ENGINEERING GEOLOGY FIELD MANUAL

SECOND EDITION

VOLUME II

ENGINEERING GEOLOGY FIELD MANUAL

SECOND EDITION

VOLUME II

2001

U.S. Department of the Interior
Bureau of Reclamation

The Mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to tribes.

The Mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Information contained in this manual regarding commercial products or firms may not be used for advertising or promotional purposes and is not an endorsement of any product or firm by the Bureau of Reclamation.

The information contained in this manual was developed for the Bureau of Reclamation; no warranty as to the accuracy, usefulness, or completeness is expressed or implied.

Acknowledgments
for Second Edition
Volume 2

The original compilation and preparation of this manual involved many engineering geologists, geophysicists, and engineers within Reclamation. Their input is greatly appreciated. This second edition incorporates comments on the first edition and technological changes since the first edition was prepared approximately 13 years ago. Without the comments and input from the Denver, Regional, and Area Offices, the revision would not have happened. Special thanks to Sam Bartlett for his support and input throughout the preparation of the second edition and to James Krulik who saw to the completion of this edition.

Although there are too many people to acknowledge individually who contributed to the revisions and the second edition, Jack Cunningham, Robert Bianchi, Jeff Farrar, David Gillette, Sandy Kunzer, Richard Markiewicz, Ronald Pearson, Peter Rohrer, Ulrich Schimschal, and Andy Viksne made especially significant contributions. Mark McKeown made notable contributions, wrote several new chapters, and edited the second edition.

Continued recognition is given to Jerry S. Dodd who initiated the manual; Jerry's successor, Newcomb Bennett, who kept the manual moving; and to Steve D. Markwell, who saw the first edition completed. We extend our thanks and appreciation to Louis R. Frei, who helped establish and document many geological standards of practice; to Richard H. Throner, who wrote much of the original manual and assembled and served on committees for preparation and review; to Sam R. Bartlett, who compiled and printed the early loose leaf version of the manual; and to Mel Hill, who completed the publication of the first edition. To the Regional Geologists and their staffs and the many geotechnical engineers who offered comments incorporated into the manual, we extend our thanks and appreciation for their work as well. The manual would not be complete without the drawings and figures; to the engineering and physical science technicians we extend our gratitude and thanks. We further acknowledge Robert Rood and Patty Alexander, the technical writers who assisted in the editing and helped prepare the manual for printing.

FOREWORD
TO THE SECOND EDITION
VOLUME 2

Approximately 13 years have gone by since the first edition of the manual was published, and technology, methodology, and missions have changed significantly. This second edition incorporates many modifications and additions. The Global Positioning System (GPS) has revolutionized how we survey and locate ourselves in the field, computers are used extensively to collect and evaluate data, and computer aided modeling, design, and drafting are almost universal. Reclamation's current mission places greater emphasis on maintenance and safety of infrastructure, dam safety analyses and modifications, and water resource management than on design and construction of new hydraulic structures. Techniques for these activities are reflected in this edition.

A few of this edition's most significant changes to the manual are the addition of a section on water testing for grouting, an expanded chapter on permeability testing, a chapter on the global positioning system, a significantly modified chapter on rip rap, a chapter on foundation preparation, treatment, and cleanup, a chapter on waxing, preserving, and shipping samples, and an index to facilitate finding relevant information. Many other suggested revisions and improvements collected since the manual was first published also are incorporated. Volume I contains material commonly needed in the field, and Volume II includes reference and supplementary information and materials.

As in the first edition, the *Engineering Geology Field Manual* presents the practices for the collection of geologic data obtained by the Bureau of Reclamation. The manual establishes common guidelines, procedures, and concepts for the collection, evaluation, and presentation of geologic information. The analysis of geologic conditions, the preparation of designs and specifications, and effective monitoring of construction require consistent, comprehensive, and timely geologic information. The use of these guidelines by all Reclamation engineering geologists collecting, documenting, evaluating, and presenting geological and geotechnical data promotes consistency, helps assure that the required evaluations and data are complete, and promotes integration and coordination of geological and engineering activities.

The *Engineering Geology Field Manual* forms the basis for the mutually beneficial exchange of ideas by geologists and engineers. Experienced geologists will find useful reminders, new procedures, and special techniques, while less experienced engineering geologists and those from other disciplines can use the manual to expand their familiarity with geology as practiced in the geotechnical field.

Review and comments on the manual are encouraged, and if you have comments or suggested additions, please forward them to the Engineering Geology Group at Reclamation's Technical Service Center.

Richard H. Throner
Chief, Geotechnical
Services Division

CONTENTS

Page

CONTENTS

CONTENTS

Page

Page

CONTENTS

CONTENTS

Page

CONTENTS

Page

Page

CONTENTS

APPENDICES

TABLES

CONTENTS

FIGURES

FIGURES (continued)

Figure *Page*

FIGURES (continued)

FIGURES (continued)

CONTENTS

FIGURES (continued)

FIGURES (continued)

Figure *Page*

CONTENTS

FIGURES (continued)

Figure *Page*

FIGURES (continued)

Chapter 13

SURFACE GEOPHYSICAL INVESTIGATIONS

Introduction

Surface geophysical surveys have been applied to mineral and petroleum exploration for many years. A magnetic compass was used in Sweden in the mid-1600s to find iron ore deposits. The lateral extent of the Comstock ore body was mapped using self-potential methods in the 1880s. A very crude type of seismic survey measured the energy resulting from blasting operations in Ireland in the late 1800s. The idea that energy travels through a material with a certain velocity came from this survey. During World War I, geophysical techniques were used to locate artillery pieces. Anti-submarine warfare in World War II led to magnetic and sonar surveys.

The main emphasis of geophysical surveys in the formative years was petroleum exploration. Technology developed for oil and gas surveys led to the use of geophysical surveys in many important facets of geotechnical investigations. Geophysical surveys have been applied to civil engineering investigations since the late 1920s, when seismic and electrical resistivity surveys were used for dam siting studies. A seismic survey was performed in the 1950s in St. Peter's Basilica to locate buried catacombs prior to a renovation project. From the late 1950s until the present time, geophysical techniques have had an increasing role in both groundwater exploration and in geotechnical investigations. Geophysical surveys are now used routinely as part of geological investigations and to provide information on site parameters (i.e., in place dynamic properties, cathodic protection, depth to bedrock) that in some instances are not obtainable by other methods. Values derived from seismic geophysical surveys are obtained at strain levels different from some site parameters obtained by other means.

All geophysical techniques are based on the detection of contrasts in different physical properties of materials. If contrasts do not exist, geophysical methods will not work. Reflection and refraction seismic methods contrast compressional or shear wave velocities of different materials. Electrical methods depend on the contrasts in electrical resistivities. Contrasts in the densities of different materials permit gravity surveys to be used in certain types of investigations. Contrasts in magnetic susceptibilities of materials permit magnetic surveying to be used in some investigations. Contrasts in the magnitude of the naturally existing electric current within the earth can be detected by self-potential (SP) surveys.

Seismic refraction surveys are used to map the depth to bedrock and to provide information on the compressional and shear wave velocities of the various units overlying bedrock. Velocity information also can be used to calculate in place small-strain dynamic properties of these units. Electrical resistivity surveys are used to provide information on the depth to bedrock and information on the electrical properties of bedrock and the overlying units. Resistivity surveys have proven very useful in delineating areas of contamination within soils and rock and also in aquifer delineation. Gravity and magnetic surveys are not used to the extent of seismic and resistivity surveys in geotechnical investigations, but these surveys have been used to locate buried utilities. Self-potential surveys have been used to map leakage from dams and reservoirs.

Geophysical surveys provide indirect information. The objective of these surveys is to determine characteristics of subsurface materials without seeing them directly. Each type of geophysical survey has capabilities and limitations and these must be understood and considered when designing a geophysical investigations program.

Geophysical interpretations should be correlated with real "ground-truth" data such as drill hole logs. It is very important that the results of geophysical surveys be integrated with the results of other geologic investigations so that accurate interpretation of the geophysical surveys can be made.

The following sections provide the theory behind and guidelines for uses of geophysical surveys, particularly in geotechnical investigations. Although this chapter does not provide all the detail necessary, the theory and interpretation methods involved in geophysical surveying are included in references in the bibliography. The references should be used to supplement the materials presented in this chapter.

Seismic Surveys

Seismic Refraction Surveys

Purpose.—Seismic refraction surveys are used to determine the compressional wave velocities of materials from the ground surface to a specified depth within the earth. For most geotechnical investigations, the maximum depth of interest will be specified by the nature of the project. In many cases the objective of a seismic refraction survey is to determine the configuration of the bedrock surface and the compressional wave velocities of the underlying materials. Bedrock may be defined by compressional wave velocities. The information obtained from a seismic refraction survey is used to compute the depths to various subsurface layers and the configurations of these layers. The thickness of the layers and the velocity contrasts between the layers govern the effectiveness and the accuracy of the survey. Seismic refraction surveys do not provide all compressional wave velocities or delineate all subsurface layers. Seismic refraction interpretation assumes that layer velocities increase with depth.

Applications.—Seismic refraction surveys have been used in many types of exploration programs and geotechnical investigations. The initial application of these surveys was mapping of salt domes in the early days of oil exploration. Seismic refraction surveys are now routinely used in foundation studies for construction projects and siting studies, fault investigations, dam safety analyses, and tunnel alignment studies. Seismic refraction surveys are also used to estimate rippability (Appendix C). Figure 13-1 is a schematic of a seismic refraction test.

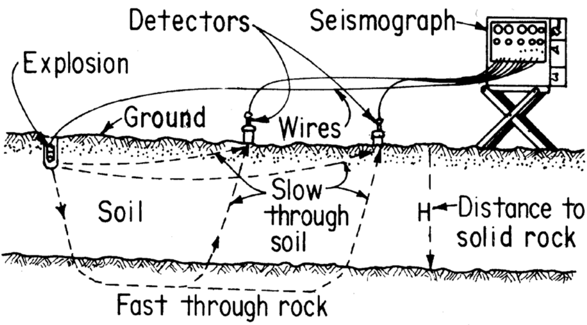


Figure 13-1.—Simplified diagram of a seismic refraction test.

Seismic Reflection Surveys

Purpose.—Seismic reflection surveys have been used successfully in petroleum and geothermal exploration projects and to investigate for shallow coal. The information obtained from seismic reflection surveys can be used to define the geometry of the different subsurface layers and structural features.

Applications.—High resolution seismic reflection surveys provide definitive information on the locations and types of faults, as well as the location of buried channels. Shallow, high resolution seismic reflection surveys are playing an increasingly important role in geotechnical investigations. When correctly used, seismic reflection surveys may provide data that seismic refraction surveys can not (e.g., velocity reversal information). However, compressional (P) wave velocity information derived from reflection surveys may not be as accurate as from refraction surveys. The compressional wave velocities are needed for the analysis of the reflection records themselves and for seismic refraction surveys, uphole velocity surveys, and sonic logs.

Shear Wave Surveys

Purpose.—Shear (S) waves travel through a medium at a slower velocity than compressional (P) waves and arrive after compressional waves. Other types of secondary arrivals also exist due to reflections, combinations of reflections and refractions, and surface waves. Field survey techniques are designed to suppress compressional and unwanted reflected or refracted wave arrivals. The field procedure optimizes shear wave generation as well as the polarity of the wave energy.

Applications.—For geotechnical investigations, shear wave velocities provide information on the low-strain dynamic properties of a given material. The relationships between compressional wave velocity, shear wave velocity, density, and in place dynamic properties of materials are shown in table 13-1. The compressional wave velocity can be determined from refraction surveys, the shear wave velocity from shear wave surveys, and the density from borehole geophysics or laboratory testing. The cross-hole seismic method is a common procedure used to determine

Table 13-1.—Determining moduli and ratios for typical velocities of earth materials from refraction surveys

V_p = Compressional wave velocity	(ft/s) (m/s)
V_s = Shear wave velocity	(ft/s) (m/s)
E = Young's Modulus	(lb/in^2) (MPa)
G = Shear Modulus	(lb/in$^{2)}$ (MPa)
K = Bulk Modulus	(lb/in$^{2)}$ (MPa)
μ = Poisson's Ratio	
ρ = Density (in situ)	(lb/ft$^{3)}$ (kg/m^3)

Shear Modulus: $G = \rho V_s^2$
Young's Modulus: $E = 2G(1+\sigma)$
Bulk Modulus: $K = \rho(V_p\text{-}4/3V_s^2)$
Velocity Ratio: V_p/V_s
Poisson's Ratio: $\mu = (.05)\ [V_p/V_s]^2\text{-}2/[V_p/V_s]^2\text{-}1\backslash$

material dynamic properties. Compressional waves and shear waves are generated in one drill hole, and the seismic wave arrivals are received in companion drill hole(s). The seismic source(s) and receiver(s) are located at equal depths (elevations) for each recording. Drill hole deviation surveys are performed in each drill hole to accurately determine the distances between each of the drill holes at all recording intervals. For typical seismic velocities of earth materials, see table 13-2.

Surface Wave Surveys

Purpose.—Surface wave surveys produce and record surface waves and their characteristics. Surface waves have lower frequencies and higher amplitudes than other seismic waves. Surface waves result from the constructive and destructive interference of refracted and reflected seismic waves. Surface waves that travel along the boundaries of a body are called Stanley waves. Surface waves are the slowest type of seismic

Table 13-2.—Typical velocities of earth materials

Material	Velocity (feet per second)	Velocity (meters per second)
Dry silt, sand, loose gravel, loam, loose rock, talus, and moist fine-grained topsoil	600-2,500	200-800
Compact till, indurated clays, gravel below water table*, compact clayey gravel, sand, and sand-clay	2,500-7,500	800-2,300
Weathered, fractured, or partly decomposed rock	2,000-10,000	600-3,000
Sound shale	2,500-11,000	800-3,400
Sound sandstone	5,000-14,000	1,500-4,000
Sound limestone, chalk	6,000-20,000	2,000-6,100
Sound igneous rock	12,000-20,000	3,700-6,100
Sound metamorphic rock	10,000-16,000	3,100-4,900
*Water (saturated materials should have velocities equal to or exceeding that of water)	4,700	1,400

wave, traveling along the boundaries between different materials. The characteristics must be determined in addition to recording the waves. Normally, surface waves are filtered out of seismic data or are ignored. The term. "ground roll," in the oil exploration industry denotes surface waves. Special care is taken in seismic reflection surveys to filter out surface waves because they can interfere with desired reflections. The different types of surface waves and their characteristic motions are shown in figure 13-2.

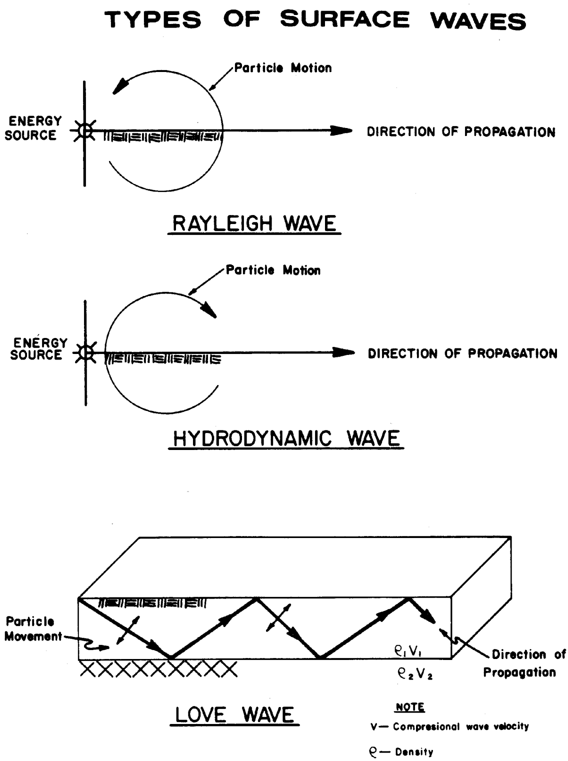


Figure 13-2.—Types of surface waves.

Applications.—The principal application of surface wave surveying for geotechnical investigations is to determine the type and characteristics of surface waves that can exist at a site. This information is used to determine preferred site frequencies and for earthquake design analysis.

Vibration Surveys

Purpose.—Vibration surveys measure the vibration levels produced by mechanical or explosive sources. Once these levels are determined, structures can be designed to reduce the possibility of vibration damage.

Applications.—Vibration surveys have been performed for quarrying and mining operations, excavations, measuring the effects of traffic on sensitive equipment, and measuring the effects of aircraft (sonic vibrations) on urban areas and historical buildings. Many manufacturing and research facilities contain extremely sensitive equipment with very small vibration tolerances. Vibration surveys can be very useful in determining the exact levels of allowable vibration and in designing procedures to reduce vibration levels produced by construction and blasting activities. The same type of vibration survey can be used in quarrying and/or mining operations to reduce vibration levels while maintaining rock breakage and fragmentation.

Electrical Resistivity Surveys

The electrical resistivity of any material depends largely on its porosity and the salinity of the water in the pore spaces. Although the electrical resistivity of a material may not be diagnostic, certain materials have specific ranges of electrical resistivity. In all electrical resistivity surveying techniques, a known electrical current is passed through the ground between two (or more) electrodes. The potential (voltage) of the electrical field resulting from the application of the current is measured between two (or more) additional electrodes at various locations. Since the current is known, and the potential can be measured, an apparent resistivity can be calculated. The separation between the current electrodes depends on the type of surveying being performed and the required investigation depth.

For representatives values of resistivity, see table 13-3.

Table 13-3.—Representative values of resistivity

Material Resistivity	(ohm-m)
Clay and saturated silt	1-100
Sandy clay and wet silty sand	100-250
Clayey sand and saturated sand	250-500
Sand	500-1,500
Gravel	1,500-5,000
Weathered rock	1,000-2,000
Sound rock	1,500-40,000

Electrical Resistivity Profiling Surveys

Electrical resistivity survey profiling is based on lateral changes in the electrical properties of subsurface materials.

Purpose.—Electrical resistivity profiling is used to detect lateral changes in the electrical properties of subsurface material, usually to a specified depth. Electrode spacing is held constant.

Applications.—Electrical resistivity has been used to map sand and gravel deposits, determine parameters for cathodic protection, map contamination plumes in hazardous waste studies, and used in fault studies.

Electrical Resistivity Sounding Surveys

Electrical resistivity sounding surveys measure vertical changes in the electrical properties of subsurface materials. The electrode spacing used for resistivity sounding is variable, with the center point of the electrode array remaining constant. The depth of investigation increases as the electrode spacing increases.

Purpose.—Resistivity soundings are used to investigate variations of resistivity with depth. Electrode spacing is varied.

Applications.—Electrical resistivity soundings are commonly used for aquifer and aquaclude delineation in groundwater investigations. The technique has been used for bedrock delineation studies where there is not a sufficient velocity contrast to permit seismic surveying. Vertical electrical soundings have been used for large scale mineral investigations, geothermal investigations, cathodic protection and toxic waste studies, and in conjunction with self-potential surveys for seepage investigations.

Electrical Resistivity Dipole-Dipole Surveys

Dipole-dipole surveying potential electrodes may have any position with respect to the pair of current electrodes. When the current and potential electrodes are positioned along the same line, the array is referred to as an axial dipole array (figure 13-3). The current electrodes are separated from the potential electrodes by an interval, n, which is some multiple of the current and potential electrodes separation, a. The separation of the current and potential electrodes is normally equal.

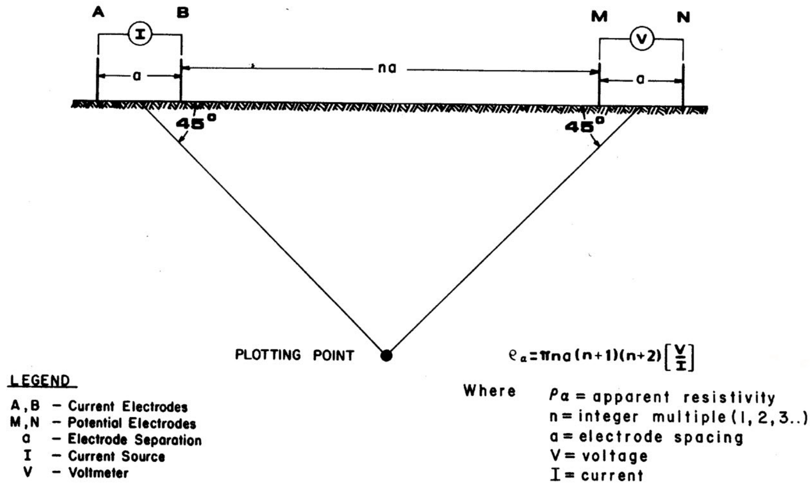


Figure 13-3.—Dipole resistivity array.

Purpose.—Dipole-dipole arrays are used to determine both the lateral and vertical changes in electrical properties of subsurface materials with one electrode array.

Applications.—The dipole-dipole array has limited applications in engineering and groundwater geophysics. This type of electrode array has been used primarily in mineral and geothermal exploration. The method has been used to delineate abandoned mines, mapping saltwater/fresh water interfaces, and mapping buried stream channels.

Electromagnetic Conductivity Surveys

Electromagnetic Conductivity Profiling Surveys

Electromagnetic (EM) surveying uses time-varying, low frequency, electromagnetic fields induced into the earth. A transmitter, receiver, and a buried conductor are coupled by electrical circuitry through electromagnetic induction. The characteristics of electromagnetic wave propagation and attenuation by a material can permit interpretation of the electrical conductivities of the subsurface materials.

Purpose.—Since electrical conductivity is the reciprocal of electrical resistivity, electromagnetic surveys are used to provide resistivity information on subsurface materials. Electromagnetic conductivity profiling surveys are specifically used to determine lateral changes in conductivity of the subsurface materials.

Applications.—Electromagnetic surveys have been used primarily for mineral exploration; and with the exception of magnetic surveys, EM surveys are the most commonly used geophysical surveys for minerals. EM surveys have been used in engineering and groundwater investigations. The method is found to be particularly useful in mapping contaminant plumes and buried metallic waste such as metal drums containing hazardous chemicals. EM surveys are suited to hazardous waste studies because the surveying procedure does not require equipment to touch potentially contaminated ground. The method has been used to locate buried pipes and cables and to locate landmines.

Electromagnetic Conductivity Sounding Surveys

Purpose.—Electromagnetic conductivity sounding surveys are used to determine vertical changes in conductivity of subsurface material.

Applications.—Electromagnetic sounding surveys can locate areas of permafrost, gravel deposits, map bedrock topography, and provide general geological information. EM sounding and profiling surveys have been applied to mapping areas of salt water intrusion, archaeological investigations, and fault studies.

Ground Penetrating Radar Surveys

Purpose

Ground penetrating radar (GPR) surveys have the same general characteristics as seismic surveys. The depth of investigation with GPR is extremely shallow when compared to seismic surveys. This disadvantage is partially offset by the much better resolution of GPR.

Applications

Ground penetrating radar surveys can be used for a variety of very shallow geotechnical investigations, including the locations of pipes or other buried manmade objects such as timbers, very high resolution mapping of near-surface geology, and detecting cavities, piping, and leakage in dams. The applications are limited by the very shallow penetration depth of the very high radar frequencies. Silts, clays, salts, saline water, the water table, and any other conductive materials in the subsurface severely restrict or stop penetration of radar.

Self-Potential Surveys

Purpose

Self-potential ([SP] spontaneous potential or natural potential) is the natural electrical potential existing within the earth due to many causes. These causes can be classified broadly into two groups (excluding manmade causes):

Mineralization Potential.—Mineralization potential is commonly the result of chemical concentration cells formed when conductive mineral deposits, such as graphite or sulfide, are intersected by the water table. Mineralization potentials are almost always negative and may have values up to several hundred millivolts. Background potentials can be either positive or negative and usually have values of only a few tens of millivolts.

Background Potential.—Background potential is commonly the result of (a) two electrolytes of different concentration being in contact with one another, (b) electrolytes flowing through a capillary system or porous media, (c) an electrolyte in contact with a solid, and (d) electromagnetically induced telluric (large scale flow in the earth's crust) currents.

The background potentials developed by electrolytes flowing through a capillary system or porous media (called electro-filtration or streaming potentials) are used to study seepage. Water flowing through a capillary system collects and transports positive ions from the surrounding materials. The positive ions accumulate at the exit point of the capillary, leaving a net positive charge. The untransported negative ions accumulate at the entry point of the capillary leaving a net negative charge. If the streaming potentials developed by this process are of sufficient magnitude to measure, the entry point and the exit point of zones of concentrated seepage may be determined from the negative and positive self potential anomalies.

Applications

Self-potential surveys have been used to map the lateral extent of mineral deposits and, in some instances, provide information of the configuration of the deposits. Another exploration application has been to map the depth to and configuration of certain geothermal areas. Normally, in geothermal applications, self-potential surveying is used in conjunction with other geophysical surveys (gravity, seismic, and electrical).

In geotechnical investigations, self-potential surveys have been used to map leakage paths from dams. Self-potential surveying has also been used to map leaks from canals and buried pipelines that transport liquids. Detachment walls and lateral limits of some landslide masses have been mapped with self-potential surveys.

Self-potential surveying may play an important role in hazardous waste investigations and in monitoring leakage from dams.

Magnetic Surveys

Purpose

Magnetic surveys measure anomalous conditions within the Earth's magnetic field. The Earth's magnetic field resembles the field of a bar magnet. The field is twice as strong in the polar regions than at the equator. The intensity of the field in the polar regions is approximately 60,000 gammas; while at the equator, the intensity is approximately 30,000 gammas. The Earth's magnetic field is not symmetrical but contains many large perturbations due to local variations in magnetic materials and larger magnetic features. Within the Earth's field, anomalies on the order of one gamma to several thousand gammas are detected by magnetic surveys. The smaller anomalies can be detected with complex instruments, and the larger anomalies can be detected with simpler instruments and field techniques.

Applications

Magnetic surveys have their widest applications in petroleum and mineral exploration programs. For applications in petroleum exploration, the application is somewhat simpler because the sources of most magnetic anomalies lie within the basement complex and the overlying sediments are often "transparent" to magnetic surveys.

In geotechnical investigations, magnetic surveys have been used to detect buried barrels of contaminated materials and to detect and map buried pipelines. Magnetic surveys have also been used in archaeological investigations.

Gravity Surveys

Purpose

Gravity anomalies are the result of contrasts in densities of materials in the Earth. If all the materials within the Earth were layered horizontally and were of uniform density, there would be no density contrasts. Density contrasts of different materials are controlled by a number of different factors; the most important are the grain density of the particles forming the material, the porosity of the material, and the interstitial fluids within the material. Generally, soil and shale specific gravities range from 1.7 to 2.2. Massive limestone specific gravities average 2.7. While this range of values may appear to be fairly large, local contrasts will be only a fraction of this range. A common order of magnitude for local density contrasts is 0.25. Density contrasts can be determined by calculating the gravity effect of a known model and comparing that effect with the observed gravity determined from a gravity survey.

Applications

Gravity surveys provide an inexpensive determination of regional structures that may be associated with groundwater aquifers or petroleum traps. Gravity surveys have been one of the principal exploration tools in regional petroleum exploration surveys. Gravity surveys have somewhat limited applications in geotechnical investigations. Gravity surveys have been used to obtain information on bedrock depths and the top of rock configuration in areas where it has not been possible or practical to use other geophysical techniques. Microgravity (high-precision) surveys have been used in a few instances to obtain information on the success of grouting programs. In these cases, microgravity surveys are performed before and after grouting operations and density contrasts of the two surveys are compared. Microgravity surveys have been used for archaeological investigations.

Glossary

A

Accelerometer – Transducer with output proportional to acceleration. A moving coil geophone (type of transducer) with a response proportional to frequency may operate as an accelerometer.

Acoustic Logging – borehole log of any of several aspects of acoustic-wave propagation (e.g., a sonic, amplitude, character, or three-dimensional log).

Air Wave – Energy from a shot which travels in the air at the velocity of sound.

Amplitude – The size of a signal, either in the ground or after amplification. Usually measured from the zero or rest position to a maximum excursion. The amplitude of a signal has units based on the measurement of the signal (e.g., acceleration (inch per square second [in/sec^2]), velocity (inches per second [in/sec]) or displacement (inches [in]).

Analog – (1) A continuous physical variable (such as voltage or rotation) that has a direct relationship to another variable (such as acceleration) with one proportional to the other. (2) Continuous as opposed to discrete or digital.

Anomaly – A deviation from normal or the expected. For example, a travel time anomaly, Bouger anomaly, free-air anomaly.

Apparent Velocity – (1) The velocity that a wave-front registers on a line of geophones. (2) The inverse slope of a time-distance curve.

Automatic Gain Control (AGC) – The output amplitude controls the gain of a seismic amplifier, usually individual for each channel; but sometimes, multi-channel devices are used.

B

Basement (Complex) – (1) generally of igneous and metamorphic rocks overlain unconformably by sedimentary strata. (2) Crustal layer beneath a sedimentary layer and above the Mohorovicic discontinuity.

Bedrock – Any solid rock exposed at the surface of the earth or overlain by unconsolidated material.

Bit – A binary digit; either a 1 (one) or 0 (zero).

Body Waves – The only waves that travel through the interior of a body consisting of P-waves and S-waves.

Byte – Word.

C

Cable – The assembly of electrical conductors used to connect geophone groups or other instruments.

Casing – Tubing or pipe used to line drill holes or shot-holes to keep them from caving.

Cathodic Protection – Electrical corrosion protection for pile foundations, electrical grounding mats, buried pipelines, or any metal subject to corrosion.

Channel – (1) A single series of interconnected devices through which geophysical data can flow from source to recorder. (2) An elongated depression or geological feature. (3) An allocated portion of the radio frequency spectrum.

Channel Wave – An elastic wave propagated in a layer of lower velocity than layers on either side. Energy is largely prevented from escaping from the channel because of repeated total reflection at the channel boundaries or because rays that tend to escape are refracted back toward the channel.

Character. – (1) The recognizable aspect of a seismic event, usually displayed in the waveform that distinguishes the event from other events. Usually, a frequency or phasing effect, often not defined precisely and dependent upon subjective judgment.

Common Depth Point (CDP). – The same portion of subsurface that produces reflections at different offset distances on several profiles.

Compressional Wave – An elastic body wave with particle motion in the direction of propagation. Same as P-waves, longitudinal wave, dilatation wave, irrotational.

Converted Wave – A wave that is converted from longitudinal to transverse, or vice versa, upon reflection or refraction at oblique incidence from an interface.

Critical Angle – Angle of incidence, θ_c, for which the refracted ray grazes the contact between two media (of velocities V_1 and V_2):

$$\sin \theta_c = V_1/V_2$$

Critical Distance – (1) The offset at which a refracted event becomes the first break; cross-over distance. (2) The offset at which the reflection time equals the refraction time (i.e., the offset at which the reflection occurs at the critical angle).

Crossfeed (Crosstalk) – Interference resulting from the unintentional pickup of information or noise from another channel.

D

Datum – (1) The arbitrary reference level to which measurements are corrected. (2) The surface from which seismic reflection times or depths are measured, corrections having been made for local topographic and/or weathering variations. (3) The reference level for elevation measurements, often sea level.

Delay Time – (l) In refraction work, the additional time taken for a wave to follow a trajectory to and along a buried marker over that which would have been taken to follow the same marker hypothetically at the ground surface or at a reference level. Normally, delay time exists separately under a source and under a detector and depends on the depth of the marker at wave incidence and emergence points. Shot delay time plus geophone delay time equals intercept time. (2) Delay produced by a filter. (3) Time lag introduced by a delay cap.

Diffraction – (1) Scattered energy which emanates from an abrupt irregularity, particularly common where faults cut reflecting interfaces. The diffracted energy shows greater curvature than a reflection (except in certain cases where there are buried foci), although not necessarily as much as the curve of maximum convexity. Diffraction frequently blends with a reflection and obscures the fault location or becomes confused with dip. (2) Interference produced by scattering at edges. (3) Causes energy to be transmitted laterally along a wave crest. When a portion of a wave train is interrupted by a barrier, diffraction allows waves to propagate into the region of the barrier's geometric shadow.

Digital – Representation of quantities in discrete units. An analog system represents data as a continuous signal.

Dipole – A pair of closely spaced current electrodes that approximates a dipole field from a distant pair of voltage detecting electrodes.

E

Elastic Constants –

(1) Bulk modulus, $\boldsymbol{k}$. – The stress-strain ratio under hydrostatic pressure.

$$\boldsymbol{k} = \frac{\Delta \boldsymbol{P}}{\Delta \boldsymbol{V} / \boldsymbol{V}}$$

where ΔP = pressure change, V = volume, and ΔV = volume change.

(2) Shear modulus, μ. – rigidity modulus, Lame's constant. The stress-strain ratio for simple shear.

$$\mu = \frac{\boldsymbol{F_t} / \boldsymbol{A}}{\Delta\boldsymbol{L} / \boldsymbol{L}}$$

where F_t = tangential force, A = cross-sectional area, L = distance between shear planes, and ΔL = shear displacement. Shear modulus can also be expressed in terms of other moduli as:

$$\mu = \frac{\boldsymbol{E}}{(1+\sigma)}$$

where E = Young's modulus, and σ = Poisson's ratio.

(3) Young's modulus, E. – The stress-strain ratio when a rod is pulled or compressed.

$$\boldsymbol{E} = \frac{\Delta\boldsymbol{F} / \boldsymbol{A}}{\Delta\boldsymbol{L} / \boldsymbol{L}}$$

where $\Delta F/A$ = stress (force per unit area), L = original length, and ΔL = change in length.

(4) Lame' constant, λ. – a tube is stretched in the up-direction by a tensile stress, S, giving an upward strain, s, and S' is the lateral tensile stress needed to prevent lateral contraction, then:

$$\lambda = \frac{\boldsymbol{S}'}{\boldsymbol{s}}$$

This constant can also be expressed in terms of Young's modulus, E, and Poisson's ratio, σ:

$$\lambda = \frac{\sigma E}{(1+\sigma)(1-2\sigma)}$$

Electromagnetic – Periodically varying field, such as light, radio waves, radar.

End Line – Shotpoints that are shot near the end of the spread.

F

Fan Shooting – An early use of the refraction seismograph to find salt domes within a thick, low-velocity section; detectors located in widely spaced fan-like arrays radiating from the shot locations.

Filter – (1) A device that discriminates against some of the input information. The discrimination is usually on the basis of frequency, although other bases such as wavelength or moveout (see velocity filter) may be used. The act of filtering is called convolution. (2) Filters may be characterized by their impulse response or, more usually, by their amplitude and phase response as a function of frequency. (3) Band-pass filters are often specified by successively listing their low-cutoff and high-cutoff points. (4) Notch filters reject sharply at a particular frequency. (5) Digital filters filter data numerically in the time domain. Digital filtering can be the exact equivalent of electrical filtering. Digital filtering is very versatile and permits easy filtering according to arbitrarily chosen characteristics.

First Break or First Arrival – The first recorded signal attributable to seismic wave travel from a source. First breaks on reflection records provide information about weathering. Refraction work is based principally on the first breaks, although secondary (later) refraction arrivals are also used.

Frequency – The repeat rate of a periodic wave, measured in hertz. Angular frequency, measured in radians per second.

G

Galvanometer – A coil suspended in a constant magnetic field. The coil rotates through an angle proportional to the electrical current flowing through the coil. A small mirror on the coil reflects a light beam proportional to the galvanometer rotation.

Geophone (Seismometer, Jug) – Instrument used to convert seismic energy (vibration) into electrical energy.

Geophone Station – Location of a geophone on a spread.

Gravimeter. – An instrument for measuring variations in gravitational attraction.

Gravity Survey – A survey performed to measure the gravitational field strength, or derivatives, that are related to the density of different rock types.

Group Velocity – The velocity at which most of the energy in a wave train travels. In dispersive media where velocity varies with frequency, the wave train changes shape as it progresses so that individual wave crests appear to travel at velocities (the phase velocity) different from the overall energy as approximately enclosed by the envelope of the wave train. The velocity of the envelope is the group velocity. Same as dispersion.

H

Head Wave – See refraction wave.

Hidden Layer – A layer that cannot be detected by refraction methods; typically, a low velocity layer beneath a high velocity layer.

Hydrodynamic Wave – A seismic surface wave propagated along a free surface. The particle motion is elliptical and prograde (M2 type Rayleigh or Sezawa) (e.g., Rayleigh-type wave dependent upon layering).

Hydrophone (Pressure Detector) – A detector sensitive to variations in pressure, as compared to a geophone, which is sensitive to motion. Used when the detector can be placed below a few feet of water such as in marine or marsh applications or as a well seismometer. The frequency response of the hydrophone depends on the depth beneath the surface.

I

In-line Offset – Shot points that are in line, but offset to a spread.

L

Lead – An electrical conductor (wire cable) used to connect electrical devices. Geophones are connected to cables at the takeouts via leads on the geophones.

Love Wave – A surface seismic wave associated with layering. This wave is characterized by horizontal motion perpendicular to the direction of propagation, with no vertical motion.

Low-velocity Layer (LVL) – The surface layer that has low seismic velocity.

M

Magnetic Survey – A survey to measure the magnetic field or its components as a means of locating concentrations of magnetic materials or determining depth to basement.

Mis-tie – (1) The time difference obtained on carrying a reflection event or some other measured quantity around a loop; or the difference of the values at identical points on intersecting lines or loops. (2) In refraction shooting, the time difference from reversed profiles that gives erroneous depth and dip calculations.

Multiple – Seismic energy that has been reflected more than once (e.g., long-path multiple, short-path multiple, peg-leg multiple, and ghosts).

N

Noise – (1) Any undesired signal or disturbance that does not represent any part of a message from a specified source. (2) Sometimes restricted to random energy. (3) Seismic energy that is not resolvable as reflections. Noise can include microseisms, shot-generated noise, tape-modulation noise, and harmonic distortions. Sometimes divided into coherent noise (including non-reflection coherent events) and random noise (including wind noise, instrument noise, and all other energy which is non-coherent). (3) Random noise can be attenuated by compositing signals from independent measurements. (4) Sometimes restricted to seismic energy not derived from the shot explosion. (5) Disturbances in observed data due to more or less random inhomogeneities in surface and near-surface material.

Noise Analysis – A profile or set of profiles used to gather data for an analysis of coherent noise. Used to design geophone arrays in reflection surveys.

Noise Survey (Ground Noise Survey) – A survey of ambient, continuous seismic noise levels within a given frequency band. This technique is a useful tool for detecting some geothermal reservoirs because they are a source of short-period seismic energy.

O

On-line – Shot points that are not at any point on a spread other than at the ends.

Original Data – (1) Any element of data generated directly in the field in the investigation of a site. (2) A new element of data resulting from a direct manipulation or compilation of field data.

Oscillograph – A device that records oscillations as a continuous graph of corresponding variation in an electric current or voltage.

Oscilloscope – A device that visually displays an electrical wave on a screen (e.g., on a cathode ray tube).

P

Phase Velocity – The velocity of any given phase (such as a trough or a wave of single frequency); it may differ from group velocity because of dispersion.

Plant – The manner in which a geophone is placed or the coupling to the ground.

Primacord – An explosive rope that can be used to either connect explosive charges or to detonate separately as a primary energy source.

Profile – The series of measurements made from several shotpoints to a recording spread from which a seismic data cross section or profile can be constructed.

R

Radar – An exploration method where microwaves are transmitted into a medium and are reflected back by objects or layers. The reflected microwaves are received and processed to provide an image of the subsurface. May be used for shallow penetration surveys.

Rayleigh Wave – A seismic wave propagated along a free surface. The particle motion is elliptical and retrograde.

Ray Path – The path of a seismic wave. A line everywhere perpendicular to wave-fronts (in isotropic media).

Reconnaissance – (1) A general examination of a region to determine its main features, usually preliminary to a more detailed survey. (2) A survey to determine regional geological structures or to determine whether economically prospective features exist, rather than to map an individual structure.

Reflection Survey – A survey of geologic structure using measurements made of arrival times of seismic waves that have been reflected from acoustic impedance change interfaces.

Refraction Survey – A survey of geologic structure using compressional or head waves. Head waves involve energy that enters a high-velocity medium (refractor).

