptions with diesel generator(s) in a noise reduction enclosure. The power systems must be able to automatically switch from oneto the other in the event that the operating systems fails for any reason. The backup system must be capable of providing power for maintaining pumping on the entire dewatering system and be able to operate unattended for the length of period that the construction site may be without dewatering personal present.

3.2.5 Maintenance Records

Establish and conduct a regularly scheduled maintenance program to keep the dewatering system fully operational on a continuous basis for the entireperiod of time it is required. The maintenance program must conform toequipment manufacturer's recommendations and instructions, and must include not only operating components, but standby/backup equipment and materials as well. The maintenance program must include, as a minimum:

a. Testing pumps on a monthly basis. Pumps testing less than 75 percent of manufacturer's rated characteristics must be promptly repaired or replaced.

b. Each piece of backup power equipment and backup pumps must be tested weekly. Any equipment failing to perform must be immediately repaired or replaced.

3.3 VERIFICATION AND MONITORING

3.3.1 Verification and Monitoring of Water Levels

The use of temporary observation wells and existing piezometers will be used to monitor groundwater levels and to verify that water level is lowered and maintained at or below the required elevation during all excavation and backfill operations.

3.3.1.1 Temporary Observation Wells

Provide a minimum of ten temporary observation wells at Auxiliary Dam and five temporary observation wells at Main Dam to monitor groundwater levels within the excavation limits on a daily basis. Locate the observations wells within a five (5) foot horizontal radius of the locations shown in Table 1 below:

TABLE 1. Minimum Observation Well Locations		
Horizontal Datum: California Coordinate System, Zone 5, NAD83		
Auxiliary Dam	Northing (feet)	Easting (feet)
1.	2420485	6421250
2.	2420340	6421530
3.	2420245	6421815
4.	2420160	6422100
5.	2420360	6421685
6.	2419900	6423384
7.	2419915	6423160
8.	2419955	6422885
9.	2420045	6422515
10.	2420115	6422260
1.	2421400	6418025

2.	2421350	6418090
3.	2421300	6418405
4.	2421330	6418190
5.	2421370	6418020

Install temporary observation wells at the downstream Borel Conduit monolith/outlet works excavation if this excavation occurs separately fromthe Auxiliary Dam foundation excavation. Place the temporary observation wells for this excavation and dewatering within a 5 foot horizontal radiusof the four interior corners of the bottom of the excavation pit and one located along the centerline of the Borel Canal excavation a maximum horizontal distance of 5 feet from the southern end of the dam outflow conduit.

All temporary observation well tip elevations must be at least 10 feet below the working excavation grade. Holes for the temporary observation wells must be drilled in accordance to the requirements in Section 31 09 15EARTH EMBANKMENT AND EARTH FOUNDATION DRILLING REQUIREMENTS. Complete well using 2-inch diameter Schedule 40 PVC with 0.020-inch slotted screen. Wells must be screened from the bottom to about 5 feet below the ground surface. Place Zone 2A Filter Sand, specified in Section 35 73 13 EMBANKMENT FOR EARTH DAMS, for a filter pack from the bottom of hole to 3feet below the ground surface. Place bentonite from the top of the sand filter pack up to the ground surface and then hydrate. Develop the temporary observation wells as needed. Install vibrating wire transducerin each temporary observation well in accordance to Section 31 09 13 GEOTECHNICAL INSTRUMENTATION AND MONITORING.

3.3.1.2 Monitoring of Temporary Observations and Existing Piezometers

Monitor water levels in temporary observation wells and all existing piezometers listed in the drawings as to be preserved. This includes any piezometers and monitoring wells located along the crest, slope, toe of the embankments and includes any piezometers, monitoring wells, and observation wells installed by the Contractor within and outside the excavation limits. Install or reinstall each observation well in such a fashion that continuous daily readings of the groundwater level are available during the entire duration of dewatering and rewatering operations for each phase of the dewatering efforts. Monitor the water level in each instrument continuously with vibrating wire transducer in accordance to Section 31 09 13 GEOTECHNICAL INSTRUMENTATION AND MONITORING with reading recorded digitally at quarter hour (15 minute) intervals. In addition take manual water level measurements in each instrument daily. Provide digital readouts of groundwater levels daily during dewatering and rewatering operations until all dewatered excavated areas are backfilled and the dewatering system has been removed. This data, including digital files, must be provided in the Daily Dewatering Performance Report to the Contracting Officer. The Contractor must grant Government representativeaccess to all instruments for Government inspection and water level monitoring at all times. Provide protection devices around each observation well at every excavation level. Reinstall observation wells damaged during construction within five (5) days to accommodate excavationand refilling operations.

3.3.2 Monitoring Flows

Continuous flow measurements in each header pipe, outflow measurements from each dewatering pump, inflows into each settling basin and outflows into the discharge point(s) for each phase must be available at any time and must be recorded digitally at hourly intervals during the entire dewateringand rewatering operations. Provide the data, including digital files, in the Daily Dewatering Performance Report and at the request of the Contracting Officer.

3.4 DISCHARGE OF WATER FROM DEWATERING OPERATIONS

3.4.1 Discharge of Water

Dewatering effluent must be either utilized as construction water, discharged to surface waters, or discharged using evaporation or percolation ponds. Discharging to surface waters, including but not limited to the Kern River and Isabella Lake, must be accomplished in accordance to Section 01 57 20 ENVIRONMENTAL PROTECTION. When dewatering effluent is utilized as construction water, the effluent must meet the water quality requirements for that use. Locate percolation ponds, evaporation ponds, settling basin, and storage ponds a minimum distance of 200 feet from the top of excavation limits to prevent re-infiltration ofthe discharge effluent back into the system.

3.4.2 Temporary Settling Basin System

Provide a temporary dewatering outflow settling basin system for each phaseof dewatering at the site prior to the commencement of dewatering operations. The settling basin system must consist of any necessary pump(s) and all associated piping and appurtenance necessary to transfer the flow from the dewatered area to the settling basin system. Operate andmaintain the settling basin system until dewatering operations are complete. Design the settling basins with an impervious lined earthen dikeor an impermeable metal dewatering settling tank at the locations approved by the Contracting Officer. Construct earthen dikes in a manner that prevents seepage loss, dike internal erosion and surficial dike erosion from the settling basins. Encapsulate all dike embankments constructed of soil or rockfill with a puncture resistant impermeable geomembrane to prevent discharge of dewatering effluent onto the surrounding ground surface or into waterways during and between the dewatering operation periods. Locate settling basins and storage ponds a minimum 200 feet from the top of excavation limits to prevent re-infiltration of discharge effluent back into the system.

3.4.3 Treatment of Dewatering Operation Effluent

Direct dewatering effluent from all dewatering phases through a conduit to the designated settling basin to settle suspended solids. For turbidity control requirements refer to paragraph Water Resources of Section 01 57 20 ENVIRONMENTAL PROTECTION. Conduct chemical and biologic testing on groundwater samples taken at the beginning of each dewatering phase to determine treatment requirements for unacceptably high levels of contamination. Continue to periodically test dewatering effluent from the excavation outflow conduit as required in Section 01 57 20 ENVIRONMENTAL PROTECTION. Direct outflow from the settling basins into the portable water treatment plant identified in Section 01 57 20 ENVIRONMENTAL PROTECTION. The treatment plant is designed to remove constituents identified in the groundwater at the site which have unacceptable levels ofcontamination. Those constituents of interest and their concentrations in tested samples at the site are identified in Section 01 57 20 ENVIRONMENTALPROTECTION. Perform chemical tests on grab water samples from the treatment plant holding tank water effluent at the frequency identified in Section 01 57 20 ENVIRONMENTAL PROTECTION. When test show unacceptably high concentrations of chemical constituents identified in Section 01 57 20 ENVIRONMENTAL PROTECTION, recycle the holding tank effluent through the treatment plant until the effluent meets the State water quality standards identified in Section 01 57 20 ENVIRONMENTAL PROTECTION. Discharge effluent from the treatment plant holding tank into the outfall pipe to thedischarge point(s) only after it meets the water quality standards of Section 01 57 20 ENVIRONMENTAL PROTECTION.