Refraction Wave (Headwave, Mintrop Wave, Conical Wave) – Wave travel from a point source obliquely downward to and along a relatively high velocity formation or marker and then obliquely upward. Snell's law is obeyed throughout the trajectory. Angles of incidence and of emergence at the marker are critical angles. Refracted waves typically following successively deeper markers appear as first arrivals with increasing range (shot to detector distance). Refracted waves following different markers may have different arrival times for any given range. Refracted waves cannot arise for angles of incidence less than the critical angle for any given marker. At the critical angle, the refracted wave path (and travel time) coincides with that of a wide angle reflection.

Refractor (Refraction Marker) – An extensive, relatively high-velocity layer underlying lower velocity layers that transmits a refraction wave nearly horizontally.

Resistivity – An electrical property of rock reflecting the conductivity of an electrical current. Resistivity is the ratio of electric field intensity to current density.

Resistivity Meter – A general term for an instrument used to measure the in place resistivity of soil and rock materials.

Resistivity Survey – A survey that measures electric fields and earth resistivity of a current induced into the ground.

Reverse Profile – A refraction seismic profile generated by shooting both ends of a spread.

Roll-along – A mechanical or electrical switch that connects different geophones (or geophone groups) to the recording instruments. The use of a roll-along switch permits common depth point reflection data to be easily acquired in the field.

S

Seismic Amplifier – An electronic device used to increase the electrical amplitude of a seismic signal.

Seismic Camera – A recording oscillograph used to make a seismic record.

Seismic Cap – A small explosive designed to be detonated by an electric current that detonates another explosive. These caps are designed to provide very accurate time control on the shot.

Seismic Velocity – The rate of propagation of seismic waves through a medium.

Seismogram – A seismic record.

Self-potential (Spontaneous Potential, Natural Potential, SP) – The direct current or slowly varying natural ground voltage between nearby non-polarizing electrodes.

Shear Wave – A body wave with the particle motion perpendicular to the direction of propagation. (Same as S-wave, equivoluminal, transverse wave.)

Shooter – The qualified, licensed individual (powder-man) in charge of all shot point operations and explosives on a seismic crew.

Shot Depth – The distance from the surface down to the explosive charge. The shot depth is measured to the center or bottom of the charge with small charges. The distances to both the top and bottom of the column of explosives are measured with large charges.

Shot Instant (Time Break, TB, Zero Time) – The instant at which a shot is detonated.

Shot Point – Location of the energy source used in generating a particular seismogram.

Signal Enhancement – A process used in seismographs and resistivity systems to improve the signal-to-noise ratio by real-time adding (stacking) of successive waveforms from the same source point. Random noise tends to cancel out, and the coherent signal tends to add or stack.

Snell's Law – When a wave crosses a boundary between two isotropic mediums, the wave changes direction such that the sine of the angle of incidence of the wave divided by the velocity of the wave in the first medium equals the sine of the angle of refraction of the wave in the second medium divided by the velocity of the wave in the second medium (see critical angle).

Spread – The layout of geophone groups from which data from a single shot are simultaneously recorded.

Stanley Wave – A type of seismic surface wave propagated along an interface.

Surface Wave – Energy that travels along or near the surface (ground roll).

T

Takeout – The connections on a multiconductor cable for connecting geophones. May refer to the short cable leads from the geophones (pigtails).

Time Break – The mark on a seismic record that indicates the shot instant or the time the seismic wave was generated.

Trace – A record of one seismic channel. This channel may contain one or more geophones.

V

Vibration Monitor – A sensitive, calibrated recorder of ground and structural acceleration and velocity. Measurements are made to determine vibration amplitudes and modal frequencies of buildings, towers, etc., under ambient conditions, as well as to detect potentially damaging vibrations caused by blasting, pile driving, etc.

Vibroseis – A seismic energy source consisting of controlled frequency input into the earth by way of large truck mounted vibrators. Trademark of Continental Oil Company (Conoco).

W

Wave Length – The distance between successive similar points on two adjacent cycles of a wave measured perpendicular to the wavefront.

Wave Train – (1) The sum of a series of propagating wave fronts emanating from a single source. (2) The complex wave form observed in a seismogram obtained from an explosive source.

Weathering Spread – A short-spaced geophone interval refraction spread used to provide corrections to refraction data caused by delay times in near-surface, low-velocity materials.

WWV(B) – The National Institute of Standards and Technology (NIST) radio stations that broadcast time and frequency standards.

For the definition of other geophysical terms used in this manual, refer to: *Glossary of Terms Used in Geophysical Exploration,* Society of Exploration Geophysicists, Tulsa, Oklahoma, 1984.

Bibliography

Mooney, H.M., *Handbook of Engineering Geophysics, Volume 1: Seismic*, Bison Instruments, Inc., Minneapolis, Minnesota, 1984.

Telford, W.M., et al., *Applied Geophysics*, Cambridge University Press, New York, New York, 1976.

U.S. Corps of Engineers, *Geophysical Exploration for Engineering and Environmental Investigations*, Engineering Manual EM 111-1-1802, 1995.

U.S. Geological Survey, *Application of Surface Geophysics to Ground-Water Investigations*, Chapter D1, Book 2, Techniques of Water-Resources Investigations of the United States Geological Survey, 1974.

Van Blaricom, R., compiler, *Practical Geophysics II for the Exploration Geologist*, Second Edition, Northwest Mining Association, Spokane, Washington, 1992.

Ward, S.H., editor, *Geotechnical and Environmental Geophysics*, Society of Exploration Geophysics, Tulsa, Oklahoma, 1990.

Chapter 14

BOREHOLE GEOPHYSICAL AND WIRELINE SURVEYS

Introduction

Wireline surveys determine physical properties in and beyond the wall of a borehole by devices attached to a cable, or wireline. Subsurface geologic conditions and engineering characteristics can be derived directly or indirectly from the wide variety of measurable properties available by wireline surveying. Wireline logging techniques commonly are classified by the kind of energy that is input (active systems) or received (passive systems), including electric, seismic or acoustic, nuclear, magnetic, gravity, or optical. Logging tools are also classified according to whether they are for use in open holes or cased holes. Data from several methods are often combined to evaluate a single geologic or engineering characteristic. This chapter describes the methods and discusses the application of borehole wireline surveying to geotechnical exploration and investigation.

Electric Logging Techniques

An electric log is a continuous record of the electrical properties of the fluids and geologic materials in and around a borehole. Electric logging is performed in the uncased portion of a borehole by passing electric current through electrodes in the logging device, or sonde, and out into the borehole and the geologic medium. Other electrodes located on the surface or in the borehole complete the circuit to the source and recording device. Electric logging surveys are efficient and cost effective because the process is automated and several electrical properties are measured simultaneously by combining several electrode configurations in the same tool.

Electric logging techniques can be used in geotechnical investigations to assess the variation with depth of geologic materials and associated fluids. Electric logs from two or more boreholes are used to correlate and determine the continuity of geologic strata or zones that have similar electrical properties. Since the electrical properties depend on physical characteristics, the porosity, mineral composition, water content (saturation), water salinity, lithology, and continuity of the bedding can often be deduced. Borehole electric logs are also the best source of control for surface electrical surveys by providing subsurface layer resistivities. Electric log correlation of continuous layers from borehole to borehole is relatively simple. The interpretation of physical properties from electric logs can be ambiguous and complicated. Different combinations of water content, salinity, mineralogy, porosity, and borehole peculiarities can produce similar electric logs.

Spontaneous Potential (SP) Log

The spontaneous potential or SP logging device records the difference in potential in millivolts between a fixed electrode at the surface and an electrode in a borehole (figure 14-1). The measured potential difference changes as the electrical potential between the borehole fluid and the fluids in the various strata opposite the sonde changes. The spontaneous potential device commonly is incorporated into the multiple-electrode resistivity sonde so that resistivity and SP logs are acquired simultaneously. Figure 14-2 illustrates a typical SP resistivity log. The recording is a relative measurement of the voltage in the borehole. Readings opposite shales or clays are relatively constant and form the shale baseline, or "shale line." The SP curve typically deflects to the left or right opposite permeable formations depending on the salinity of the drilling mud. Sandstone and other porous

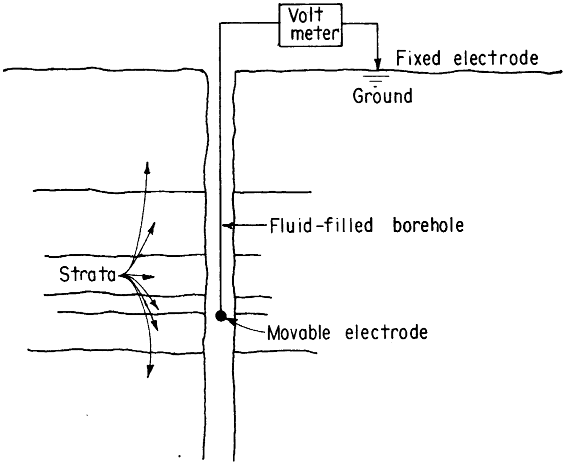


Figure 14-1.—Spontaneous potential survey elements.

and permeable strata allow the different fluids to mix and produce salinity of the pore fluid with respect to the borehole fluid and the clay content of the strata. The SP peaks are commonly displayed to the left of the shale line because negative potentials are more commonly encountered. In fresh water, SP anomalies are typically inverted (i.e., they appear to the right of the shale baseline).

SP logs are used in combination with resistivity logs to define strata boundaries for correlation and to infer the lithology of the strata as a function of permeability and resistivity. The shape of the SP curve depends on the drilling mud used and the geologic strata encountered. For example, the SP curve for a shale bed would indicate zero deflection (the shale line in figure 14-2), and the

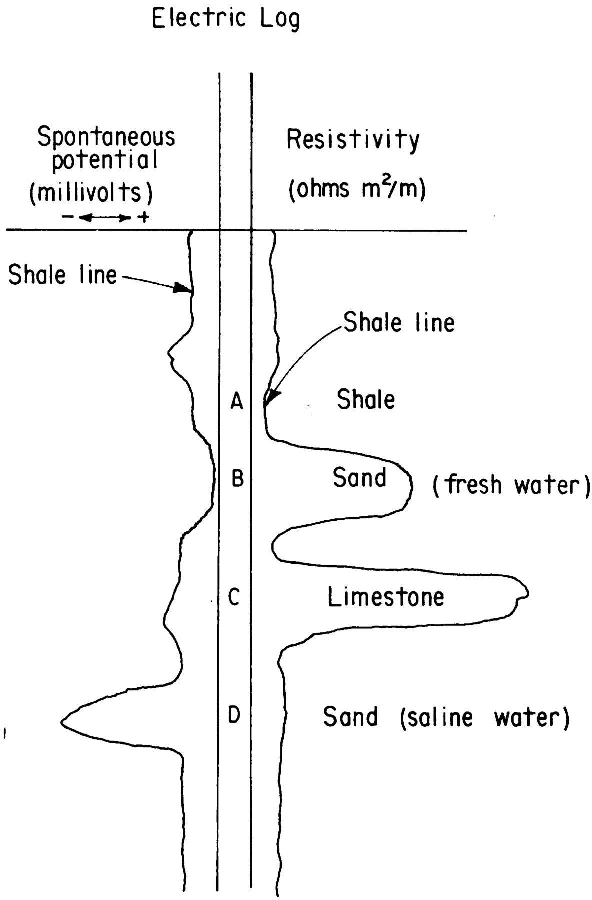


Figure 14-2.—Electric log showing SP and resistivity in different beds.

resistivity curve would indicate low resistivity because of bound water in the clay and trapped water in the shale. Generally, the SP and resistivity curves converge opposite shale strata (A in figure 14-2) and diverge opposite permeable fresh water sands (B in figure 14-2). A limestone bed, which usually is highly resistive but develops little spontaneous potential, would produce a large resistivity deflection but a subdued SP peak (C in figure 14-2). A permeable sand bed with pore water of high salinity typically would produce little resistivity deflection but a large negative SP peak (D in figure 14-2). The determination of lithologies from SP logs should be done cautiously because many factors contribute to the magnitudes and directions of the curve deflections. The SP log is best used with resistivity and other logs to detect strata boundaries and to correlate strata between boreholes. The SP log is also used to determine formation water resistivities.

Single-Point Resistance Log

Single-point resistance loggers are the simplest electrical measuring devices. A current can flow between a single electrode placed in a borehole and another electrode at the ground surface (figure 14-3). The earth between the electrodes completes the circuit. Single-point resistance logs are not commonly used in modern logging operations but illustrate the principal of down-hole electric logging. The resistance, R, of the circuit (electrodes plus earth) can be calculated from Ohm's Law, $R = V/I$, where V is the measured voltage drop and I is the current through the circuit. The resistance of a circuit is determined by the conductor's size, shape, and resistivity, p, an intrinsic property of a material. Resistance of the conductor (the earth) is a convenient and useful property determined by pore fluid content, pore fluid composition, lithology, and continuity of strata. The resistivity of the measured

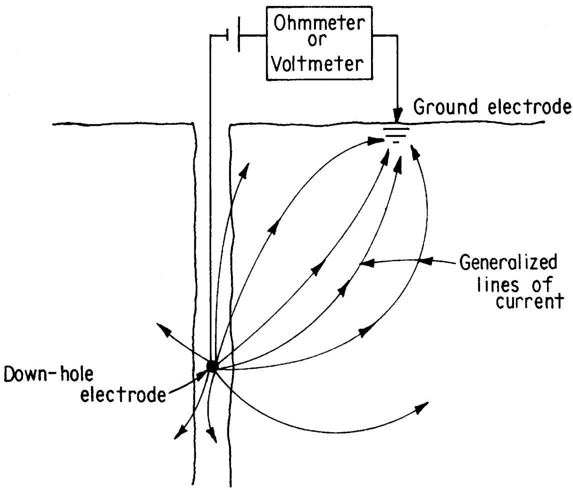


Figure 14-3.—Single-point resistivity array.

section of earth is determined from $p = KR$, where K is a geometric factor which differs for different electrode configurations and R is the measured resistance V/I. A single point resistance log measures an apparent resistance of a section made up of borehole fluid and the various individual strata between the borehole and the ground surface electrode. The multiple electrode arrays discussed below are designed to narrow and better define the measured sections so that individual strata are represented more accurately by the logs.

Multiple Electrode Array Log

Wireline electrical systems using multiple electrode arrays in the borehole provide better resolution of resistivity and associated properties of individual strata within the subsurface than can be achieved with the single-point array. Multiple electrode arrays include the short- and long-normal array, lateral array, and focused-current or guard logging systems. Current electrodes usually are designated A and B, and potential electrodes are designated M and N. "Normal" arrays place the in-hole current electrode far away, at effective infinity (figure 14-4A). "Lateral" devices place the two potential electrodes close together with respect to the in-hole current electrode (figure 14-4B). Conventional modern logging systems use a sonde made up of two normal devices and one lateral device to produce three resistivity logs and the SP log simultaneously.

The normal resistivity arrays (figure 14-4A) are called short normal or long normal depending on the spacing of the in-hole current (A and B) and potential (M and N) electrodes. The industry standard AM electrode spacing is 16 inches (in) for the short normal and 64 in for the long normal. The normal arrays measure the electrical potential at the in-hole electrode M. The greater the spacing between the current and measuring electrodes, the greater the effective penetration (effective penetration distance = ½ AM) of the device into the surrounding geologic medium.

The lateral resistivity array is shown in figure 14-4B. The actual positions of the electrodes may vary from the circuit shown, but the resistivity measurements are the same. The lateral array spacing is measured from the A current electrode to the center (0) of the potential electrode configuration and is called the AO spacing. The lateral array measures the potential difference, or gradient, between the two in-hole potential electrodes and is also called the gradient array. The influence of the

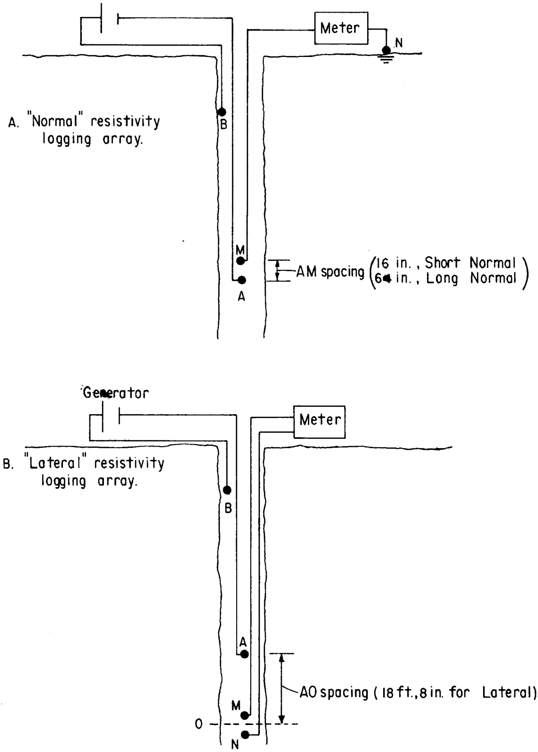


Figure 14-4.—Multiple-electrode resistivity arrays. (A and B are current electrodes. M and N are potential electrodes.)

geologic medium away from the borehole (the effective penetration of the system) is greatest for the lateral array with an 18-foot 8-in spacing and least for a 16-in short normal array.

The focused-current, or guard, resistivity device is a modified single-electrode array in which "guard" current electrodes are placed above and below a central current electrode and two pairs of potential electrodes (figure 14-5). Current in the central and guard electrodes is adjusted so that zero potential exists between the potential electrodes and the current is thus forced or focused to flow out into the geologic medium in a narrow band. The device then measures the potential between the sonde electrodes and a reference electrode at the ground surface. Focusing the current into a band of predetermined thickness gives the focused array much greater thin bed and stratum boundary resolution than the other arrays.

Geotechnical applications for multiple-electrode resistivity arrays include apparent resistivity and an approximation of true resistivities of individual strata or zones. Resistivity data aid in the interpretation of surface electrical surveys, which generally are less expensive to conduct than down-hole surveys and can be applied over large areas. Since the depth of penetration of the measuring circuit into the geologic medium varies with the electrode spacing, logs of the various arrays (short- and long-normal and lateral) can be analyzed collectively to evaluate the effects of bed thickness, borehole fluids, drilling mudcake, variations in fluids, and permeabilities of strata at various distances from the borehole. The curve shapes of the logs for several boreholes permit correlation of strata and zones from borehole to borehole. Normal devices produce better boundary definition of thick strata than do lateral devices, but lateral logs are more effective in delineating thin strata and thin, highly resistive zones. The focused logs discriminate more sharply between different strata, define strata boundaries better, and give a better approximation of true stratum resistivity than do normal or lateral logs. Porosity and

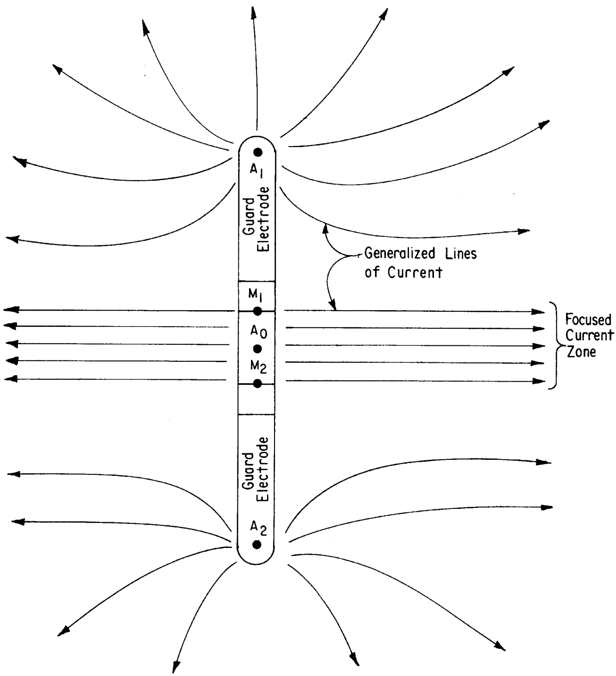


Figure 14-5.— Focused current, or guard, resistivity array.

water content of strata and salinity of pore fluids can be calculated from the logs if sufficient information is obtained.

Microlog

Special purpose electric logging devices are available to supplement or, in some cases, replace standard resistivity logs. The microlog (also called contact log, microsurvey, and minilog) records variations in resistivity of a narrow, shallow zone near the borehole wall. The microlog sonde is equipped with three closely spaced electrodes placed on a rubber pad which is pressed against the borehole wall (figure 14-6). Microlog electrode spacings are about 1 to 2 in (2.5 to 5 centimeters [cm]). Depth of investigation for the microlog is about 3 in (7.5 cm) from the borehole wall. Normal, lateral, and guard arrays can be configured in the microlog survey. Porous permeable zones allow the mud cake to penetrate the strata, and the resulting microlog will indicate resistivities close to that of the mud.

Induction Log

Induction logging is performed in dry boreholes or boreholes containing nonconducting fluids. Induction logging can be done in holes cased with polyvinyl chloride (PVC) casing. The induction logger is housed in a sonde that uses an induction coil to induce a current in the geologic medium around a borehole. The induction logger is a focused device and produces good thin-bed resolution at greater distances from the borehole than can be achieved with normal spacing devices. A primary advantage over other devices is that the induction logger does not require a conductive fluid in the borehole.

Nuclear Radiation Logging Techniques

A nuclear radiation log is a continuous record of the natural or induced radiation emitted by geologic materials near the borehole. Nuclear radiation logging is performed by raising a sonde containing a radiation detector or a detector and a radioactive source up a borehole and recording electrical impulses produced by a radiation detector.

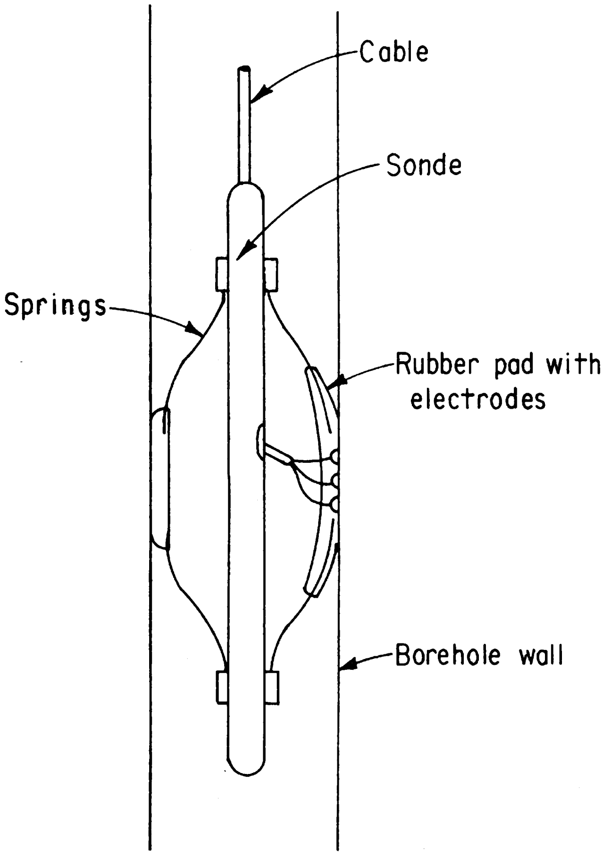


Figure 14-6.—Microlog resistivity logging device.

Nuclear radiation logging can be used in geotechnical investigations to correlate strata between boreholes, aid in determining lithology, and derive or measure directly many physical parameters of the subsurface materials, including bulk density, porosity, water content, and relative clay content. Most nuclear radiation logging systems can be operated in cased boreholes, giving them an advantage over electrical wireline logging systems. Nuclear logs can also be run in old boreholes, which are more likely to be cased than recent boreholes. The logs are among the simplest to obtain and interpret, but the calibrations required for meaningful quantitative interpretations must be meticulously performed. Nuclear radiation logging tools should be calibrated in controlled calibration pits. Under favorable conditions, nuclear borehole measurements approach the precision of direct density tests of rock cores. The gamma-gamma density log and the neutron water content log require the use of isotopic sources of nuclear radiation. Potential radiation hazards necessitate thorough training of personnel working around the logging sources.

Three common wireline nuclear radiation systems are the gamma ray or natural gamma logger, the gamma-gamma or density logger, and the neutron logger. The gamma-gamma and neutron devices require the use of a radioactive source material. The gamma ray device is a passive system and does not use a radioactive source. All the systems employ an in-hole sonde containing the source and detector. Most devices require a borehole diameter of at least 2 in (5 cm) and investigate a zone extending 6 to 12 in (15 to 30 cm) from the borehole. Applications differ for the three systems and depend on the properties measured.

Gamma Ray (Natural Gamma) Log

The gamma ray or natural gamma radiation device provides a continuous record of the amount of natural gamma radiation emitted by geologic materials near the sonde. The gamma-ray detector converts incoming gamma radiation to electrical impulses proportional to the number of rays detected and sends the amplified electrical impulses to the surface. Gamma ray logs generally reveal the presence of shale or clay beds because clay minerals commonly contain the potassium isotope K^{40},which is the primary source of natural gamma radiation in most sediments. Shales and clays usually produce a high gamma ray count, or a peak on the log.

The relative density of a stratum is also indicated by natural gamma ray logs because gamma radiation is absorbed more by dense materials than by less dense materials. Gamma ray logging can be conducted in cased boreholes and is sometimes used in place of SP logging to define shale (clay) and non-shale (non-clay) strata. The natural gamma ray and neutron logs are usually run simultaneously from the same sonde and recorded side-by-side. The gamma ray log also infers the effective porosity in porous strata with the assumption that high gamma counts are caused by clay filling the pores of the strata. Gamma ray logs are used to correct gamma-gamma density logs. Gamma ray logs are standard in radiation logging systems.

Natural Gamma Spectral Log

Spectral logging permits the identification of the naturally occurring radioactive isotopes of potassium, uranium, and thorium which make up the radiation detected by the natural gamma ray detector. Spectral logging technology allows the detection and identification of radioactive isotopes that contaminate water resources, or are introduced as tracer elements in hydrologic studies.

Density or Gamma-Gamma Log

The gamma-gamma or density logging device measures the response of the geologic medium to bombardment by gamma radiation from a source in the sonde. Electrons in the atoms of the geologic medium scatter and slow down the source gamma rays, impeding their paths to the detector. Figure 14-7 illustrates the operation of the gamma-gamma device. Since electron density is proportional to bulk density for most earth materials, strata with high bulk densities impede the source gamma rays more than low density strata and produce correspondingly lower counts at the detector. The primary use of the gamma-gamma log is determining bulk density. If bulk density and grain density are known from samples and if fluid density can be determined, the porosity can be calculated. Gamma-gamma logs can also be used for correlation of strata between boreholes.

Gamma-gamma logging is best conducted in uncased boreholes using decentralizing devices to keep the sonde

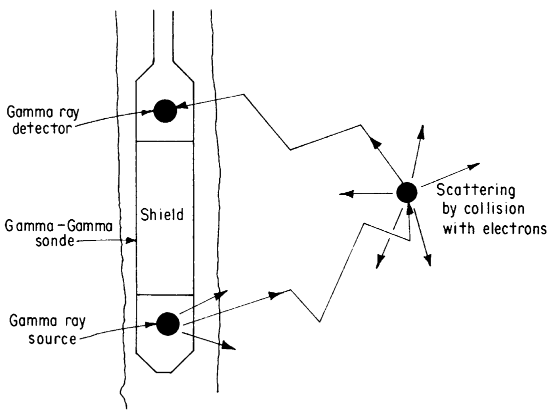


Figure 14-7.—Gamma-gamma logging sonde.

against the borehole wall. The system is affected by borehole diameter and wall irregularities and must be carefully calibrated for the effects of hole diameter, decay of the strength of the radioactive source, natural gamma radiation in the geologic medium, and borehole fluid density. A caliper log is commonly run to permit calibration for hole diameter. Some logging systems simultaneously display the gamma ray count on one side of the log and bulk density on the other side, with the bulk density log automatically corrected for the effects of mud cake and wall irregularity.

Neutron Log

Neutron logging is an active system that measures the response of the geologic medium to emission of neutron radiation from a source located in the sonde. Neutrons emitted by the source are captured by certain atoms, especially hydrogen nuclei, in the strata near the sonde. Figure 14-8 illustrates the operation of the neutron logging device. Collision of neutrons with atoms of the geologic medium causes secondary emission of neutrons and gamma rays, a portion of which are picked up by the detector. A high concentration of hydrogen atoms near the source captures a greater number of neutrons and produces a smaller counting rate at the detector.

Neutron detectors may detect gamma (neutron-gamma) or neutron (neutron-neutron) radiation. Water and hydrocarbons contain high concentrations of hydrogen atoms. If the fluid in the strata is assumed to be water, the neutron log is an indicator of the amount of water present. Determination of volumetric water content (weight of water per unit volume measured) is a principal use of neutron logging. In saturated materials, the neutron log can be an indicator of porosity, the ratio of the volume of void space to the total volume. Bulk densities from gamma-gamma logs, grain densities from samples, caliper

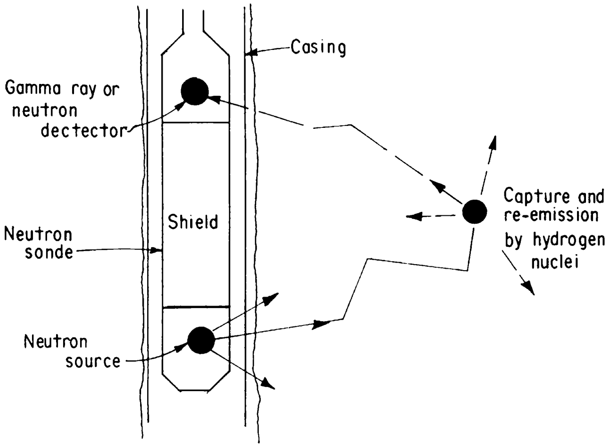


Figure 14-8.—Neutron logging sonde.

log data, and natural gamma log information can be combined with the volumetric water contents from neutron logs to calculate the more commonly used geotechnical parameters of water content by weight, wet and dry density, void ratio, porosity, saturation, and lithology.

Neutron logs and natural gamma radioactive logs can be obtained through thick borehole casing and are usually run together. Figure 14-9 illustrates the format of the tandem logs and typical responses for several lithologies. The response of different lithologies to the natural gamma and neutron devices is controlled directly by the amount of radioactive material present (especially the clay content) and the fluid content and indirectly by the porosity and bulk density of the material. For example, the saturated sand of figure 14-9 absorbs neutron radiation, produces a correspondingly low neutron log count, and produces a low natural gamma count because of the lack of clay in its pores.

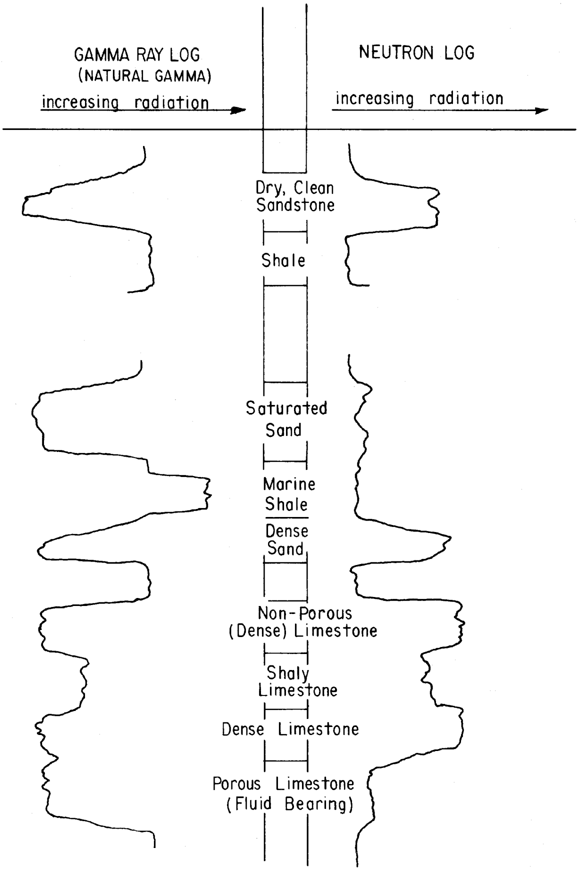


Figure 14-9.—Typical curve responses for nuclear radiation logs.

Neutron activation or spectral logs provide spectral analyses of the geologic medium near the borehole to detect the presence of certain elements. The spectral logging sonde activates a neutron source in bursts of short duration and activates the gamma ray detector only during the source bursts. As a result, only certain radiation ("prompt" gamma rays) is detected. The system produces a plot of counts versus gamma ray energy. Peaks on the energy/count plot identify specific elements, for example carbon, silicon, magnesium, or chlorine. Spectral logs may have application to groundwater quality investigations.

Neutrino Log

Neutrino logs are not necessarily a borehole log, but the method can and often is performed in drill holes. Neutrino sources are located throughout the universe with the closest and most useful source being the sun. Using the sun as a neutrino source eliminates the need for a special source, and there are no handling and licensing problems that are often associated with other nuclear logs. Neutrinos penetrate the earth with little to no impediment, so neutrino logs can be run at any time of day or night. Logging tools generally consist of strings of widely spaced photomultiplier tubes (PMT) placed into the borehole. High-energy neutrinos passing through the rock will occasionally interact with the rock and create a muon. These muons emit Cherenkov light when passing through the array and are tracked by measuring the arrival times of these Cherenkov photons at the PMTs. Resolution is high because the source (Sun) only subtends half a degree at this distance (93 million miles from Earth). Neutrino logging is particularly useful in cased holes and when fluid in the hole is not a factor. New or old holes can be logged no matter what the hole contains. These logs are often used for tomography. Tomography is particularly useful and convenient because, as the Earth rotates, the source penetrates a full 360 degrees, similar to CAT scan or Magnetic Resonance Imaging. A drawback is that 24 hours is required to run a complete neutrino tomography log.

Acoustic/Seismic Logging Techniques

Wireline acoustic/seismic logging systems use medium- to high-frequency acoustic (sonic) energy emitted from a sonde to image the borehole wall or to obtain seismic velocities of the geologic material in the wall or in the formation away from the borehole. Acoustic systems use sonic energy generated and propagated in a fluid such as water. Seismic systems use sonic energy propagated through the ground in geologic materials. The three systems discussed in this section are the acoustic velocity logger, an acoustic and seismic energy system; a borehole imaging device, an acoustic system; and the cross-hole seismic technique, a seismic technique.

The borehole imaging and acoustic velocity systems are used in geotechnical investigations to evaluate geologic conditions including attitude and occurrence of discontinuities and solution cavities and in determining the effective porosity and elastic properties of rock. The down-hole systems also supply subsurface information for interpreting surface-applied geophysical surveys. The frequency of the energy pulse produced by the sonde's transducer determines whether the energy penetrates the borehole wall for acoustic/seismic velocity surveys. Medium frequencies are used in acoustic/seismic velocity surveys. The energy is reflected from the wall for imaging surveys. High frequencies are used in acoustic/seismic velocity surveys. Borehole imaging velocity devices generate compressional (P-wave) energy. The acoustic velocity device generates primarily compressional energy but also generates shear (S-wave) energy if the shear wave velocity exceeds the fluid velocity. The cross-hole seismic technique generates P- and S-wave energies and determines their velocities.

Acoustic Velocity Log

The continuous acoustic velocity (sonic) logger measures and records the velocity of acoustic energy (seismic waves) in the material adjacent to the borehole. A transmitter located at one end of the sonde generates an electromechanical pulse that is transmitted through the borehole fluid into the borehole wall and by refraction to a receiver located at the other end of the sonde (figure 14-10). The propagation velocities of the seismic waves can be calculated by travel times and distance traveled from transmitter to receiver. The devices must be operated in a fluid-filled borehole. Compressional wave oscillations set up by the sonde's transmitter in the borehole fluid set up, in turn, oscillations in the borehole wall. Compressional, shear, and surface waves propagate in and along the wall and then, as compressional waves, back through the borehole fluid to the receiver. Compressional waves have the highest velocity and arrive first, followed by the shear waves and the surface waves. Devices with two receivers cancel the borehole fluid travel times so that only the refracted wave paths through the borehole wall are measured. Single-receiver devices require the borehole fluid acoustic velocity and borehole diameter for calculation of seismic velocities. The acoustic pulses generated at the transmitter are in the lower ultrasonic range around 20 kilohertz (kHz).

The simplest acoustic logging devices display a single graphical trace or waveform of the arrival of each pulse

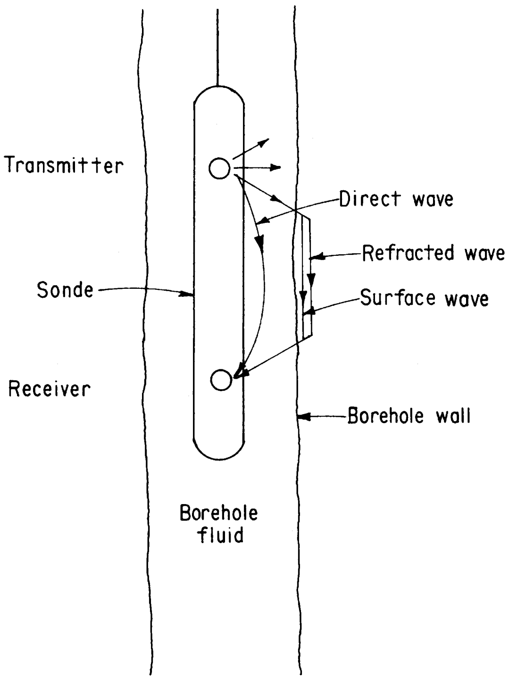


Figure 14-10.—Elements of a simple wireline acoustic velocity device.

and are called amplitude-modulated devices. Intensity-modulated acoustic logging devices record the entire continuous acoustic wave (see figure 14-11). Intensity-modulated, or three-dimensional (3-D), devices express wave frequency by the width of light and dark bands and

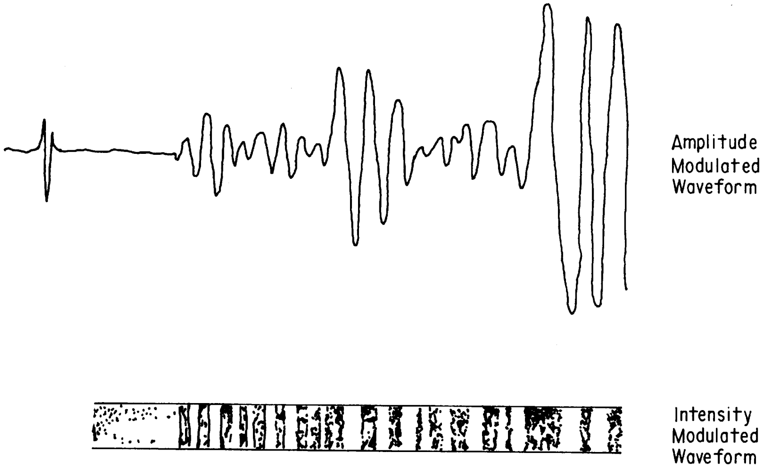


Figure 14-11.—Acoustic log presentations. The intensity modulated waveform wave frequency by the width of the band, amplitude by the shading intensity of the band.

amplitude by the degree of light or dark shading (figures 14-11 and 14-12). The P-wave, S-wave, and surface wave arrivals can be discerned on the 3-D records. Strata or zones of different seismic velocity produce contrasting signatures on the 3-D log. The phyllite zones in figure 14-12 represent softer materials of lower seismic velocity than the metabasalts and produce correspondingly later arrival times. Fractures or other discontinuities can often be seen as disruptions in the bands.

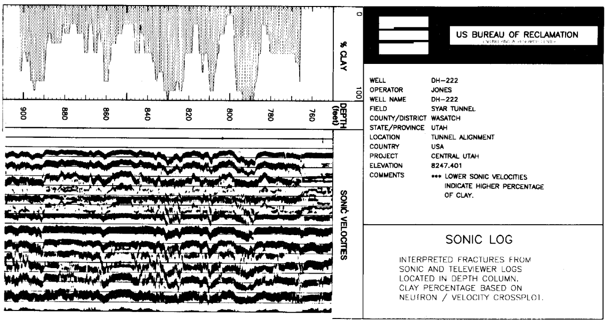


Figure 14-12.—Sample of intensity modulated acoustic log. Lithologies from core samples.