3.5 REMOVAL

3.5.1 Decommissioning Dewatering Wells and Temporary Observation Wells

Decommission all dewatering wells and temporary observations wells after backfill operations and after receiving approval from the Contracting Officer. Decommissioning must be performed according to local, county, and State requirements. With exception of subsurface dewatering conduits or wells required to maintain groundwater levels below the workingbackfill grade, remove and dispose of all temporary subsurface conduits andwells off site once these operations are complete. Fill all dewatering system subsurface conduits and wells left in place, due to their necessity to dewater during backfilling operations, well holes, and temporary observation wells with Portland cement grout. Accomplish grout placement by means of a tremie pipe and apply in one continuous operation from the bottom of the conduit, well, or casing to the top. Record quantities of grout injection within each conduit or well and compare to volume of well to verify there are no voids. Periodically place additional grout to conduit or well (top off) as the grout settles and cures. Cut well casing flush with the ground surface and remove any surface completions.

3.5.1.1 Portland Cement Grout Mix

Provide cement grout with a mixture of a maximum of 7 gallons of approved water per 94 lb bag of portland cement, conforming to ASTM C150/C150M. Addno more than 5 percent by weight of bentonite powder to reduce shrinkage, hold the cement in suspension prior to the grout set. Use sodium bentonitepowder and/or granules for high-solids bentonite grout. Mix water from an approved source with these powders or granules to form a thick bentonite slurry, consisting of a mixture of bentonite and the manufacturer's recommended volume of water to achieve an optimal seal. The slurry must contain at least 20 percent solids by weight and have a density of 9.4 lb per gallon of water or greater. Perform grout mixing thoroughly with a mechanical mixer, high shear mixer, or re-circulating through a pump to ensure that the mixture is uniform and there are no lumps.

3.5.2 Settling Basins

Upon completion of dewatering operations, dewater settling basins using pumps. Test and treat, if need be, the effluent from settling basin dewatering to meet the water quality discharge standards identified in Section 01 57 20 ENVIRONMENTAL PROTECTION. Test and, if need be, treat allsediment within the settling basins according to the requirements of Section 01 57 20 ENVIRONMENTAL PROTECTION. After treatment, dispose of thetreated sediments from the settling basins in the Engineer Point Disposal Site indicated on the drawings. Remove all temporary settling basin systems after temporary dewatering operations are complete. Restore the areas of the Auxiliary and Main Dam settling basins to the original conditions in an orderly manner.

3.5.3 Water Treatment Plants

Upon completion of dewatering operations, decontaminate, disassemble and remove water treatment plants and appurtenances from the site as prescribedin Section 01 57 20 ENVIRONMENTAL PROTECTION. Restore the areas of the Auxiliary and Main Dam portable treatment plants to their original conditions in an orderly manner.

3.6 SAFETY

3.6.1 Inspections

Excavation bottoms and slopes must be inspected daily by the Contractor for concentrations of seepage into the excavations, sloughing, sand boils, cracking, or other adverse conditions indicative of insufficient drawdown and/or instability. If any of these conditions occur, immediately cease excavation and report the adverse condition to the Contracting Officer and the USACE Geotechnical Engineer. A plan for corrective action must be prepared to correct any deficiency in design/operation of the dewatering system.

3.6.2 Alarm System

An alarm system must be in place to provide warning if the groundwater pumping system stops working. The system must be designed to notify the Contractor 24 hours a day, including weekends and holidays.

3.6.3 Night Lighting

Lighting must be in place at active excavations to allow staff to inspectthe excavated areas, equipment, and discharge of water. Lighting must beshielded and directed downward toward the work site.

3.6.4 Personnel and Equipment

During time when excavations are open and continuous pumping is being performed, the Contractor must always have at least one person on site, 24 hours per day, including weekends and holidays to verify the operations of the surface control and dewatering systems. The person on site must make a physical inspection of the power source(s), pumps, discharge, etc. not less than every 4 hours. The Contractor must provide two people if simultaneous dewatering is occurring at Auxiliary Dam and Main Dam with each person responsible for the operations at one dam.

Provide sufficient personnel and equipment readily available to change and repair pumps, make pipe connections, connect standby/backup power supplies, move materials, and perform other operations and maintenance on the dewatering systems to ensure uninterrupted operations.

The following minimum personnel are required to be on call in the event ofan emergency, and must be able to arrive on site within 90 minutes, 24 hours per day including weekend and holidays:

a. One Dewatering Specialist (Responsible Supervisor).

b. One Dewatering Assistant.

c. One Earthmoving Foreman (Responsible Supervisor).

d. One Heavy Equipment Operator.

-- End of Section –

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SECTION 31 23 19.00 12

DEWATERING AND UNWATERING

PART 1 GENERAL

1.1 SCOPE

The work provided for herein consists of furnishing all plant, labor, material and equipment and performing all operations required for designing, furnishing, installing and operating a system or systems to dewater the excavation area; maintaining the area free from water during construction operations; rewatering the area under controlled conditions atthe termination of the dewatering; and removing the system. High water levels on the Ohio River will threaten the stability of the excavation area during construction. These operations will also include emergency floodingof the excavation area by controlled backflooding if the system become inadequate. Once the water level on the Ohio River drops to an acceptable level as determined by the Contracting Officer, the excavation area would be unwatered again so that construction operations could proceed.

1.2 SUBMITTALS

Government approval is required for submittals with a "G" designation; submittals not having a "G" designation are for information only. When used, a designation following the "G" designation identifies the office that will review the submittal for the Government. The following must besubmitted according to Section 01 33 00 SUBMITTAL PROCEDURES:

SD-05 Design Data

Dewatering System Design; G

The submittal that outlines the proposed dewatering system design must include a drawing that delineates the locations of temporary piezometers (see paragraph 3.1 Installation in SECTION 33 28 00.00 11 PIEZOMETERS), deep well and/or well point locations, and system design calculations and assumptions. The submittal must also address proposed dewatering system details to include deep well and/or well point tip depths, screen lengths,and diameters; required pumping rates; and discharge point(s) for the collected ground and surface water. Additional requirements for the Dewatering System Design submittal are provided in paragraph 3.2 REVIEW OFSYSTEM DESIGN AND PERFORMANCE, below

1.3 QUALITY CONTROL

The Contractor must establish and maintain quality control for all dewatering operations to assure compliance with contracting requirementsand maintain records of his quality control for all construction operations, including but not limited to the following:

(1) Fabrication and workmanship.

(2) Installation, operation and removal.

(3) Monitoring free water surface and piezometric elevations.

(4) Measuring effluent from dewatering system.

(5) Monitoring of sanding.

A copy of these records and tests, as well as the corrective action taken, must be furnished the Government. Reports of operation and inspection must include the following data: piezometer elevations, river stages, timeof operation of each well, effluent discharge, sanding rates during pump test, problems encountered, proposed actions, and any other pertinent data.

1.4 GENERAL

All work under this contract except as otherwise specified must be carriedon in areas free of water. The Contractor must design, furnish, install, operate and maintain such facilities necessary to accomplish the following:

(1) Collect and dispose of all surface water in the protected area regardless of source.

(2) Control and dispose of all surface water around the periphery of the excavation areas to prevent such water from entering the excavation.

(3) Lower and maintain the water table at least 3 feet below the bottom of the excavation and adjacent areas, and at least 3 feet below the side slopes. No upward or lateral flow of ground water will be permitted at any time.

(4) Install and monitor three construction piezometers.

1.5 DEFINITIONS

1.5.1 DEWATERING

Dewatering defines the lowering of the ground water below the slopes and bottom of the excavation and adjacent areas to ensure dry, firm working conditions and the reduction to safe levels of any hydrostatic uplift pressures in any confined foundation strata and/or aquifers which is necessary to ensure the stability and integrity of the foundation.