In addition to determining seismic velocities to aid interpretation of seismic surveys, 3-D logs show lithologic contacts, geologic structure, and solution features. The P- and S-wave velocities can be used directly to calculate the dynamic elastic properties of rock, including Poisson's ratio, Young's modulus, and bulk and shear moduli. For example, Poisson's ratio is calculated using:

$$\mu = \frac{1/2\,\boldsymbol{V_p}^2 - \boldsymbol{V_s}^2}{\boldsymbol{V_p}^2 - \boldsymbol{V_s}^2}$$

where V_p and V_s are the compressional and shear wave velocities, respectively. The effective porosity of the strata can also be evaluated.

Acoustic velocity devices require a borehole diameter of 3 in or greater. The in-place geologic materials must have compressional wave velocities higher than the velocity of the borehole fluid (approximately 4,800 feet per second [ft/sec] [1,450 meters per second (m/sec)] for water) for the waves to be refracted and detected. Soils generally do not have sufficiently high seismic velocities for acoustic velocity logging. The wave train (trace) type velocity loggers often are equipped with a radiation logging device and caliper on the same sonde for simultaneous acoustic and natural gamma ray logging and borehole diameter measurement.

Acoustic Borehole Imaging Log

The acoustic borehole imaging device uses high-frequency acoustic waves to produce a continuous 360-degree image of the borehole wall. Physical changes in the borehole wall are visible as changes in image intensity or contrast. Proprietary trade names for the acoustic borehole imaging device include "Televiewer" and "Seisviewer." Figure 14-13 illustrates the operation of an acoustic borehole imaging device. A piezoelectric transducer and direction indicating magnetometer are rotated by a motor inside the tool housing at approximately three revolutions per second (rps). The transducer emits pulses of acoustic energy toward the borehole wall at a rate of about 2,000 pulses per second at a frequency of about 2 megahertz (MHz). The high-frequency acoustic energy does not penetrate the borehole wall, and the acoustic beams are reflected from the borehole wall back to a transducer. The intensity of the reflected energy is a function of the physical condition of the borehole wall,

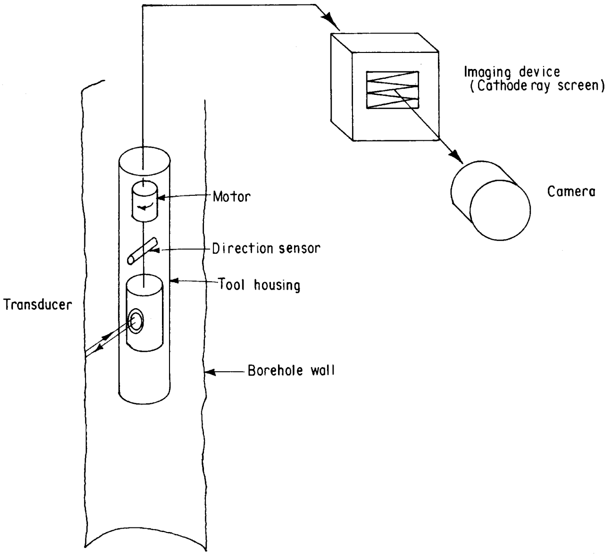


Figure 14-13.—Acoustic borehole imaging system.

including texture (rough or smooth) and hardness (a function of the elasticity and density of the material). The image is oriented to magnetic north with every revolution of the transducer. Fractures, cavities, and other open discontinuities produce low-amplitude acoustic reflections and are readily discerned in their true orientation, width, and vertical extent on the image.

The acoustic borehole imaging device can detect discontinuities as small as 1/8 in (3 millimeters [mm]) wide and

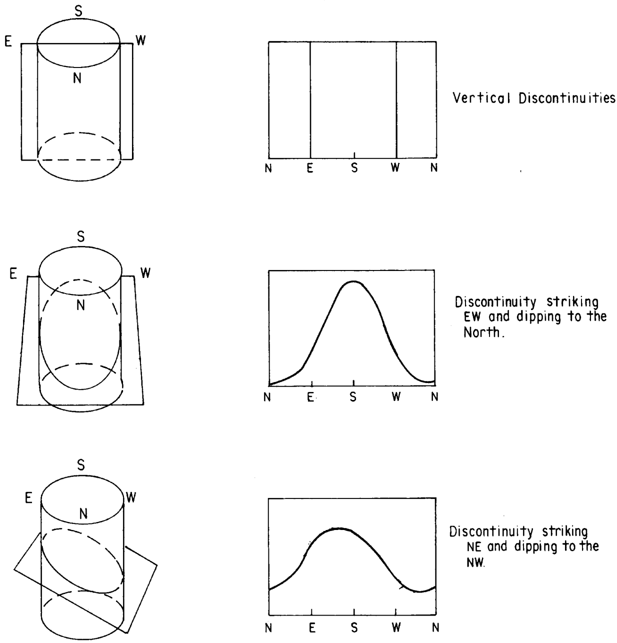


Figure 14-14.—Traces of planar discontinuities intercepting the borehole (left) as they appear on the acoustic borehole imaging record (right).

can sometimes distinguish contrasting lithologies. Discontinuities, contacts, and other linear features intercepting the borehole at an angle other than 90 degrees produce a characteristic sinusoidal trace that can be used to determine the strikes and dips of the features. Figure 14-14 shows how planar discontinuities of different orientations appear on the acoustic imaging log. Vugs and solution cavities are displayed as black areas, and their traces can be analyzed to determine their size (in two dimensions) and percentage of the borehole. The device can also be used to inspect casing and well screens for defects. The acoustic imaging device must be operated in a fluid-filled borehole.

Cross-Hole Seismic Test

Cross-hole tests are conducted by generating a seismic wave in a borehole and recording the arrival of the seismic pulse with geophones placed at the same depth in another (receiver) borehole. Source and geophones are placed at several regular depth intervals in the boreholes to determine seismic wave velocities of each material. Compressional and shear wave velocities can be determined with the cross-hole test. Figure 14-15 illustrates the essential features of a cross-hole seismic test. The seismic source may be either an explosive or a mechanical device. A vertical hammer device, clamped to the borehole wall, is the typical source arrangement. Cross-hole geophones configured in triaxial arrays are used because directional detectors are necessary to identify the S-wave arrival. Two or more receiver holes are sometimes used.

The raw data of cross-hole tests are the times required for P- and S-waves to travel from the seismic source in one borehole to the detectors in the receiver hole or holes. The corresponding P- and S-wave arrival times can be used to calculate seismic velocities as the ratios of distance to travel time, assuming the arrivals are direct (non-refracted) arrivals. If refraction through a faster zone occurs, true velocities must be calculated, similar to surface refraction seismic calculations.

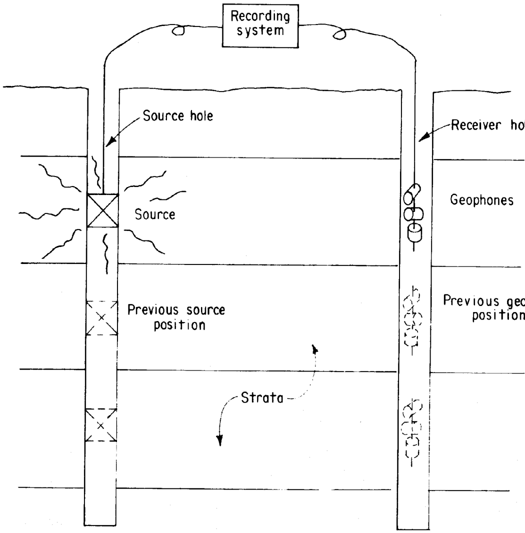


Figure 14-15.—Cross-hole seismic test.

Borehole spacing (distance) is critical in cross-hole tests. Generally, spacing should be no greater than 50 feet (ft) (15 meters [m]) or less than 10 ft (3 m). Borehole deviation surveys should be conducted prior to testing to determine precise spacing between holes at each shotpoint depth.

Cross-hole tests have several applications in engineering geology. S-wave cross-hole tests provide data on material properties for static and dynamic stress analysis. In addition to providing true P- and S-wave velocities at different depths, cross-hole surveys can detect seismic anomalies such as zones of low velocity underlying a zone of higher velocity or a layer with insufficient thickness or velocity contrast to be detected by surface refraction tests.

Seismic Tomography

Tomography is a method of constructing an image of some physical property inside an object from energy sent through the object in many directions. Much like medical tomography (such as a CAT scan) is used to create images of features inside the human body; geophysical tomography is used to create images of features beneath the ground or within engineered structures. There are several types of geophysical tomography. Cross-hole seismic tomography is one of the most common types of geophysical tomography. Cross-hole seismic tomography involves sending seismic energy from one borehole to another. A transmitter is lowered into one borehole, and the transmitted seismic energy is recorded by a string of receivers located in the second borehole. The positions of the transmitter and receivers are varied so that the seismic energy is transmitted between the two boreholes over a large depth range and at many different angles. The arrival times of the transmitted seismic energy are used to construct an image of seismic P-wave velocity of the geologic materials between the two boreholes. In addition, the amplitudes of the transmitted signals may sometimes be used to construct an image of the apparent attenuation of the geologic materials. Attenuation is a measure of the amount of energy loss of the seismic signal and is related to such factors as material type, degree of compaction or cementation, porosity, saturation, and fracturing. Cross-hole seismic tomography may be used to image geologic features such as solution cavities, fracture zones, and lithologic contacts. Tomography may also be used to evaluate engineered structures such as concrete cut-off walls, grout curtains, and concrete dams.

Borehole Optical Systems

Borehole optical logging systems include borehole film-recording cameras and borehole television cameras. Optical logging systems permit viewing and recording visible features in the walls of dry or water-filled boreholes, wells, casing, pipelines, and small tunnels.

Borehole cameras are an important supplement to geotechnical drilling and sampling programs because the cameras show geologic and construction features in place and in relatively undisturbed condition. The aperture (opening), true orientation (strike and dip), and frequency (number per foot of borehole) of discontinuities in the rock mass can be determined with optical logging systems. The effective or fracture porosity associated with open discontinuities can be derived also. The occurrence and size of solution cavities, the rock type from visual examination of rock and mineral textures and color of the borehole wall, and the location of stratigraphic contacts can be determined. Other geotechnical applications include inspecting contacts of rock or soil against concrete and of the interior of rock or concrete structures. Optical logging systems permit viewing soft or open zones in rock that may be overlooked in normal drilling and sampling operations. Zones of poor sample recovery or of drilling fluid loss can be investigated.

Borehole Television Camera.—Borehole television systems provide real-time viewing of the interior of a dry or water-filled borehole. The typical system (figure 14-16) consists of a down-hole probe and surface-mounted cable winch, control unit, power supply, television monitor, and video tape unit. The down-hole probe contains a television camera facing vertically downward toward a partially reflecting mirror inclined 45 degrees to the axis

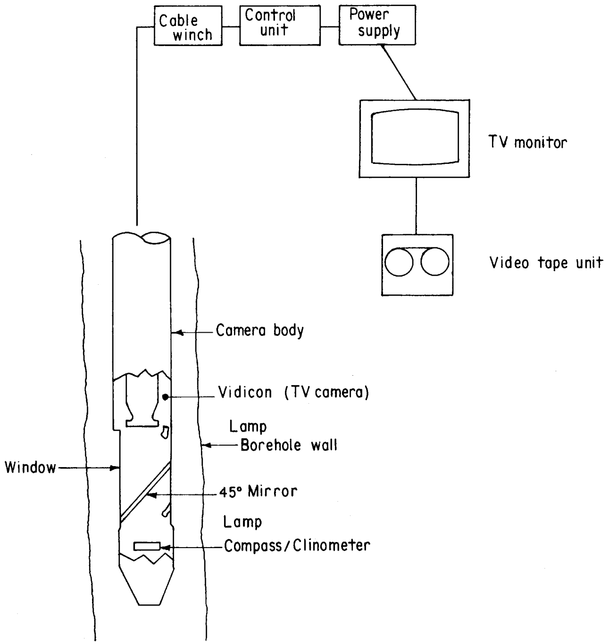


Figure 14-16.—Borehole television logging system.

of the probe. A compass/clinometer used to determine the orientation of features in the borehole wall and the inclination of the borehole can be viewed selectively through the mirror by varying the relative intensities of lamps located above and below the plane of the mirror. A motor within the probe rotates the camera and mirror assembly (or the mirror only) 380 degrees in a reciprocating fashion (clockwise, then counterclockwise) as the probe is lowered or raised within the hole. The operator controls the motion and logging speed of the probe. A full image of the borehole wall is obtained with each rotation.

A viewing head can be attached to the probe to allow axial viewing of the hole directly ahead. The probe ascent or descent and camera rotation can be slowed or stopped for examination of borehole features. The entire viewing sequence is usually videotaped.

Borehole television systems are commonly used to inspect large concrete structures and rock for defects, determine the effectiveness of grouting operations, check the condition of concrete joints, estimate the volume of cavities within a rock or soil mass, and inspect the screens of water wells. Television images can be used to determine attitudes of discontinuities but are less effective for complete borehole mapping than the borehole film camera systems that portray the borehole walls as discrete, full-color, separate images more suitable for detailed study and measurement.

Borehole Film Camera.—The borehole film camera produces consecutive, over-lapping, 16-mm color still photographs of 360 degree, 1-in (25-mm) sections of the walls of NX or larger boreholes. The system consists of a down-hole probe and surface- mounted lowering device, metering units, and power supply. Figure 14-17 illustrates the down-hole probe. The probe consists of a 16-mm camera mounted in line with the probe axis and facing a truncated conical mirror, which is surrounded by a cylindrical quartz window. A film magazine and film drive mechanism are mounted above the camera in the probe housing. A ring-shaped strobe illuminates the borehole wall in synchronization with the camera's film advance. Film advance, probe movement, and strobe lighting are synchronized to expose a frame every 3/4 in

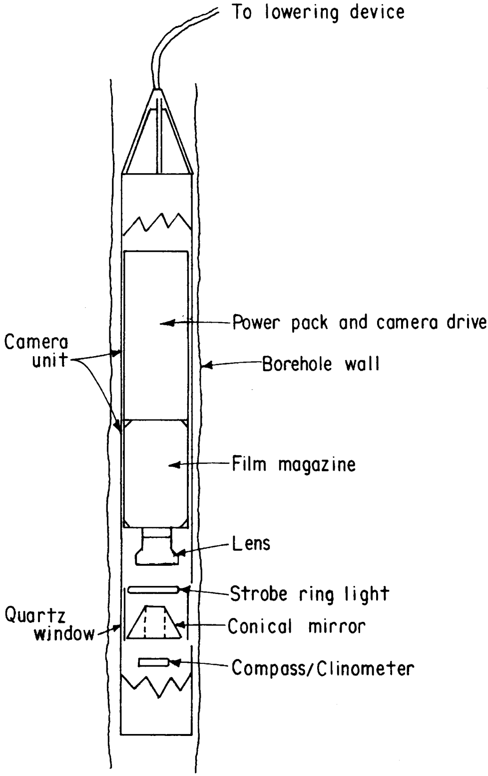


Figure 14-17.—Borehole film camera.

(20 mm) along the hole, so that a photograph with 1/4-in (6-mm) overlap of every 360-degree, 1-in (25-mm) section is taken. The truncation of the conical mirror allows the camera simultaneously to photograph a compass/clinometer located directly beneath the mirror. Photographs produced by the camera are viewed individually (still frame) using a special projector. The image of the mirror face is projected onto a flat film plane. The outer ring of the image "doughnut" represents the base, and the inner ring is the top of the 1-in section of the borehole wall. Planar discontinuities intersecting the borehole wall produce a trace on the image like the shaded curve of figure 14-18 (top). In a vertical borehole, the outer and inner rings of the image "doughnut" are horizontal traces. Points of intersection of planar discontinuities with a ring thus define the strike of the feature in vertical boreholes. The compass area in the center of the image of figure 14-18 shows the bearing of intersecting planes. The dip of the feature can be determined graphically by counting the number of successive frames in which the feature appears. A steeply dipping plane intersects several frames, a gently dipping plane only a few frames, and a horizontal plane intersects one frame. The thickness or width of the discontinuity can also be measured directly from the image. Data from inclined boreholes must be corrected to true strike, dip, and width. The NX borehole film camera is a proven tool for supplying subsurface data in difficult drilling and sampling situations and for providing complete quantitative information on discontinuities and borehole anomalies. Strike can be measured accurately to about 1 degree, dip (or inclination) to about 5 degrees, and thickness or width of discontinuities to a hundredth of an inch (0.1 mm) or better.

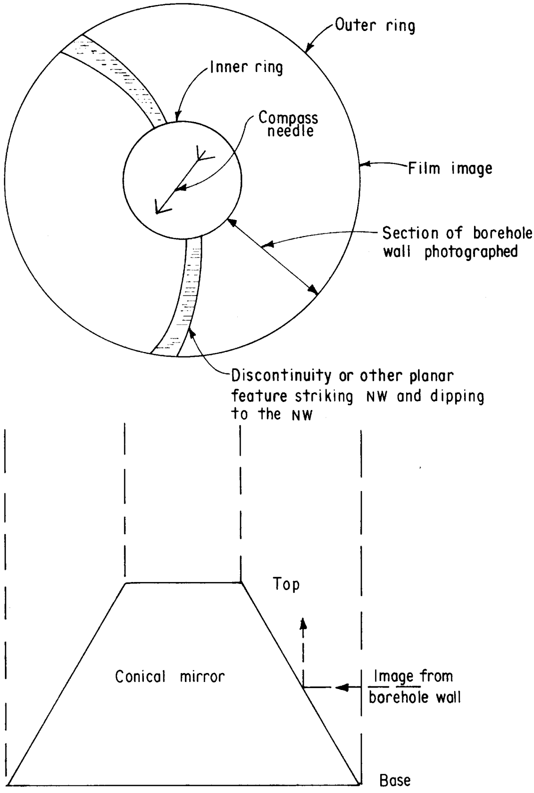


Figure 14-18.—Projection of borehole wall image into the film plane from the conical mirror of the borehole film camera.

In operation, the borehole film camera probe is lowered to the bottom of the hole and raised by a hand-cranked winch at a logging speed of between 5 and 10 feet per minute (ft/min). A single load of film can photograph 75 to 90 ft of borehole. Since the camera has no shutter or light metering capability, the aperture must be preset to satisfy lighting requirements of hole diameter and reflectivity of the borehole wall. The lens aperture ranges from f 2.8 for dark rock to f 22 for light colored rock. The maximum range (depth of field) is about 12 in. Boreholes larger than NX require centering of the probe in the hole.

Borehole Image Processing System (BIPS)

The Borehole Image Processing System (BIPS) uses direct optical observation of the borehole wall in both air and clear-fluid filled holes. The BIPS tool has a small fluorescent light ring that illuminates the borehole wall. A conical mirror in a clear cylindrical window projects a 360-degree optical slice of borehole wall into the camera lens. The tool also contains a digital azimuth sensor that determines the orientation of the image. The optical images are digitized and stored.

The BIPS analysis provides color images of the borehole wall. In a two-dimensional (2-D) projection, planar features that intersect the borehole wall appear as sinusoids. Processing allows the borehole wall to be viewed like a core, regardless of whether core was actually recovered. Strike and dip of planar features intersecting the borehole wall are determined during processing.

Other Wireline Systems

This section discusses wireline devices that are available for supplementary or special purpose applications. These systems include the borehole caliper log, directional surveys, borehole temperature log, borehole gravity log, magnetic log, and flowmeter log.

Borehole Caliper Log

The borehole caliper log provides a continuous record of changes in borehole diameter determined by a probe equipped with tensioned mechanical arms or an acoustic transducer. Caliper or borehole diameter logs are one of the most useful and simple of all logs obtained in borehole geophysics. Caliper logs provide the physical size of a drill hole and should be run in all borings in which other geophysical logging is anticipated. Caliper logs provide indirect information on subsurface lithology and rock quality. Borehole diameter varies with the hardness, fracture frequency, and cementation of the various materials penetrated. Borehole caliper surveys can be used to accurately identify washouts or swelling or to help determine the accurate location of fractures or solution openings, particularly in borings with core loss. Caliper logs can also identify more porous zones in a boring by locating the intervals in which excessive mud filter cake has built up on the walls of the borehole. One of the major uses of borehole caliper logs is to correct for borehole diameter effects. Caliper logs also can be used to place water well screens, position packers for pressure testing in foundation investigations for dams or other large engineering structures, and help estimate grout volumes in solution or washout zones.

Mechanical calipers are standard logging service equipment available in one-, two-, three-, four-, or six-arm probe designs. Multiple-arm calipers convert the position of feelers or bow springs to electrical signals in the probe. The electrical signals are transmitted to the surface through an armored cable. Some caliper systems average the movement of all the arms and record only the change in average diameter with depth, and others provide the movements of the individual arms as well as an average diameter. The shape or geometry of the borehole cross section can be determined with the individual caliper arm readings. A six-arm caliper capable of detecting diameter changes as small as 1/4 inch in 6-in to 30-in diameter boreholes produces a record like that shown on figure 14-19. The six arms are read as three pairs so that the diameter in three directions is recorded in addition to the average diameter.

Mechanical calipers are lowered to the bottom of the borehole in closed position, the arms are released, and the tool is raised. The calipers must be calibrated against a known minimum and maximum diameter before logging.

The acoustic caliper measures the distance from an acoustic transducer to the borehole wall. Individual diameter readings for each of four transducers mounted 90 degrees apart are obtained and also the average of the four readings. A special purpose acoustic caliper designed for large or cavernous holes (6 ft to 100 ft in diameter) uses a single rotating transducer to produce a continuous record of the hole diameter.

Directional Surveys

In some situations, borehole deviation must be accurately determined. Several methods of accomplishing this have been devised. Borehole survey instruments initially consisted of single or multiple picture (multishot) cameras that photographed a compass and plumb bob at selected locations in the borehole. Depths were based on the length of cable in the borehole. The camera was retrieved from the hole, and the film developed and interpreted.

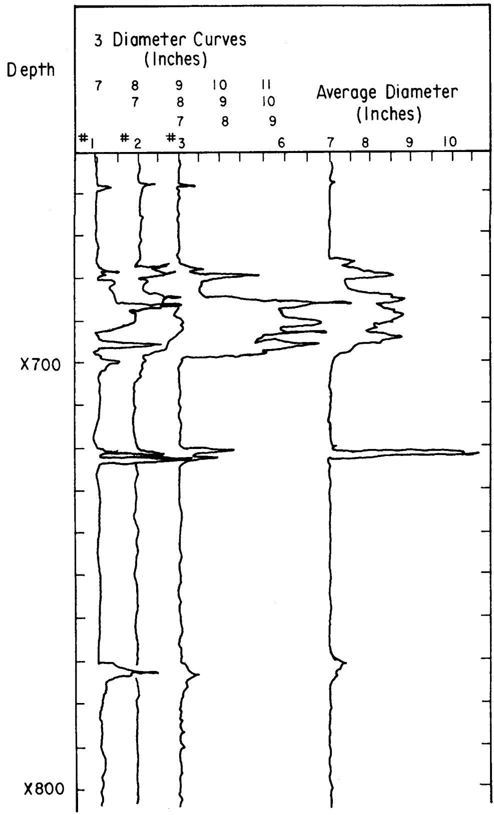


Figure 14-19.—Log of six-arm mechanical caliper.

A fluxgate compass or gyroscope is commonly used today to measure azimuth and an inclinometer to measure deviation from vertical. The information is then electronically transmitted to the surface via a cable. This method provides a continuous survey of the hole and not just checks at intervals. Systems are also available that use accelerometers and gyroscopic sensors to survey boreholes and provide real time data via a cable or ultrasonically via the mud.

Borehole Fluid Temperature Log

The standard borehole fluid temperature logging device continuously records the temperature of the water or drilling fluid in an open or cased borehole as a sonde is raised or lowered in the borehole. The standard, or gradient device uses a single thermistor (thermal resistor) that responds to temperature variations in the fluid. A small change in temperature produces a large change in resistance, which is converted to a change in electrical current. A differential device is sometimes used to record differences in temperature between two positions in the borehole by the use of two thermistors or one thermistor and a memory unit within the control module. The log produced by the temperature device shows temperature as a function of depth.

Temperature logging is usually performed before standard geophysical logging operations to permit correction of other logs (e.g., the resistivity logs) for the effects of borehole fluid temperature. Temperature logs can also be used to locate the source and movement of fluids into or out of the borehole and identify zones of waste discharge or thermal pollution in groundwater. Grouted zones behind casing can be located by the cement heat of hydration.

The temperature probe must be calibrated against a test fluid of known temperature immediately before temperature logging. Borehole fluids must be allowed to reach a stable temperature after drilling operations before logging can be conducted. Open boreholes take longer to stabilize because of the differences in thermal conductivity of the various materials encountered.

Borehole Gravity Log

A borehole gravity meter (or gravimeter) detects and measures variations in the force of gravity. The device determines the bulk density of a large volume of soil or rock between a pair of measuring stations in the borehole (figure 14-20). The bulk density (ρ) is proportional to the measured difference in gravity (G) between the two stations and inversely proportional to the vertical distance (Z) between the stations:

$$\Delta \boldsymbol{G} = \boldsymbol{G}_1 - \boldsymbol{G}_2 \quad \rho = \frac{\Delta \boldsymbol{G}}{\Delta \boldsymbol{Z}}$$

The borehole gravity meter has been used primarily by the oil industry to determine the porosity (especially porosity because of fractures and vugs). Porosity is calculated using standard equations relating bulk density, fluid density, and specific gravity of solids (matrix density). Although density and porosity are also provided by the gamma-gamma density logger (see "Nuclear Radiation Logging Techniques"), because of the large volume and radius of investigation, gravity meter data are less affected by borehole effects such as mudcake and irregular hole diameter. Gravity meter density determinations are more representative of the geologic medium away from the borehole than are gamma-gamma logs. The radius of investigation of the gravity meter is an estimated five times the difference in elevation between the measuring stations (see figure 14-20).

The borehole gravity meter is not a continuous recording instrument. The tool must be positioned at a preselected measurement station, allowed to stabilize, and then a

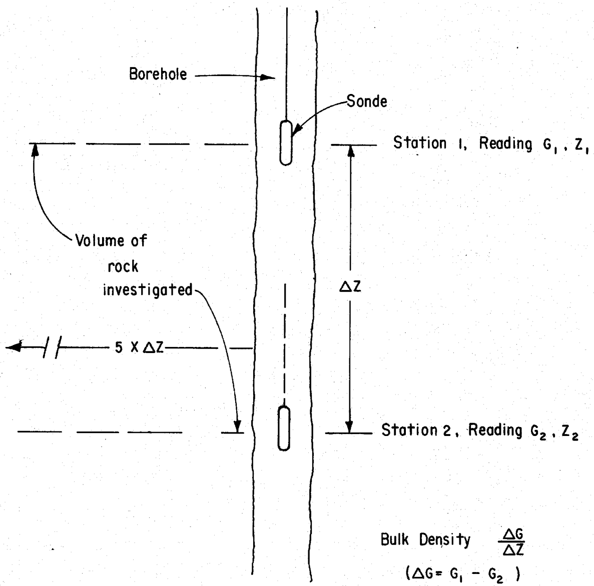


Figure 14-20.—Elements of borehole gravity logging.

measurement taken. Instrument readings are converted to gravity values using a conversion table. Bulk density is then calculated using the difference in gravity values and depth between the two stations. Usually two or three traverses are made in the hole at repeated stations and the values averaged. The precision of the method increases with decreasing distance between measuring stations.

Surveys may be conducted in cased or uncased holes. Several corrections to the data are necessary, including corrections for earth-moon tides, instrument drift, borehole effects, subsurface structure, gravity anomalies, hole deviation from vertical, and the free-air vertical gravity gradient. The instrument should be calibrated to correct for drift and tides. The borehole gravity logger is available commercially but is not a standard log. An apparent advantage over other bulk density/porosity logging methods is the reduction of borehole effects and the greater sample volume.

Magnetic Log

The magnetic field in an uncased borehole is the result of the Earth's magnetic field and any induced or remnant magnetism. These fields can be directly measured. Magnetic susceptibility is the degree that a material is effected by a magnetic field and is the basis for the logging technique. Susceptibility logs measure the change of inductance in a coil caused by the adjacent borehole wall. The magnetic susceptibility is proportional to the amount of iron-bearing minerals in the rock.

Flowmeter Log

A flowmeter measures fluid flow in boreholes. A flowmeter log provides information about locations where fluid enters or leaves the borehole through permeable material or fractures intersecting the borehole.

Conventional flowmeters employ a spinner or propellor driven by fluid moving through the borehole. High resolution flowmeters such as the heat pulse flowmeter can measure very low flow rates. Heat pulse flowmeters have a heating element that heats borehole fluid. The flow rate and direction of the heated fluid pulse is measured by detectors located above and below the heat element. Electromagnetic flowmeters induce a voltage in electrically conductive borehole fluid moving at a right angle through a magnetic field. The induced voltage is proportional to the velocity of the conductive borehole fluid.

Bibliography

Hearst, J. R, and Nelson, P.H., *Well Logging for Physical Properties*, McGraw-Hill Book Co., New York, 1985.

Labo, J., *A Practical Introduction to Borehole Geophysics*, Society of Exploration Geophysicists, Tulsa, Oklahoma, 1987.

Chapter 15

REMOTE SENSING TECHNIQUES

Introduction

This chapter briefly summarizes the capabilities, limitations, and requirements of typical remote sensing techniques. Depending on the nature of the data and the objective of the study, geologic interpretation of remotely sensed data may be simple or complicated. Remote sensing is a tool that makes some tasks easier, makes possible some tasks that would otherwise be impossible, but is inappropriate for some tasks. Depending on the individual situation, remote sensing may be extremely valuable. Some remote sensing interpretations can stand on their own with confidence, but for most, establishing ground truth is essential.

Imaging Systems

There are three main types of imaging systems, two of which are widely used in terrestrial applications:

Photographic.—Cameras and film are used. Photography provides the best spatial resolution but less flexibility in spectral data collection and image enhancement. Spatial resolution is dependent on altitude, focal length of lenses, and the types of film used. Spectral resolution is limited to visible and near infrared wavelengths.

Electronic Spectral Sensors.— Detectors are used, usually scanners, that may have less spatial resolution than photographs but can gather spectral data over wide spectral ranges that enable a wide variety of imaging processing, mineral, and geologic identification. These remote sensing systems include satellites, such as Landsat, airborne sensors carried on aircraft, and spectrometers carried on the ground. This type of remote sensing has several categories, including multi-spectral, hyperspectral, and imaging spectroscopy.

Vidicon. – A television-type system. Vidicon systems generally are inferior to other types both spatially and spectrally. They are used mostly on space probes because of operational constraints.

Resolution

Two types of resolution are important to remote sensing.

Spatial.— The sharpness of an image and the minimum size of objects that can be distinguished in the image are a function of spatial resolution.

Spectral.—The width or wavelength range of the part of the electromagnetic spectrum the sensor or film can record and the number of channels a sensor uses defines spectral resolution. The electromagnetic spectrum is an ordered array of electromagnetic radiation based on wavelength. Certain portions of the electromagnetic spectrum, including the visible, reflective infrared, thermal infrared, and microwave bands, are useful for remote sensing applications. Rocks, minerals, vegetation, and manmade materials have identifying spectral signatures and distinctive absorptive and reflective spectral characteristics. Given sufficient spectral data, digital image processing can generally identify unique spectral signatures. In practice, instrument limitations or cost limits on computer processing may preclude identification of some materials.

REMOTE SENSING TECHNIQUES

Photography

The uses of aerial photography in engineering geology are discussed in Volume 1, chapter 6. A brief description of types is presented below.

Panchromatic photography records images essentially across the entire visible spectrum and, with proper film and filters, also can record into the near-infrared spectrum. In aerial photography, blue generally is filtered out to reduce the effects of atmospheric haze.

Natural color images are recorded in the natural colors seen by the human eye in the visible portion of the spectrum.

False-color infrared images are recorded using part of the visible spectrum and part of the near-infrared, but the colors in the resultant photographs are arbitrary and not natural. Infrared film commonly is used and is less affected by haze than other types.

Multispectral photographs acquired by multiple cameras simultaneously recording different portions of the spectrum can facilitate interpretation.

Terrestrial photographs acquired on the ground or at low altitude with photogrammetric cameras can be used to map geologic features such as strike and dip of joints, faults, and geologic contacts. This type of geologic photogrammetry can also be done from historic construction photographs if enough survey control features can be located in the photographs and tied to present control or known points. Terrestrial photogrammetry has proven useful on several dam projects in which mapping of geologic features by standard methods was very difficult or impossible.

Thermal Infrared Imagery

Thermal infrared systems create images by scanning and recording radiant temperatures of surfaces. Some characteristics of thermal image data are generic to digital image data, and enable computer image processing. Other characteristics are unique to thermal infrared images and make thermal image data valuable interpretive tools.

Thermal infrared imagery can be interpreted using conventional photogeologic techniques in conjunction with the thermal properties of materials, instruments, and environmental factors that affect the data. Where thermal characteristics of a material are unique, thermal infrared imagery can be easy to interpret and can be a great help in geologic studies. Thermal characteristics of a material can vary with moisture content, differential solar heating, and topography making interpretation more difficult and ambiguous.

Multispectral Scanner Imagery

Multispectral scanner (MSS) images are a series of images of the same target acquired simultaneously in different parts of the electromagnetic spectrum. MSS images are an array of lines of sequentially scanned digital data. They may have unique distortions and may or may not have high resolution or information content. Scanner systems consist of scanning mechanisms, spectral separators, detectors, and data recorders.

A digital image is actually an array of numerical data and can be computer processed for a variety of purposes. Geometric distortions caused by sensor characteristics can be removed or distortions can be introduced.

Computer processing can be used to precisely register a digital image to a map or another image. Various types of data (e.g., thermal and visible imagery or a digital image and digitized gravity data) can be merged into a single image. Subtle information, difficult to interpret or even to detect, can sometimes be extracted from an image by digital processing.

Airborne Imaging Spectroscopy

The "image," or data, consist of an array of reflected spectra collected in a two-dimensional array of pixels. Specific spectral features may be identified and pixels with the same spectral wavelength feature may be mapped in colors. Different colors are assigned to different reflectance spectra, and an image is generated from these classifications. A number of different airborne systems are becoming available with various spatial and spectral characteristics. A few commercially available systems are capable of recording 10 or more spectral bands simultaneously, ranging from ultraviolet to thermal infrared wavelengths. The key factor in spectral identification of materials, geologic features, or vegetation is the system's spectral resolution or number of channels. Currently, the premier sensor for imaging spectroscopy is the National Aeronautics and Space Administration (NASA) Jet Propulsion Laboratory airborne visible and infrared imaging spectrometer (AVIRIS). AVIRIS simultaneously collects data over a spectral range of 0.4 micron to 2.4 microns in 224 channels. The number of channels and very high quality of data allow discrimination of very small spectral absorption features and, therefore, the user can distinguish and map hundreds of types of minerals or vegetation. The AVIRIS instrument is operated by Jet Propulsion Laboratory (JPL) onboard a NASA ER-2 (modified U-2). Commonly operating at an altitude of 63,000 feet, the spatial resolution is approximately 20 meters. The AVIRIS instrument is also operated by JPL onboard a twin-engine aircraft at lower altitudes. The spatial resolution is approximately 1 meter. The size of data sets and the number of separate spectral bands on some airborne systems may require data consolidation or careful selection of data subsets for special processing. Because the Earth's atmosphere introduces it's own spectral absorption features along the light path from the ground to the sensor, removal of atmospheric spectral features is essential to proper analysis of spectroscopy or hyperspectral data. Ground calibration spectra using a field spectrometer operating over the same wavelength ranges and resolution collected at the same time as the overflight is generally required to achieve the highest quality mapping and discrimination of materials. Interpretation usually involves normal photogeologic techniques along with knowledge of spectral characteristics and the data manipulations applied. The spectral imaging data can be analyzed with specialized commercially available software, which relies primarily on statistical analyses and projection of a few spectral features in an image. Software being developed at the U.S. Geological Survey (USGS) Imaging Spectroscopy Laboratory is based on a comparison of laboratory spectral features of several hundred minerals and materials in a spectral library. This software, known as "Tetracorder," rapidly identifies and maps all the key spectral features of known minerals or materials in each pixel of an AVIRIS or other hyperspectral image. The companion field spectrometer can also be used in its own right to collect both laboratory and field spectra of materials for identification. Applications may include distinguishing different types of clay minerals (e.g., expansive or not) as the geologist maps in the field in real time.

High resolution can be obtained and, with proper band selection and processing, complex geochemistry, mineralogy, vegetation, or other materials information can be mapped from the imagery. These technologies have been used very successfully in mapping large areas for several environmental/engineering geology related projects. Because of the complexity of the geology and the size of the area studied, the calibration and analysis of imaging spectroscopy data is not trivial. The volume of high quality data that can be collected using imaging spectroscopy over immense areas in one flight may provide an advantage over any other traditional method of mapping in certain applications.

Satellite Multispectral Scanner Imagery

Landsat is the NASA satellite for civilian remote sensing of the Earth's land surface. The Landsat MSS records seven bands at 30-meter resolution. The MSS is useful for some geological applications, although it is oriented primarily toward agricultural uses. The Landsat 7 is equipped also with a thematic mapper (TM) that has increased spatial and spectral resolution compared to the MSS.

SPOT, a French satellite, provides panchromatic imagery with up to 10-meter spatial resolution. MSS imagery is capable at 20-meter resolution of stereo imaging. Other commercially available satellites designed for rapid and repeated imagery of customer-selected sites are also being put in orbit to produce imagery of any area. In addition, archives of once classified satellite imagery gathered by United States spy satellites have become available to the public and government agencies and may be a source of data and images.

Satellite imagery provides a synoptic view of a large area that is most valuable for regional studies. With increasing resolution and new sensors, site-specific geologic mapping with satellite images is also of value to engineering geology.

Radar Imagery

Radar is an active remote sensing method (as opposed to passive methods like photography and thermal infrared) and is independent of lighting conditions and cloudiness. Some satellite radar imagery is available, like Landsat, and coverage may be useful for regional geologic studies. Side-looking airborne radar (SLAR) produces a radar image of the terrain on one side of the airplane equivalent to low-oblique aerial photography. Radar interferometry is a quickly emerging field of radar remote sensing. Radar interferometry techniques will detect very small changes in topography, such as those caused by landslide movements, fault displacements, erosion, or accretion, and can be mapped remotely over large areas.

Radar imagery has some unusual distortions that require care in interpretation. Resolution is affected by several factors, and the reflectivity of target materials must be considered. The analysis and interpretation of radar imagery requires knowledge of the imaging system, the look direction, and the responses of the target materials.

Radar can penetrate clouds and darkness and, to some extent, vegetation or even soil. Distortions, image geometry, and resolution can complicate interpretation, as can a lack of multispectral information.

Side Scan Sonar

Side Scan Sonar (SSS) is used for underwater surveys and not remote sensing in the typical sense of using aerial or space platforms to image the Earth's surface. SSS produces images underwater and may have useful applications to engineering geologic studies of reservoirs, dams, and other structures or surfaces hidden by water. SSS works by sending out acoustic energy and sensing the return of the acoustic signal. Materials on the bottom surface under water reflect the signal at different strengths depending on the material properties and the angles of incidence and reflection of the signal. The varying strength of signal return is then used to form an image. An image of the reflected signal that looks like a black and white aerial photograph is the output. The acoustic signal is generated by a torpedo-like instrument that is towed behind a boat by cable. Resolution of objects or features on the bottom of a reservoir varies with the frequency of the signal generated and the height of the torpedo above the bottom of the water body.

Single- and Multi-Beam Sonar

Other types of sonar devices that can be used to map the topographic surface of a reservoir bottom include single-beam and multi-beam sonar. These devices send a direct acoustic signal and measure the time of return. This is translated to a depth to the bottom or feature. By sending continuous signals along a line of survey, a profile of the bottom is developed. When multiple lines are tied together in criss-crossing patterns with Global Positioning System (GPS) positions being recorded simultaneously, a topographic map of the bottom can be generated. Multi-beam systems send out multiple beams at different angles simultaneously, thereby generating a topographic surface over much larger areas in greater detail than single-beam systems.

Applications to Engineering Geology

The most useful form of remote sensing for engineering geology applications is aerial photography because of its high resolution, high information content, and low cost. Various scales of aerial photography are available for regional and site studies, for both detecting and mapping a wide variety of geologic features.

Applications of other forms of remote sensing to engineering geology depend on the nature of the problem to be solved and the characteristics of the site geology. Some problems can best be solved using remote sensing; for others, remote sensing is of little value. In some cases, the only way to find out if remote sensing can do the job is to try it.

Appropriate remote sensing data are often not available for the area of interest, and the data must be acquired specifically for the project. Mission planning and time and cost estimating are critical to a remote sensing data collection program. Numerous factors make planning a remote sensing mission more complicated than planning a conventional aerial photographic mission. Remote sensing mission planning is best done in consultation with someone with experience in the field.

Estimating the cost of data acquisition is relatively easy. Estimating the cost of interpretation is a function of the time and image processing required and is much harder. Some forms of remote sensing are inexpensive to acquire and require little processing to be useful. Other forms may be expensive to acquire or may require much processing, perhaps needing experimentation to determine what will work in an untried situation. The cost must be weighed against the benefits.

Bibliography

Two excellent reference books on remote sensing and applications to geology are:

Sabins, F.F., *Remote Sensing Principles and Interpretation,* 2nd edition: Freeman, New York, 449 p., 1987.

Siegal, B.S., and Gillespie, A. R., editors, *Remote Sensing in Geology,* Wiley & Sons, New York, 702 p., 1980.

For a thorough treatment of remote sensing ("more than you ever wanted to know about remote sensing"), including a 287-page chapter on geological applications with abundant color images, see:

Colwell, R N., editor, *Manual of Remote Sensing,* 2nd edition, Falls Church, Virginia, American Society of Photogrammetry, 2724 p., 1983.

Chapter 16

WATER TESTING FOR GROUTING

Introduction

Water testing is necessary for evaluating seepage potential and for determining whether grouting is necessary or practical. Water testing for designing a grout program is often secondary to the main purpose of the water testing program, which is to determine permeabilities for seepage evaluation or control. Design of an exploration program for water testing for grouting can be significantly different from permeability testing for the design of a dam or tunnel because of the desired results, the restricted area of a damsite, or the often high-cover linear tunnel site. Regardless of the main reason for the investigations, design of a program for water testing for grouting should consider:

(1) The type of structure to be built. Water testing for evaluating and grouting a dam foundation is significantly different than water testing for a tunnel. More drill holes are generally available for testing in dam foundations where seepage potential is the primary concern and grouting is secondary. Relatively few drill holes are available for evaluating tunnel alignments because of the great depths often necessary and the linear nature of a tunnel. A great deal of judgment is necessary in evaluating the data for tunnels because of the small sample size. Ground conditions are of primary interest, and permeability and groutability are secondary, especially if the tunnel will be lined.

(2) The geologic conditions at the site and the variations between areas or reaches. Generalizations based on other sites are usually inaccurate because geologic conditions depend on the

interrelationship of the local depositional, tectonic, and erosional history that uniquely determine geologic conditions important to the permeability and groutability of a site. Damsite foundation permeabilities can vary over short distances because of lithology and fracture changes, faults, or stress relief in the abutments. Tunnel studies compound the difficulties because of the linear nature of the structure, the often high cover, and access problems. Proper evaluation of water test results requires that the values be correlated with geologic conditions. The permeability values should be noted and plotted on the drill logs along with the water takes and test pressures. The test interval should be drawn on the log so that the water test data can be related to fracture data.

(3) The level of seepage control desired. Seepage quantities beneath a dam in the arid West may be insignificant compared to those in an area with abundant rainfall. Infiltration into a tunnel may be insignificant in one area but, because of overlying cultural and environmental features, may be the controlling factor in a dewatering, excavation, or lining design. The economic value of the water may also be a significant factor.

(4) Groutability and permeability are not necessarily related. A highly fractured rock or a gravel may be very permeable but essentially ungroutable using standard cement grout. Water test data must be correlated with geologic data to properly assess groutability.

(5) Groutability depends on fracture openness and number. Connectivity is not necessarily important because of the relatively limited travel of grout.

(6) Permeability depends on fracture openness, number, and connectivity. Highly fractured rock with low connectivity will have low permeability, and a slightly fractured rock with high connectivity can have high permeability.

(7) Exploratory drill hole orientations introduce a significant bias into water test results. The orientation of the drill hole relative to the fractures has a direct effect on the number of fractures intercepted by the hole. Vertical boreholes intercept very few vertical fractures and can provide very misleading water test information on rock mass permeability and groutability. A vertical hole drilled in a material that has predominantly vertical fractures like flat-bedded sediments will not intercept the fractures that control the rock mass permeability. Drill holes should be oriented to cross as many fractures as possible not only for more meaningful permeability tests but also to get more meaningful rock mass design parameters. If a preferred hole orientation is not practical, the results may be adjusted for the orientation bias.

(8) Water test calculation results can be very misleading. Water test calculations based on a 10-foot (3-meter [m]) interval with one 1/4-inch (6-millimeter [mm]) fracture taking water can have a significantly different seepage and grout potential than a 10-foot (3 m) interval with dozens of relatively tight fractures taking the same amount of water. Each water test must be evaluated individually to determine what the data really mean.

(9) Different rock types, geologic structures, or in situ stresses have different jacking and

hydrofracture potential and, therefore, different maximum acceptable water testing and grouting pressures. Dam foundations are more sensitive to jacking and hydro-fracturing than tunnels. Dam foundations can be seriously damaged by jacking and hydrofracturing. A dam foundation in interbedded sedimentary rock with high horizontal in situ stresses is very sensitive to jacking. A dam foundation in hard, massive granite is almost unjackable and may have to be hydrofractured before any movement can take place. Tunnels are less susceptible to jacking because of overburden depth and may benefit from jacking open fractures to provide grout travel and closure of ungrouted fractures.

(10) Rock mass permeability or groutability cannot be determined by drill holes alone. Mapping and analysis of the fractures are necessary factors in determining seepage potential and groutability. Drill holes usually do not provide a realistic characterization of fracture orientations and connectivity. All these data should be integrated to determine rock mass permeability or groutability.

(11) Rules of thumb are not good substitutes for using data and judgment in making decisions in grouting unless specifically developed for the site conditions.

(12) Hydraulic models are a tool that can be used to evaluate seepage potential and groutability but depend on realistic design and data input parameters. Water test-derived permeability and groutability are important parameters for hydraulic models. Water tests must be carefully evaluated to ensure that bad test data are not used in models.

Models large enough to approach characterizing a site are usually very large and expensive. The tendency is to build models that are small and economical and, therefore, have a limited connection with reality. Realistic parameters are difficult to obtain in quality or quantity. Few exploration programs provide a statistically significant sample size to fully characterize a site, especially tunnels. Model input parameters and design should be part of any modeling report so the output can be properly evaluated.

Procedure

Calculations

Permeabilities can be calculated in lugeons, feet per year (ft/yr), centimeters per second (cm/sec), or other units from the same basic field data. Lugeons are used in the following discussion because these units are commonly used in the grouting industry. Chapter 17 has a thorough discussion of testing and calculating in other permeability units. One Lugeon equals:

(1) 1 liter per minute per meter (l/min/m) at a pressure of 10 bars

(2) 0.0107 cubic feet per minute (ft^3/min) at 142 pounds per inch (psi)

(3) 1 x 10^{-5} cm/sec

(4) 10 ft/yr

The formula for calculating Lugeons (lu) is:

$$\textit{take} \times \frac{10}{\textit{test}}\,\frac{\textit{bar}}{\textit{pressure}}\,\frac{}{(\textit{bars})} \qquad \textit{take} \times \frac{}{\textit{test}}\,\frac{1.55}{\textit{pressure}}\,\frac{}{(\textit{psi})}$$

Take is in liters per meter per minute or cubic feet per foot per minute. One bar equals 14.5 psi. Takes should be measured after flows have stabilized and should be run for 5-10 minutes at each pressure step.

Geologic Data

The geologic data should be examined to determine the optimum drill hole orientations and locations. Geologic structure, such as bedding and rock type, can be used to set the initial maximum water test pressures. Easily jacked or hydrofractured rock should initially be water tested at a pressure of 0.5 psi per foot (2.2 kilogram per square centimeter per meter [kg/cm^2/m]) of overburden and increased pressures based on stepped pressure tests or jacking tests.

Stepped Pressure Tests

Stepped pressure tests are the best method of conducting water tests. Pressures are stepped up to the maximum pressure and then stepped down through the original pressures. Comparison of the calculated permeability values and the pressure versus flow curves for the steps can provide clues as to whether the flow is laminar, if jacking or hydrofracturing is occurring, and if fractures are being washed out or plugged. Single pressure tests can be misleading because of all the unknown pressure and flow variables affecting the test. The Lugeon value for the test interval should be determined by analysis of the individual test values and not necessarily by an average. The individual tests are used to determine what is happening in the rock, and one value of the five tests is usually the appropriate value to use.

Figures 16-1 through 16-5 are bar chart plots showing the relationship of pressure to Lugeon values for the

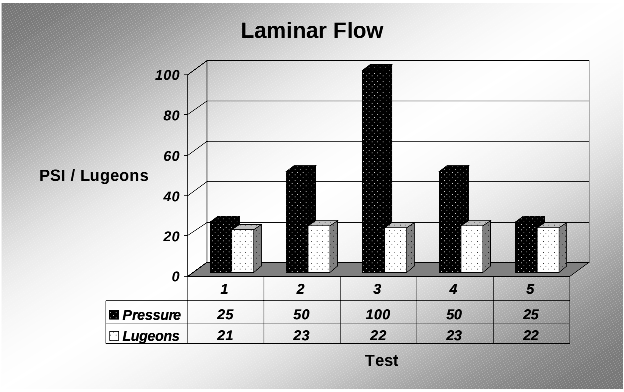


Figure 16-1.—Bar chart showing relationship of test pressure and Lugeons in laminar flow.

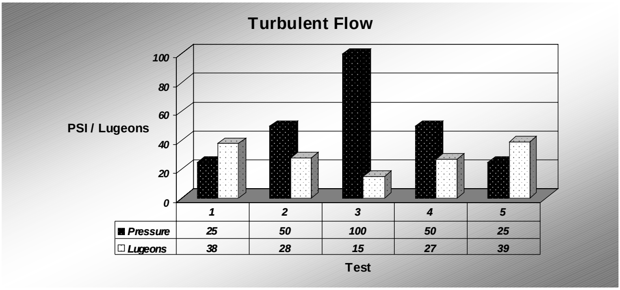


Figure 16-2.—Bar chart showing relationship of test pressure and Lugeons in turbulent flow.

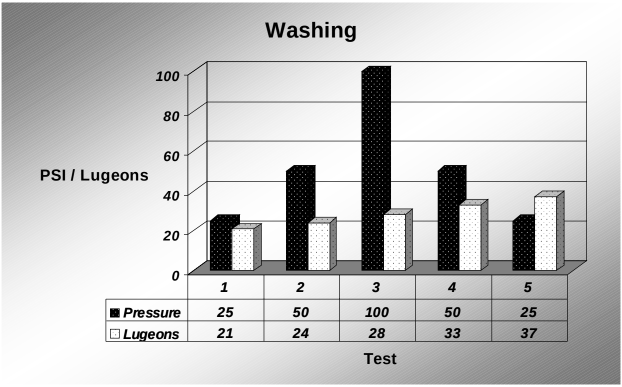


Figure 16-3.—Bar chart showing relationship of test pressure and Lugeons when fractures are washing out.

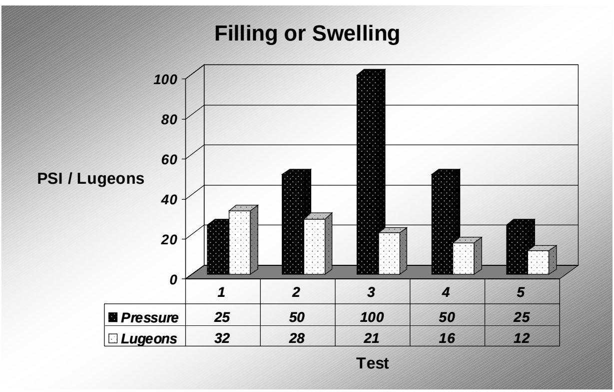


Figure 16-4.—Bar chart showing relationship of test pressure and Lugeons when fractures are filling or swelling.

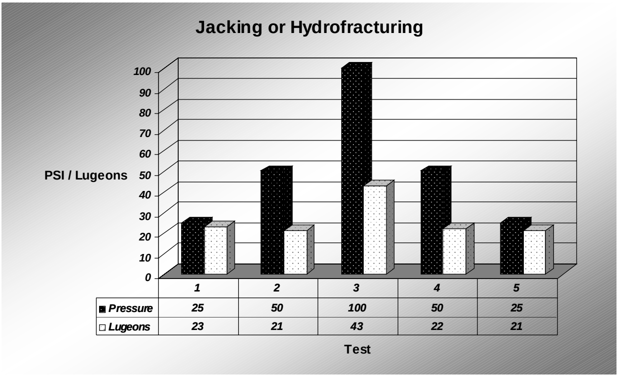


Figure 16-5.—Bar chart showing relationship of test pressure and Lugeons when rock is hydrofractured or joints are jacked open.

more common types of water test results. Figure 16-1 is a plot of laminar flow in the fractures. The permeability is essentially the same no matter what the pressure and resultant water take. Figure 16-2 is a plot of turbulent flow in the fractures. Permeability decreases as the pressure and resultant flow increases because of the turbulent flow in the fractures. Figure 16-3 is a plot of flow in fractures that increase in size as the water washes material out of the openings. Permeability increases because fractures are enlarged by the test. Figure 16-4 is a plot of flow in fractures that are being filled and partially blocked as water flows or the fractures are in swelling rock that closes fractures over time because of the introduction of water by the test. Figure 16-5 is a plot of testing in rock that is being jacked along existing fractures or rock that is being fractured by the highest water test pressure. Flow is laminar at the lower pressures.

Combinations of these types of flow can occur and require careful analysis. If the pressures are increased to where jacking or hydrofracturing is occurring, the design grout pressures can be set as high as possible to get effective grout injection yet preclude fracturing or, if appropriate, induce fracturing. Hydrofracture tests are easier to analyze if a continuous pressure and flow recording is obtained. The resolution of a step test may not be adequate to separate hydrofracturing from jacking. Figure 6 is a plot of a continuously recorded hydrofracture/jacking test.

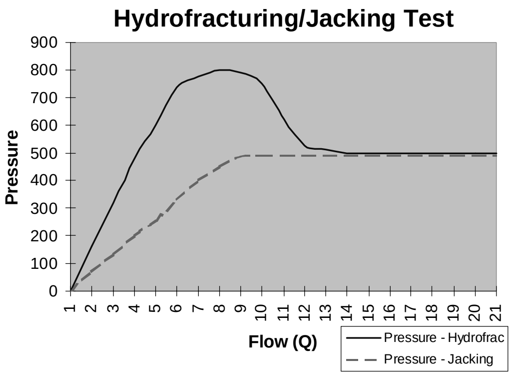


Figure 16-6.—Continuously recorded plot of pressure and flow.

Back Pressures

Back pressures should be measured after every test to determine if the hole holds pressure and if fracturing or jacking has occurred. If the hole holds pressure and backflow occurs while releasing the pressure, hydro-fracturing or jacking may have occurred at the test pressure.

Test Equipment

Test equipment can affect the test results. At moderate to high flows, the friction loss caused by the piping and the packer should be considered. Significant loss of pressure occurs between the gauge and the packer. At high flows, the plumbing system "permeability" can be the controlling factor and not the permeability of the test interval. If meters and gauges are located in optimum relation to each other and close to the hole, the arrangement of pipe, hose, etc., will not seriously influence shallow tests although sharp bends in hose, 90-degree fittings on pipes, and unnecessary changes in pipe and hose diameters should be avoided. Laying the system out on the ground and pumping water through the plumbing to determine the capacity of the system is a good idea, especially if using small-diameter piping or wireline packers.

Water Takes Relative to Grout Takes

Water takes alone may be an indication of whether grouting is necessary, and Lugeon calculations may not be necessary after enough water tests with subsequent grouting provide sufficient data to determine a correlation. This shortcut is usually used during the actual grouting because not enough data are available from exploration unless a test grout section is constructed. The correlation can change if the geologic domain in a foundation changes, so any correlation must be continuously checked.

Depth of Grouting

Grouting depth for dam foundations is commonly determined by a rule of thumb related to dam height. A better method is to base the depth on water test data and geologic data. Weathered rock and fracture surfaces are an indication of permeable rock. Grouting to tight, unweathered rock, where practical, makes more sense than trying to grout tight, impermeable rock or not grouting pervious rock because of a rule-of-thumb approach.

Grouting in tunnels should be based on the thickness of the disturbed zone around the periphery of the tunnel and the depth necessary to get the desired results when grouting natural fractures. A machine-bored tunnel will have a much shallower disturbed zone than a conventionally excavated (drill-blast) tunnel and may require less or no grouting.

Bibliography

Ewart, Friedrich-Karl, *Rock Permeability and Groutability Related to Dams and Reservoirs*, lecture notes, University of Paderborn, Germany, September 1994.

Water Resources Commission N.S.W., *Grouting Manual*, 3rd edition, Australia, 1980.

Chapter 17

WATER TESTING FOR PERMEABILITY

General

Most rock and soil contains numerous open spaces where water may be stored and through which water can move. Permeability, or hydraulic conductivity, is a measure of the ease of movement of fluid and gas through the open spaces and fractures. The properties of soil and rock have significant impact on water movement through the interstitial spaces. Water movement through soil and rock significantly impacts the ability to control water during construction. The movement of water through slopes must be known to understand the stability of slopes. Permeability in this chapter is considered synonymous with the term hydraulic conductivity and is a measurement of the groundwater flow through a cross-sectional area. This chapter discusses how to measure permeability and how to best determine or estimate appropriate permeability values.

In addition to permeability, there are other hydrologic parameters that may impact the understanding of groundwater but are either more difficult to determine or are not a significant consideration in most engineering situations. These other hydraulic parameters of subsurface materials are transmissivity, porosity, and storage. For a more detailed explanation of these hydrologic parameters and the methods used to obtained the values, see chapters 5, 6, and 10 of the Reclamation *Groundwater Manual*.

Transmissivity

Transmissivity is the average permeability multiplied by the saturated thickness. Transmissivity is particularly important in areas with multiple aquifers. Determining the aquifer thickness may not be practical or necessary if permeability and recharge are relatively low and the need for groundwater control is short term. Transmissivity values are often based on individual permeability values or averaged permeability values. It is important that any permeability value used with the saturated thickness of an aquifer be appropriate and representative. Since most aquifers are rarely homogeneous and isotropic, any field derived data must be qualified by indicating if averaging or selecting the highest or lowest permeability value for a particular site is appropriate.

Porosity

Porosity is the percentage of interstitial space within the soil or rock relative to the total volume of soil or rock. Porosity is not necessarily directly proportional to permeability. Porosity is a significant factor in understanding the stability of soil and rock, but only the effective porosity, the interconnected pores, contribute to permeability. Not all pores or interstitial spaces are connected. Porosities are typically high for sands and gravels (30-40 percent) with high permeabilities (10 to 10^{-2} cm/sec). Clays have higher porosities (45-55 percent) but have very low permeabilities (10^{-6} to 10^{-8} cm/sec). Effective porosity is important in high permeability materials, but the total porosity is rarely relevant except in contaminant transport modeling. In the field, porosity is typically estimated by borehole geophysics.

Storage

Storage is a dimensionless term defined as the volume of water released from or taken into interstitial spaces in the soil or rock. Storage is often interchanged with the terms specific yield, effective porosity, coefficient of storage, and storativity. The water stored within the effective porosity is controlled by the material's ability to retain and to release water. Understanding storage is particularly important in high permeability materials or localized zones connected with a large source of water.

A glossary of abbreviations and definitions is shown in table 17-1 that are used in the various permeability calculations. Not all the parameters listed in the table are necessary for every calculation. These parameters are derived from the *Groundwater Manual*, chapter 10. Abbreviations used in the figures and text of this chapter are consistent throughout the chapter. Definitions apply to angled holes as well as vertical wells.

Geologic Conditions

Understanding and measuring the above parameters is important, but understanding the hydrogeologic conditions is essential. Obtaining representative and appropriate hydrologic values is critical in any site investigation. Identifying the water bearing zones and selecting the appropriate test method is very important. Improper test methods, poor well construction, and improper isolation can significantly impact any test design. It is essential that the various aquifers and boundaries in heterogeneous settings be identified and that the various water surfaces existing at the site be located. Obtaining values without a good understanding of subsurface conditions can be misleading and can result in surprises once the actual site conditions are exposed. Obtaining water level data and permeability values has little value without an understanding of the factors controlling the groundwater. Proper evaluation of permeability values requires that the values be correlated with geologic conditions.

Table 17-1.—A glossary of abbreviations and definitions used in permeability calculations

K	=	Coefficient of permeability in feet (meters) per year under a unit gradient.
Q	=	Steady flow into the well in ft^3/sec [m^3/sec].
H	=	The effective head of water in the well in feet (m). For packer tests, determining the effective head is defined in figure 17-5. The effective head may be natural or induced.
a	=	Exposed surface area of the test section in ft^2 (m^2). Note that this area in an uncased borehole or using a single packer for some tests would include the exposed area at the bottom of the borehole.
ℓ	=	Length of test (packer tests) or screened (perforated) section of well isolated from the adjacent material in feet (m).
r	=	The radius to the borehole sidewall.
r_e	=	The effective radius to borehole sidewall that is reduced because of an obstruction such as a slotted riser (perforated casing), caved material, gravel, or sand pack.
r_1	=	The outer radius of a riser or casing.
r_c	=	The inside radius of a riser or casing.
D	=	Distance from the ground surface to the bottom of the test section.
U	=	Thickness of unsaturated materials above the water surface, including the capillary zone.
T_u	=	U-D+H = The distance from the induced water surface in the well to the static natural water surface.
S	=	Thickness of saturated material overlying a relatively impermeable layer.
C_u	=	Conductivity coefficient for unsaturated materials with partially penetrating cylindrical test wells.
C_s	=	Conductivity coefficient for saturated materials with partially penetrating cylindrical test wells.

Selecting the Appropriate Test

There are numerous ways to determine or estimate permeabilities. The appropriate method for estimating permeabilities must be based on the subsurface conditions and how significantly the values obtained will impact the project. The required level of understanding of the subsurface water conditions should be weighed relative to the cost and the impact on stability and constructability of the feature and on the changes in quality and quantity of water important to the site. Soil classification and Standard Penetration Testing (SPT) blow counts provide crude estimates of soil permeabilities. There are numerous geophysical methods for estimating permeabilities by using flowmeters, acoustic velocities, and gamma borehole logging techniques. The Reclamation *Ground Water Manual*, chapters 8 and 10, provides detailed explanations of various test methods for determining permeabilities. Except for the aquifer tests, most methods described in this chapter determine the vertical or horizontal permeability.

The most accurate test method for determining permeability is conducting a relatively long-term aquifer test. This method is not covered in this chapter. A full-term aquifer test (pumping test as described in chapter 8 of the Reclamation *Ground Water Manual*) is rarely justified during initial site investigations because of the cost and time required to perform an aquifer test and possibly handle the discharged water. This chapter discusses the less costly methods used to determine permeability. These methods are less accurate, primarily because these tests are too short in duration and because the interval being tested is not necessarily "truly" undisturbed, open, dimensioned as assumed, and representative.

Stable Boreholes

Packer tests are commonly performed in boreholes because the tests are inexpensive and can be performed quickly, with minimal disruption to the primary task of most site investigations, which is determining the subsurface materials and geology. The Gravity Permeability Test Method 1 for consolidated material is used only within the vadose zone and is the preferred method if the packer test method cannot be used. The principal problem of any gravity permeability test method is that a uniform supply of water is necessary so that a constant head can be maintained above the static water surface. The falling head tests should be used in stable boreholes if the packer test and gravity tests (both above and below the static water surface) cannot be performed.

Unstable Boreholes

There are a number of permeability tests that can be used in unstable materials such as soils that are noncohesive, uncemented, or unindurated or fractured rock that collapses into the borehole. Slug tests are used primarily when water availability or usage is a problem. Piezometer tests are good where water surfaces are high and where water bearing units are relatively thin layered but significant. Unfortunately, the piezometer test is limited to relatively shallow depths (around 20 feet [6 m]) and is rarely successful in cohesionless or gravelly or coarser soils. Gravity permeability tests are used in unstable soils or in rock. Gravity permeability tests require costly construction techniques, delay or prevent further advancement of the borehole, and require a supply of water.

Borehole permeabilities are appropriate for the interval tested unless the test interval is greater than 10 feet

(3 m). For intervals greater than 10 feet (3 m), the test may not be an accurate reflection of the entire saturated column. The permeability values should be plotted on the drill log, along with the water take and test pressures. The test interval should be drawn on the log so that the water test data can be related to fracture data.

Permeability Testing in Rock

Permeability tests are routinely performed in rock, particularly by pressure or packer tests. The permeability calculation assumes laminar flow in an isotropic, homogeneous medium. In reality, the test water take is effectively controlled by fractures because the intact rock permeability is effectively zero in most cases. The water may be flowing into one or into many fractures in the test interval, but the permeability calculation assumes laminar flow in an isotropic, homogeneous medium. The length of the test interval is governed by the rock characteristics. Typically, the test interval may be 10 feet (3 m) long, but the water can be going into one ¼-inch (8-mm) fracture. Test intervals greater than 20 feet (6 m) are inadvisable because, typically, there are a few fractures or a relatively small zone that controls the groundwater flow in bedrock. The calculated permeability of the packer test interval may be a magnitude different from the actual rock mass permeability. Only in the case of a highly fractured rock mass is the calculated permeability relatively reliable and the result is still a relative or effective permeability.

Orientation.—The orientation of the drill hole relative to the fractures significantly affects the number of fractures intercepted by the hole and the perceived permeability. A vertical hole drilled in a material that has predominantly vertical fractures such as flat-bedded sediments will not intercept the predominant control on the rock mass permeability. The drill holes should be oriented to cross as many fractures as possible not only for more meaningful permeability tests, but also to get meaningful rock mass design parameters. If hole orientation is not practical, the results may be corrected for the orientation bias.

Jacking.—The pressure used for the water test should consider geologic structure. Flat-lying, bedded sediments are very susceptible to jacking along bedding planes. The combination of weak bedding planes, typically low vertical confining pressures, or high horizontal in place stresses can result in jacking and apparently high permeabilities. Test pressures of half the typical pressure of 1 pound per square inch per foot (psi/ft) (0.2 kg/cm^2/m) of depth are often appropriate.

Hydrofracturing.—In place stresses in many areas are not lithostatic and horizontal stresses are significantly lower than vertical stresses. The theoretical overburden stress (roughly 1 pound per square inch per foot depth [0.2 kg/cm^2/m]) is typically used to determine the test pressure. If the horizontal stress is much lower than the vertical stress, hydrofracturing can occur, resulting in an induced high permeability value.

Stepped Pressure Tests.—Stepped pressure tests are an effective method of conducting water tests. Pressures are stepped up to the maximum pressure and then stepped down through the original pressures. Comparison of the calculated permeability values and the pressure versus flow curves for the steps can tell you whether the flow is laminar, if jacking or hydrofracturing is occurring, and if fractures are being washed out. If the pressures are increased to where jacking or hydrofracturing is occurring, the design grout pressures can be set as high as possible to get effective grout injection, yet preclude fracturing or induce fracturing if desired. Chapter 16 discusses these tests in detail.

Test Equipment.—The test equipment can affect the test results. At moderate to high flows, the friction loss and restriction because of the piping (plumbing) and the packer is important. Significant pressure loss occurs between the gauge and the packer. At high flows, the plumbing system "permeability" can be the controlling factor rather than the permeability of the test interval. If meters and gauges are located in relation to each other as recommended, the arrangement of pipe, hose, etc., will not seriously influence the tests, although sharp bends in hose, 90-degree fittings on pipes, and unnecessary changes in pipe and hose diameters should be avoided. Laying the system out on the ground and pumping water through the plumbing to determine the capacity of the system is a good idea, especially if using small diameter piping or wireline packers.

In many investigations, information on the permeability of saturated or unsaturated materials is required. Permeability within the vadose zone, including the capillary fringe, is typically estimated by permeameter or gravity permeability tests. Permeability tests within saturated materials are typically performed as a falling head, packer, or aquifer test. Water within the unsaturated zone is suspended within the material but is primarily moving downward by gravity. In unsaturated conditions, the material permeabilities are obtained by one of several field permeameter tests. Since the material is not saturated, permeability tests require more equipment and a lot of water. These tests measure the volume of water flowing laterally while maintaining a constant head.

Laboratory permeability tests of subsurface materials usually are not satisfactory. Test specimens are seldom undisturbed, and a specimen typically represents only a limited portion of the investigated material. Field tests have been devised that are relatively simple and less costly than aquifer pumping tests (although aquifer tests do provide relatively accurate permeability values). These tests are usually conducted in conjunction with exploratory drilling or monitoring existing wells.

Permeability testing of existing monitoring wells may help determine material characteristics when evaluation of existing data indicates gaps or when it is necessary to confirm previous assumptions. Properly conducted and controlled permeability tests will yield reasonably accurate and reliable data. Several locations may need to be tested to provide data on spatial variations of subsurface material characteristics.

The quality of water used in permeability tests is important. The presence of only a few parts per million of turbidity or air dissolved in water can plug soil and rock voids and cause serious errors in test results. Water should be clear and silt free. To avoid plugging the soil pores with air bubbles, use water that is a few degrees warmer than the temperature of the test section.

For some packer tests, pumps of up to 250 gallons per minute (gal/min) (950 liters per minute [L/min]) capacity against a total dynamic head of 160 feet (50 m) may be required.

The tests described below provide semiquantitative values of permeability. There have been numerous types of permeability tests devised with varying degrees of accuracy and usefulness. The tests described below are relatively simple and generally provide useful permeability data. If the tests are performed properly, the values obtained are sufficiently accurate for some engineering purposes. The tests assume laminar flow and a homogeneous medium. These conditions are not often encountered, and fracture flow is what usually occurs in rock. The equations given for computing permeability are applicable to laminar flow. The velocity where turbulent flow occurs depends, in part, on the grain size of the materials tested. A maximum average velocity for laminar flow is about 0.1 foot per second (ft/sec) (25 millimeters per second [mm/sec]). If the quotient of the water intake in cubic units per second divided by the open area of the test section in square units times the estimated porosity of the tested material is greater than 0.10, the various given equations may not be accurate or applicable. The values obtained are not absolute and can vary from the true permeability by plus or minus an order of magnitude.

Length, volume, pressure, and time measurements should be made as accurately as available equipment will permit, and gauges should be checked periodically for accuracy. Keep the accuracy of the results in mind when determining the needed accuracy of the measurements. Results should be reported in feet per year or centimeters per second for most engineering uses.

In an open hole test, the total open area of the test section is computed by:

$$a = 2\pi r\ell + \pi r^2$$

where:

a = total open area of the hole face plus the hole bottom
r = radius of the hole
ℓ = length of the test section of the hole

When perforated casing is used and the open area is small, the effective radius, r_e, is used instead of the actual radius, r.

$$r_e = \frac{a_p}{a} r$$

a_p = total open area of perforations
a = area of each perforation

If fabricated well screens are used, estimates of the screen open area generally can be obtained from the screen manufacturer.

Permeability tests are divided into four types: pressure tests, constant head gravity tests, falling head gravity tests, and slug tests. In pressure tests and falling head gravity tests, one or two packers are used to isolate the test section in the hole. In pressure tests, water is forced into the test section through combined applied pressure and gravity head or the tests can be performed using gravity head only. In falling head tests, only gravity head is used. In constant head gravity tests, no packers are used, and a constant water level is maintained. Slug tests use only small changes in water level, generally over a short time.

Pressure Permeability Tests in Stable Rock

Pressure permeability tests are run using one or two packers to isolate various zones or lengths of drill hole. The tests may be run in vertical, angled, or horizontal holes. Compression packers, inflatable packers, leather cups, and other types of packers have been used for pressure testing. Inflatable packers are usually more economical and reliable because they reduce testing time and ensure a tighter seal, particularly in rough-walled, oversize, or out-of-round holes. The packer(s) are inflated through tubes extending to a tank of compressed air or nitrogen at the surface. If a pressure sensing instrument is included, pressure in the test section is transmitted to the surface. Although this arrangement permits an accurate determination of test pressures, manual observations should still be made to permit an estimate of permeability if pressure sensors fail. When double packers are used, the hole can be drilled to total depth and then tested. When a single packer is used, the hole is advanced and tested in increments.

Methods of Testing

In stable rock the hole is drilled to the total depth without testing. Two inflatable packers 5 to 10 feet (1.5 to 3 m) apart are installed on the drill rod or pipe used for making the test. The section between the packers is perforated. The perforations should be at least ¼ inch (6 mm) in diameter, and the total area of all perforations should be more than two times the inside cross-sectional area of the pipe or rod. Tests are made beginning at the bottom of the hole. After each test, the packers are raised the length of the test section, and another test is made of the appropriate section of the hole.

In unstable rock, the hole is drilled to the bottom of each test interval. An inflatable packer is set at the top of the interval to be tested. After the test, the hole is then drilled to the bottom of the next test interval.

Cleaning Test Sections

Before testing, the test section should be surged with clear water and bailed or flushed out to clean cuttings and drilling fluid from the hole. If the test section is above the water table and will not hold water, water should be poured into the hole during the surging, then bailed out as rapidly as possible. When a completed hole is tested using two packers, the entire hole can be cleaned in one operation. Although cleaning the hole is frequently omitted, failing to clean the hole may result in a permeable rock appearing to be impermeable because the hole wall is sealed by cuttings or drilling fluid.

Alternatives to surging and bailing a drill hole in indurated or consolidated material before pressure testing include rotating a stiff bristled brush while jetting with water. The jet velocity should be at least 150 ft/sec (45 m/sec). This velocity is approximately 1.4 gal/min per 1/16-inch- (5.3 L/min per 2-mm-) diameter hole in the rod. The drill hole should be blown or bailed out to the bottom after jetting.

Length of Test Section

The length of the test section is governed by the character of the rock, but generally a length of 10 feet (3 m) is acceptable. Occasionally, a good packer seal cannot be obtained at the planned depth because of bridging, raveling, fractures, or a rough hole. If a good seal cannot be obtained, the test section length should be increased or decreased or test sections overlapped to ensure that the test is made with well-seated packers. On some tests, a 10-foot (3-m) section will take more water than the pump can deliver, and no back pressure can be developed. If this occurs, the length of the test section should be shortened until back pressure can be developed, or the falling head test might be tried.

The test sections should have an $\ell/2r$ ratio greater than 5, where r is the radius of the hole and ℓ is the length of the test section. The packer should not be set inside the casing when making a test unless the casing has been grouted in the hole. Test sections greater than 20 feet (6 m) long may not allow sufficient resolution of permeable zones.

Size of Rod or Pipe to Use in Tests

Drill rods are commonly used to make pressure and permeability tests. NX and NW rods can be used if the take does not exceed 12 to 15 gallons per minute (45 to 60 liters per minute) and the depth to the top of the test section does not exceed 50 feet (15 m). For general use, 1¼-inch (32-mm) or larger pipe is better. Figures 17-1 through 17-4 show head losses at various flow rates per 10-foot (3-m) section for different sizes of drill rod and 1¼-inch (32-mm) pipe. These figures were compiled from experimental tests. Using 1¼-inch (32-mm) pipe, particularly where holes 50 feet (15 m) or deeper are to be tested, is obviously better than using smaller pipe. The couplings on 1¼-inch (32-mm) pipe must be turned down to 1.8 inch (45 mm) outside diameter for use in AX holes.

Pumping Equipment

Mud pumps should not be used for pumping the water for permeability tests. Mud pumps are generally of the multiple cylinder type and produce a uniform but large fluctuation in pressure. Many of these pumps have a maximum capacity of about 25 gal/min (100 L/min), and if not in good condition, capacities may be as small as 18 gal/min (70 L/min). Tests are often bad because pumps do not have sufficient capacity to develop back pressure in the length of hole being tested. When this happens, the tests are generally reported as “took capacity of pump, no pressure developed.” This result does not permit a permeability calculation and only indicates that the permeability is probably high. The fluctuating pressures of multiple cylinder pumps, even when an air chamber is used, are often difficult to read accurately because the high and low readings must be averaged to determine the approximate true effective

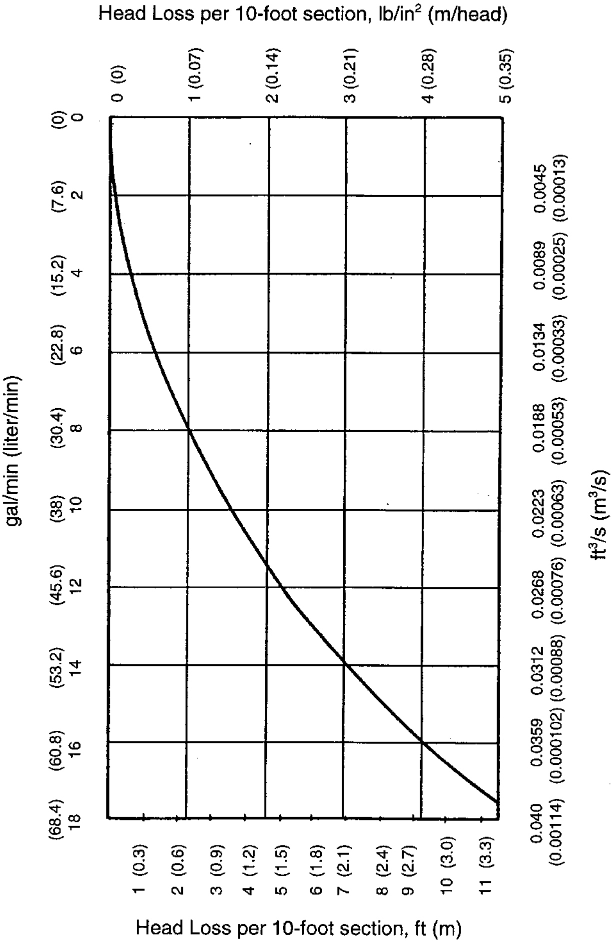


Figure 17-1.—Head loss in a 10-foot (3-m) section of AX (1.185-inch- [30.1-mm-] inside diameter [ID] drill rod.

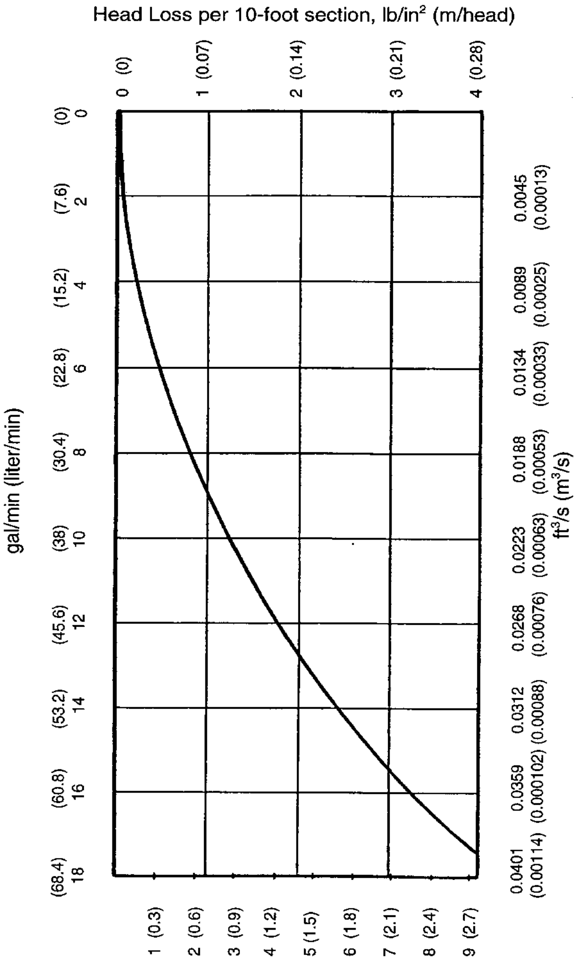


Figure 17-2.—Head loss in a 10-foot (3-m) section of BX (1.655-inch [42.0-mm] ID) drill rod.