1.5.2 DEWATERING SYSTEM

Dewatering System defines the machinery, equipment and appurtenances necessary for and related to the accomplishment of dewatering, and the collection and disposal of all surface water within the protected area.

1.5.3 EMERGENCY FLOODING

Emergency flooding of the protected area is defined as the controlled process of filling the excavation and adjacent areas with water to a specified elevation and at a specified rate for the purpose of ensuring thestability and integrity of the protected area, the cofferdam, the levee, and the foundation. The emergency flooding must be performed at the rates and to the elevations directed by the Contracting Officer.

1.5.4 UNWATERING

Unwatering is defined as the process of removing all water within an excavation.

1.5.5 REWATERING

Rewatering is defined as the controlled process of allowing the ground water to return to its natural occurring elevation at a specified rate when construction is completed and the dewatering system is no longer required.

1.6 DESIGN

The dewatering system must be designed by a licensed professional engineerthat is registered in the state in which the work is to be performed. Accepted professional methods of engineering design consistent with the best current practice must be used in system design. The Contractor mustperform necessary tests and/or analyses of the water and soil environment at the site to satisfy himself that the materials used in his system will not corrode or otherwise deteriorate to such an extent that the system willnot perform satisfactorily during the life of the contract. The Contractor must gather the data needed to design a dewatering system that will meet the contract requirements. The Contractor will be required to submit a complete and detailed Dewatering System Design to the Contracting Officer for approval. No part of the dewatering system must be installed without prior approval of the Contracting Officer.

1.7 DEWATERING REQUIREMENTS

The dewatering system for the protected area which includes the culvert excavation and adjacent areas must be of a type and capacity to accomplishall requirements specified herein.

(1) The dewatering system must be designed, installed and operated to dewater and lower the piezometric levels within the protected area for an Ohio River elevation of at least 333.3 feet NAVD88 at the culvert site.

(2) The system must be of such capacity that it will lower and maintain the free water and piezometric levels, to an elevation at least 3 feet below all earth slopes and excavation surfaces lying within the area, inclusive of the interior slopes of the cofferdam embankments proper. The system must have sufficient capacity to accomplish this desired result allowing for normal variations in soil properties and foundation conditions.

(3) The water level must be maintained continuously at or below the elevation specified above so that construction operations can be performed without interruption due to wet conditions. The groundwater table in the project area is expected to fluctuate depending on the water level in the Ohio River, seasonal variations, and weather conditions.

(4) No upward or vertical or lateral flow of ground water into the work area will be permitted at any time. The dewatering system must be designed, constructed and operated as necessary throughout all stages of the construction process, including unwatering, rewatering, dewatering, and/or emergency flooding, so as to prevent movement and/or piping of the foundation, excavation slopes and fill materials. The

system must be operated as necessary during dewatering, unwatering, emergency flooding and rewatering so as to maintain piezometric levels, within the dewatered area, at or beneath the elevation of the water level in the excavation.

(5) The required dewatering system must consist of deep wells, wellpoints, pumps, sumps, sump pumps, ditches and necessary appurtenances capable, at all water levels less than or equal to the design water levels specified in paragraph (1) above. In any case, protection of allslopes will be required to prevent erosion under normal surface runoff and construction conditions.

(6) Rewatering and/or emergency flooding of the area must be accomplished by directing surface and ground water into the area. The dewatering system must be kept operating at full capacity during such conditions, with dewatering effluent being directed into the excavation.
Protection of slopes and excavation surfaces must be provided as necessary to prevent erosion during these operations. No upward or vertical or lateral flow of ground water into the excavation will be permitted.

(7) Burying of headers will be allowed only in areas and to depthsabsolutely necessary for protection against damage at construction equipment crossings. The effluent from the dewatering system will berequired to be discharged over the top of the flood side cofferdam atelevation 333.3 feet NAVD88 and extend to the cofferdam toe before release from the discharge pipe(s). The discharge water must be controlled to prevent erosion or damage to the cofferdam or the existing natural ground.

(8) A system of construction piezometers will be required to monitorfree water surface elevations and piezometric elevations to evaluate the effectiveness of the dewatering system in fulfilling the requirements specified herein. Piezometers must be of adequate numbers and in suitable arrangements and depths for determining the free water surface elevations and piezometric elevation over the area. A minimum of three construction piezometers must be installed near thecritical areas of the excavation with general locations (1) near the culvert outlet structure (2) near the gatewell structure excavation and
(3) at the existing landside levee toe near the inlet structure. The piezometers must be located as far as practical from dewatering units (deep wells or well points) to accurately measure the groundwater levelin the excavation. The construction piezometers must be installed andreadings taken according to paragraph 3.3 MONITORING AND READING PIEZOMETERS, SECTION 33 28 00.00 11 PIEZOMETERS. The piezometer readings, along with corresponding water surface elevations in the OhioRiver gage at Cairo, IL, must be recorded on an approved form and reported to the Contracting Officer within 12 hours after they are obtained. If, in the opinion of the Contracting Officer, more frequent readings are required, the Contractor will be directed as to the numberand time that these readings are required. If additional readings are directed, an equitable adjustment in the contract unit price for dewatering will

be made.

(9) The system must include mechanical means for measuring the effluent from each well as well as the total effluent of the dewatering system. Devices and techniques used in measurement must be acceptable to the Contracting officer. The Contractor must make a minimum of one reading per instrument, per 24-hour period, a minimum of 20 hours apart, based on a 7-day week. These instrument readings, along with corresponding river stage readings, must be recorded on an approved form and reported to the Contracting Officer within 12 hours after theyare obtained. If, in the opinion of the Contracting Officer, more frequent readings are required, the Contractor will be directed as to the number and time that these readings are required. If additional readings are directed, an equitable adjustment in the contract unit price for dewatering will be made.

(10) The system must be designed, installed and operated in a manner which will preclude removal of materials from the foundation by the pumping operation (hereafter referred to as "sanding"). After installation, each well and wellpoint segment must be individually pump-tested at maximum design flow to verify acceptability with respectto sanding. Any well or wellpoint segment found sanding at a rate exceeding one pint per 25,000 gallons of effluent during the individualpump-test of maximum design flow must be replaced in a manner acceptable to the Contracting Officer, and at no additional cost to theGovernment. During pumping operations, a Rossum Sand Content Tester must be used daily to check each deep well and wellpoint segment for sanding. The results must be reported to the Contracting Officer with the effluent report described in (10) above.

(11) The rate of unwatering the excavation must not exceed 5 feetper day for the first 10 feet and one foot per day thereafter until completely unwatered.

(12) The grain size data available for design of the dewatering system is provided in the boring logs provided on the plan sheets. TheContractor must gather any additional data needed to design a dewatering system that will meet the contract requirements.

(13) All collected readings, from paragraphs (8), (9), and (10) must be entered into a spreadsheet. The spreadsheet must contain all readings from the start of the job to completion. The spreadsheet mustbe made available to the Contracting Officer if requested.

1.8 UNWATERING REQUIREMENTS

The unwatering system must be of a type and capacity for collection and disposal of all ground and surface water in the protected area. This includes leakage from the cofferdam, rain water, water from influent ditches, and any runoff from adjacent areas. The water level in the protected area must be maintained continuously at an elevation which will allow for construction operations to be performed without interruption due to wet conditions. During these operations, the Contractor must comply with all applicable laws and regulations regarding discharge of stormwater and collected groundwater from a construction site.

PART 2 PRODUCTS (NOT APPLICABLE)

PART 3 EXECUTION

3.1 INITIAL TESTING

Upon installation of the system, the Contractor must test and evaluate the completed system to demonstrate to the satisfaction of the Contracting Officer that the system is, in fact, capable of performing the intended dewatering operation as outlined herein. This testing must includecomplete falling head tests to be conducted on each piezometer.