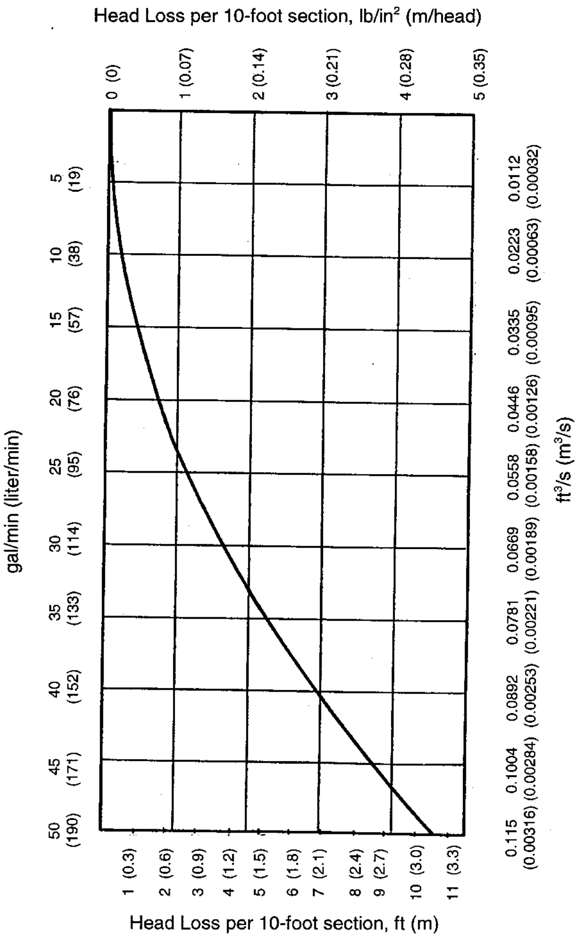


Figure 17-3.—Head loss in a 10-foot (3-m) section of NX (2.155-inch [54.7-mm] ID) drill rod.

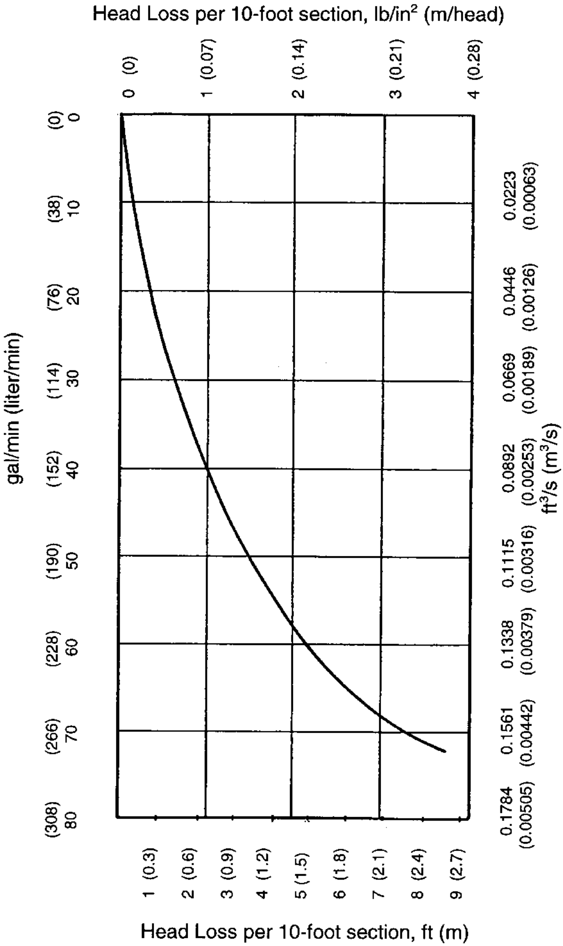


Figure 17-4.—Head loss in a 10-foot (3-m) section of 1¼-inch (32-mm) steel pipe.

pressure. In addition, mud pumps occasionally develop high peak pressures that may fracture the rock or blow out a packer.

Permeability tests made in drill holes should be performed using centrifugal or positive displacement pumps (Moyno type) having sufficient capacity to develop back pressure. A pump with a capacity of up to 250 gal/min (950 L/min) against a total head of 160 feet (48 m) is adequate for most testing. Head and discharge of these pumps are easily controlled by changing rotational speed or adjusting the discharge valve.

Swivels for Use in Tests

Swivels used for pressure testing should be selected for minimum head losses.

Location of Pressure Gauges

The ideal location for a pressure gauge is in the test section, but as close to the packer as possible.

Water Meters

Water deliveries in pressure tests may range from less than 1 gal/min (3.8 L/min) to as much as 400 gal/min (1,500 L/min). No one meter is sufficiently accurate at all ranges. Two meters are recommended: (1) a 4-inch (100-mm) propeller or impeller-type meter to measure flows between 50 and 350 gal/min (200 and 1,300 L/min), and (2) a 1-inch (25-mm) disk-type meter for flows between 1 and 50 gal/min (4 and 200 L/min). Each meter should be equipped with an instantaneous flow indicator and a totalizer. Water meters should be tested frequently.

Inlet pipes should be available to minimize turbulent flow into each meter. The inlet pipes should be at least 10 times the diameter of the meter inlet.

Length of Time for Tests

The minimum length of time to run a test depends on the nature of the material tested. Tests should be run until three or more readings of water take and pressure taken at 5-minute intervals are essentially equal. In tests above the water table, water should be pumped into the test section at the desired pressure for about 10 minutes in coarse materials or 20 minutes in fine-grained materials before making measurements.

Stability is obtained more rapidly in tests below the water table than in unsaturated material. When multiple pressure tests are made, each pressure theoretically should be maintained until stabilization occurs. This procedure is not practical in some cases, but good practice requires that each pressure be maintained for at least 20 minutes, and take and pressure readings should be made at 5-minute intervals as the pressure is increased and for 5 minutes as pressure is decreased.

Pressures Used in Testing

The pressure used in testing should be based on the rock being tested. Relatively flat-lying, bedded rock should be tested at 0.5 lbs/in^2 per foot (0.1 kg/cm^2/m) of depth to the test interval to prevent uplift or jacking of the rock. Relatively homogeneous but fractured rock can be tested at 1 lb/in^2 per foot (0.2 kg/cm^2/m) of test interval depth. Relatively unfractured rock can be tested at 1.5 lb/in^2 per foot (0.3kg/cm^2/m) of test interval depth.

Arrangement of Equipment

The recommended arrangement of test equipment starting at the source of water is: source of water; suction line; pump; waterline to settling and storage tank or basin, if required; suction line; centrifugal or positive displacement pump; line to water meter inlet pipe; water meter; short length of pipe; valve; waterline to swivel; sub for gauge; and pipe or rod to packer. All connections should be kept as short and straight as possible, and the number of changes in hose diameter, pipe, etc., should be kept as small as possible.

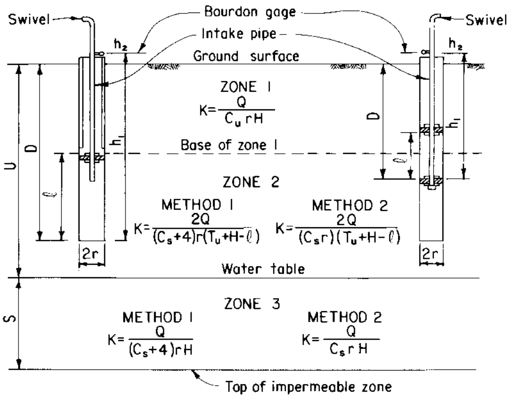
All joints, connections, and hose between the water meter and the packer or casing should be tight, and there should be no water leaks.

Pressure Permeability Tests

A schematic of the following two methods is shown in figure 17-5.

Method 1: The hole is drilled, cleaned, the tools are removed, a packer is seated the test interval distance above the bottom of the hole, water under pressure is pumped into the test section, and readings are recorded. The packer is then removed, the hole is drilled the test interval length deeper, cleaned, the packer is inserted using the length of the newly drilled hole as the test section, and the test performed.

Method 2: The hole is drilled to the final depth, cleaned, and blown out or bailed. Two packers spaced on pipe or drill stem to isolate the desired test section are used. Tests should be started at the bottom of the hole. After each test, the pipe is lifted a distance equal to l, shown on figure 17-5, and the test is repeated until the entire hole is tested.



K = coefficient of permeability, feet per second under a unit gradient

Q = steady flow into well, ft^3/s

$H = h_1 + h_2 - L$ = effective head, ft

h_1 (above water table) = distance between Bourdon gage and bottom of hole for method 1 or distance between gage and upper surface of lower packer for method 2, ft

h_1 (below water table) = distance between gage and water table, ft

h_2 = applied pressure at gage, 1 lb/in^2 = 2.307 ft of water

L = head loss in pipe due to friction, ft; ignore head loss for Q<4 gal/min in $1\frac{1}{4}$ - inch pipe; use length of pipe between gage and top of test section for computations

$X = \frac{H}{T_u}(100)$ = percent of unsaturated stratum

ℓ = length of test section, ft

r = radius of test hole, ft

C_u = conductivity coefficient for unsaturated materials with partially penetrating cylindrical test wells

C_s = conductivity coefficient for semi-spherical flow in saturated materials through partially penetrating cylindrical test wells

U = thickness of unsaturated material, ft

S = thickness of saturated material, ft

$T_u = U - D + H$ = distance from water surface in well to water table, ft

D = distance from ground surface to bottom of test section, ft

a = surface area of test section, ft^2; area of wall plus area of bottom for method 1; area of wall for method 2

Limitations:

$Q/a \leq 0.10$, $S \geq 5\ell$, $\ell \geq 10r$, thickness of each packer must be $\geq 10r$ in method 2

Figure 17-5.—Permeability test for use in saturated or unsaturated consolidated rock and well indurated soils.

Data required for computing the permeability may not be available until the hole has encountered the water table or a relatively impermeable bed. The required data for each test include:

- Radius, r, of the hole, in feet (meters).
- Length of test section, ℓ, the distance between the packer and the bottom of the hole, Method 1, or between the packers, Method 2, in feet (meters).
- Depth, h_1, from pressure gauge to the bottom of the hole, Method 1, or from gauge to the upper surface of lower packer in Method 2. If a pressure transducer is used, substitute the pressure recorded in the test section before pumping for the h_1 value.
- Applied pressure, h_2, at the gauge, in feet (meters), or the pressure recorded during pumping in the test section if a transducer is used.
- Steady flow, Q, into well at 5-minute intervals, in cubic feet per second (ft^3/sec) (cubic meters per second [m^3/sec]).
- Nominal diameter in inches (mm) and length of intake pipe in feet (m) between the gauge and upper packer.
- Thickness, U, of unsaturated material above the water table, in feet (m).
- Thickness, S, of saturated material above a relatively impermeable bed, in feet (m).
- Distance, D, from the ground surface to the bottom of the test section, in feet (m).
- Time that the test is started and the time measurements are made.

- Effective head, the difference in feet (m) between the elevation of the free water surface in the pipe and the elevation of the gauge plus the applied pressure. If a pressure transducer is used, the effective head in the test section is the difference in pressure before water is pumped into the test section and the pressure readings made during the test.

The following examples show some typical calculations using Methods 1 and 2 in the different zones shown in figure 17-5. Figure 17-6 shows the location of the zone 1 lower boundary for use in unsaturated materials.

Pressure permeability tests examples using Methods 1 and 2:

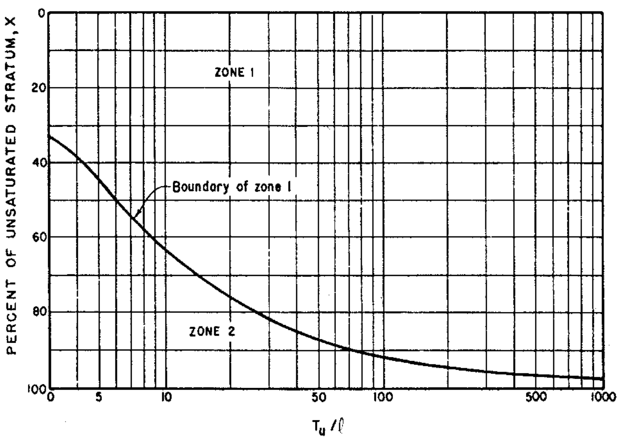


Figure 17-6.—Location of zone 1 lower boundary for use in unsaturated materials.

Zone 1, Method 1

Given: U = 75 feet, D = 25 feet, ℓ = 10 feet, r = 0.5 foot, h_1 = 32 feet, h_2 = 25 lb/in^2 = 57.8 feet, and Q = 20 gal/min = 0.045 ft^3/sec

From figure 17-4: head loss, L, for a 1¼-inch pipe at 20 gal/min is 0.76 foot per 10-foot section. If the distance from the gauge to the bottom of the pipe is 22 feet, the total head loss, L, is (2.2) (0.76) = 1.7 feet.

$H = h_1 + h_2 - L = 32 + 57.8 - 1.7 = 88.1$ feet of effective head, $T_u = U - D + H = 75 - 25 + 88.1 =$ 138.1 feet

$$X = \frac{H}{T_u}(100) = \frac{88.1}{138.1}(100) = 63.8\%$$

$$\frac{C_u}{\ell} = \frac{138.1}{10} = 13.8$$

The values for X and T_u / ℓ lie in zone 1 (figure 17-6). To determine the unsaturated conductivity coefficient, C_u, from figure 17-7:

$$\frac{H}{r} = \frac{88.1}{0.5} = 176.2$$

$$\frac{\ell}{H} = \frac{10}{88.1} = 0.11 \quad \textit{also} \quad C_u = 62$$

then:

$$K = \frac{Q}{C_u r H} = \frac{0.045}{(62)\ (0.5)\ (88.1)} = 0.000016 \ \textit{ft/s}$$

$$K = 0.000016 \ \textit{ft/s} \ X \ 3.15X10^7 = 504 \ \textit{ft/yr}$$

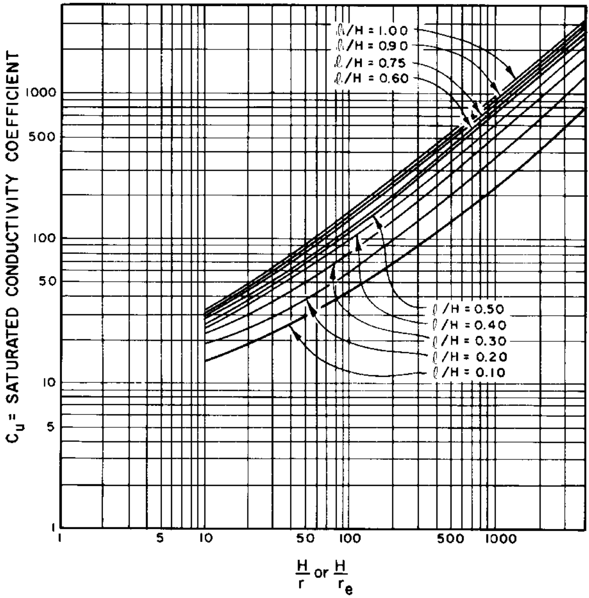


Figure 17-7.—Conductivity coefficients for permeability determination in unsaturated materials with partially penetrating cylindrical test wells.

Zone 2

Given: U, ℓ, r, h_2, Q, and L are as given in example 1, D = 65 feet, and h_1 = 72 feet

If the distance from the gauge to the bottom of the intake pipe is 62 feet, the total L is (6.2) (0.76) = 4.7 feet.

H = 72 + 57.8 - 4.7 = 125.1 feet

T_u = 75 - 65 + 125.1 = 135.1 feet

$$X = \frac{125.1}{135.1}(100) = 92.6\% \quad also \quad \frac{T_u}{\ell} = \frac{135.1}{10} = 13.5$$

The test section is located in zone 2 (figure 17-6). To determine the saturated conductivity coefficient, C_s, from figure 17-8:

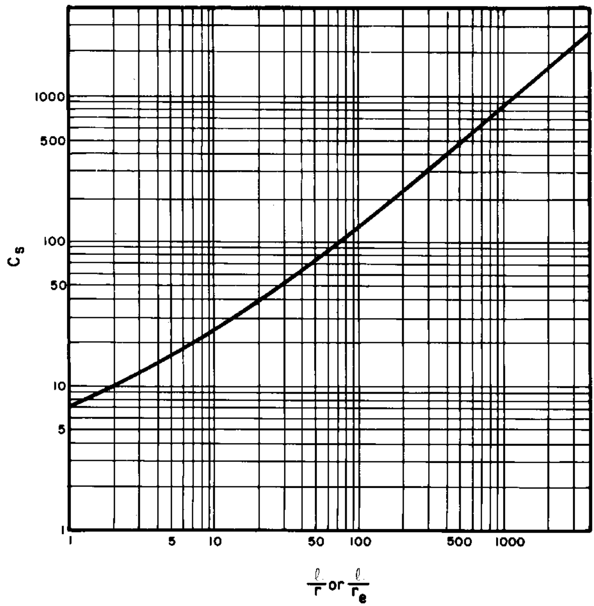


Figure 17-8.—Conductivity coefficients for semispherical flow in saturated materials through partially penetrating cylindrical test wells.

$$\frac{\ell}{r} = \frac{10}{0.5} = 20 \quad also \quad C_s = 39.5$$

Method 1:

$$K = \frac{2Q}{(C_s + 4)r(T_u + H - \ell)}$$

$$K = \frac{(2)(0.045)}{(39.5 + 4)(0.5)(135.1 + 125.1 - 10)}$$

$$K = 0.000016\ ft/s$$

$$K = 0.000016 ft/s\ X\ 3.15X10^7 = 504\ ft/yr$$

Method 2:

$$K = \frac{2Q}{(C_s\ r)(T_u + H - \ell)}$$

$$K = \frac{(2)(0.045)}{(39.5)(0.5)(135.1 + 125.1 - 10)} = 0.000018\ ft/s$$

$$K = 0.000018\ ft/s\ X\ 3.15X10^7 = 567\ ft/yr$$

Zone 3

Given:

U, ℓ, r, h_2, Q, and L are as given in example 1, D = 100 feet, h_1 = 82 feet, and S = 60 feet

If the distance from the gauge to the bottom of the intake pipe is 97 feet, the total L is (9.7) (0.76) = 7.4 feet.

H = 82 + 57.8 - 7.4 = 132.4 feet

$$\frac{\ell}{r} = \frac{10}{0.5} = 20 \quad also \quad C_s = 39.5 \ \textit{from figure} \ 17-8$$

Method 1:

$$K = \frac{Q}{(C_s + 4)rH} = \frac{0.045}{(39.5 + 4)(0.5)(132.4)} = 0.000016 \ ft/s$$

$$K = 0.000016 \ ft/s \ X \ 3.15X10^7 = 504 \ ft/yr$$

Method 2:

$$K = \frac{Q}{C_s \ rH} = \frac{0.045}{(39.5)(0.5)(132.4)} = 0.000017 \ ft/s$$

$$K = 0.000017 ft/s \ X \ 3.15X10^7 = 536 \ ft/yr$$

Multiple Pressure Tests

Multiple pressure tests are pressure permeability tests that apply the pressure in three or more approximately equal steps. For example, if the allowable maximum differential pressure is 90 lb/in^2 (620 kilopascal [kPa]), the test would be run at pressures of about 30, 60, and 90 lb/in^2 (210 kPa, 415 kPa, and 620 kPa).

Each pressure is maintained for 20 minutes, and water take readings are made at 5-minute intervals. The pressure is then raised to the next step. After the highest step, the process is reversed and the pressure maintained for 5 minutes at the same middle and low pressures. A plot of take against pressure for the five steps is then used to evaluate hydraulic conditions. These tests are also discussed in chapter 16.

Hypothetical test results of multiple pressure tests are plotted in figure 17-9. The curves are typical of those often encountered. The test results should be analyzed using confined flow hydraulic principles combined with data obtained from the core or hole logs.

Probable conditions represented by plots in figure 17-9 are:

1. Very narrow, clean fractures. Flow is laminar, permeability is low, and discharge is directly proportional to head.

2. Practically impermeable material with tight fractures. Little or no intake regardless of pressure.

3. Highly permeable, relatively large, open fractures indicated by high rates of water intake and no back pressure. Pressure shown on gauge caused entirely by pipe resistance.

4. Permeability high with fractures that are relatively open and permeable but contain filling material which tends to expand on wetting or dislodges and tends to collect in traps that retard flow. Flow is turbulent.

5. Permeability high, with fracture filling material which washes out, increasing permeability with time. Fractures probably are relatively large. Flow is turbulent.

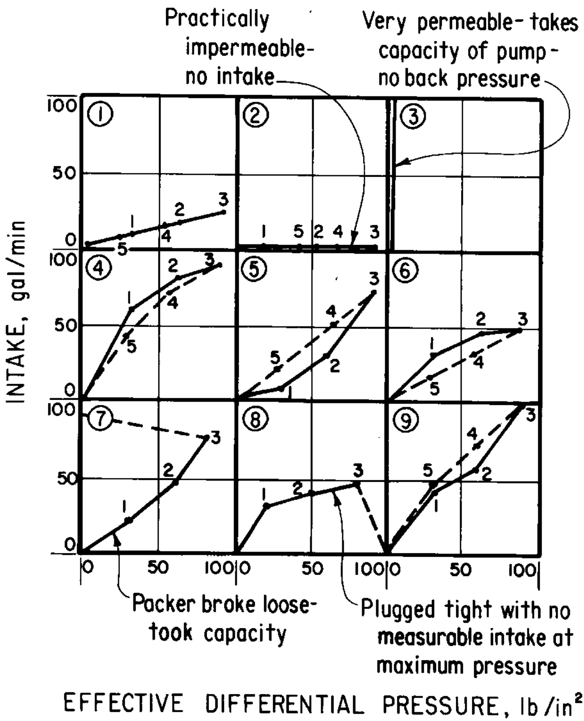


Figure 17-9.—Plots of simulated, multiple pressure permeability tests.

6. Similar to 4, but fractures are tighter and flow is laminar.

7. Packer failed or fractures are large, and flow is turbulent. Fractures have been washed clean; highly permeable. Test takes capacity of pump with little or no back pressure.

8. Fractures are fairly wide but filled with clay gouge material that tends to pack and seal when under pressure. Takes full pressure with no water intake near end of test.

9. Open fractures with filling that tends to first block and then break under increased pressure. Probably permeable. Flow is turbulent.

Gravity Permeability Tests

Gravity permeability tests are used primarily in unconsolidated or unstable materials. Gravity tests are performed in unconsolidated materials but are typically performed at greater depths. Gravity tests can be run only in vertical or near-vertical holes. A normal test section length is 5 feet (1.5 m). If the material is stable, stands without caving or sloughing, and is relatively uniform, sections up to 10 feet (3 m) long may be tested. Shorter test sections may be used if the length of the water column in the test section is at least five times the diameter of the hole. This length to diameter ratio is used in attempting to eliminate the effect of the bottom of the borehole. The diameter of the borehole sidewalls is the outside of any screen and annular packing of sand and or gravel. After each test, the casing for open hole tests is driven to the bottom of the hole, and a new test section is opened below the casing. If perforated casing is used, the pipe can be driven to the required depth and cleaned out, or the hole can be drilled to the required depth and the casing driven to the bottom of the hole and the hole cleaned out.

Cleaning and Developing Test Sections

Each newly opened test section should be developed by surging and bailing. Development should be done slowly and gently so that a large volume of loosely packed material is not drawn into the hole and only the compaction caused by drilling is broken down and the fines will be removed from the formation.

Measurement of Water Levels Through Protective Pipe

Measuring water depths inside a ¾- to 1½-inch- (20- to 40-mm-) small-diameter perforated pipe in the hole dampens wave or ripple action on the water surface caused by the inflow of water resulting in more accurate water level measurements. Water may also be introduced through the pipe and water level measurements made in the annular space between the pipe and the casing.

In an uncased test section in friable materials liable to wash, the end of the pipe should rest on a 4- to 6-inch (100- to 150-mm) cushion of coarse gravel at the bottom of the hole. In more stable material, the pipe may be suspended above the bottom of the hole, but the bottom of the pipe should be located at least 2 feet (0.6 m) below the top of the water surface maintained in the hole.

Pumping Equipment and Controls

Pressure is not required in the test, but pump capacity should be adequate to maintain a constant head during the test.

Accurate control of the flow of water into the casing is a problem on many gravity tests. The intake of the test section necessary to maintain a constant head is sometimes so small that inflow cannot be sufficiently controlled using a conventional arrangement. A precise method of controlling low flows, such as using needle valves, is important. Many meters are inaccurate at very low flows.

Water Meters

Water deliveries in gravity tests may range from less than 1 gal/min (3.8 L/min) to several hundred gallons per minute. No one meter is sufficiently accurate at all ranges. Two meters are recommended: (1) a 4-inch (100-mm) propeller- or impeller-type meter to measure flows between 50 and 350 gal/min (200 and 1,300 L/min) and (2) a 1-inch (25-mm) disk-type meter for flows between 1 and 50 gal/min (4 and 200 L/min). Each meter should be equipped with an instantaneous flow indicator and a totalizer. Water meters should be tested frequently.

Length of Time for Tests

As in pressure tests, stabilized conditions are very important if good results are to be obtained from gravity tests. Depending on the type of test performed, one of two methods is used. In one method, the inflow of water is controlled until a uniform inflow results in a stabilized water level at a predetermined depth. In the other method, a uniform flow of water is introduced into the hole until the water level stabilizes.

Arrangement of Equipment

A recommended arrangement of test equipment, starting at the source of water, is: suction line; pump; waterline to settling and storage tank or basin, if required; suction line; centrifugal or positive displacement pump; line to water meter inlet pipe; water meter; short length of pipe; valve; and the waterline to the casing. All connections should be kept as short and straight as possible, and the changes in diameter of hose, pipe, etc., should be kept to a minimum. If a constant head tank is used, the tank should be placed so that water flows directly into the casing.

Gravity Permeability Test - Method 1

For tests in unsaturated and unstable material using only one drill hole, Method 1 is the most accurate available. Because of mechanical difficulties, this test cannot be economically carried out at depths greater than about 40 feet (12 m) when gravel fill must be used in the hole. When performing the test, after the observation and intake pipes are set, add gravel in small increments as the casing is pulled back; otherwise, the pipes may become sandlocked in the casing. For tests in unsaturated and unstable material at depths greater than about 12 meters (40 feet), Method 2 should be used (described later).

The procedures for testing soil conditions are:

Unconsolidated Materials – A 6-inch (150-mm) or larger hole is drilled or augered to the test depth and then carefully developed. A cushion of coarse gravel is placed at the bottom of the hole, and the feed pipe

(I) and the observation pipe (O) are set in position (figure 17-10). After the pipes are in position, the hole is filled with medium gravel to a depth at least five times the diameter of the hole. If the drill hole wall material will not stand without support, the hole must be cased to the bottom. After casing, the gravel cushion and pipes are put in, and the casing is pulled back slowly as medium gravel is fed into the hole. The casing should be pulled back only enough to ensure that the water surface to be maintained in the hole will be below the bottom of the casing. About 4 inches (100 mm) of the gravel fill should extend up into the casing.

A metered supply of water is poured into the feed pipe until three or more successive measurements of the water level taken at 5-minute intervals through the observation pipe are within 0.2 foot (5 mm). The water supply should be controlled so that the stabilized water level is not within the casing and is located more than five times the hole diameter above the bottom of the hole. The water flow generally has to be adjusted to obtain the required stabilized level.

Consolidated Materials – The gravel fill and casing may be omitted in consolidated material or unconsolidated material that will stand without support even when saturated. A coarse gravel cushion is appropriate. The test is carried out as in unconsolidated, unstable materials.

Tests should be made at successive depths selected so that the water level in each test is located at or above the bottom of the hole in the preceding test. The permeability coefficients within the limits ordinarily employed in the field can be obtained from figures 17-7 and 17-8. The test zone and applicable equations are shown in figures 17-6 and 17-10.

Data required for computing the permeability may not be available until the hole has penetrated the water table. The required data are:

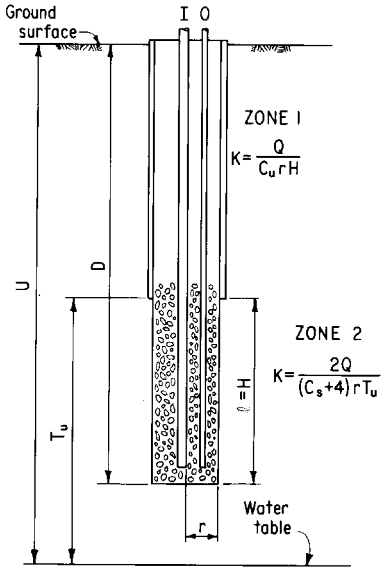
- Radius of hole, r, in feet (meters)
- Depth of hole, D, in feet (meters)
- Depth to bottom of casing, in feet (meters)
- Depth of water in hole, H, in feet (meters)
- Depth to top of gravel in hole, in feet (meters)
- Length of test section, ℓ, in feet (meters)
- Depth-to-water table, T_u, in feet (meters)
- Steady flow, Q, introduced into the hole to maintain a uniform water level, in ft^3/sec (m^3/sec)
- Time test is started and time each measurement is made

Some examples using Method 1 are:

Zone 1, Method 1

Given:

$H = \ell = 5$ feet, $r = 0.5$ foot, $D = 15$ feet, $U = 50$ feet, and $Q = 0.10$ ft^3/sec



K = coefficient of permeability, feet per second under a unit gradient
Q = uniform flow into well, ft^3/s
r = radius of test section, ft
H = height of column of water in well, ft
ℓ = length of test section, ft (for this method, ℓ = H)
C_u and C_s = conductivity coefficients
$X = \frac{H}{T_u}(100)$ = percent of unsaturated stratum
$T_u = U - D + H$ = distance from water surface in well to water table, ft
U = thickness of unsaturated permeable bed, ft
D = distance from ground surface to bottom of test section, ft
I = feed pipe for pouring water into well (a 2-inch standard pipe is usually satisfactory)
O = observation pipe ($1\frac{1}{4}$-inch o.d. pipe is satisfactory)
a = surface area of test section (area of wall plus area of bottom), ft^2
Limitations:
$\ell \geq 10r$ and $\frac{Q}{a} \leq 0.10$

Figure 17-10.—Gravity permeability test (Method 1).

$T_u = U\text{-}D + H = 50 - 15 + 5$ = 40 feet, also $T_u\ /\ \ell$ = $40/5 = 8$

$$X = \frac{H}{T_u}(100) = \frac{5}{40}(100) = 12.5\%$$

The values for X and $T_u\ /\ \ell$ lie in zone 1 (figure 17-6). To determine the unsaturated conductivity coefficient, C_u, from figure 17-7:

$$\frac{H}{r} = \frac{5}{0.5} = 10, \quad \frac{\ell}{H} = \frac{5}{5} = 1, \quad also\ C_u = 32$$

$$K = \frac{Q}{C_u r H} = \frac{0.10}{(32)(0.5)(5)} = 0.00125\ ft/s$$

$$K = 0.00125\ ft/sec\ X\ 3.15X10^7 = 39{,}400\ ft/yr$$

Zone 2, Method 1

Given:

H, ℓ, r, U, and *Q* are as given in example 1, *D* = 45 feet

$$T_u = 50 - 45 + 5 = 10\ ft \quad also \quad \frac{T_u}{\ell} = \frac{10}{5} = 2$$

$$X = \frac{5}{10}(100) = 50\%$$

Points $T_u\ /\ \ell$ and X lie in zone 2 on figure 17-6.

To determine the saturated conductivity coefficient, C_s, from figure 17-8:

$$\frac{\ell}{r} = \frac{5}{0.5} = 10 \quad also \quad C_s = 25.5$$

From figure 17-10:

$$K = \frac{2Q}{(C_s + 4)rT_u} = \frac{(2)(0.10)}{(25.5 + 4)(0.5)(10)} = 0.00136 \; ft/sec$$

Gravity Permeability Test - Method 2

This method may give erroneous results when used in unconsolidated material because of several uncontrollable factors. However, it is the best of the available pump-in tests for the conditions. The results obtained are adequate in most instances if the test is performed carefully. When permeabilities in streambeds or lakebeds must be determined below water, Method 2 is the only practical gravity test available.

A 5-foot (1.5-m) length of 3- to 6-inch- (75- to 150-mm-) diameter casing uniformly perforated with the maximum number of perforations possible without seriously affecting the strength of the casing is best. The bottom of the perforated section of casing should be beveled on the inside and case hardened for a cutting edge.

The casing is sunk by drilling or jetting and driving, whichever method will give the tightest fit of the casing in the hole. In poorly consolidated material and soils with a nonuniform grain size, development by filling the casing with water to about 3 feet (1 m) above the perforations and gently surging and bailing is advisable before making the test. A 6-inch (150-mm) coarse gravel cushion is poured into the casing, and the observation pipe is set on the cushion.

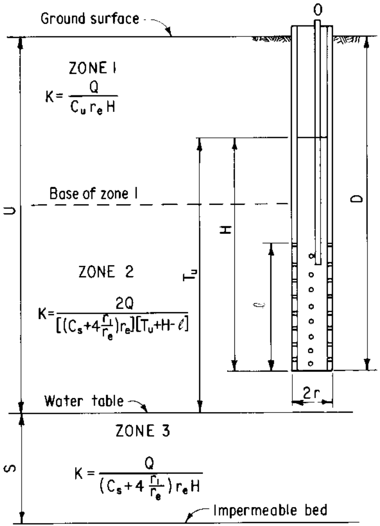
A uniform flow of water sufficient to maintain the water level in the casing above the top of the perforations is poured into the well. The water should be poured through a pipe and measurements made between the pipe and casing or reversed if necessary. Depth of water measurements are made at 5-minute intervals until three or more measurements are within ±0.2 foot (60 mm) of each other.

When a test is completed, the casing is sunk an additional 5 feet (1.5 m), and the test is repeated.

The test may be run in stable consolidated material using an open hole for the test section. Because the bottom of the casing is seldom tight in the hole and significant error may result from seepage upward along the annular space between the casing and the wall of the hole, this is not recommended. Measurements should be made to the nearest 0.01 foot (3 mm). The values of C_u and C_s within the limits ordinarily employed in the field can be obtained from figures 17-7 and 17-8. The zone in which the test is made and applicable equations can be found in figures 17-6 and 17-11, respectively.

Some data required for computing permeability are not available until the hole has encountered the water table.

The recorded data are supplemented by this information as the data are acquired. The data recorded in each test are:



K = coefficient of permeability, feet per second under a unit gradient
Q = steady flow into well, ft^3/s
H = height of water in well, ft
ℓ = length of perforated section, ft
r_1 = outside radius of casing (radius of hole in consolidated material), ft
r_e = effective radius of well = r_1(area of perforations)/(outside area of perforated section of casing) ; $r_1 = r_e$ in consolidated material that will stand open and is not cased
C_u and C_s = conductivity coefficients
T_u = distance from water level in casing to water table, ft
a = surface area of test section (area of perforations plus area of bottom), ft^2; where clay seal is used at bottom, a = area of perforations
S = thickness of saturated permeable material above an underlying relatively impermeable stratum, ft
$X = \frac{H}{T_u}(100)$ = percent of unsaturated stratum
U = thickness of unsaturated material above water table, ft
D = distance from ground surface to bottom of test section, ft
O = observation pipe (1 to $1\frac{1}{4}$-inch pipe)

Limitations:

$S \geq 5\ell$, $\ell \geq 10r$, and $\frac{Q}{a} \leq 0.10$

Notes:

In zone 3, H is the difference in elevation between the normal water table and the water level in the well. In zones 2 and 3, if a clay seal is placed at the bottom of the casing, the factor $4\frac{r_1}{r_e}$ is omitted from the equations. Where the test is run with "ℓ" as an open hole, $\frac{r_1}{r_e} = 1$ and $(C_s + 4\frac{r_1}{r_e}) = (C_s + 4)$.

Figure 17-11.—Gravity permeability test (Method 2).

- Outside radius of casing, r_1
- Length of perforated section of casing, ℓ
- Number and diameter of perforations in length ℓ
- Depth to bottom of hole, D
- Depth-to-water surface in hole
- Depth of water in hole, H
- Depth-to-water table, U
- Thickness of saturated permeable material above underlying relatively impermeable bed, S
- Steady flow into well to maintain a constant water level in hole, Q
- Time test is started, and measurement is made

Some examples using Method 2 are:

Zone 1, Method 2

Given:

H = 10 feet, ℓ = 5 feet, r_1 = 0.25 foot, D = 20 feet, U = 50 feet, Q = 0.10 ft^3/sec, 128 0.5-inch-diameter perforations, bottom of the hole is sealed

Area of perforations = $128\pi\, r_1{}^2 = 128\pi\,(0.25)^2 =$ $25.13\ in^2 = 0.174\ ft^2$

Area of perforated section = $2\pi r_\ell = 2\pi\,(0.25)\,(5) =$ $7.854\ ft^2$

$$r_e = \frac{0.174}{7.854}(0.25) = 0.00554\ ft$$

$$T_u = U - D + H = 50 - 20 + 10 = 40\ ft$$

$$also\ \frac{T_u}{\ell} = \frac{40}{5} = 8$$

$$X = \frac{H}{T_u}(100) = \frac{10}{40}(100) = 25\%$$

Points T_u / ℓ and X lie in zone 1 on figure 17-6.

Find C_u from figure 17-7:

$$\frac{H}{r_e} = \frac{10}{0.00554} = 1{,}805\ \ also\ \frac{\ell}{H} = \frac{5}{10} = 0.5$$

then, C_u = 1,200

From figure 17-11:

$$K = \frac{Q}{C_u r_e H} = \frac{0.10}{(1{,}200)(0.00554)(10)} = 0.0015\ ft/s$$

$$K = 0.0015\ ft/s\ X\ 3.15X10^7 = 47{,}300\ ft/yr$$

Zone 2, Method 2

Given:

Q, H, ℓ, r_1, r_e, U, ℓ/H, and H/r_e same as Zone 1, Method 2, D = 40 feet

$$T_u = 50 - 40 + 10 = 20\,ft \quad also \quad \frac{T_u}{\ell} = \frac{20}{5} = 4$$

$$X = \frac{10}{20}(100) = 50\%$$

Points T_u / ℓ and X lie in zone 2 on figure 17-6.

Find C_s from figure 17-8:

$$\frac{\ell}{r_e} = \frac{5}{0.00554} = 902 \quad also \quad C_s = 800$$

From figure 17-11:

$$K = \frac{2Q}{\left[\left(C_s + 4\frac{r_1}{r_e}\right)r_e\right](T_u + H - \ell)} =$$

$$\frac{0.20}{(5.43)(20 + 10 - 5)} = 0.0015\,ft/s$$

$$K = 0.0015\,ft/s \; X \; 3.15X10^7 = 47{,}300\,ft/yr$$

Zone 3, Method 2

Given:

Q, H, ℓ*,* r_1*,* r_e*,* ℓ/H*,* H/r_e*, U,* C_s, and ℓ/r_e are as given in Zone 2, Method 2, *S* = 60 feet

From figure 17-11:

$$K = \frac{Q}{\left(C_s + 4\frac{r_1}{r_e}\right) r_e H} = \frac{0.10}{(980.5)(0.00554)(10)} = 0.0018\ ft/s$$

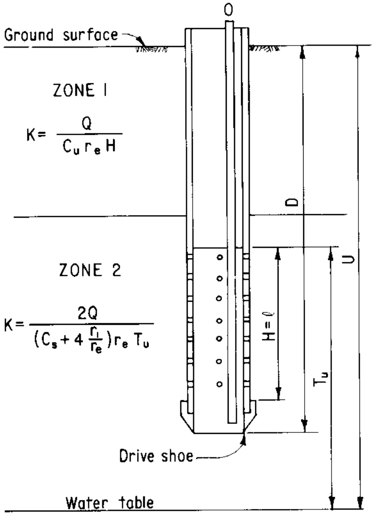
$$K = 0.0018\ ft/s\ X\ 3.15X10^7 = 56{,}700\ ft/yr$$

Gravity Permeability Test - Method 3

This method is a combination of gravity permeability test Methods 1 and 2. The method is the least accurate, but is the only one available for gravelly or coarse soils (figure 17-12).