3.2 REVIEW OF SYSTEM DESIGN AND PERFORMANCE

The Contractor must submit to the Contracting Officer, for review, details of his proposed dewatering facilities, including the type of system, planned layout and sizes of wells, headers, including all lengths requiring burial, collectors, ditches, piezometers, sumps and pumps; capacities of standby pumping and power supply facilities; number, type, location, proposed method of installation, and proposed methods of testing of piezometers; facilities for measuring the flow of water pumped from each well of the dewatering system; facilities for monitoring of sanding; provisions for disposal of water riverside of the floodside cofferdam from the dewatering system; and plan of operation including flooding and rewatering plans. This submittal must include the design capacity of each well at the design stage, and must be submitted no later than 30 days prior to installation of the system. The Contractor's proposed dewatering facilities will be reviewed for general design concept, gross capacity at design stages, and flooding and rewatering plans. The Contractor retains full responsibility for design, installation, operation and performance of the system, facilities, and its components. The Contractor must install the entire dewatering system and must make no reduction to the planned system without the prior written approval of the Contracting Officer. If during the progress of the work, the installed dewatering system proves inadequate to meet the requirements specified, including piezometers, the Contractor must, at his expense, furnish, install and operate such additional dewatering facilities and/or make such changes, either in features of the system or the plan of operation, as may be necessary to perform the required dewatering in a satisfactory manner. Such changes andadditions must be approved in writing by the Contracting Officer prior to being made.

3.3 OPERATION

The Contractor will be required to perform such dewatering and to maintain the work areas in a dry condition as long as is necessary for the work under this contract to be completed. The piezometers and dewatering system must be installed and fully functional prior to the start of excavation operations. Once the area is dewatered, it must be maintained in a dewatered condition until all work is completed, unless emergency flooding is directed or approved by the Contracting Officer. In the event that emergency flooding is deemed necessary, the protected area must be flooded according to the sequence of emergency flooding proposed by the Contractor and approved by the Contracting Officer. If emergency flooding is required, the Contractor may have to construct a landside cofferdam as directed by the Contracting Officer. However, the Contractor must not flood the protected areas without approval to do so by the Contracting Officer. If emergency flooding is directed by the Contracting Officer, based on factors other than inadequate performance of the Contractor's dewatering system or the Contractor's fault or negligence, the Contractor will be compensated for damages to permanent work according to the "Damage to Work" clause in the Special Clauses. Also, an equitable adjustment will be made in the contract for repair of damages to the cofferdam and dewatering systems provided such damages are not due to the fault or negligence of the Contractor or poor performance of the Contractor's dewatering system. However, all costs including backflooding of the excavation resulting from flooding necessitated because of the Contractor's fault, negligence, inadequate dewatering system performance, or convenience will be borne by the Contractor. If flooding is directed bythe Contracting Officer for reasons other than those above all extra costs will be borne by the Government and an equitable adjustment in the contractprice will be made for the costs according to the CONTRACT CLAUSE entitled "Changes".

3.4 MAINTENANCE AND SERVICING

The Contractor must be responsible for the maintenance, servicing and repairs of the entire dewatering system and appurtenances during the life of the contract, including replacement of any and all wells, and piezometers found performing unsatisfactorily.

3.5 STANDBY PUMPING EQUIPMENT POWER

The Contractor must furnish standby pumping equipment power as follows:

(1) Diesel or liquid petroleum gas prime movers for pumpsmust have 50% standby equipment.

(2) Portable electric generators must have 100% connectedstandby equipment.

(3) Commercial electric power, if available, must have100% standby electric generating equipment.

3.6 REMOVAL

The dewatering facilities required to maintain a dry condition within the protected area must be maintained until completion of the work within the protected area, and then must be completely removed. However, no dewatering facilities of any kind must be removed without prior approval of the Contracting Officer. All wells, pumps and appurtenances employed inthe dewatering system and all materials other than earth must remain the property of the Contractor, and must be removed by him from the site of the work. All holes must be plugged as follows: The screens and riser pipes of the dewatering system(s) must be plugged. Plugging must be accomplished by inserting a grout pipe to the full depth of the well or well point and the grout either poured or pumped in. The grout for pluggingthe hole must consist of a mixture of portland cement, bentonite, and water proportioned as directed by the Contracting Officer. The water percentage may be varied for a more effective plugging job. The riser pipesmust be cut 2 feet below the ground surface at the well location and backfilled with compacted impervious fill. The temporary piezometers must be plugged according to paragraph 3.4 PLUGGING ABANDONED HOLES in SECTION 33 28 00.00 11 PIEZOMETERS.

All ditches and discharge pipes associated with the dewatering and unwatering system must be completely removed. Any ditches and depressions used for the dewatering and unwatering system must be filled and compacted in accordance to the plans and specifications.

-- End of Section --

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DEWATERING

PART 1 GENERAL

1.1 DEWATERING

Provide materials, equipment, and labor to install and maintain pumps, piping, drains, well points, wells, and other facilities required to effectively control, collect, and dispose of groundwater to permit safe and proper completions of the work. Use appropriate equipment and methods for dewatering based on existing site conditions.

1.2 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

ASTM INTERNATIONAL (ASTM)

ASTM C33/C33M (2016) Standard Specification for ConcreteAggregates

ASTM D1785 (2012) Standard Specification for Poly(Vinyl Chloride) (PVC), Plastic Pipe,Schedules 40, 80, and 120

U.S. ARMY CORPS OF ENGINEERS (USACE)

ER 1110-1-1807 (2006) Procedures for Drilling in EarthEmbankments

1.3 DEFINITIONS

1.3.1 Dewatering

Removal and control of groundwater from pores or other open spaces in soil or rock formations to allow construction activities to proceed as intended in the dry. Includes relief of groundwater pressure and discharge of effluent water. This does not include runoff of surface water into excavated areas.

1.3.2 Hydrostatic Groundwater Level

The groundwater level at any location during construction and before dewatering. The hydrostatic groundwater level will fluctuate during construction as a result of precipitation and pool elevations.

1.3.3 Temporary Piezometer

Temporary instrument to monitor groundwater levels during construction and evaluate performance of dewatering system.

1.3.4 Wellpoint Dewatering System

Dewatering system consisting of a series of small-diameter wells (known as well points) connected via a header pipe to the suction side of a suitable vacuum pump on the surface.

1.3.5 Surging

The result of lowering the pumping water level in a well to the level of the pump intake such that the pump draws in air in a cyclic pattern causing the discharge to come out in surges.

1.3.6 Dewatering Specialist

Professional Engineer, Certified Engineering Geologist, or Certified Hydrogeologist in the State of [redacted] responsible for developing the dewatering plan and operation oversight of the dewatering system.

1.4 SUBMITTALS

Government approval is required for submittals with a "G" designation; submittals not having a "G" designation are for information only. When used, a designation following the "G" designation identifies the office that will review the submittal for the Government. Submit the following according to Section 01 33 00 SUBMITTAL PROCEDURES:

SD-01 Preconstruction Submittals

Dewatering Plan; G, DO

SD-06 Test Reports

Dewatering Reports; G, DODrilling
Logs; G, DO Completion Reports; G,
DO
Performance Tests Report; G, DO

1.5 DEWATERING PLAN

Submit the Dewatering Plan, prepared by the Dewatering Specialist, a minimum of 45 days prior to any dewatering activities. Include the following:

a. Schedule of mobilization and delivery of personnel, equipment, and materials for drilling, installing and testing the dewatering system.

b. Sequence of the dewatering system construction and operation includingrequired tests, initial drawdown period, duration, and schedule(s) or period(s) of active dewatering operation for the system.

c. Products and Materials Certifications including but not limited to:

    (1) Well Screen manufacturer: Certifying well slot width, open area,and slot cleanliness of screen.

(2) Filter Sand Supplier: Certifying gradation data, source of wellpoint sand pack material, and physical characteristics of sandpack material.