In some materials, a casing that is beveled and case hardened at the bottom will not stand up to the driving. This is particularly true in gravelly materials where the particle size is greater than about 1 inch (25 mm). Under these conditions, Method 3 would probably not be satisfactory because a drive shoe must be used. Using a drive shoe causes excessive compaction of the materials and forms an annular space around the casing.

On completion of each test, a 3- to 6-inch-diameter (90- to 150-millimeter) perforated casing is advanced 5 feet (1.5 m) by drilling and driving. After each new test section is developed by surging and bailing, a 6-inch (150-mm) gravel cushion is placed on the bottom to support the observation pipe. A uniform flow of water sufficient to maintain the water level in the casing just at the top of the perforations is then poured into the well. The water is poured directly into the casing, and measurements are made through a 1¼-inch (32-mm) observation pipe. The test should be run until three or



K = coefficient of permeability, feet per second under a unit gradient
Q = steady flow into well, ft^3/s
r_1 = outside radius of casing
r_e = effective radius of casing = r_1(area of perforations)/(outside area of ℓ)
ℓ = length of perforated section, ft
C_u and C_s = conductivity coefficients
H = height of column of water in perforated section, ft
T_u = distance from water level in casing to water table, ft
$X = \frac{H}{T_u}(100)$ = percent of unsaturated stratum
O = observation pipe ($1\frac{1}{4}$-inch o.d. pipe is satisfactory)
U = tickness of unsaturated material above water table, ft
D = distance from ground surface to bottom of test section, ft
a = surface area of test section (area of perforations plus area of bottom), ft^2; where clay seal is used at bottom, a = area of perforations

Limitations:

$\frac{Q}{a} \leq 0.10$, $\ell \geq 10r$

Note:

In zone 2, if clay seal is placed at bottom of casing, the factor $4\frac{r_1}{r_e}$ is omitted from equation.

Figure 17-12.—Gravity permeability test (Method 3).

more measurements taken at 5-minute intervals are within ±0.2 foot (60 mm) of the top of the perforations.

The values of C_u and C_s, within the limits ordinarily employed in the field, can be obtained from figures 17-7 and 17-8. The zone in which the test is made and applicable equations can be found on figures 17-6 and 17-12, respectively.

The data recorded in each test are:

- Outside radius of casing, r_1, in feet (meters)

- Length of perforated section of casing, ℓ, in feet (meters)

- Number and diameter of perforations in length ℓ

- Depth to bottom of hole, D, in feet (meters)

- Depth-to-water surface in hole, in feet (meters)

- Depth of water in hole, in feet (meters)

- Depth-to-water table, in feet (meters)

- Steady flow into well to maintain a constant water level in hole, Q, in ft^3/sec (m^3/sec)

- Time test is started and time each measurement is made

Zone 1, Method 3

Given: Q = 10.1 gal/min = 0.023 ft^3/sec, $H = \ell = 5$ feet, D = 17 feet, U = 71 feet, T_u = 54.5 feet, r_e = 0.008 foot, and r_1 = 1.75 inches = 0.146 foot (nominal 3-inch casing)

$$\frac{T_u}{\ell} = \frac{54.4}{5} = 10.9$$

$$Also\ X = \frac{H}{T_u}(100) = \frac{5}{54.5}(100) = 9.2\%$$

These points lie in zone 1 (figure 17-6).

Find C_u from figure 17-7:

$$\frac{H}{r_e} = \frac{5}{0.008} = 625 \ also\ \frac{\ell}{H} = 1$$

then, $C_u = 640$

From figure 17-12:

$$K = \frac{Q}{C_u r_e H} = \frac{0.023}{(640)(0.008)(5)} = 0.0009\ ft/s$$

$$K = 0.0009\ ft/s\ X\ 3.15X10^7 = 28{,}400\ ft/yr$$

Zone 2, Method 3

Given: Q, H, ℓ, U, r_e, and r_1 are as given in Zone1, Method 3, D = 66 feet and T_u = 10 feet

$$\frac{T_u}{\ell} = \frac{10}{5} = 2 \quad also \quad X = \frac{5}{10}(100) = 50\%$$

These points lie in zone 2 (figure 17-6).

Find C_s from figure 17-8:

$$\frac{\ell}{r_e} = \frac{5}{0.008} = 625 \quad also \quad C_s = 595$$

From figure 17-12:

$$K = \frac{2Q}{\left(C_s + 4\frac{r_1}{r_e}\right) r_e T_u} = \frac{(2)(0.023)}{(668)(0.008)(10)} = 0.00086\ ft/s$$

$$K = 0.00086\ ft/s \ X\ 3.15X10^7 = 27{,}100\ ft/yr$$

Gravity Permeability Test - Method 4

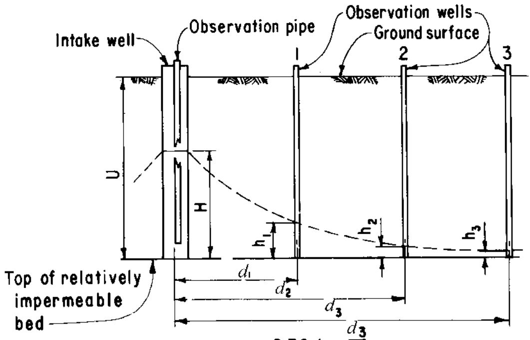
This method can be used to determine the overall average permeability of unsaturated materials above a widespread impermeable layer. The method does not detect permeability variations with depth. The method is actually an application of steady-state pumping test theory.

A well, preferably 6 inches (150 mm) or larger in diameter, is drilled to a relatively impermeable layer of wide areal extent or to the water table. If the saturated thickness is small compared to the height of the water column that can be maintained in the hole, a water table will act as an impermeable layer for this test. The well is uncased in consolidated material, but a perforated casing or screen should be set from the bottom to about 5 feet (1.5 m) below the ground surface in unconsolidated material. The well should be developed by pouring water into the hole while surging and bailing before testing.

Before observation wells are drilled, a test run of the intake well should be made to determine the maximum height, *H*, of the column of water that can be maintained (figure 17-13) above the top of the impermeable layer. A 1- to 1¼-inch (25- to 32-mm) observation pipe should be inserted to near the bottom of the intake well to for water-level measurements. The spacing of the observation wells can be determined from this test run.

A minimum of three observation wells should be installed to the top of the impermeable layer or water table. Suitable pipe, perforated for the bottom 10 to 15 feet (3 to 4.5 m), should be set to the bottom of these wells. The observation wells should be offset from the intake well by distances equal to multiples of one-half the height, *H*, of the water column that will be maintained in the intake well.

The elevations of the top of the impermeable layer or water table in each well are determined, and the test is started. After water has been poured into the intake at a constant rate for an hour, measurements are made of water levels in the observation wells. Measurements are then made at 15-minute intervals, and each set of measurements is plotted on semi-log paper. The square



$$K = \frac{2.3Q \log \frac{d_3}{d_2}}{\pi(h_2^2 - h_3^2)}$$

K = coefficient of permeability, feet per second under a unit gradient

Q = uniform flow into intake well, ft^3/s

d_1, d_2, and d_3 = distance from intake well to observation holes, ft

h_1, h_2, and h_3 = height of water in observation holes d_1, d_2, and d_3 respectively, above elevation of top of impermeable layer, ft

H = height of column of water in intake pipe above top of impermeable stratum, ft

U = distance from ground surface to impermeable bed, ft

Figure 17-13.—Gravity permeability test (Method 4).

of the height of the water level above the top of the impermeable bed, h^2, is plotted against the distance from the intake well to the observation holes, d, for each hole (figure 17-14). When the plot of a set of measurements is a straight line drawn through the points within the limits of plotting, conditions are stable and the permeability may be computed.

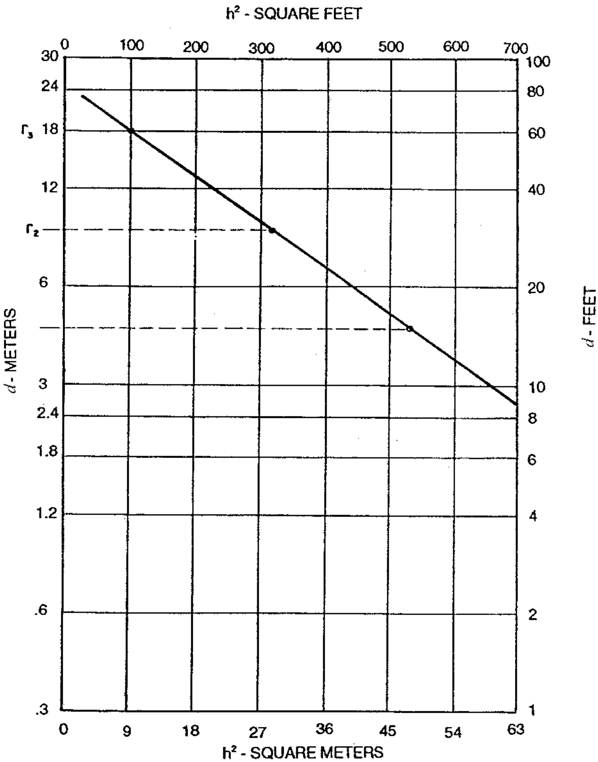


Figure 17-14.—Plot of h^2 versus d for gravity permeability test (Method 4).

The data recorded in each test are:

- Ground elevations at the intake well and the observation wells
- Elevations of reference points at the intake well and the observation wells, in feet (meters)

- Distances from center of the observation wells to center of the intake well, d_1, d_2, and d_3, in feet (meters)
- Elevation of top of impermeable bed at the intake well and the observation wells, in feet (meters)
- Depths of water below reference point in the intake well and the observation wells at 15-minute intervals, in feet (meters)
- Uniform flow of water, Q, introduced into well, in ft^3/sec (m^3/sec)
- Time pumping is started and time each measurement is made

Method 4

Given: U = 50 feet, Q = 1 ft^3/sec, H = 30 feet, d_1 = 15 feet, d_2 = 30 feet, d_3 = 60 feet, h_1 = 23.24 feet, h_2 = 17.89 feet, h_3 = 10.0 feet, h_1^2 = 540 ft^2, h_2^2 = 320 ft^2, and h_3^2 = 100 ft^2

A plot of d against h^2, as shown on figure 17-14, shows that a straight line can be drawn through the plotted points, meaning that stable conditions exist and the permeability may be computed.

From figure 17-13:

$$\log \frac{d_3}{d_2} = 0.3010,\ \log \frac{d_3}{d_1} = 0.6021,\ \textit{also}\ \log \frac{d_2}{d_1} = 0.3010$$

$$K = \frac{2.3\ Q \log \dfrac{d_3}{d_2}}{\pi(h_2^2 - h_3^2)} = \frac{2.3\ Q \log \dfrac{d_3}{d_1}}{\pi(h_1^2 - h_3^2)} = \frac{2.3\ Q \log \dfrac{d_2}{d_1}}{\pi(h_1^2 - h_2^2)}$$

$$K = \frac{(2.3)(1)(0.3010)}{\pi(220)} =$$

$$\frac{(2.3)(1)(0.6020)}{\pi(440)} = \frac{(2.3)(1)(0.3010)}{\pi(220)} = 0.001\ ft/s$$

$$K = 0.001\ ft/s\ X\ 3.15X10^7 = 315{,}000\ ft/yr$$

Falling Head Tests

Falling head tests are used primarily in open holes in consolidated rock. Falling head tests use inflatable packers identical to those used for pressure testing and can be used as an alternate method if the pressure transducer or other instrumentation fails. The method of cleaning the hole is the same as that described under pressure testing.

Tests Below the Static Water Level

Tests below the static water level should be done as follows:

- Use inflatable straddle packers at 10-foot (3-m) spacing on a 1¼-inch (32-mm) drop pipe (inside diameter = 1.38 inches [35 mm]). Set packers initially at the bottom of the hole and inflate to 300 lb/in^2 (2,000 kPa) of differential pressure.

- After packers are inflated, measure the water level in the drop pipe three or more times at 5-minute intervals until the water level stabilizes. The stabilized level will be the static water level in the test section.

- Pour 2 gallons (8 liters [L]) or more of water as rapidly as possible into the drop pipe after the water level stabilizes. One gallon (4 l) of water will raise the water level in a 1¼-inch (32-mm) pipe 13 feet (4 m) if the section is tight.

- Measure the water level as soon as possible after the water is poured in. Measure the initial depth to water, record the time as soon as possible, and repeat twice at 5-minute intervals. If the rate of decline exceeds 15 feet (4.5 m) in 13 minutes, the transmissivity of a 10-foot (3-m) test section is greater than 200 ft^2 (18 m^2) per year, and the average permeability is greater than 20 feet (6 m) per year.

The transmissivity value determined is only an approximation, but the value is sufficiently accurate for many engineering purposes.

The equation for analysis is:

$$T = \frac{V}{2\pi s \Delta t}$$

where:

T = transmissivity of test section, in ft^2/sec (m^2/sec)

V = volume of water entering test section in period Δt, in ft^3 (m^3). (A 1-foot [30-cm] decline in 1¼-inch [32-mm] pipe = 0.01 ft^3 [0.000283 m^3])

s = decline in water level in period Δt, in feet (meters)

Δt = period of time, in seconds, between successive water level measurements (i.e., $t_1 - t_o$, $t_2 - t_1$, etc.)

- If the log indicates the test section is uniform and without obvious points of concentrated leakage, the average permeability of the test section, in feet per second (m/sec), can be estimated from $K = T / \ell$, where ℓ is the length of the test section, in feet (m). If the log indicates a predominantly impervious test section, but includes a zone or zones of concentrated flow, the average K of the zones can be estimated from $K = T / \ell'$, where ℓ' is the thickness of the permeable zone or zones, in feet (m).

- After each test, deflate the packers, raise the test string 10 feet (3 m), and repeat the test until the entire hole below the static water level has been tested.

Tests Above the Water Table

Tests above the water table require different procedures and analyses than tests in the saturated zone. Tests made in sections straddling the water table or slightly above the water table will give high computed values if the equations for tests below the static water level are used and low computed values if the following equations are used. For tests above the water table, the following procedure is used:

- Install a 10-foot- (3-m-) spaced straddle packer at the bottom of the hole, if the hole is dry, or with the top of the bottom packer at the water table if the hole contains water. Inflate the packer.

- Fill the drop pipe with water to the surface, if possible, otherwise, to the level permitted by pump capacity.

- Measure the water level in the drop pipe and record with time of measurement. Make two or more similar measurements while the water-table declines.

- Upon completion of a test, raise the packer 10 feet (3 m) and repeat this procedure until all of the open or screened hole is tested.

The equation for analysis is:

$$K = \frac{r_1^2}{2\ell\Delta t}\left[\frac{\sinh^{-1}\frac{\ell}{r_e}}{2}\ln\left(\frac{2H_1 - \ell}{2H_2 - \ell}\right) - \ln\left(\frac{2H_1H_2 - \ell H_2}{2H_1H_2 - \ell H_1}\right)\right]$$

where:

r_1 = inside radius of drop pipe, in feet (mm) (0.0575 foot [17.25 mm] for 1¼-inch [32-mm] pipe)

r_e = effective radius of test section, in feet (mm) 0.125 foot [37.5 mm] for a 3-inch [75-mm] hole)

Δt = time intervals (t_1-t_0, t_2-t_1), in seconds

$sinh^{-1}$ = inverse hyperbolic sine

ln = natural logarithm

H = length of water column from bottom of test interval to water surface in standpipe, in feet (m) (H_0, H_1, H_2 lengths at time of measurements t_0, t_1, t_2, etc.)

- For the particular equipment specified and a 10-foot (3-m) test section, the equation may be simplified to:

$$K = \frac{1.653 \times 10^{-4}}{\Delta t}\left[2.5\ln\left(\frac{H_1 - 5}{H_2 - 5}\right) - \ln\left(\frac{H_1H_2 - 5H_2}{H_1H_2 - 5H_1}\right)\right]$$

Slug Tests

Slug tests are performed by "rapidly" changing water levels in a borehole. The rapid change is induced by adding or removing small quantities of water, air, or an object that displaces the groundwater. The time required to restore the water level to its original level is used to calculate the permeability. Slug tests are typically performed in areas where access or budget is limited or the extraction or addition of water has a potential impact to the surrounding area. Slug tests are appropriate where the aquifer will not yield enough water to conduct an aquifer test or the introduction of water could change or impact the water quality. A major factor in conducting slug tests is ensuring that the water level during the test reflects the aquifer characteristics and is not unduly affected by the well construction. Where the water surface is shallow and clean water is readily available, water injection or bailing is often the easiest method. The displacement or air injection method may be desirable at locations where the water level is deep and rapid injection or removal of water is difficult.

A number of slug test methods exist. The choice depends on the hydrologic and geologic conditions and the well size and construction.

Selecting the Slug Test

The method of inducing a rise or fall in the water surface depends on the purpose of the test and conditions at the site. Where the water level is shallow and clean water is readily available, water injection or bailing is often the easiest method. One limitation of the test accuracy is the initial water flowing down the inside wall of the well. Causing a rapid rise by displacing water in the well by dropping a pipe or weight or injecting air may be preferable at remote sites where water may not be readily available. The displacement or air injection method may also be desirable at locations where the water level is deep and rapid injection or removal of water may require collection and treatment or at well sites that are being used for water chemistry studies.

Conducting the Slug Test

Before introducing the slug, the well's sidewall, screen, or filter pack needs to be clear of any obstructions that will impede the movement of water from the riser to the surrounding materials. The riser diameter and length need to be measured as accurately as possible, and the water level must be stabilized and recorded as accurately as possible. The water level measurements must begin immediately following the introduction of the slug. Where the water level changes slowly, measurements may be made by using a water level indicator. For most tests, water levels should be recorded by a pressure transducer and automatic data logger.

Hvorslev Slug Test

The Hvorslev slug test is the simplest method of analysis. This analysis assumes a homogeneous, isotropic, infinite medium in which both the soil and water are incompressible. It neglects well bore storage and is not accurate where a gravel pack or thick sand pack is present. Figure 17-15 is a sketch of the geometry of this test and the method of analysis. This method is used where the slug test is conducted below an existing water surface.

The test analysis requires graphing the change in head versus time. Head changes are given by $(H\text{-}h)\,/\,H\text{-}H_0$ and

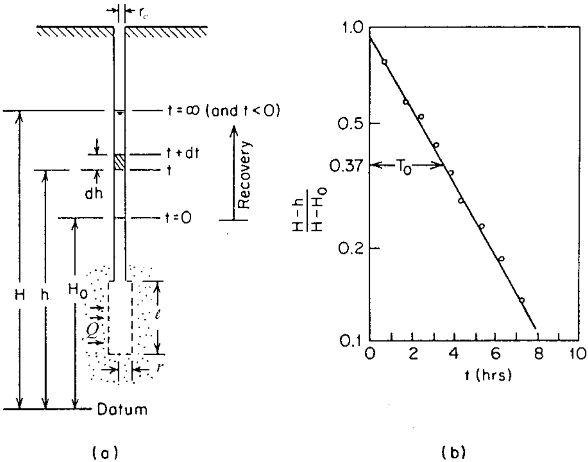


Figure 17-15.—Hvorslev piezometer test.

should be plotted semi-log against time. The graph should approximate a straight line; and at the point 0.37, the T_0 value is defined as the basic time lag. Using the graphical solution for T_0, the dimension of the cavity, and the appropriate shape factor, F, a solution for permeability can be found by:

$$K = \frac{\pi r_c^2}{F T_0}$$

Figure 17-16 provides shape equations for various well geometries.

Bouwer Slug Test

The Bouwer slug test assumes no aquifer storage and finite well bore storage. The well can be partially penetrating and partially screened. The method was

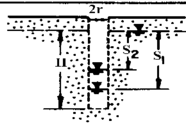
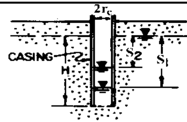
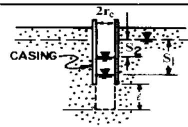
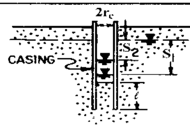
	CONDITION	DIAGRAM	SHAPE FACTOR, F	PERMEABILITY, K BY VARIABLE HEAD TEST	APPLICABILITY
OBSERVATION WELL OR PIEZOMETER IN SATURATED ISOTROPIC STRATUM OF INFINITE DEPTH	(A) UNCASED HOLE		$F = 16\pi H F_s r$	(FOR OBSERVATION WELL OF CONSTANT CROSS SECTION) $K = \frac{r}{16 H F_s} \times \frac{(S_2 - S_1)}{(t_2 - t_1)}$ FOR $\frac{H}{r} < 50$	SIMPLEST METHOD FOR PERMEABILITY DETERMINATION. NOT APPLICABLE IN STRATIFIED SOILS. FOR VALUES OF F_s, SEE FIGURE 17-16b.
	(B) CASED HOLE, SOIL FLUSH WITH BOTTOM.		$F = \frac{11 r_c}{2}$	$K = \frac{2\pi r_c}{11(t_2 - t_1)} \ln\left(\frac{S_1}{S_2}\right)$ FOR $6'' \leq H \leq 60''$	USED FOR PERMEABILITY DETERMINATION AT SHALLOW DEPTHS BELOW THE WATER TABLE. MAY YIELD UNRELIABLE RESULTS IN FALLING HEAD TEST WITH SILTING OF BOTTOM OF HOLE.
	(C) CASED HOLE, UNCASED OR PERFORATED EXTENSION OF LENGTH "ℓ".		$F = \frac{2\pi \ell}{\ln\left(\frac{\ell}{r_c}\right)}$	$K = \frac{r_c^2}{2L(t_2 - t_1)} \ln\left(\frac{\ell}{r_c}\right) \ln\left(\frac{S_1}{S_2}\right)$ FOR $\frac{\ell}{r_c} > 8$	USED FOR PERMEABILITY DETERMINATIONS AT GREATER DEPTHS BELOW WATER TABLE.
	(D) CASED HOLE, COLUMN OF SOIL INSIDE CASING TO HEIGHT "ℓ"		$F = \frac{11\pi r_c^2}{2\pi r_c + 11\ell}$	$K = \frac{2\pi r_c + 11\ell}{11(t_2 - t_1)} \ln\left(\frac{S_1}{S_2}\right)$	PRINCIPAL USE IS FOR PERMEABILITY IN VERTICAL DIRECTION IN ANISOTROPIC SOILS.

Figure 17-16a.—Shape factors for computing permeability from variable head tests.

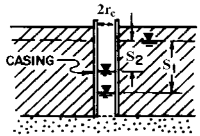
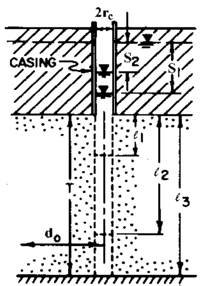
	CONDITION	DIAGRAM	SHAPE FACTOR, F	PERMEABILITY, K BY VARIABLE HEAD TEST	APPLICABILITY
OBSERVATION WELL OR PIEZOMETER IN AQUIFER WITH IMPERVIOUS UPPER LAYER				(FOR OBSERVATION WELL OF CONSTANT CROSS SECTION)	
	(E) CASED HOLE, OPENING FLUSH WITH UPPER BOUNDARY OF AQUIFER OF INFINITE DEPTH		$F = 4r_c$	$K = \frac{\pi r_c}{4(t_2-t_1)} \ln\left(\frac{S_1}{S_2}\right)$	USED FOR PERMEABILITY DETERMINATION WHEN SURFACE IMPERVIOUS LAYER IS RELATIVELY THIN. MAY YIELD UNRELIABLE RESULTS IN FALLING HEAD TEST WITH SILTING OF BOTTOM OF HOLE.
	(F) CASED HOLE, UNCASED OR PERFORATED EXTENSION INTO AQUIFER OF FINITE THICKNESS: (1) $\frac{\ell_1}{T} \leq 0.2$ (2) $0.2 < \frac{\ell_2}{T} < 0.85$ (3) $\frac{\ell_3}{T} = 1.00$ NOTE: d_o EQUALS EFFECTIVE RADIUS TO SOURCE AT CONSTANT HEAD.		(1) $F = C_s r_c$	$K = \frac{\pi r_c}{C_s(t_2-t_1)} \ln\left(\frac{S_1}{S_2}\right)$	USED FOR PERMEABILITY DETERMINATIONS AT DEPTHS GREATER THAN ABOUT 5FT. FOR VALUES OF C_s, SEE FIGURE 17-16b.
			(2) $F = \frac{2\pi \ell_2}{\ln(\ell_2/r_c)}$	$K = \frac{r_c^2 \ln\left(\frac{\ell_2}{r_c}\right)}{2\ell_2(t_2-t_1)} \ln\left(\frac{S_1}{S_2}\right)$ FOR $\frac{\ell}{r_c} > 8$	USED FOR PERMEABILITY DETERMINATIONS AT GREATER DEPTHS AND FOR FINE GRAINED SOILS USING POROUS INTAKE POINT OF PIEZOMETER.
			(3) $F = \frac{2\pi \ell_3}{\ln\left(\frac{d_o}{r_c}\right)}$	$K = \frac{R^2 \ln\left(\frac{d_o}{r_c}\right)}{2\ell_3(t_2-t_1)} \ln\left(\frac{S_1}{S_2}\right)$	ASSUME VALUE OF $\frac{d_o}{r_c}$ = 200 FOR ESTIMATES UNLESS OBSERVATION WELLS ARE MADE TO DETERMINE ACTUAL VALUE OF d_o.

Figure 17-16a (cont.).—Shape factors for computing permeability from variable head

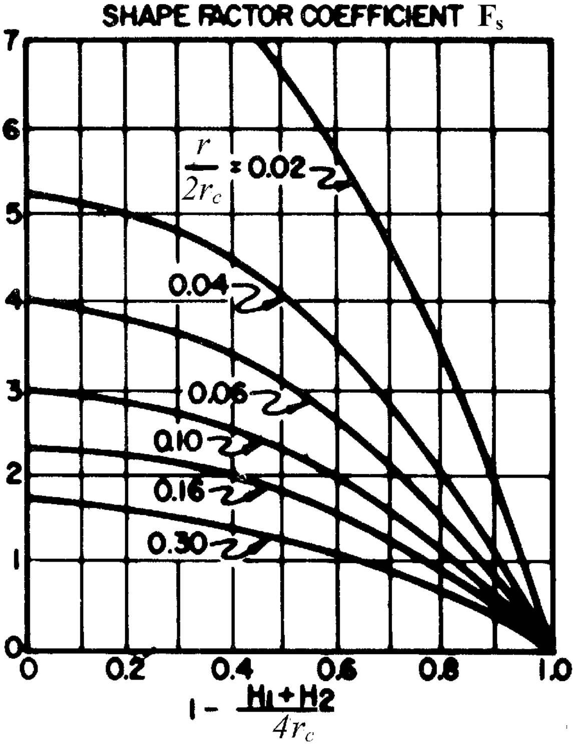


Figure 17-16b.—Shape factor coefficient F_s.

originally developed for unconfined aquifers but can also be used for confined or stratified aquifers if the top of the screen or perforated section is located some distance below the upper confining layer. The analysis is based on the Thies equation and determines permeability, *K*, of the aquifer around the well from the equation:
where:

$$K = \frac{r_c^2 \ln\left(\frac{d_e}{r}\right)}{2\ell} \frac{1}{t} \ln\left(\frac{s_0}{s_t}\right)$$

ℓ = length of the perforated section of the casing or riser

s_0 = the vertical difference between the water surface inside the well and the static water level at time zero

s_t = s at time t

t = time of reading

r_c = inside radius of the riser or casing

r = the effective radius of the well, including the perforated casing, sand or gravel pack, and any remaining annular space to the sidewall of the borehole

d_e = effective radial distance; the distance between the well and the observation well; the distance over which the water level, s, returns to the static level

H = the height of water within the well

S = aquifer thickness

See figure 17-17 for the configuration definition.

The values of d_e were determined using an electrical resistance analog network. The effective radial distance is influenced by the well diameter, well screen length, well depth, and the aquifer thickness. Various values for r_c, ℓ, H, and S were used in the analog network for analysis of their impacts on d_e.

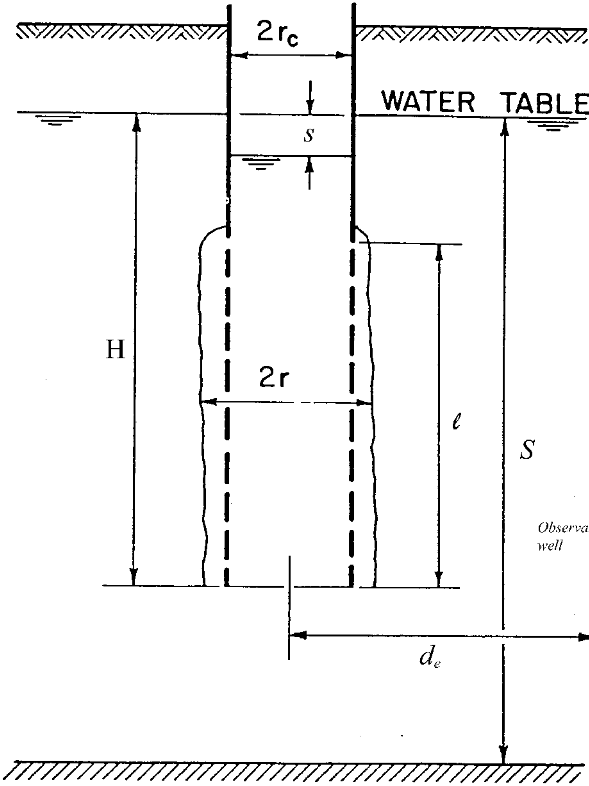


Figure 17-17.—Slug test on partially penetrating, screened well in unconfined aquifer with gravel pack and developed zone around screen.

The term, $\ln(d_e/r)$, is related to the geometry of the test zone and the amount of aquifer penetration of the well. Two separate solutions are required to address partially penetrating wells and fully penetrating wells. For partially penetrating wells, an empirical equation relating *ln* (d_e/r) to the geometry of the test zone is:

$$\ln\frac{d_e}{r} = \left[\frac{1.1}{\ln\left(\frac{H}{r}\right)} + \frac{A + B\ln\left(\frac{S-H}{r}\right)}{\frac{\ell}{r}}\right]^{-1}$$

In this equation, A and B are dimensionless coefficients that can be read on figure 17-18. An effective upper limit of $ln[(S\text{-}H)/r]$ is 6. If the computed value of $ln[(S\text{-}H)/r]$ is greater than 6, then 6 should be used in the equation for $ln(d_e/r)$. When $S = H$, the well is fully penetrating, and the value of C should be used from figure 17-18 in the equation.

$$\ln\frac{d_e}{r_e} = \left[\frac{1.1}{\ln\left(\frac{H}{r_e}\right)} + \frac{C}{\frac{\ell}{r_e}}\right]^{-1}$$

Values of the field test data should be plotted as recovery, s, versus time for each data point reading. The value s should be plotted on a y-axis log scale, and values for corresponding time should be plotted on the x-axis. The points should approximate a straight line, which indicates good test data. Areas of the data that plot as curves (usually at the beginning of the test or near the end of the test) should not be used in the computation.

Piezometer Test

The piezometer test measures the horizontal permeability of individual soil layers below a water

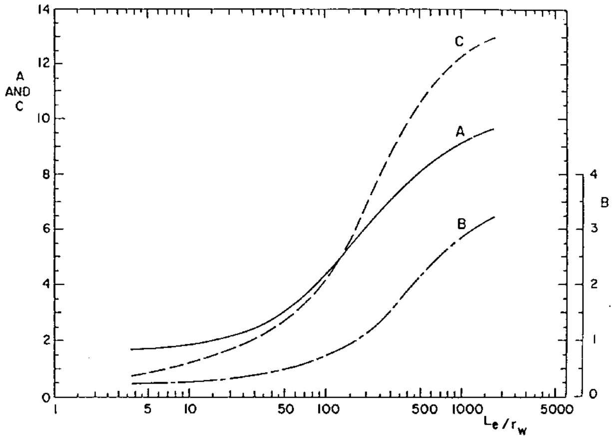


Figure 17-18.—Dimensionless parameters *A, B*, and *C* as a function of ℓ/r_e (F for calculation of $ln(d_e/r_e)$).

surface. This test may apply to large diameter direct push technology, as well as to any depth that an open hole can be maintained. This test is preferred over the auger-hole test described in section 10-6 of the *Ground Water Manual*, especially when the soils tested are less than 1.5 feet (0.5 m) thick and are below the water table. This method is particularly good for determining which layer below the capillary zone is an effective barrier.

Equipment

The following items are suggested equipment for the piezometer test.

- Riser between 1- to 2-inch (24- to 50-mm) inside diameter for a depth of around 15 feet (4.5 m) and black iron pipe with smooth inside wall for depths greater than 15 feet (4.5 m)

- Equipment capable of installing the riser and cavity below the riser

- Pump with hose and controller or bailer that will fit inside the casing

- Bottle brush for cleaning soil film from the inside of the test riser if the riser is driven without a protective point that can prevent soils from filling the inside of the tube

- Water level detector, stop watch, and transducer and data logger

Procedure

The test layer should be at least 1 foot (30 cm) thick so that a 4-inch (10-cm) length of cased hole or cavity can be located in the middle of the layer. This placement is especially important if a marked difference in the layers exists above and below the test layer. The differences may be changes in the percentages of fines (>15 percent), overall changes in the soil gradations, soil structure, or degree of cementation or induration within the soil horizon. After selecting an appropriate interval based on the soil investigation, an adjacent hole within 2 feet (0.6 m) of the test hole is advanced to around 2.0 feet (0.6 m) above the bottom of the 4-inch (10-cm) test interval, if using direct push technology, or to the top of the 4-inch (10-cm) interval. Then use a smaller diameter auger to ream the final 4-inch (10-cm) interval for the riser pipe. The final 2.0 feet (0.6 m) are driven to ensure that a reasonably good seal is obtained and also to minimize the disturbance. The lower 4 inches (10 cm) of the borehole are exposed, and the cavity must remain open. After some recovery has occurred, the riser should be cleaned with a brush to remove any soil film unless the riser was advanced by direct push. The end of the riser should be screened within the lower cavity and then cleaned out by gently pumping or bailing water and sediment from the piezometer.

After the water surface has stabilized, the transducer is installed. The riser is bailed or pumped, depending on the diameter of riser and the depth to water. It is not necessary to remove all the water in the screened interval, but the water level should be lowered enough that at least three readings during the first half of the water rise will give consistent results.

Calculations

After completing the piezometer test, the permeability is calculated from:

$$K = \frac{3600 \pi r^2 \ln\left(\frac{s_1}{s_2}\right)}{C_a(t_2 - t_1)}$$

where:

s_1 and s_2	=	distance from static water level at times t_1 and t_2, in inches (cm)
$t_2 - t_1$	=	time for water level change from s_1 to s_2, in seconds
C_a	=	a constant for a given flow geometry, in inches (cm)
ℓ	=	length of open cavity, in inches (cm)
d	=	H-ℓ, distance from the static water level to the top of the cavity, in inches (cm)
b	=	distance below the bottom of the cavity to the top of next layer, in inches (cm)

A sample calculation using this equation is shown in figure 17-19. The constant, C_a, may be taken from curves shown in figures 17-19 or 17-20. The curve on figure 17-19 is valid when *d* and *b* are both large compared to ℓ. When $b = 0$ and d is much greater than ℓ, the curve will give a C_a factor for $\ell = 4$ and d = 1 that will be about 25 percent too large.

The chart on figure 17-20 is used for determining C_a when upward pressure exists in the test zone. When pressures are present, additional piezometers must be installed. The tip of the second piezometer should be placed just below the contact between layers in layered soil (figure 17-21). In deep, uniform soils, the second piezometer tip should be placed an arbitrary distance below the test cavity.

After installing the second piezometer, the following measurements should be made:

- Distance, $\triangle$d, in feet (m), between the ends of the riser pipes

- Difference, $\triangle$H, in feet (m), between water surfaces in the two piezometers at static conditions

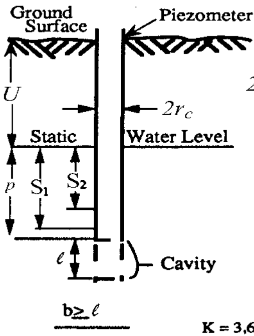
- Distance, d', in feet (m), between the center of the lower piezometer cavity and the contact between soil layers in the layered soils

The C_a value from figure 17-20 is used in the equation to determine the permeability.

Limitations

Installation and sealing difficulties encountered in coarse sand and gravel are the principal limitations of the

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



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

$2r_c$ = 2.54 centimeters (1.00 inch)
Inside diameter of piezometer and cavity

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

ℓ = 10.16 centimeters (4 inches)
Length of cavity

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

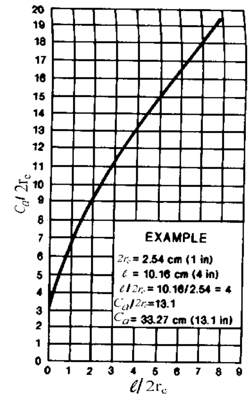
K = Hydraulic conductivity, centimeters per hour (inches per hour)

b = depth to texture change

S_1, S_2 = Distance from static water level at time t_1 and t_2 in centimeters (inches).

$(t_2 - t_1)$ = Time for water to change from S_1 to S_2 (seconds)

$$K = \frac{3{,}600\,\pi\,(r_c)^2 \log_e (S_1/S_2)}{C_a(t_2 - t_1)}$$, centimeters per hour (inches per hour)



C_a as a function of $2r_c$ and ℓ

Redrawn from LUTHIN & KIRKHAM (1949).
Revised by USBR (Mantei, 1972)

Time (seconds) Initial (t_1)	Time (seconds) Final (t_2)	Y, centimeters (inches) Initial (S_1)	Y, centimeters (inches) Final (S_2)	C_a, cent. (inches)	t_2-t_1	S_1/S_2	$\text{Log}_e\ S_1/S_2$	$3{,}600\pi(r_c)^2$ cm^2sec/hr (in^2sec/hr)	K cm/hr (in./hr)
0	30	218.44 (86.00)	197.87 (77.90)	33.27 (13.1)	30	1.104	0.099	18241.47 (2827.44)	1.80 (0.71)
30	60	197.87 (77.90)	178.44 (70.25)	33.27 (13.1)	30	1.109	0.103	18247.47 (2827.44)	1.88 (0.74)
60	90	178.44 (70.25)	160.02 (63.00)	33.27 (13.1)	30	1.115	0.109	18241.47 (2827.44)	1.99 (0.78)
90	120	160.02 (63.00)	145.47 (57.27)	33.27 (13.1)	30	1.100	0.095	18241.47 (2827.44)	1.74 (0.68)
120	150	145.47 (57.27)	131.17 (51.64)	33.27 (13.1)	30	1.109	0.103	18241.47 (2827.44)	1.88 (0.74)

Average for 5 readings = 1.86 (0.73)

Figure 17-19.—Data and computation sheet for piezometer permeability test.

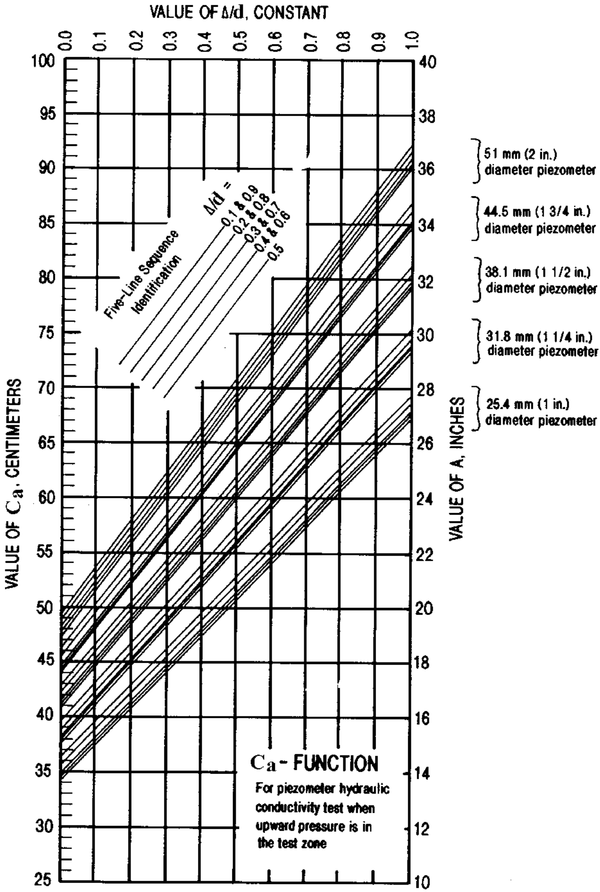


Figure 17-20.—Chart for determining C_a if upward pressure exists in the test zone.