(3) Pump Manufacturer and/or supplier: Test results verifying pumpanticipated yields at total dynamic heads (TDH).

d. Equipment data including types, sizes, capacities, sources, locations, and other identifying characteristics for pumps, drive units, standby equipment, monitoring equipment, flow meters, valves, discharge piping,and other dewatering system components.

e. Dewatering System Installation details to include drilling method forwellpoints and temporary piezometers and methods of construction for wellpoints, discharge manifolds, settling basins, and other erosion control features.

f. Qualifications. Provide credentials for personnel to fulfill thefollowing:

(1) Dewatering Specialist

(2) Dewatering Operations Superintendent

(3) Drill Crew Foreman

g. Company Experience. Provide project experience for contractor and/orsubcontractor(s) working on dewatering system installation and/or operation near a constant groundwater recharge source. Include a minimum of 3 projects similar to those of this contract. Submit the following:

(1) Name of Project

(2) Location of Project

(3) Description of dewatering system for each project

(4) Duration of dewatering system operation for each project

h. Superintendence plan and schedule, indicating who will be responsiblefor observing the dewatering system and the proposed schedule describing when personnel will be on site to observe and maintain thesystem.

i. Operations and maintenance plan for dewatering system.

j. Quality Control (QC) Plan for dewatering system.

k. Emergency Plans. Include plans for emergencies that may arise duringoperation of dewatering system. Include list and hierarchy of individuals to notify in the event of an emergency. Emergencies include, but are not limited to, failure of dewatering system components (power system, pumps, pipes, etc) during operation, dameto system from construction equipment, a sudden rise in groundwater flows as a result of precipitation and/[redacted] pool elevation, and clogging of well points or header lines.

l. Required Permits. Include all permits required by local, state, and federal law including approved Drilling Program Plan (DPP).

1.6 AVAILABLE DATA

Refer to the Geotechnical Data Report for test boring logs and historical piezometer data available in areas to be dewatered.

1.7 PERSONNEL QUALIFICATIONS

a. Dewatering Specialist(s)

(1) Professional Engineer, Certified Engineering Geologist, or Certified Hydrogeologist responsible for developing the dewateringplan and operation oversight of the dewatering system.

(2) In charge of the design, construction, operation, and maintenanceof at least 3 successful projects in the last 5 years similar in nature to that required by this contract with similar subsurface soil site conditions, dewatering depths, construction techniques,and project durations.

b. Dewatering Operations Superintendent(s)

(1) Specifically trained and under supervision of dewateringspecialist.

(2) Involved with the construction, operation, and maintenance of at least 3 successful projects in the last 5 years similar in natureto that required by this contract with similar subsurface soil site conditions, dewatering depths, construction techniques, and project durations.

c. Drill Crew Foreman

(1) Serves as the lead driller for installation of wellpoints andtemporary piezometers.

(2) Must have a minimum of 5 years experience drilling on or around embankment dams with equipment and procedures to be used during installation of wellpoints and temporary piezometers including demonstrated experience installing wellpoint dewatering systems.

(3) Must hold a current State[redacted] well-driller license and meetall driller requirements of ER 1110-1-1807.

1.8 COMPANY EXPERIENCE

Minimum of 5 years of experience in dewatering designs and at least 3 successful projects similar in nature to that required by this contract.

1.9 GOVERNMENT RESPONSIBILITY

a. The Government has designed the dewatering system, including thewellpoint locations, depth, spacing, layout, pipe size, pump requirements/locations, and discharge locations.

b. The Government will evaluate the originally designed dewatering systemperformance based on the submitted performance tests report. If the system was successful during the performance test in achieving groundwater drawdown of at least 3 feet below the base of the excavation, the Contracting Officer will provide final approval of the Dewatering System. If the system was not successful during performancetesting, the Government will take up to 10 days to evaluate the performance test reports and contractor-provided guidance to improve the system and achieve required drawdown. By the 10th calendar day, the Contracting Officer will provide the contractor written notification of approved alterations to the dewatering system, which may include installation of supplementary wellpoints.

1.10 CONTRACTOR RESPONSIBILITY

a. Furnish and install Government designed dewatering facilities accordingto Plans and other specified requirements. Provide completion reports with location, surface elevation, final drilling depth and screened interval for installed wellpoints and temporary piezometers

b. Dewatering specialist(s) and/or dewatering operations superintendent may not be rotated, transferred, or otherwise replaced without priornotification to the Contracting Officer.

c. At designated discharge locations, design, construct, and maintainsedimentation controls to meet requirements of 01 57 19 TEMPORARY ENVIRONMENTAL CONTROLS.

d. Maintain and operate the dewatering system on a 24-hour per day, 7-day per week basis so that the groundwater level is maintained at a depth 3feet below the base of the excavations for the installation of the seepage collection trench, removal of existing waterlines, and removal of existing seepage collection systems at all times when excavations are open.

e. Provide an on-site backup pump for each dewatering pump in the system during all times when the system is in operation. Install and make thebackup pump fully operable within 60 minutes in event of a pump failure. If a backup pump is used, the original must be repaired to operable condition or replaced within 24 hours.

f. Monitor and evaluate effectiveness of dewatering systems while in operation. Provide daily dewatering reports of water levels in temporary piezometers and monitoring sumps, and flowmeter measurements. Record the time and measurement of the piezometer, monitoring sump, and flowmeter. The minimum frequency of readings required is dependent on the pool elevation o[redacted] as shownin Table 1.

Table 1	
Pool Elevation (feet)	Frequency of temporary piezometer readings

528 and below	Hourly
528 and above	1 reading every 30 minutes

g. Provide the following minimum required personnel:

(1) Dewatering specialist: On site a minimum of one shift per day during wellpoint dewatering system installation and on-call 24 hours per day, 7 days per week and able to reach the site within 8hours while dewatering systems are in operation.

(2) Dewatering operations superintendent(s): On site daily, 7 days perweek while dewatering systems are in operation. The dewatering operations superintendent is responsible for all QC related to thedewatering system installation and operation.

(3) Personnel experienced in pump operation and maintenance: On site 24 hours per day, 7 days per week while the dewatering system isin operation. The dewatering specialist and/or dewatering operations superintendent can fulfill this requirement if qualified.

(4) If dewatering is occurring concurrently at multiple locations underthis contract, only one set of personnel is required

h. The dewatering specialists and the dewatering operations superintendent(s) only responsibility is installation, operation,maintenance, repair, and monitoring of the dewatering systems.

i. In the event of an unsuccessful performance test, provide suggested alterations to improve system. Begin approved alterations to dewatering system, including installation of supplementary wellpoints,within 7 days after written notification is received from the Government to proceed with alterations. Complete installation within 21 days after receipt of notification.

j. Operate and monitor the fully installed dewatering system during the performance and rebound testing. Upon Government approval of the dewatering system, and prior to any excavation work commencing, the system will be operated until groundwater levels are a minimum of 3 feet below the excavation depth and have been maintained at this levelfor a minimum of 3 days. The Contracting Officer will provide notification when excavation work may begin.

k. Notify the Contracting Officer immediately in the event of a pumpfailure or other damage that makes the system inoperable.

l. Remove pumps, equipment, and materials associated with the dewatering systems from the site at the end of the project including related powersystems. Removed materials and equipment will remain the property of the Contractor.

m. Provide spare parts and other equipment required to repair dewatering system in the event of damage. Damaged or destroyed dewatering systemsor any component of those systems, resulting from construction activities and/or human error will be repaired or replaced by the Contractor at the Contractor's expense.

PART 2 PRODUCTS

2.1 DEWATERING SYSTEM

The dewatering system consists of a single stage of wellpoints designed by the Government with locations and depths given in the Plans.

2.1.1 Wellpoints and Header Pipe

a. Dewatering Wellpoints are 1.5-inch diameter, Schedule 40 ASTM D1785 PVCpipe and screen installed in a drilled hole with a minimum diameter of 6-inches. Use the same supplier for the casing and screen with compatible watertight threaded joints to permit any combined makeup of casing and screen. Slots must be 0.025-inch in width and spaced 3/16 inch center-to-center for a maximum open area of 4.24 square inches perfoot of slotted pipe. Use only slots installed by the manufacturer. Donot install slots in the field. Surround the entire slotted section ofthe pipe with silica sand of gradation specified in Subpart 2.3.
Required hole depths and screened intervals are shown on the Plans. Install a bentonite seal from the ground surface to a depth of 3 feet.Construct the bentonite plug using the same technique described in Section 31 09 13.00 INSTRUMENTATION AND MONITORING

b. Header pipe consists of 6-inch diameter smooth interior wall PVC or HDPE pipe with saddle connections spaced on 4-foot centers. Each wellpoint is connected to the header pipes via the saddle with flexibleswing connections and associated valves allowing for control of flow inindividual wellpoints. Use clear pipe for the swing connection to allow for flow observations during operation. Plug all unused saddle connections with an airtight plug.

2.1.2 Pumps

Use 4-inch intake diameter vacuum-assisted dewatering pumps. Pumps must be capable of handling a range of flow values and total dynamic heads and must have sufficient air-handling capability to maintain pumping rate and achieve drawdown. Equip pumps with sand-trap or similar device to prevent soil material from entering pump. Connect to the 6-inch header pipe using a PVC Schedule 40 reducer. Pumps must be capable of pumping flows of approximately 40 GPM at a Total Dynamic Head (TDH) of 10 feet to 300 GPM at a TDH of 35 feet, require a maximum Net Positive Suction Head (NPSH) of 8 feet at the flow and head conditions specified, and operate at a minimum efficiency of 60%. In the event of lower than anticipated wellpoint system yields, throttle pumps or employ recirculation systems.

2.1.3 Back-Up pumps

Use backup pumps with the same capacity and pumping capabilities as the pump they are replacing. If the primary pumps use an electric power source, use diesel-powered backup pumps.

2.1.4 Flowmeters

Install flowmeters with an accuracy of plus or minus 10 percent on the pump side of each wellpoint segment shown on the Plans. Use propeller, turbine, acoustic, orifice, or similar type of meter installed in the discharge line according to manufacturer's suggestions. Do not use Pitot tube devices. Calibrate meters by volumetric means before operation and at other times if accuracy is questioned. Replace meters that do not correctly calibrate.

2.2 TEMPORARY PIEZOMETERS

a. Install temporary piezometers at the locations and depths indicated inthe Plans.

b. Temporary piezometers must be 2-inch diameter, Schedule 40, PVC pipe installed in 6 inch diameter drilled hole. Provide slots for the bottom 5 feet of the pipe. Slots must be 0.01 inch in width and spacedat a maximum of 3/16 inch center-to-center for a maximum open area of 2.51 square inches per foot of pipe. Use only screens with slots installed by the manufacturer. Do not install slots in the field. Plug the bottom of the pipe with a PVC plug. Place a removable cap atthe top. Backfill around the entire slotted section and casing with piezometer sand pack of the gradation and characteristics given in Section31 09 13.00. Place a bentonite plug using method described in Section 31 09 13.00 with a minimum thickness of 3 feet above the sand backfill to the surface elevation. Prepare completion reports for each piezometer installation. Report forms will be provided by the Government.

c. Test temporary piezometers by adding or removing water from the well risers and subsequently monitoring the stabilization of the water levels in the wells to demonstrate that temporary piezometers are functioning properly before taking water level readings. Replace anytemporary piezometer that does not function properly.

2.3 WELLPOINT SAND PACK

Select wellpoint sand pack consisting of washed, clean, silica sand composed of hard, tough, and durable particles free from any coating. Filter sand must fit the gradation given in Table 2 with sieve sizes from ASTM C136/C136M. The filter material must not contain detrimental quantities of organic matter or soft, friable, thin, or elongated particles according to the quality requirements in ASTM C33/C33M. Crushed limestone is not allowed.

Table 2

U.S. Standard Sieve Size	Allowable Percentage Passing by weight
3/8"	100
#8	80-100
#16	60-100
#20	25-95
#40	0-35
#60	0-15

#80	0-5

2.4 SUPPLEMENTARY WELLPOINTS

Install supplementary wellpoints using the same design and methods as used for the initial design. Connect supplementary wellpoints to the existing pumps and header lines. Location, depth, and other pertinent information about supplementary wellpoints, if needed, will be provided by the Contracting Officer.

2.5 EXPLORATION BORINGS

Provide drilling logs for at least every 5th boring completed for installation of wellpoints to be used as verification of site geology. A licensed Professional Geologist or licensed Geotechnical Engineer must log the borings.

2.6 MONITORING SUMPS

Monitoring sumps consist of 12-inch borings drilled to a depth of 5 feet below seepage collection trench subgrade along the centerline of each excavation. Install 10-inch diameter PVC casing with a maximum screen open area of 17.45 square inches/linear foot of casing. Fill the area between the edge of the boring and casing with ASTM C33 fine aggregate. Provide 2 monitoring sumps for every 100 feet of open excavation. Abandon monitoring sumps by removing casing and backfilling with ASTM C33 fine aggregate.

PART 3 EXECUTION

3.1 GENERAL

a. Furnish, install, maintain, and operate the Government designed dewatering system that prevents loss of fines, boiling, quick conditions, or softening of foundation strata and maintain stability ofbottom of excavations so that every phase of the work can be performed in the dry. The dewatering operations must be operated such that excavation bottoms are firm, suitably dry, and free from standing waterat all times.

b. Lowering of the groundwater level a minimum of 3 feet below the maximumexcavation limits for removal of existing waterlines and seepage collection systems and installation of the seepage collection trench must be verified by temporary piezometers readings before commencement of excavations.

c. During construction, provide devices to remove water entering excavations promptly and dispose of properly. Keep bottoms of excavations firm and free of standing water until construction is completed and backfill is placed. Conduct pumping and dewatering such that no disturbance to foundation subgrade materials or to fill materials supporting any other work will result. Pipe discharged waterto an approved area. Do not discharge water into the reservoir. Dewatering discharge must not cause siltation or other negative environmental impact on natural waterways or other property; such discharge must be according to applicable federal, state, and local permit regulations. At dewatering discharge locations, install control measures as specified in Section 01 57 19TEMPORARY ENVIRONMENTAL CONTROLS.

d. Discharge water from the dewatering systems may be used forconstruction activities if approved by Contracting Officer.

3.2 INSTALLATION AND OPERATION

a. Obtain all required drilling permits according to federal, state,and local regulations prior to commencement of drilling and installation of the dewatering system (including wellpoints and temporary piezometers).

b. Use drilling method(s) from the approved Dewatering Plan for installation of wellpoints and temporary piezometers. The use of jetting for wellpoint installation is not permitted for work under thiscontract.

c. Install the dewatering system in the following sequence:

(1) Install temporary piezometers at locations indicated in the Plans.

(2) Drill 6-inch exploration borings to shale interface using methodsfrom approved Dewatering Plan. Exploration borings surrounding agroup of wellpoint holes must be completed and logged prior to drilling holes for installation of wellpoints.

(3) Any borehole deemed by the Contracting Officer to not meet requirements or observed to be caving, sloughing, or unstable mustbe abandoned. Abandon disapproved boreholes according to requirements found in Subpart 3.3 and applicable local, state, andfederal regulations. Drill a replacement borehole in close proximity to the abandoned hole as approved by Contracting Officer. No payment will be made for abandoned boreholes.