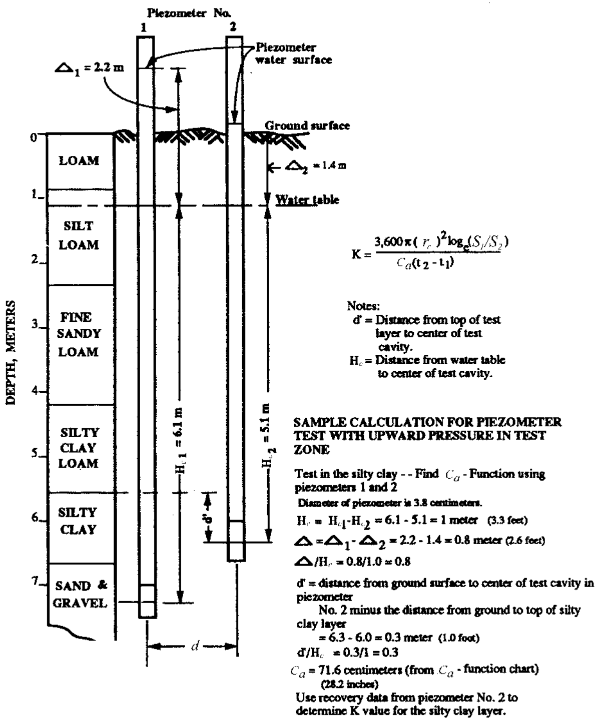


Figure 17-21.— Sample calculation for the piezometer test with upward pressure in the test zone.

piezometer test for permeability. Also, when the riser bottoms in gravel, a satisfactory cavity cannot be obtained. The practical limit of hole depth is about 20 feet (6 meters). Deeper holes require larger diameters (greater than 2 inches [5cm]), and driving the riser deeper is difficult.

Details on the equipment requirements and procedures are provided in chapter 3 of the *Drainage Manual*. Permeameter tests are typically restricted to shallow boreholes or excavations and require time to set up the equipment and perform the test. Where the permeability is relatively high (>1,000 feet per year [10^{-3} cm per second]), these tests require large amounts of water. With material having permeability values greater than 10^6 feet per year (10 cm/sec), laboratory permeability testing is more cost effective. The samples are collected and tested in the laboratory in accordance with Reclamation Procedure 5605.

Bibliography

Bureau of Reclamation, U.S. Department of the Interior, *Drainage Manual*, 1993.

Bureau of Reclamation, U.S. Department of the Interior, *Ground Water Manual*, 1995.

Chapter 18

RIPRAP

Introduction

Riprap is preferably a relatively thin layer of large, approximately equidimensional, durable rock fragments or blocks placed on bedding to dissipate water energy and protect a slope, channel bank or shore from erosion caused by the action of runoff, currents, waves or ice (figure 18-1). Bedding is usually a layer of sand and gravel placed under the riprap to prevent erosion of the material from under the riprap. Most dam embankments contain at least one zone that uses rock. Rock is used as riprap for protection against erosion or as rockfill and filter zones that strengthen or drain the embankment.

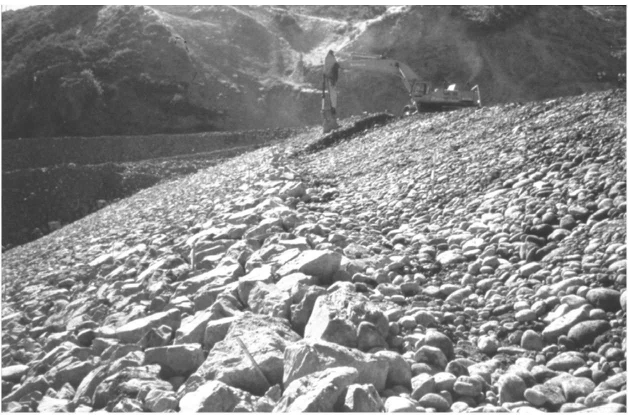


Figure 18-1.—Riprap properly placed on bedding. The riprap is angular, quarried rock, and the bedding is rounded stream gravel. The backhoe is placing and arranging the rock on the bedding.

The terms “slope protection” and “riprap” are often used interchangeably, but not all slope protection is riprap. Soil cement is also commonly used as slope protection. Riprap is an assemblage of rocks “nested” together to protect a structure or area from the action of water. The stability of an assemblage of rocks is a function of the individual rock’s size, shape, weight, and durability. An assemblage of rocks depends on the individual rock characteristics for stability and also on the site conditions, grading, and thickness. The assemblage of rocks is designed to minimize voids and thickness of the riprap layer to keep the volume of material as low as possible. Proper placement interlocks the individual fragments into a layer of rocks that resists the action of water. Figure 18-2 shows what can happen if riprap is not designed, obtained, and placed properly.

Riprap should be “hand” placed to reduce the void space and maximize the interlocking arrangement, but rarely is this economical (figure 18-3). Most riprap is dumped and falls into place by gravity with little or no additional adjustment (figure 18-4). Because of this, individual pieces of riprap must have appropriate characteristics so that the rocks can be processed, handled, and placed so that the layer remains intact for the life of the project.

This chapter discusses: (1) riprap source evaluation, (2) onsite inspection to ensure that the samples are appropriate and that specified material is being produced from the source, (3) presentation of information to designers and estimators, and (4) waste factors in riprap production. A geologic background, a knowledge of blasting methods and types of explosives, and an understanding of the equipment involved in the processing, hauling, and placing of riprap is important to riprap evaluation, production, and placement. Most of the following discussion applies to rock adequate for aggregate and to larger

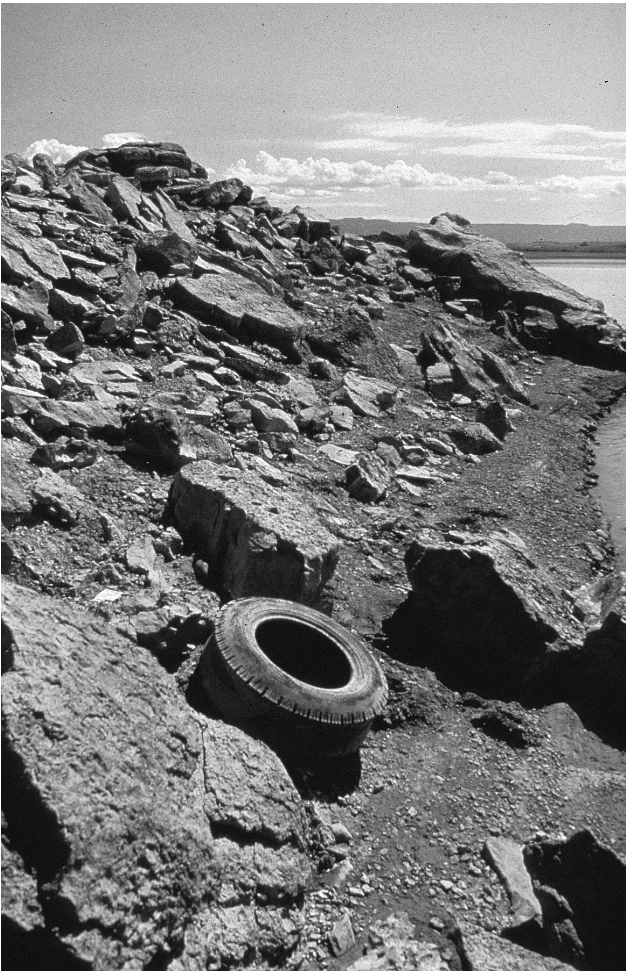


Figure 18-2.—Improperly designed, obtained, and placed riprap.

Figure 18-3.—Hand-placed riprap.

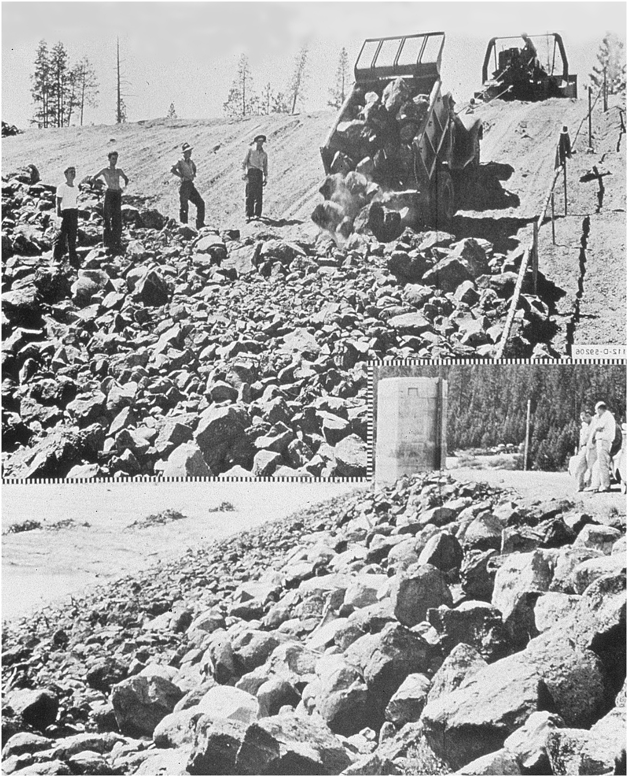


Figure 18-4.—Dumped riprap.

rock fragments used for roads, breakwaters, and jetties. However, this chapter is oriented toward acquiring suitable material for riprap.

This chapter should be used in conjunction with *USBR Procedure 6025, Sampling and Quality Evaluation Testing of Rock for Riprap Slope Protection*, and

USBR Design Standard No. 13 for Embankment Dams (DS13). Riprap design is discussed in "Chapter 7, Riprap Slope Protection," of USBR Design Standard DS13. Other documents, such as the U.S. Army Corps of Engineers' Engineering Manual 1110-2-2301, *Engineering and Design - Test Quarries and Test Fills*, and Engineering Manual 1110-20-1601, *Engineering and Design - Hydraulic Design of Flood Control Channels*, also provide information on design and source evaluation. Note: Test procedures developed to test similar riprap characteristics by different organizations are not necessarily the same. The appropriate test procedure should be selected based on the actual test and the available test equipment.

Evaluation

Much of the following discussion is more guidance than hard and fast rules or requirements. What is unacceptable riprap at one location may be acceptable at another site. Remember that a riprap source must be capable of providing suitable material in sufficient quantities at a reasonable cost. The three elements in every source evaluation are: quality, quantity, and cost.

Quality

Rock quality is determined by laboratory testing, but field personnel input and selection of the samples for testing are critical in determining the riprap quality. There are numerous quarries and pits capable of producing aggregate, but not all sources are suitable for the production of riprap. Riprap sources must produce riprap of the necessary weight, size, shape, gradation, and durability to be processed and placed and then remain "nested" for the life of the project. Performance on existing structures is a valuable method of assessing riprap quality from a particular source.

Shape

The shape of individual rock fragments affects the workability and nesting of the rock assemblage. Natural “stones” from alluvial and glacial deposits are usually rounded to subrounded and are easier to obtain, handle, and place and, therefore, are more workable. Rounded stones are less resistant to movement.

The drag force on rounded stones is less than between angular rock fragments. Rounded stones interlock more poorly than do equal-sized angular rock fragments. As a result, a rounded stone assemblage is more likely to be moved or eroded by water action. Angular-shaped rocks nested together resist movement by water and make the best riprap. The rock fragments should have sharp, angular, clean edges at the intersections of relatively flat faces.

Glacial or alluvial deposits are used as riprap sources only if rock quarries are unavailable, too distant, or incapable of producing the appropriate sizes. Unless the design slope is at an angle to the wave direction or wave energy and the erosive action of water on the slope is low, rounded to subrounded stones are typically used only on the downstream face of embankments, in underlying filters, or as the packing material in gabions.

No more than 30 percent of the riprap fragments should have a 2.5 ratio of longest to shortest axis of the rock. Stones having a ratio greater than 2.5 are either tabular or elongated. These tabular or elongated particles (figure 18-5) tend to bridge across the more blocky pieces or protrude out of the assemblage of rocks. During handling, transporting, and placement, these elongated or

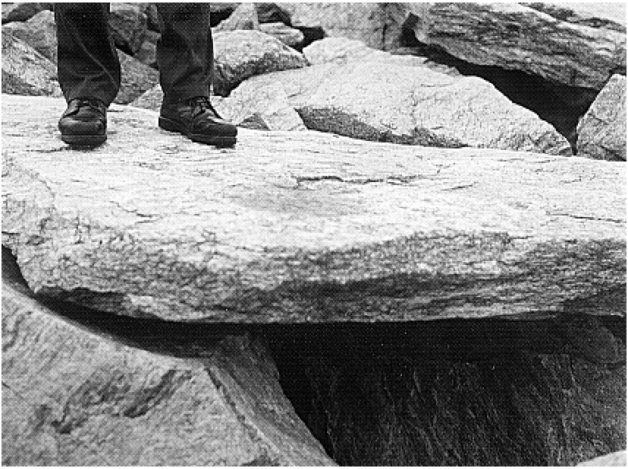


Figure 18-5.—Tabular rock fragment.

tabular rock fragments tend to break into smaller fragments and could significantly change the gradation or thickness of the protective layer.

Nearly all durable rock types can provide appropriately shaped material, but not all rock types can be blasted and processed economically into suitable shapes. Mineral alignment and fractures within the rock mass are the primary factors affecting the development of the shape. Most igneous and some sedimentary rocks are capable of making suitably shaped fragments. However, secondary fracturing or shearing will affect the shape. Rocks having closely spaced discontinuities tend to produce fragments that are too small. Sedimentary rocks that have bedding plane partings tend to produce flat shapes. Metamorphic rocks tend to break along jointing, rock cleavage, or mineral banding and often produce elongated shapes.

Weight and Size

The weight and size of individual riprap pieces are essential factors in resisting erosive water forces. The weight of the rock fragment is one design element for riprap but is difficult to obtain in the field for the larger sizes. The relationship between weight and size is approximately:

$$W_n = 0.75\ \gamma D_n^3$$

where: W_n - Percentage of total weight of rock where n percent is smaller
γ - Unit weight of rock
D_n - Representative diameter of rock where n percent is smaller

This formula assumes the shape of the rock fragment is between a sphere and a cube. The weight and size may be determined in the laboratory or in the field. The unit weight of riprap generally varies from 150 to 175 pounds per cubic foot (2.4 to 2.8 g/cm^3) and correlates with surface saturated dry specific gravity (SSSG). Rock having an SSSG above 2.6 is typically suitable for riprap.

Determination of the relationship between weight and size is difficult. Rock is either graded by size or counted and weighed, but rarely are weight and size correlated. Rarely does rock break into perfect cubical shapes; and because of the various shapes and sizes, weighing and sorting the individual pieces is difficult. The American Society for Testing and Materials Procedure D-5519 provides three methods for obtaining size and weight data.

Typically, for sizes up to 36 inches (1 meter) minimum diameter, rock pieces are sorted by size with a sieve or template, and the number of individual pieces is counted within each group. These piles can then be weighed and individual pieces adjusted to determine size. For individual pieces larger than 36 inches, the size is typically determined by using a tape to measure the maximum and minimum size of each piece. The weight is determined from a chart that assumes the shape is between a cube and sphere.

Most rock sources are capable of producing suitable weights and sizes. The size rarely impacts use as a riprap source unless more than 30 percent of the rocks are elongated or flat. In special circumstances, the rock mineralogy and porosity control the weight. The porosity of some sedimentary and extrusive volcanic rock could affect the weight. Rock having an SSSG under 2.3 is typically not considered for riprap. Generally, rock having a low unit weight is weak and tends to break down with handling.

Gradation

The desired gradation consists of size fractions of the individual particles that will nest together and withstand environmental conditions. The gradation design is based on the ability of the source(s) to produce appropriate sizes. Inherent rock mineralogy, cleavage, and fractures control the size of the rock fragments. Blasting, excavating, and processing also affect the size. Most acceptable riprap gradations are obtained by understanding the inherent rock characteristics, by proper blasting techniques, and by processing. Rarely can blending rock sizes achieve an appropriate grading for riprap because the larger fragments tend to separate from the smaller fragments during handling and processing. Processing is typically limited to running the rock fragments over a stationary grizzly (figure 18-6) or sorting with a rock bucket or rock rake (figure 18-7). Rarely is rock processed with jaw or

Figure 18-6.—Stationary grizzly. Rock is dumped on the sloping rails, and the larger material slides off and is separated from the smaller material which falls through.

Figure 18-7.—Rock rake. A dozer-mounted rock rake separates the larger fragments from the smaller material.

gyratory crushers except for testing. Segregation of large and small sizes is controlled by reducing the number and amount of drops during handling and processing. Handling should be kept to a minimum.

Most coarse-grained sedimentary and igneous rock quarries are capable of producing suitable riprap gradations. The range of gradations from sedimentary sources depends on the depositional environment. Rock derived from rapid depositional environments is more likely to produce well-graded riprap.

Size range is controlled by discontinuities in the rock. Columnar basalt, some fine-grained sedimentary rock, and metamorphic rock commonly have inherent planes of weakness that limit larger riprap sizes. Intensely to moderately fractured rock rarely produces suitable riprap gradations.

Durability

Riprap durability affects the ability of a source to provide a consistent shape, size, and gradation and the ability to resist weathering and other environmental influences. Durability is typically determined by laboratory test; but durability can be assessed by observing surface exposures, talus, and waste piles or by examining riprap applications already using the potential source or similar source materials. Cracking, spalling, delaminating, splitting, disaggregating, dissolving, and disintegrating are common forms of rock deterioration. Durability is a function of the rock's mineralogy, porosity, weathering, discontinuities, and site conditions. In rare instances, environmental considerations such as abnormal pH of the water may be a controlling factor in selecting an appropriate riprap source. A high or low pH may accelerate disintegration of the rock.

Alteration of minerals, such as feldspars to softer clays, will impact rock durability. Fine-grained rock types, rocks having high porosity, and chemically altered rocks may tend to slake after cyclic wetting and drying or freezing and thawing. Some rocks tend to break up because of discontinuities such as bedding plane parting, cementation or secondary mineralization, unstable minerals, banding, or foliation. Jointing, rock cleavage, and bedding plane partings often result in excessive finer sizes or tabular and elongated rock fragments.

Rock that breaks down either physically or chemically should be avoided. Obvious examples are most weathered or altered rocks, rock containing soluble or expansive minerals, vesicular basalts, shale, claystone, siltstone, weakly cemented or porous sandstone, schist, or phyllite. Even durable rocks such as slate and some gneisses may generally be unusable because other physical characteristics (cleavage and foliation) will not allow production of large, nearly equidimensional blocks.

Mechanical breakdown and weathering may be accelerated by microfracturing from the blasting, handling, weak cementation or may be the result of alteration of more stable minerals to clay. In addition, there appears to be a significant correlation between porosity, absorption, and durability of rock. Rock that has more than 2 percent absorption is commonly impacted by freezing and thawing and by wetting and drying processes.

Quantity

Every riprap source investigation must provide the estimated quantity required. Estimating realistic quantities depends on an understanding of subsurface geologic conditions. The uniformity of rock and discontinuities within a source area must be assessed. This estimate (often referred to as the reserve) provides not only the amount of riprap available but also provides an understanding of wastage resulting from blasting, handling, processing, haulage, and placement. In stratified deposits such as limestones or sandstones, uniformity must be evaluated because individual beds often differ in character and quality. The dip of stratified rocks and contacts between dissimilar rock types, such as igneous intrusions, must also be considered. The larger the individual pieces required, the more difficult it is for any rock type to supply suitable quantities. Zones or layers of undesirable clay or shale seams may be so large or prevalent that selective quarrying or wasting of undesirable material is required. The geologic conditions, ability of the rock to produce suitable sizes, and the potential reserve should be determined.

Existing commercial sources may be capable of producing riprap but may not be capable of expanding their operation into similar quality rock. In any new source, the amount of burden that must be removed, stability of the cutslopes, uniformity of the rock, depth to water, and ability to blast or process the rock into the appropriate gradation must be evaluated. Since riprap is a surface layer, a smaller sized riprap of increased thickness may be acceptable, or a less durable riprap may be used with the understanding that the riprap may require replacement.

Cost

A primary factor in determining a suitable riprap source is cost. Design and environmental requirements, access, subsurface conditions, testing, depth to water, quantity of suitable rock, and location also affect the cost and should be assessed early in any source investigation.

Producing sources should be located first. Using existing quarries or pits is generally cheaper because there is considerably less cost associated with permitting, developing, and evaluating an existing source. An existing source provides easier access to rock; a history of the source provides an understanding of the source's ability to provide suitable rock; regulatory requirements are often more easily met; development and processing costs are often known; and often, some testing of material has been performed so that the quality is known. Although existing sources may be known, each of these elements should be evaluated to ensure that information is representative and appropriate for the particular requirement.

In areas where existing sources are not economical, evaluating the surrounding undeveloped areas or abandoned pits or quarries should be considered. Evaluating new or abandoned sources typically involves considerable expense. A new quarry or pit investigation involves understanding subsurface conditions; obtaining, evaluating, and testing subsurface samples; and evaluating subsurface conditions to determine if appropriate riprap can be produced. Factors such as the haul distance, grade, width, and type of roadway should also be assessed.

Investigation Stages

The complexity of investigations for suitable sources of riprap is governed by the development stage and design requirements of the project. Projects are normally developed in four stages: reconnaissance, feasibility, design, and construction.

Reconnaissance

Initial exploration involves field surface reconnaissance using topographic maps, geologic and groundwater maps and reports, and aerial photographs. Supplemental information is provided by records of known developed sources of material. Areas having steep topography could have the best rock exposures. Geology maps provide generalized locations of rock types. Groundwater maps provide indications of rock permeability, depths to water, and information on the need for dewatering or unwatering within the source area. During field reconnaissance, the countryside should be examined for exposed rock outcrops or talus piles. Roadcuts and ditches may also provide useful exposures. Existing sources and any projects that previously used the rock source should be examined.

Service records are an excellent indication of the potential durability of rock. Federal (Reclamation, U.S. Army Corps of Engineers, Department of Transportation), State (highway, environmental quality), and county or local (highway or building) agencies usually maintain lists of sources. The local telephone "Yellow Pages," Internet, and construction companies may also provide information.

Data obtained should define the major advantages or disadvantages of potential material sources within reasonable haul distance. A reconnaissance construction material report should be prepared at this stage.

Feasibility

Information acquired during the feasibility stage is used to prepare preliminary designs and cost estimates. Sufficient information concerning potential sources should be gathered to determine whether the rock should be obtained from an existing source or a new source.

Selection of sources should be limited to those that may eventually be used in specifications. Core drilling and blast tests may be required to confirm fragment size and quantity of material available in each source. The potential material sources should be examined to determine size and character, and particularly to observe joint and fracture spacing, resistance to weathering, and variability of the rock. The spacing of joints, fractures, schistosity, banding, bedding, and other planes of weakness may control the rock fragment sizes and shapes. Weathering resistance of the rock will provide a good indication of durability. Quarry or pit development and the impacts of groundwater should be addressed. Particular attention should be given to location and distribution of unsound seams or beds that must be avoided or wasted during the quarry operation. A general location map and detailed report describing the potential sources and containing estimates of available quantities, overburden, haul roads, and accessibility should be prepared. Representative samples of riprap material from the most promising potential sources should be submitted to the laboratory for testing. A feasibility construction material report should be prepared at this stage.

Design

Investigations during the design stage furnish data and information required for the specifications. Sources indicated by feasibility investigations to be suitable are further investigated to establish quantities, determine the capability to produce the required gradation, and to determine uniformity. Depending on project needs, service records may be used in conjunction with or instead of laboratory testing. Blast and processing testing should be considered for new sources. All sampling and testing and the laboratory's Riprap Quality Evaluation Report should be completed at this stage. If additional sources are necessary, the new sources must be investigated as throughly as the original sources.

Construction

Investigations during construction provide field and design personnel with additional detailed information for proper source development. This information should be obtained sufficiently ahead of quarrying or excavating to provide for proper processing and placing of material. If unforeseen changes occur in the quality of material in the source, sampling and quality evaluation testing of the material may be required to confirm material suitability or to delineate unsuitable areas.

Reports

Reporting the results of any investigation is important. The level of detailed information requirements increases with each successive stage. Adequate information must be available by the feasibility stage to develop realistic cost estimates and to properly select sources. A suggested outline for reports for rock or riprap obtained from any potential quarry or pit is as follows:

a. Ownership
b. Location of source and project shown on a map
c. General description of site
d. General hydrologic and geologic descriptions
e. Structural geology information (distribution and arrangement of rock types and discontinuities within the deposit.)
f. Manner and sizes of rock breakage
g. Estimate of uniformity and wastage

h. Shape and angularity of source material
i. Hardness and density of source material
j. Degree and extent of weathering
k. Any abnormal properties or conditions not covered above
l. Estimate of extent, volume, and depth of suitable deposit(s)
m. Accessibility
n. Photographs
o. Geophysical and geologic data (e.g., drill logs, borehole geophysical logs, and seismic refraction or reflection survey data)

If commercial quarry or pit deposits are considered, obtain, as appropriate, the following information in addition to the data needed for a new source.

- Name, address, and phone number of the plant operator
- Location of the plant relative to quarry
- Description of the operation and plant with emphasis on capabilities for additional riprap production and maintaining current operation capabilities
- Blasting methods and problems related to production of riprap
- Transportation facilities and any potential difficulties
- Actual or estimated riprap gradations achieved or achievable by current or adjusted operations
- Location of scales
- Estimate of reserve and wastage

- Approximate prices of riprap material
- Service history of material produced
- Any other pertinent information

Sampling

Sampling is often the weak link in any source evaluation. The samples should represent the nature and condition of the materials and be appropriate for testing. Sampling is initiated at the specifications stage of the project. Sampling should cover the entire riprap source. The sample size should be at least 600 pounds (275 kilograms) and represent the quality range from poor to best as found at the source in the same proportions as the source can supply. If the material quality is quite variable, it may be preferable to obtain three samples that represent the poorest to best quality material available. The minimum size of individual fragments selected should be at least 0.5 foot (15 cm) square. An estimate of the relative percentages of material at each quality level should be made.

Representative samples may be difficult to obtain. Overburden may limit the areas where material can be obtained and obscure the true characteristics of the deposit. Outcrops will often be more weathered than the subsurface deposits. Samples obtained from talus piles or outer surfaces of rock outcrops are seldom representative of quality, quantity, or gradation. Fresh material may be obtained by breaking away the outer surfaces, or by trenching, blasting, or core drilling. If coring is the only method of obtaining samples, the preferred size is 6 inches (15 cm).

Shipping

Samples of rock fragments can be shipped by conventional transport such as motor freight. Large rock fragments should be securely banded to shipping pallets. Smaller fragments should be transported in bags or containers to preclude loss, contamination, or damage from mishandling during shipment.

Testing

The Riprap Quality Evaluation Report is based on laboratory testing of the shipped representative samples. The quality evaluation tests include detailed petrographic examination, determination of physical properties and absorption, and a rapid freeze-thaw durability evaluation.

Petrographic Examination.—The petrographic examination follows USBR Procedure 4295 or ASTM Procedure C 295, which were developed for concrete aggregate. The decisions concerning specific procedural methods and specimen preparation depend on the nature of the rock and the intended use of the rock.

The rock pieces are visually examined and the different rock facies and types are segregated for individual evaluation. The following are evaluated:

- Size range
- Fragment shape
- Shape and size control by discontinuities such as joints, banding, or bedding
- Surface weathering
- Secondary mineralization or alteration
- Hardness, toughness, and brittleness

- Voids and pore characteristics and their variations
- Texture, internal structure, grain size, cementation, and mineralogy of the various facies and rock types
- Thin sections, sometimes supplemented by X-ray diffraction as required

Freeze-Thaw Test.— For freeze-thaw durability testing, two 7/8-inch (73 millimeter) cubes are sawed from rock fragments selected by visual inspection to represent the range from poorest to best quality rock for each rock facies or type. Because the rock pieces could have significant physical or structural discontinuities, the number of cubes obtained for testing will vary from sample to sample. The samples are photographed, the cubes are immersed in water for 72 hours, and specific gravities (bulk, SSSG, and apparent) and absorptions are determined by USBR Procedure 4127 or ASTM Procedure C 127. The cubes are reimmersed in water to maintain a saturated condition for freeze-thaw testing.

Rapid freezing and thawing durability tests are performed on riprap samples according to USBR Procedure 4666 or ASTM Procedure D5312. The rock failure criterion is 25 percent loss of cube mass calculated from the difference in mass between the largest cube fragment remaining after testing and the initial cube mass.

Sodium Sulfate Soundness Test.—Sodium sulfate soundness tests are performed on riprap samples according to USBR Procedure 4088 or ASTM Procedure D5240. The loss after an interval of screening is determined after at least five cycles of saturation and drying of the samples. The test is a good indicator of resistance to freeze-thaw deterioration.

Physical Properties.—Material remaining after the petrographic examination and freeze-thaw testing is crushed into specific size fractions (USBR Procedure 4702). Representative samples of each size fraction are tested for bulk, SSGS, and absorption following USBR Procedure 4127 or ASTM Procedure C 127; abrasion is tested using the Los Angeles abrasion test following USBR Procedure 4131 or ASTM Procedure C 535. Both the Los Angeles abrasion and sodium sulfate soundness tests are durability tests. The Los Angeles abrasion test is used to determine the ability of the rock to withstand handling and processing and water action. The sodium sulfate soundness test simulates weathering of the rock pieces.

Waste in Riprap Production

Production of riprap generally requires drilling, blasting, and processing to obtain the desired sizes. This section is a guide to help estimate the amount of waste that can be expected from riprap production.

Numerous factors in the parent rock contribute to waste in quarrying operations. The natural factors include:

- Weathering
- Fracturing (joints, shears, and faults)
- Bedding, schistosity, and foliation
- Recementing of planar features

Other important, somewhat controllable contributors to waste are:

- Construction inspection
- Size and gradation requirements
- Drilling and blasting
- Processing, hauling, and placement

Factors a through d relate to the geology in the quarry and probably are the most important factors that govern what sizes can be produced. Weathering can extend 20 to 60 feet (6 to 18 meters) below the original ground surface. Weathering breaks down the rock and weakens existing planar features such as bedding, schistosity, and jointing. In rocks such as limestone and dolomite, secondary deposits of calcium carbonate can cement existing joints. When first examined, this cementation appears to be sound; but processing the rock can refracture these planes. Existing quarries, or quarries that have been in operation for many years, probably will produce material with less waste because excavations are partly or completely through the zone of weathering. New quarries, or quarries where rock production has been limited, must contend with the weathered zone and will likely produce a less desirable product.

Gradation Requirements

Gradation requirements and inspection control are governed by the agency issuing the construction specifications. Adjustments in gradation or inspection requirements can drastically change the waste quantities produced. Except in isolated cases, it becomes more difficult to produce riprap when rock sizes are increased and gradations are tightly controlled.

Production Methods

Production methods that include drilling, blasting, processing, and hauling also play an important role in the sizes that can be obtained. Rock that is well-graded and has a large maximum size can be produced more readily when using large diameter, widely spaced shot holes. Close spaced, small diameter shot holes tend to maximize fragmentation. Blasting agents, delays, and loading methods vary considerably and have a significant effect on how the rock fractures. The most efficient and economical drilling and blasting methods must be determined by test blasting and performing gradations on the blasted product. Test shots should be modified to achieve the desired product. Production should not start until it is proven that the required product can be produced with a minimum of waste.

Many rock types, especially those that are banded (bedded or schistose) or contain healed joints, can break down significantly during processing. Some limestones are especially susceptible to breakdown when the rock is dropped during blasting and processing operations. Rock from most quarries will fracture badly when dropped more than 50 feet (15 meters).

Quarries must tailor their blasting techniques to get the required gradations. Quarries that normally produce aggregates for concrete, road metal, and base course usually have a very difficult time producing a reasonably well-graded riprap. This is because their normal operation already has shattered the face at least 100 feet back. To obtain good riprap, a working face or ledge should be reserved for riprap production.

The quantity of quarry waste shown in table 18-1 is typical of riprap quarries. Items that should be considered when using the table include:

- Waste includes undersize and excessive intermediate sizes. Oversize riprap is reprocessed to the proper size.

- Rock produced is reasonably well graded from 6± to 36± inches (.15 to 1 m), and the inspection control is very strict. Much less waste will be incurred if smaller rock sizes

Table 18-1.—Rock types and typical usable quantities of riprap

Rock type	Estimated percent waste to produce suitable riprap	Remarks
IGNEOUS		
Intrusive	25 to 75% Average 50%±	
Extrusive	40 to 85% Average 60%±	
METAMORPHIC		
Gneiss	40 to 75% Average 55%±	Based on limited data
Schist	50 to 75% Average 65%±	Based on limited data Very little riprap would be salvaged in the weathered zone
SEDIMENTARY		
Limestone/ Dolomite	55 to 85 % Average 65%±	Based on several good quarry sites
Sandstone	Average 60%±	Based on limited data

are required or if the deposit is shot for rockfill or the specific rock product.

- Drilling, blasting, processing, hauling, and placing are accomplished by a typical contractor.

- Rock quarried is the best material available and is not severely fractured or weathered.

- Riprap production is generally limited to new quarries or unshot ledges or benches.

Chapter 19

BLAST DESIGN

Introduction

This chapter is an introduction to blasting techniques based primarily on the *Explosives and Blasting Procedures Manual* (Dick et al., 1987) and the *Blaster's Handbook* (E.I. du Pont de Nemours & Co., Inc., 1978). Blast design is not a precise science. Because of widely varying properties of rock, geologic structure, and explosives, design of a blasting program requires field testing. Tradeoffs frequently must be made when designing the best blast for a given geologic situation. This chapter provides the fundamental concepts of blast design. These concepts are useful as a first approximation for blast design and also in troubleshooting the cause of a bad blast. Field testing is the best tool to refine individual blast designs.

Throughout the blast design process, two overriding principles must be kept in mind:

(1) Explosives function best when there is a free face approximately parallel to the explosive column at the time of detonation.

(2) There must be adequate space for the broken rock to move and expand. Excessive confinement of explosives is the leading cause of poor blasting results such as backbreak, ground vibrations, airblast, unbroken toe, flyrock, and poor fragmentation.

Properties and Geology of the Rock Mass

The rock mass properties are the single most critical variable affecting the design and results of a blast. The rock properties are very qualitative and cannot be sufficiently quantified numerically when applied to blast design. Rock properties often vary greatly from one end of a construction job to another. Explosive selection, blast design, and delay pattern must consider the specific rock mass being blasted.

Characterizing the Rock Mass

The keys to characterizing the rock mass are a good geologist and a good blasting driller. The geologist must concentrate on detailed mapping of the rock surface for blast design. Jointing probably has the most significant effect on blasting design. The geologist should document the direction, density, and spacing between the joint sets. At least three joint sets—one dominant and two less pronounced—are in most sedimentary rocks. The strike and dip of bedding planes, foliation, and schistosity are also important to blast design and should be documented by the geologist. The presence of major zones of weakness such as faults, open joints, open beds, solution cavities, or zones of less competent rock or unconsolidated material are also important to blast design and must be considered. Samples of freshly broken rock can be used to determine the hardness and density of the rock.

An observant blasting driller can be of great help in assessing rock variations that are not apparent from the surface. Slow penetration, excessive drill noise, and vibration indicate a hard rock that will be difficult to break. Fast penetration and a quiet drill indicate a softer, more easily broken zone of rock. Total lack of resistance to penetration, accompanied by a lack of cuttings or return water or air, means that the drill has hit a void. Lack of cuttings or return water may also indicate the presence of an open bedding plane or other crack. A detailed drill log indicating the depth of these various conditions can be very helpful in designing and loading the blast. The log should be kept by the driller. The driller should also document changes in the color or nature of the drill cuttings which will tell the geologist and blaster the location of various beds in the formation.

Rock Density and Hardness

Some displacement is required to prepare a muckpile for efficient excavation. The density of the rock is a major factor in determining how much explosive is needed to displace a given volume of rock (powder factor). The burden-to-charge diameter ratio varies with rock density, changing the powder factor. The average burden-to-charge diameter ratio of 25 to 30 is for average density rocks similar to the typical rocks listed in table 19-1. Denser rocks such as basalt require smaller ratios (higher powder factors). Lighter materials such as some sandstone or bituminous coal can be blasted with higher ratios (lower powder factors).

The hardness or brittleness of rock can have a significant effect on blasting results. If soft rock is slightly under-blasted, the rock probably will still be excavatable. If soft rock is slightly overblasted, excessive violence will not usually occur. On the other hand, slight underblasting of hard rock will often result in a tight muckpile that is difficult to excavate. Overblasting of hard rock is likely to cause excessive flyrock and airblast. Blast designs for hard rock require closer control and tighter tolerances than those for soft rock.

Voids and Zones of Weakness

Undetected voids and zones of weakness such as solution cavities, "mud" seams, and shears are serious problems in

Table 19-1.—Typical rocks, densities, and unit weights

Rock type	Density range (g/cm³)[1]	Range of unit weights: U.S. customary (lb/ft³)[2]	Range of unit weights: Metric (kg/m³)[3]
Limestone	2.5 to 2.8	156 to 174.7	2,500 to 2,800
Schist	2.6 to 2.8	162.2 to 174.7	2,600 to 2,800
Rhyolite	2.2 to 2.7	137.2 to 168.5	2,200 to 2,700
Basalt	2.7 to 2.9	168.5 to 181	2,700 to 2,900
Sandstone	2/0 to 2.6	124.8 to 162.2	2,000 to 2,600
Bituminous coal	1.2 to 1.5	74.9 to 93.6	1,200 to 1,500

[1] Grams per cubic centimeter.
[2] Pounds per cubic foot.
[3] Kilograms per cubic meter.

blasting. Explosive energy always seeks the path of least resistance. Where the rock burden is composed of alternate zones of hard material, weak zones, or voids, the explosive energy will be vented through the weak zones and voids resulting in poor fragmentation. Depending on the orientation of zones of weakness with respect to free faces, excessive violence in the form of airblast and flyrock may occur. When the blasthole intersects a void, particular care must be taken in loading the charge, or the void will be loaded with a heavy concentration of explosive resulting in excessive air-blast and flyrock.

If these voids and zones of weakness can be identified and logged, steps can be taken during borehole loading to improve fragmentation and avoid violence. The best tool for this is a good drill log. The depths of voids and zones of weakness encountered by the drill should be documented. The geologist can help by plotting the trends of "mud" seams and shears. When charging the blasthole, inert stemming materials rather than explosives should be loaded through these weak zones. Voids should be filled with stemming. Where this is impractical because of the size of the void, it may be necessary to block the hole just above the void before continuing the explosive column.

If the condition of the borehole is in doubt, the top of the powder column should be checked frequently as loading proceeds. A void probably exists if the column fails to rise as expected. At this point, a deck of inert stemming material should be loaded before powder loading continues. If the column rises more rapidly than expected, frequent checking will ensure that adequate space is left for stemming.

Alternate zones of hard and soft rock usually result in unacceptably blocky fragmentation. A higher powder factor seldom will correct this problem; it will merely cause the blocks to be displaced farther. Usually, the best way to alleviate this situation is to use smaller blastholes with smaller blast pattern dimensions to get a better powder distribution. The explosive charges should be concentrated in the hard rock.

Jointing

Jointing can have a pronounced effect on both fragmentation and the stability of the perimeter of the excavation. Close jointing usually results in good fragmentation.

Widely spaced jointing, especially where the jointing is pronounced, often results in a very blocky muckpile because the joint planes tend to isolate large blocks in place. Where the fragmentation is unacceptable, the best solution is to use smaller blast holes with smaller blast pattern dimensions. This extra drilling and blasting expense will be more than justified by the savings in loading, hauling, and crushing costs and the savings in secondary blasting.