(4) Install the 1.5-inch diameter wellpoints in the 6-inch borings ensuring that the screened interval is placed in the sand layer. Estimated screened intervals are given in the Plans but the Government will provide the final location based on boring logs from the nearest exploration borings. Backfill the boring aroundthe wellpoint screen and riser pipe with sand pack. Place a bentonite seal with a minimum thickness of 3 feet from the top ofthe sand pack to the ground surface elevation.

d. Locations of dewatering wellpoints, header lines, pumps, discharge points, and temporary piezometers are given in the Plans. Minimize interference of excavation and construction activity with the locationof other features related to the dewatering system. Locations are subject to approval by the Contracting Officer.

e. Performance Test: After installation of the dewatering system, including wellpoints, header lines, pumps, discharge lines, sediment control features, and temporary piezometers, operate the system for 7 consecutive days or until temporary piezometer readings indicate that adrawdown of the groundwater elevation of 3 feet below the bottom of theexcavation has been achieved and maintained for 3 days.

Record the water level in the temporary piezometers and flowmeter readings every
15 minutes for the first 4 hours of the test and hourly for the remainder of the test. If the first Performance Test indicates thatthe system design is insufficient and alterations are made to the design, perform a second Performance Test to evaluate the changes. Provide the piezometer readings and flow measurements in the performance tests report.

f. Rebound Test: After the completion of performance testing, monitor andreport the recovery of groundwater levels after the system is shut
off. Record the water level in each temporary piezometer on 15 minute intervals for the first 4 hours after the system is shut off, and hourly thereafter for 20 hours or until the water level is within 5% ofthe pre-pumping initial condition. Provide the piezometer measurementsduring the rebound test as a part of the performance tests report.

g. Operate the dewatering system continuously 24 hours per day, 7 days perweek. Maintain the groundwater levels at least 3 feet below the maximum excavation limits and do not allow the water level to rise until excavations are completely backfilled. The Contracting Officer must approve the cessation of operation of part or all of the dewatering system(s). The Contractor is responsible for any damage resulting from failure to maintain the dewatering system. In the eventemergency backfilling is required, disruption of dewatering operations is not permitted.

h. Provide complete standby equipment and power sources available for immediate operation as may be required, to adequately maintain the dewatering in the event that all or any part of the dewatering systembecomes inadequate or fails.

j. Contact the Contracting Officer immediately if two consecutive measurements from the temporary piezometers monitoring holes indicate that the groundwater level is within 30 inches of the maximum excavation limits or if any single temporary piezometer or monitoring hole measurement indicates the groundwater level is within 2 feet of the maximum excavation limits of any open excavation. Record the waterelevation in the temporary piezometers every 30 minutes after either ofthe above mentioned events has occurred and maintain that monitoring frequency until directed otherwise by the Contracting Officer.

k. If wells are not constructed with specified materials and to specifieddimensions, remove and replace well(s) at contractor's expense.

l. When the dewatering system does not meet the specified requirements as a result of Contractor's failure to adequately operate or maintain the system, and as a consequence, loosening or disturbance of the foundations strata, instability of the slopes, or damage to the foundations or structures occurs, the Contractor is responsible for supplying all materials and labor and performing all work for restoringfoundation soils, slopes, foundations, and structures, to the satisfaction of the Contracting Officer, at no additional expense to the Government.

m. In the event that dewatering system fails due to Contractor's failure to adequately operate or maintain the system, or due to the system being made inoperable due to damage from construction equipment and thegroundwater level rises to the levels necessitating emergency backfill as specified in 31 00 00 EARTHWORK, provide all materials and labor foremergency backfill at Contractor's expense.

n. Purchase, rental, installation, or mobilization to the site any elements of the dewatering system before approval is at the Contractorsrisk and will not be reimbursed or compensated if not used. The drill rig used for the wellpoint borings must remain on the project site through the wellpoint installation, dewatering system setup and for a minimum of 14 calendar days following the completion of the first
Performance Test. This requirement applies to both Seepage Area 1 and 2.

3.3 REMOVAL

a. Obtain written approval from the Contracting Officer before discontinuing operation of any portion of the dewatering system(s).

b. Remove all equipment from the dewatering system(s) and temporarypiezometers from the site at the completion of dewatering work.

c. Abandon temporary piezometers and wellpoints by overdrilling with ahollow stem auger and backfilling holes within the footprint of the blanket filter with ASTM C33/C33M fine aggregate. Backfill holesoutside of the blanket filter limits with approved grout shown in02 41 00 DEMOLITION.

-- End of Section --

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Glossary

Abbreviations and Terms.

ASTM ASTM International, formerly American Society for Testing Materials

ATP adenosine triphosphate test

AWWA American Water Works Association

cfs cubic feet per second

cm centimeters

cm/sec centimeters per second

DSM deep soil mixing

EM Engineer Manual

ETL Engineer Technical Letter

ft feet

ft-lbs foot-pounds

ft/min feet per minute

ft/sec feet per second

gpm gallons per minute

HDPE high density polyethylene

HPC heterotrophic plate count

HSA Hollow Stem Auger

k hydraulic conductivity

kh horizontal hydraulic conductivity

kv vertical hydraulic conductivity

LSI Langelier Saturation Index

mm millimeter(s)

MMS multimedia messaging service

mV millivolts

ORP oxidation reduction potential

NOAA National Oceanic and Atmospheric Administration

PFDS Precipitation Frequency Data Server

ppm parts per million

psi pounds per square inch

PVC polyvinyl chloride

PVD prefabricated vertical drain

Reclamation U.S. Department of Interior Bureau of Reclamation

SMS short message service

Sy specific yield

TDS total dissolved solids

USACE U.S. Army Corps of Engineers

WES U.S. Army Engineer Waterways Experiment Station

Anisotropy – Variability of a soil causing the horizontal hydraulic conductivity to be different than the vertical hydraulic conductivity. Typically, natural deposits and manmade fill will have greater horizontal than vertical hydraulic conductivity because they are placed in a horizontal fashion, causing them to be stratified.

Arching – The soil property in which stresses distribute onto stiffer elements, such as rock formation or a concrete structure, in such a way that the vertical stresses over softer areas are less than the overburden pressure.

Artesian flow – Seepage through the pervious aquifer is confined between two or more impervious strata, and the piezometric head within the pervious aquifer is above the top of the pervious aquifer.

Bedrock – A general term that includes any of the generally indurated or crystalline materials that make up part of the Earth's crust. Individual stratigraphic units or units significant to engineering geology within bedrock may include poorly or nonindurated materials such as beds, lenses, or intercalations.

Blowout – When a relatively impermeable soil (or confining layer) is present downstream of an embankment dam, the pressure associated with seepage moving through the underlying pervious layer may increase until it exceeds the weight of the overlying soil. This can cause the confining layer to lift off of the underlying soil and/or rupture. Also known as "Uplift".

Clean – A soil gradation that contains less than 5 percent fines by weight.

Conduit – Typically a pipe, box, or horseshoe structure that is constructed by means of "cut and cover." A conduit can convey water or house other conduits, pipes, cables, wires, etc.

Cutoff – A vertical barrier, usually constructed in a deep vertically sided trench. The backfill in the trench can be a variety of materials including concrete, soil-bentonite, and soil-cement-bentonite. A wall of impervious material (e.g., concrete, timber, steel sheet piling) located in the foundation, which forms a water barrier to reduce underseepage.

Dam – An artificial barrier that has the ability to impound water, wastewater, or any liquid-borne material for the purpose of storage or control of water.

Deep Well – A dewatering well equipped with its own submersible pump.

Dewatering – The process of removing water from an embankment or foundation to be excavated.

Discharge point – The end of a drain system where flow is discharged into some other watercourse or drainage way.

Drain – Typically, a second stage of a filter/drain system consisting of gravel. A feature designed to collect water and convey it to a discharge location. Typically, a drain is intended to relieve excess water pressures.

Drain pipe – A system of pipe used to collect seepage from the excavation and convey it to a discharge point.

Equipotential Lines – Represents segments of constant piezometric head in flow net diagrams. These lines pass through the flow lines and are spaced such that the flow lines and equipotential lines create squares.

Erosion – Removal of soil grains by either surface water flow or seepage through the ground.

Filter – A zone of material designed and installed to provide drainage, yet prevent the movement of soil particles due to flowing water. A material or constructed zone of earthfill that is designed to permit the passage of flowing water through it but prevents the passage of significant amounts of suspended solids through it by the flowing water.

Fines – The soil grain sizes that are smaller than the No. 200 sieve (0.075 mm) as used in the USCS.

Flexible pipe – A pipe that derives its load carrying capacity by deflecting at least 2 percent into the surrounding medium upon application of load.

Flood – A temporary rise in water surface elevation resulting in inundation of areas not normally covered by water.

Flow Net – An illustration of seepage conditions under given geometry and boundary conditions. It shows how pressures are distributed and where flow is being directed based on a given set of assumptions that simplify the real life situation.

Geotextiles – Any fabric or textile (natural or synthetic) when used as an engineering material in conjunction with soil, foundations, or rock. Geotextiles have the following uses: drainage, filtration, separation of materials, reinforcement, moisture barriers, and erosion protection.

Gradation – The distribution of particles of granular material among standard sizes usually expressed in terms of cumulative percentages larger or smaller than each of a series of sieve openings.

Gradient – The change in head loss of a given distance. Also the property used to evaluate the potential for seepage water to move (erode) a soil particle.

Grain size distribution – A visual representation of the percentage of specified soil particle sizes relative to one another.

Gravel – Materials that will pass a 3-inch (76.2-millimeter [mm]) and be retained on a No. 4 (4.75-micrometer [µm]) U.S. standard sieve.

Gravity flow – The surface of the water table is below the top of the pervious aquifer.

Grout – A fluidized material that is injected into soil, rock, concrete, or other construction material to seal openings and to lower the hydraulic conductivity and/or provide additional structural strength. There are four major types of grouting materials: chemical, cement, clay, and bitumen.

Grout mix – The proportions or amounts of the various materials used in the grout, expressed by weight or volume (the words “by volume” or “by weight” should be used to specify the mix).

Head – The vertical difference, typically expressed in feet, between two water surface elevations.

Heave – Specific type of internal erosion caused by seepage moving vertically upward through granular soil (no confining layer). When effective stress equals zero.

Heterogeneous – Consisting of dissimilar constituents. For soils, consisting of several soil types.

Homogeneous – Consisting of similar constituents. For soil, consisting of a single soil type.

Hydraulic conductivity – The proportionality constant, sometimes referred to as the coefficient of permeability, is a measure of the ease with which water will flow through a porous material such as soil. Hydraulic conductivity has units of length divided by time; commonly expressed as centimeters per second (cm/sec) for soils or feet per day (ft/day) for open graded gravels. See related term permeability.

Hydraulic fracture – A separation in a soil or rock mass that occurs if the applied water pressure exceeds the lateral effective stress in the mass. Hydraulic fracture may occur in vertical cracks transverse to the dam axis or other defects. Soils compacted dry of optimum water content are more susceptible to hydraulic fracture.

Hydraulic gradient – The slope of the hydraulic grade line. The hydraulic gradient is the slope of the water surface in an open channel.

Hydrostatic pressure – The pressure exerted by water at rest.

Impervious – Not permeable; not allowing liquid to pass through easily.

Instrumentation – An arrangement of devices installed into or near the dewatering system and excavation that provide for measurements that can be used to evaluate the structural behavior and performance parameters of the dewatering system and excavation.

Isotropic – Uniformity of a soil in that the horizontal hydraulic conductivity is the same as the vertical hydraulic conductivity.

Joint – A natural fracture that forms by tensile-loading walls. Walls of fracture move apart slightly as joints develop.

Leakage – Uncontrolled loss of water by flow through a hole or crack.

Levee – An embankment whose primary purpose is to furnish flood protection from seasonal high water.

Open cut – An excavation through rock or soil made through topographic features.

Perforated pipe – A pipe intended to collect seepage through holes or slots on its exterior.

Permeability – A property of the structure and composition of a porous media that is an indication of the ability of water to flow through it. Permeability is expressed in units of m^2 or the darcy (D). See related term hydraulic conductivity.

Pervious – Permeable, having openings that allow water to pass through.

Phreatic surface – The planar surface between the zone of saturation and the zone of aeration. Also known as free-water surface, free-water elevation, ground water surface, and ground water table.

Piezometer – An instrument for measuring fluid pressure (air or water) within soil, rock, or concrete. A device for measuring the pore water pressure at a specific location in earthfill or foundation materials. Also called an observation well.

Pore pressure – The interstitial pressure of a fluid (air or water) within a mass of soil, rock, or concrete.

Quality control – A planned system of inspections that is used to directly monitor and control the quality of a construction project. Construction quality control is normally performed by the contractor and is necessary to achieve quality in the constructed system. Construction quality control refers to measures taken by the contractor to determine compliance with the requirements for materials and workmanship as stated in the plans and specifications for the project. An example of a quality control activity is the testing performed on compacted earthfill to measure the dry density and water content. By comparing measured values to the specifications for these values based on the design, the quality of the earthfill is controlled.

Relative density – A numerical expression that defines the relative denseness of a cohesionless soil. The expression is based on comparing the density of a soil mass at a given condition to extreme values of density determined by standard tests that describe the minimum and maximum index densities of the soil. Relative density is the ratio, expressed as a percentage, of the difference between the maximum index void ratio and any given void ratio of a cohesionless, free-draining soil to the difference between its maximum and minimum index void ratios.

Relief well – A vertical well near the downstream toe of the dam or levee used to relieve pressure in a deeper foundation layer that is under high pressure.

Risk – A measure of the likelihood and severity of adverse consequences.

Rock – Lithified or indurated crystalline or noncrystalline materials. Rock is encountered in masses and as large fragments, which have consequences to design and construction differing from those of soil.

Sand – Particles of rock that will pass the No. 4 (4.75-µm) sieve and be retained on the No. 200 (0.075-mm) U.S. standard sieve.

Seepage – The infiltration or percolation of water through rock or soil or from the surface.

Segregation – The tendency of particles of the same size in a given mass of aggregate to gather together whenever the material is being loaded, transported, or otherwise disturbed. Segregation of filters can cause pockets of coarse and fine zones that may not be filter compatible with the material being protected.

Silt – Material passing the No. 200 (75-µm) U.S. standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air dried.

Slope – Inclination from the horizontal. Sometimes referred to as batter when measured from vertical.

Slotted pipe – See "Perforated pipe."

Slough – See "Slump."

Slump – Movement of a soil mass downward along a slope.

Specific yield of aquifer – The volume of water that can be drained by gravity from a saturated unit volume of material.

Stability – The resistance to sliding, overturning, or collapsing.

Storage – The retention of water or delay of runoff either by planned operation, as in a reservoir, or by temporary filling of overflow areas, as in the progression of a flood wave through a natural stream channel.

Storativity – Volume of water released from, or taken into, storage per unit surface area of the aquifer per unit change in the component of hydraulic head normal to that surface. This term is nondimensional and expressed as a percent. For artesian aquifers, storativity is equal to the water forced from storage by compression of a column of the aquifer by the additional load created by lowering the artesian pressure in the aquifer by pumping or drainage. For gravity flow aquifers, storativity is equal to the specific yield of the material being dewatered plus the water forced from the saturated portion of the aquifer by the increased surcharge caused by lowering the groundwater table.)

Uniform gradation or uniformly graded – A soil gradation consisting primarily of soils grains that are near the same size. Also known as narrowly graded or poorly graded.

Uplift – The pressure in the upward direction against the bottom of a structure such as an embankment dam or conduit or a soil stratum.

Vadose Zone – also known as the unsaturated zone, region between the top of the phreatic zone and the ground surface.

Well-graded – A soil gradation consisting of several soil sizes that form a smooth gradation curve when plotted on a logarithmic scale.

Wellpoint – Commonly used dewatering method as they are applicable to a wide range of excavation and groundwater conditions. Wellpoints differ from wells only in the way they are pumped, and wellpoint systems can be designed to match the performance of the well system.