Where possible, the perimeter holes of a blast should be aligned with the principal joint sets. This produces a more stable excavation, whereas rows of holes perpendicular to a primary joint set produces a more ragged, unstable perimeter (figure 19-1). The jointing will generally determine how the corners at the back of the blast will break out. To minimize backbreak and flyrock, tight corners, as shown in figure 19-2, should be avoided.

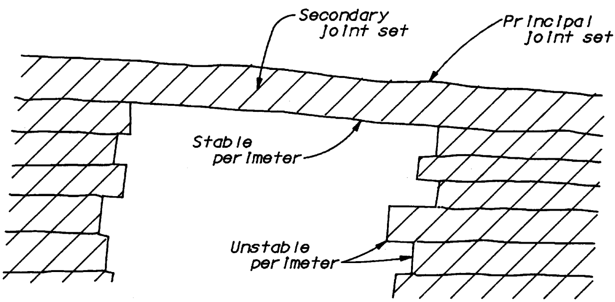


Figure 19-1.—Effect of jointing on the stability of an excavation (plan view).

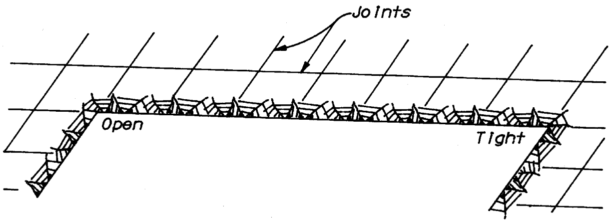


Figure 19-2.—Tight and open corners caused by jointing (plan view).

The open corner at the left of figure 19-2 is preferable. Given the dominant jointing in figure 19-2, more stable conditions will result if the first blast is opened at the far right and designed so that the hole in the rear inside corner contains the highest numbered delay.

Bedding/Foliation

Bedding has a pronounced effect on both the fragmentation and the stability of the excavation perimeter. Open bedding planes, open joints, or beds of weaker materials should be treated as zones of weakness. Stemming, rather than explosive, should be loaded into the borehole at the location of these zones as shown in figure 19-3. In a bed of hard material (greater than 3 feet [1 m] thick), it is often beneficial to load an explosive of higher density than is used in the remainder of the borehole. To break an isolated bed or zone of hard rock (3 feet [1 m] thick or greater) near the collar of the blasthole, a deck charge is recommended, as shown in figure 19-4, with the deck being fired on the same delay as the main charge or one delay later. Occasionally, satellite holes are used to help break a hard zone in the upper part of the burden. Satellite holes (figure 19-4) are short holes, usually smaller in diameter than the main blastholes drilled between the main blastholes.

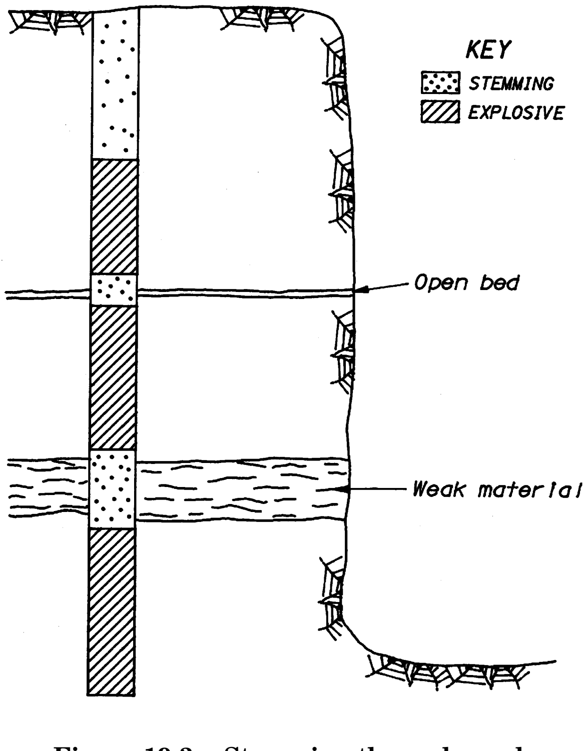


Figure 19-3.—Stemming through weak material and open beds.

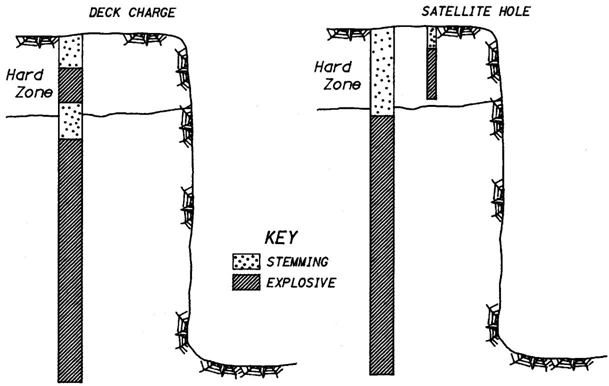


Figure 19-4.—Two methods of breaking a hard collar zone.

A pronounced foliation, bedding plane, or joint is frequently a convenient location for the bench floor. It not only gives a smoother floor but also may reduce subdrilling requirements.

Dipping beds frequently cause stability problems and difficulty in breaking the toe of the excavation. When bedding or foliation dip into the excavation wall, the stability of the slope is enhanced. When the dip is out of the wall, slip planes exist that increase the likelihood of slope deterioration or failure. Blasthole cutoffs (part of a column of explosives not fired) caused by differential bed movement are also more likely. Beds dipping out of the final slope should be avoided wherever possible.

Although beds dipping into the face improve slope stability, the beds can create toe problems because the toe rock tends to break along the bedding or foliation planes.

Dipping beds such as these require a tradeoff, depending upon which is the more serious problem—a somewhat unstable slope or an uneven toe. In some cases, advancing the opening perpendicular to dipping beds may be a compromise.

Many blasting jobs encounter site-specific geologic conditions not covered in this general discussion. Good blasting techniques require constant study of the geology to make every effort to advantageously use the geology, or at least minimize its unfavorable effects in blast designs.

Surface Blasting

Blast Hole Diameter

The blast hole size is the first consideration of any blast design. The blast hole diameter, along with the type of explosive being used and the type of rock being blasted, determines the burden (distance from the blast hole to the nearest free face). All other blast dimensions are a function of the burden. This discussion assumes that the blaster has the freedom to select the borehole size. Many operations limit borehole size based on available drilling equipment.

Practical blasthole diameters for surface construction excavations range from 3 (75 mm) to approximately 15 inches (38 cm). Large blasthole diameters generally yield low drilling and blasting costs because large holes are cheaper to drill per unit volume, and less sensitive, cheaper blasting agents can be used in larger diameter holes. Larger diameter blastholes also allow large burdens and spacings and can give coarser fragmentation. Figure 19-5 illustrates this comparison using 2- (50-mm) and 20-inch (500-mm)-diameter blastholes as an example.

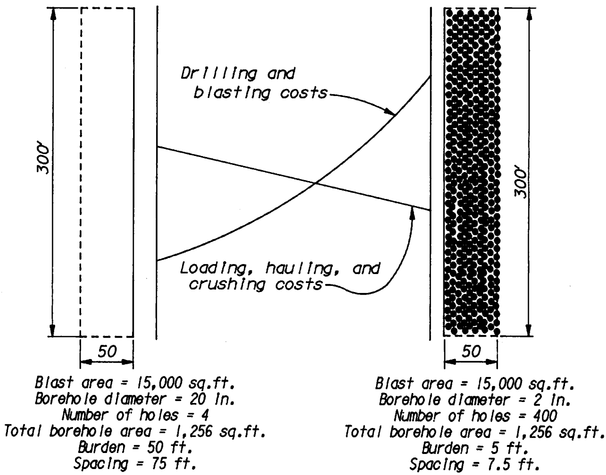


Figure 19-5.—The effect of large and small blast holes on unit costs.

Pattern A contains four 20-inch (500-mm) blast boles, and pattern B contains 400 2-inch (5-mm) blast holes. In all bench blasting operations, some compromise between these two extremes is chosen. Each pattern represents the same area of excavation—15,000 square feet (1,400 m^2)—each involves approximately the same volume of blast holes, and each can be loaded with about the same weight of explosive.

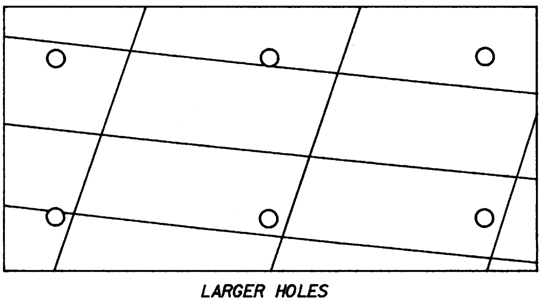
As a general rule, large diameter blast holes (6 to 15 inches [15 to 38 cm]) have limited applications on most construction projects because of the requirements for fine fragmentation and the use of relatively shallow cuts. However, borehole size depends primarily on local conditions. Large holes are most efficient in deep cuts where a free face has already been developed.

In most construction projects, small diameter drilling with high-speed equipment provides relatively low unit costs and permits fairly close spacing of holes. This close spacing provides better distribution of explosives throughout the rock mass, which in turn produces better breakage. An additional advantage of small diameter blast holes is that the reduction in the amount of each explosive used in each hole reduces ground vibrations. Construction blast hole diameters usually vary from 3.5 to 4.5 inches (90 to 114 mm), and the normal drilling depth is less than 40 feet (12 m). Reclamation generally limits blast hole diameters for structural excavation drilling to 3.5 inches (90 mm). Blasting patterns usually range from 6 by 8 feet (1.8 by 2.4 meters) to 8 by 15 feet (2.4 by 4.6 m) and are usually rectangular with the burden being less than the spacing.

In a given rock type, a four-hole pattern will give relatively low drilling and blasting costs. Drilling costs for large blastholes will be low, a low-cost blasting agent will be used, and the cost of detonators will be minimal. In a difficult blasting situation, the broken material will be blocky and nonuniform in size, resulting in higher loading, hauling, and crushing costs as well as requiring more secondary breakage. Insufficient breakage at the toe may also result.

The 400-hole pattern will yield high drilling and blasting costs. Small holes cost more to drill per unit volume, powder for small diameter blastholes is usually more expensive, and the cost of detonators will be higher. The fragmentation will be finer and more uniform, resulting in lower loading, hauling, and crushing costs. Secondary blasting and toe problems will be minimized. Size of equipment, subsequent processing required for the blasted material, and economics will dictate the type of fragmentation needed and the size of blast hole to be used.

Geologic structure is a major factor in determining the blast hole diameter. Planes of weakness (i.e., joints, shears, or zones of soft rock) tend to isolate large blocks of rock in the burden. The larger the blast pattern, the more likely these blocks are to be thrown unbroken into the muckpile. Note that in the top pattern in figure 19-6, some of the blocks are not penetrated by a blast hole. In the smaller bottom pattern, all the blocks contain at least one blast hole. Because of the better explosives distribution, the bottom pattern will give better fragmentation.



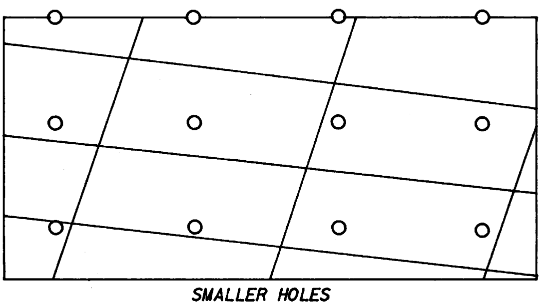


Figure 19-6.—The effects of jointing on selection of blast hole size.

Airblast and flyrock often occur because of an insufficient collar distance (stemming column) above the explosive charge. As the blast hole diameter increases, the collar distance required to prevent violence increases. The ratio of collar distance to blast hole diameter required to prevent violence varies from 14:1 to 28:1, depending on the relative densities and velocities of the explosive and rock, the physical condition of the rock, the type of stemming used, and the point of initiation. A larger collar distance is required where the sonic velocity of the rock exceeds the detonation velocity of the explosive or where the rock is heavily fractured or low density. A top-initiated charge requires a larger collar distance than a bottom-initiated charge. As the collar distance increases, the powder distribution becomes poorer, resulting in poorer fragmentation of the rock in the upper part of the bench.

Ground vibrations are controlled by reducing the weight of explosive fired per delay interval. This is done more easily with small blast holes than with large blast holes. In many situations where large diameter blast holes are used near populated areas, several delays, along with decking, must be used within each hole to control vibrations.

Large holes with large blast patterns are best suited to an operation with: (1) a large volume of material to be moved, (2) large loading, hauling, and crushing equipment, (3) no requirement for fine, uniform fragmentation, (4) an easily broken toe, (5) few ground vibration or airblast problems (few nearby buildings), and (6) a relatively homogeneous, easily fragmented rock without many planes of weakness or voids. Many blasting jobs have constraints that require smaller blast holes.

The final selection of blast hole size is based on economics. Savings realized through inappropriate cost cutting in the drilling and blasting program may well be lost through increased loading, hauling, or crushing costs.

Blast Patterns

The three drill patterns commonly used are square, rectangular, and staggered. The square drill pattern (figure 19-7) has equal burdens and spacing, and the rectangular pattern has a larger spacing than burden. In both the square and rectangular patterns, the holes of each row are lined up directly behind the holes in the preceding row. In the staggered pattern (figure 19-8), the holes in each are positioned in the middle of the spacings of the holes in the preceding row. In the staggered pattern, the spacings should be larger than the burden.

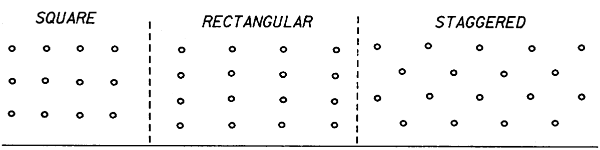


Figure 19-7.—Three basic types of drill patterns.

Square or rectangular drilling patterns are used for firing V-cut (figure 19-9) or echelon rounds. The burdens and subsequent rock displacement are at an angle to the original free face either side of the blast round in V-cut or echelon patterns. The staggered drilling pattern is used for row-on-row firing where the holes of one row are fired before the holes in the row immediately behind them as shown in figure 19-9. Looking at figure 19-9, with the burdens developed at a 45-degree angle to the original

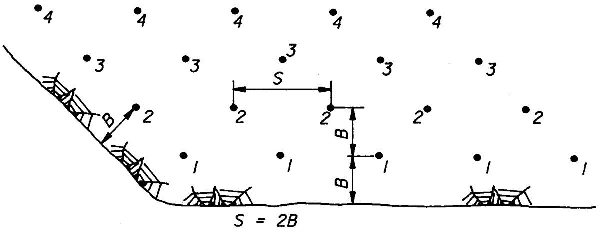


Figure 19-8.—Corner cut staggered pattern with simultaneous initiation within rows (blast hole spacing, S, is twice the burden, B).

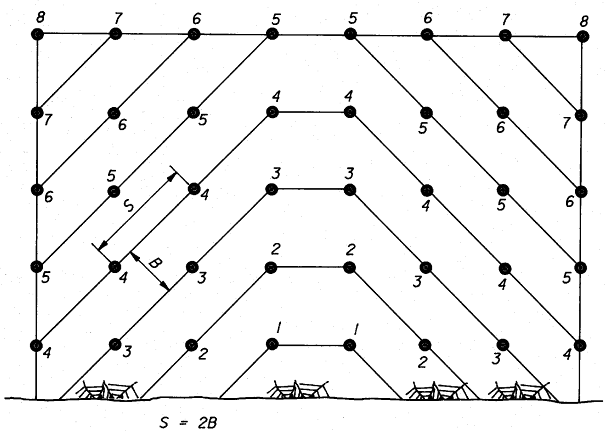


Figure 19-9.—V-Echelon blast round (true spacing, S, is twice the true burden, B).

free face, the original square drilling pattern has been transformed to a staggered blasting pattern with a spacing twice the burden. The three simple patterns discussed here account for nearly all the surface blasting.

Burden

Figure 19-10 is an isometric view showing the relationship of the various dimensions of a bench blast. The burden is defined as the distance from a blast hole to the nearest free face at the instant of detonation. In multiple row blasts, the burden for a blast hole is not necessarily measured in the direction of the original free face. The free faces developed by blast holes fired on lower delay periods must be taken into account. As an

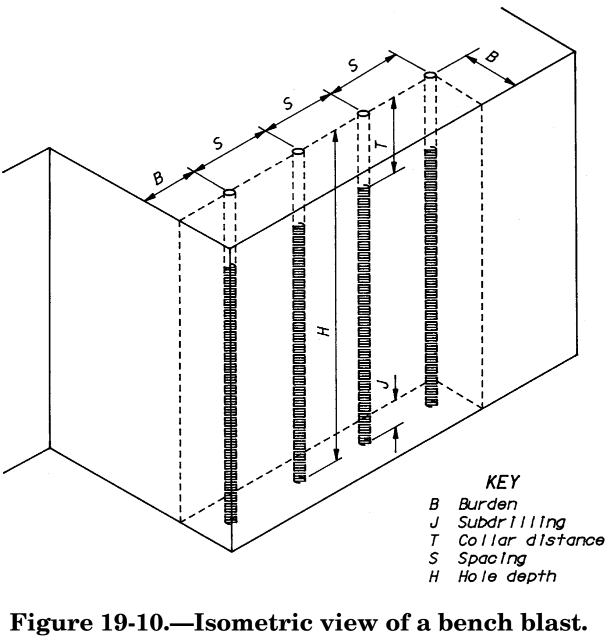


Figure 19-10.—Isometric view of a bench blast.

example, in figure 19-8, where one entire row is blasted before the next row begins, the burden is measured perpendicular between rows.

In figure 19-9, the blast progresses in a V-shape. The true burden on most of the holes is measured at an angle of 45 degrees from the original free face.

It is very important that the proper burden be calculated, accounting for the blast hole diameter, relative density of the rock and explosive, and, to some degree, the depth of the blast hole. An insufficient burden will cause excessive airblast and flyrock. Too large a burden will produce inadequate fragmentation, toe problems, and excessive ground vibrations. If it is necessary to drill a round before the previous round has been excavated, it is important to stake out the first row of the second round before the first round is fired. This will ensure a proper burden on the first row of blast holes in the second blast round.

For bulk-loaded charges (the charge is poured down the hole), the charge diameter is equal to the blast hole diameter. For tamped cartridges, the charge diameter will be between the cartridge diameter and the blast hole diameter, depending on the degree of tamping. For untamped cartridges, the charge diameter is equal to the cartridge diameter. When blasting with ANFO (ammonium nitrate/fuel oil mixture) or other low density blasting agents with densities near 53 lb/ft^3 (0.85 g/cm^3), in typical rock with a density near 170 lb/ft^3 (2.7 g/cm^3, the normal burden is approximately 25 times the charge diameter. When using denser products such as slurries or dynamites with densities near 75 lb/ft^3 (1.2 g/cm^3), the normal burden is approximately 30 times the charge diameter. These are first approximations, and field testing usually results in adjustments to these values.

The burden-to-charge-diameter ratio is seldom less than 20 or seldom more than 40, even in extreme cases. For instance, when blasting with a low density blasting agent such as ANFO in a dense formation such as basalt, the desired burden may be about 20 times the charge diameter. When blasting with denser slurries or dynamites in low density formations such as sandstones, the burden may approach 40 times the charge diameter. Table 19-2 summarizes these approximations.

Table 19-2.—Approximate burden charge diameter ratios for bench blasting

Material	Ratio
ANFO (density 53.1 lb/ft^3, 0.85 g/cm^3)	
Light rock (density ~ 137.3 lb/ft^3, 2.2 g/cm^3)	28
Average rock (density ~ 168.6 lb/ft^3, 2.7 g/cm^3)	25
Dense rock (density ~ 199.8 lb/ft^3, 3.2 g/cm^3)	23
Slurry, dynamite (density ~ 199.7 lb/ft^3, 3.2 g/cm^3)	
Light rock (density ~ 137.3 lb/ft^3, 2.2 g/cm^3,)	33
Average rock (density ~ 168.6 lb/ft^3, 2.7 g/cm^3)	30
Dense rock (density ~ 199.8 lb/ft^3, 3.2 g/cm^3)	27

High-speed photographs of blasts show that flexing of the burden plays an important role in rock fragmentation. A relatively deep, narrow burden flexes and breaks more easily than a shallow, wider burden. Figure 19-11 shows the difference between using a 6-inch (150-mm) blast hole and a 12.25-inch (310-mm) blast hole in a 40-foot (12-m) bench with a burden-to-charge-diameter ratio of 30 and appropriate subdrilling and stemming dimensions. Note the inherent stiffness of the burden with a 12.25-inch (310-mm) blast hole as compared to a 6-inch (150-mm)

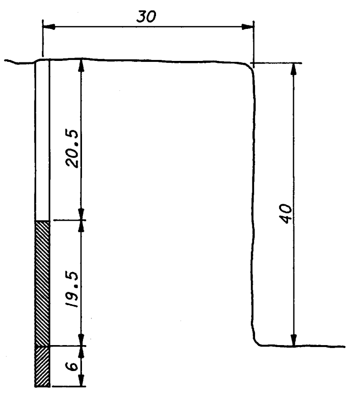


Figure 19-11.—Comparison of a 12¼-inch- (300-mm) diameter blast hole (stiff burden) on the left with a 6-inch- (150-mm) diameter blast hole (flexible burden) on the right in a 40-foot (12-m) bench.

blast hole. Based on this, lower burden-to-charge-diameter ratios should be used as a first approximation when the blast hole diameter is large in comparison to the bench height. Care must be taken that the burden ratio is not so small as to create violence. Once the ratio has been determined, it becomes the basis for calculating subdrilling, collar distance (stemming), and spacing.

Subdrilling

Subdrilling is the distance drilled below the floor level to ensure that the full face of rock is removed. Where there is a pronounced parting at floor level for the explosive charge to conveniently break, subdrilling may not be required. In coal strip mining, it is common practice to drill down to the coal and then backfill a foot or two before loading explosives, resulting in a negative subdrill. In most surface blasting jobs, some subdrilling is necessary to make sure the shot pulls to grade. In most construction blasting, subdrilling is generally limited to 10 percent or less of the bench height. In blasting for civil engineering structures where a final grade is specified, subdrilling of the final lift is severely restricted. The final lift in structural excavations is usually limited to 5 or 10 feet (1.5 to 3 m). Subdrilling is not allowed in a 5-foot (1.5-m) lift and is limited to 2 feet (0.6 m) for the 10-foot (3-m) lift. To prevent damage to the foundation, the diameter of the blast hole is limited to 3.5 inches (90 mm).

Priming the explosive column at the toe level (bottom of the drill hole) gives maximum confinement and normally gives the best breakage. Toe priming usually requires less subdrilling than collar priming. Toe priming should be restricted in structural foundations because of the potential damage to the unshot rock.

Excessive subdrilling is unnecessary, expensive, and may cause excessive ground vibrations because of the high degree of confinement of the explosive in the bottom of the blast hole, particularly when the primer is placed in the bottom of the hole. In multiple-bench operations, excessive subdrilling may cause undue fracturing in the upper portion of the bench below, creating difficulties in collaring holes in the lower bench. Insufficient subdrilling causes a high bottom, resulting in increased wear and tear on equipment and expensive secondary blasting or hand excavation in structural foundations.

Collar Distance (Stemming)

Collar distance is the distance from the top of the explosive charge to the collar of the blast hole. This zone usually is filled with an inert material called stemming to give some confinement to the explosive gases and to reduce airblast. A well-graded, crushed gravel works best as stemming, but it is common practice to use drill cuttings because of availability and economics. Collar distances that are too short result in excessive violence in the form of airblast and flyrock and may cause backbreak (breaking beyond the back wall). Collar distances that are too long create large blocks in the upper part of the muck pile. The selection of a collar distance is often a tradeoff between fragmentation and the amount of airblast and flyrock that can be tolerated. This is true especially where the upper part of the bench contains rock that is difficult to break or is of a different type. The difference between a violent blast and one that fails to fragment the upper zone properly may be a matter of only a few feet of stemming. Collar or direct priming (placing the primer at or near the collar of the blast hole with the blasting cap pointing toward the bottom of the hole) of blast holes normally causes more violence than center or toe priming and requires the use of a longer collar distance.

Field experience has shown that a collar distance equal to 70 percent of the burden is a good first approximation. Careful observation of airblast, flyrock, and fragmentation will enable further refinement of this dimension. Where adequate fragmentation in the collar zone cannot be attained while still controlling airblast and flyrock, deck charges or satellite (mid-spaced) holes may be required (figure19-4).

A deck charge is an explosive charge near the top of the blast hole, separated from the main charge by inert stemming. If large blocky materials are being created in the collar zone and less stemming would cause excessive airblast or flyrock, the main charge should be reduced slightly and a deck charge added. The deck charge is usually shot on the same delay as the main charge or one delay later. Do not place the deck charge too near the top of the blast hole, or excessive flyrock may result. Alternatively, short satellite holes between the main blast holes can be used. The diameter of the satellite holes is usually smaller than the main blast holes, and the satellite holes are loaded with a light charge of explosives.

Collar distance is a very important blast design variable. One violent blast can permanently alienate neighbors. In a delicate situation, it may be best to start with a collar distance equal to the burden and gradually reduce this if, conditions warrant. Collar distances greater than the burden are seldom necessary.

Spacing

Spacing is defined as the distance between adjacent blast holes, measured perpendicular to the burden. Where the rows are blasted one after the other as in figure 19-8, the spacing is measured between holes in a row. Where the blast progresses at an angle to the original free face, as in figure 19-9, the spacing is measured at an angle from the original free face.

Spacing is calculated as a function of the burden and also depends on the timing between holes. Spacing that is too close causes crushing and cratering between holes, large blocks in the burden, and toe problems. Spacing that is too wide causes inadequate fracturing between holes, toe problems, and is accompanied by humps on the face (figure 19-12).

When the holes in a row are initiated on the same delay period, a spacing equal to twice the burden usually will pull the round satisfactorily. The V-cut round in

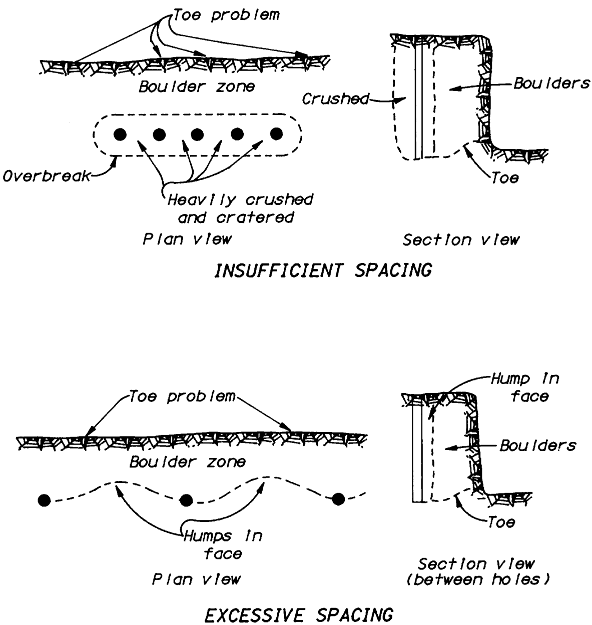


Figure 19-12.—Effects of too small and too large spacing.

figure 19-9 illustrates simultaneous initiation within a row, with the rows being the angled lines of holes fired on the same delay. The true spacing is twice the true burden, even though the holes originally were drilled on a square pattern.

Field experience has shown that the use of millisecond (ms) delays between holes in a row results in better fragmentation and also reduces ground vibrations produced by the blast. When ms delays are used between holes in a row, the spacing-to-burden ratio must be reduced to somewhere between 1.2 and 1.8, with 1.5 being a good first approximation. Various delay patterns may be used within the rows, including alternate delays (figure 19-13) and progressive delays (figure 19-14). Generally, large diameter blast holes require lower spacing-to-burden ratios (usually 1.2 to 1.5 with ms delays) than small diameter blast holes (usually 1.5 to 1.8). Because of the geology complexities, the interaction of delays, differences in explosive and rock strengths, and other variables, the proper spacing-to-burden ratio must be determined through onsite experimentation, using the preceding values as first approximations.

Except when using controlled blasting techniques such as smooth blasting and cushion blasting, described later, the spacing should never be less than the burden.

Hole Depth

In any blast design, the burden and the blast hole depth (or bench height) must be reasonably compatible. The rule of thumb for bench blasting is that the hole depth-to-burden ratio should be between 1.5 and 4.0. Hole depths

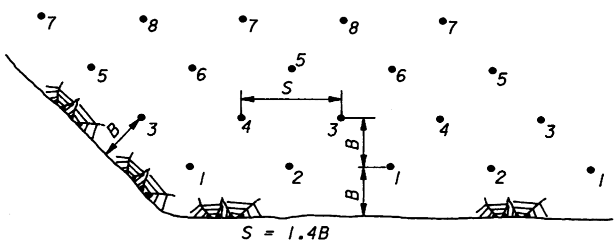


Figure 19-13.—Staggered blast pattern with alternate delays (blast hole spacing, S, is 1.4 times the burden, B).

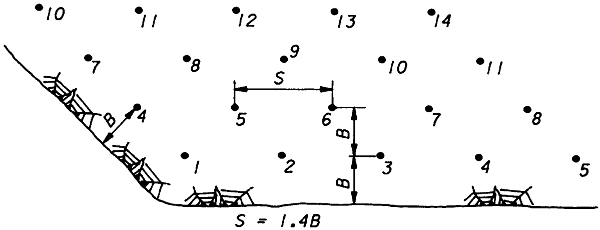


Figure 19-14.—Staggered blast pattern with progressive delays (blast hole spacing, S, is 1.4 times the burden, B).

less than 1.5 times the burden cause excessive air blast and fly rock and, because of the short, thick shape of the burden, give coarse and uneven fragmentation. Where operational conditions require a ratio of less than 1.5, a primer should be placed at the toe of the bench to assure maximum confinement. Keep in mind that placing the primer in the subdrill can cause increased ground vibrations and unacceptably irregular final grades for engineering structures. If the use of a hole depth-to-burden ratio of less than 1.5 is necessary or specified, consideration should be given to increasing the bench height or using smaller drill hole diameters.

Hole depths greater than four times the burden are also undesirable. The longer a hole is with respect to its diameter, the more error there will be in the hole location at toe level (hole wandering), the most critical portion of the blast. A poorly controlled blast will result. Extremely long, slender holes have been known to intersect.

High benches with short burdens can also create safety hazards, such as small equipment having to drill the front row of holes near the edge of a high ledge or a small shovel having to dig at the toe of a precariously high face. The obvious solution to this problem is to use a lower bench height. There is no real advantage to a high bench. Lower benches yield more efficient blasting results, lower drilling costs, less chance of cutoffs, and are safer. If it is impractical to reduce the bench height, larger rock handling and drilling equipment should be used, effectively reducing the blast hole depth-to-burden ratio.

A major problem with long slender charges is the greater potential for cutoffs in the explosive column. If it is necessary to use blast designs with large hole depth-to-burden ratios, multiple priming should be used as insurance against cutoffs.

Delays

Millisecond delays are used between charges in a blast round to:

- To ensure that a proper free face is developed to enable the explosive charge to efficiently fragment and displace the rock

- To enhance fragmentation between adjacent holes

- To reduce the ground vibrations created by the blast

Numerous delay patterns exist, several of which are covered in figures 19-8, 19-9, 19-13, and 19-14. Keep in mind the following:

- The delay time between holes in a row should be between 1 and 5 ms per foot (0.3 m) of burden. Delay times less than 1 ms per foot of burden cause premature shearing between holes, resulting in coarse fragmentation. If an excessive delay time is used between holes, rock movement from the first hole prevents the adjacent hole from creating additional fractures between the two holes. A delay of 3 ms per foot (0.3 m) of burden gives good results in many types of rock.

- The delay time between rows should be two to three times the delay time between holes in a row. To obtain good fragmentation and to control fly rock, a sufficient delay is needed so that the burden from previously fired holes has enough time to move forward to accommodate moving rock from subsequent rows. If the delay between rows is too short, movement in the back rows will be upward rather than outward (figure19-15).

- Where airblast is a problem, the delay between holes in a row should be at least 2 ms per foot of spacing. This prevents airblast from one charge from adding to that of subsequent charges as the blast proceeds down the row.

- For controlling ground vibrations, most regulatory authorities consider two charges to be separate events if they are separated by a delay of 9 ms or more.

These rules-of-thumb generally yield good blasting results. When using surface delay systems such as detonating cord connectors and sequential timers, the chances for cutoffs will be increased. To solve this problem, in-hole delays should be used in addition to the surface delays. When using surface detonating cord connectors, the use of 100-ms delays in each hole is suggested. This will cause ignition of the in-hole delays well in advance of

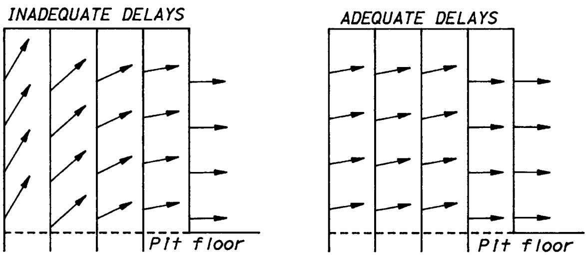


Figure 19-15.—The effect of inadequate delays between rows.

rock movement, thus minimizing cutoffs. The same effect can be accomplished with a sequential timer by avoiding the use of electric caps with delays shorter than 75 to 100 ms.

It is best if all the explosive in a blast hole is fired as a single column charge. When firing large blast holes in populated areas, two or more delays within a blast hole can be used to reduce ground vibrations. Blast rounds of this type can become quite complex.

All currently used delay detonators employ pyrotechnic delay elements that depend on a burning powder train for their delay. Although these delays are reasonably accurate, overlaps have been known to occur. When it is essential that one charge fires before an adjacent charge, such as in a tight corner of a blast, it is a good idea to skip a delay period.

Powder Factor

Powder factor, or pounds of explosive per cubic yard (kg/m^3) of rock, is not the best tool for designing blasts.

Blast designs should be based on the dimensions discussed earlier in this chapter. Powder factor is a necessary calculation for cost accounting purposes. In construction blasting where the excavated material has little or no inherent value, powder factor is usually expressed in pounds of explosives per cubic yard of material broken. Powder factors for surface blasting can vary from 0.25 to 2.5 pounds per cubic yards (lb/yd^3 [0.1 to 1.1 kg/m^3]), with 0.5 to 1.0 lb/yd^3 (0.2 to 0.45 kg/m^3) being most typical.

Powder factor for a single blast hole is calculated by the following:

$$\boldsymbol{P.F.} = \frac{\boldsymbol{L}(0.340\boldsymbol{d})(\boldsymbol{D}^2)}{27\boldsymbol{BSH}}$$

P.F. = Powder factor in pounds of explosive per cubic yard of rock
L = Length of explosive charge in feet
d = Density of explosive charge in grams per cubic centimeter
D = Charge diameter in inches
B = Burden in feet
S = Spacing in feet
H = Bench height

A comparable formula can be developed for metric equivalents. Many companies that provide explosives also publish tables that give loading densities in pounds per foot of blast hole for different combinations of d and D. A common nomograph found in most blasting handbooks can be used to calculate the density in pounds per foot of borehole. The powder factor is a function of the explosive type, rock density, and geologic structure. Table 19-3 includes typical powder factors for surface blasting.

Table 19-3.—Typical powder factors for surface blasting

Rock breakage difficulty	Powder factor	
	(lb/yd^3)	(kg/m^3)
Low	0.25-0.40	0.1-0.18
Medium	0.40- 0.75	0.18-0.34
High	0.75-1.25	0.34-0.57
Very high	1.25-2.50	0.57-1.14

Higher energy explosives, such as those containing large amounts of aluminum, can break more rock per unit weight than lower energy explosives. Most of the commonly (lower energy) used explosive products have similar energy values and, thus, have similar rock breaking capabilities. Soft, low density rock requires less explosive than hard, dense rock. Large hole patterns require less explosive per volume of rock because a larger portion of stemming is used. Poor powder distribution in larger diameter blast holes frequently results in coarser fragmentation. Massive rock with few existing planes of weakness requires a higher powder factor than a rock unit with numerous, closely spaced joints or fractures. The more free faces a blast has to break to, the lower the powder factor requirement. A corner cut (figure19-8), with two vertical free faces, requires less powder than a box cut (figure19-9) or angled cuts (figure19-16) with only one vertical free face, which, in turn, will require less powder than a shaft sinking type or parallel cut (figure19-16) where there are no free faces. In a shaft sinking cut, it is desirable to open a second free face by

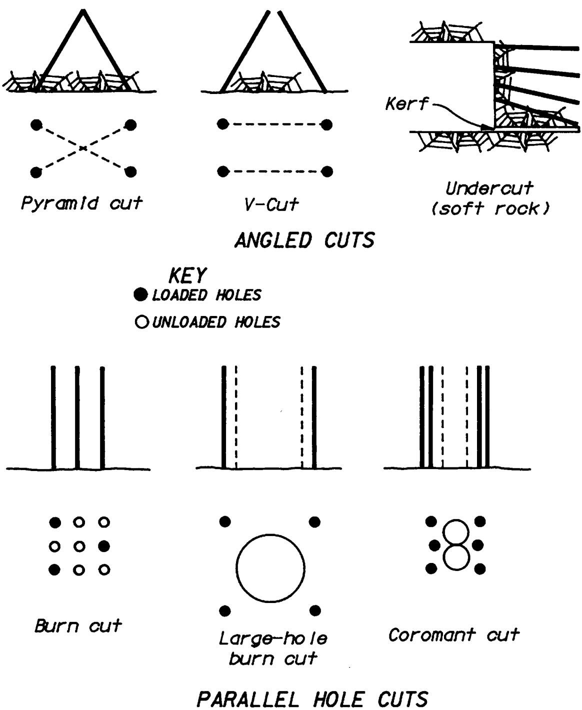


Figure 19-16.—Types of opening cuts.

creating a V-cut somewhere near the center of the round. V-cuts are discussed in the “Underground Blasting” section of this chapter.

When blasted materials have an inherent value per ton, such as aggregate or ore, powder factors are expressed as unit weight of explosives per ton of rock or tons of rock per unit weight of explosive.

Secondary Blasting

Some primary blasts will result in fragments too large to be handled efficiently by the loading equipment and will cause plugging of crushers or preparation plants. Secondary fragmentation techniques must be used to break these oversize fragments. If fragments are too large to be handled, the loader operator will set the rock aside for treatment. Identifying material large enough to cause crusher plugging is not always easy. The loader operator must be knowledgeable enough to watch for material that is small enough for convenient loading but that is too large for the crusher.

Secondary fragmentation can be accomplished by:

- A heavy ball (headache ball) suspended from a crane may be dropped repeatedly on the oversize fragment until it breaks. This is a relatively inefficient method, and breaking a large or tough (nonbrittle) rock may take considerable time. This method is adequate where the number of oversize fragments is small.

- A hole may be drilled into the oversize fragment and a hydraulic wedging device inserted to split the rock. This is also a slow method but may be satisfactory where a limited amount of secondary fragmentation is necessary. An advantage to this method is that it does not create the flyrock associated with explosives and, to some degree, with the headache ball method.

- An explosive may be packed loosely into a crack or depression in the oversize fragment then covered with a damp earth material and fired. This type of charge is called a mudcap, plaster, or adobe charge. This method is inefficient because of the limited explosive confinement and the relatively large amount of explosives required. Other results are excessive noise, flyrock, and often, inadequate fragmentation. This method is also hazardous because the primed charge lying on the surface is prone to accidental initiation by external impacts from falling rocks or equipment. External charges should be used to break boulders only where drilling is impractical.

- The most efficient method of secondary fragmentation is through the use of small (1- to 3-inch [25- to 75-mm]) blast holes. The blast hole is normally collared at the most convenient location, such as a crack or depression in the rock, and is directed toward the center of the mass. The hole is drilled two-thirds to three-fourths of the way through the rock. Because the powder charge is surrounded by free faces, less explosive is required to break a given amount of rock than in primary blasting. One-quarter pound per cubic yard (0.1 kg/m^3) usually is adequate. Careful location of the charge is more important than its precise size. When in doubt, it is best to estimate on the low side and under load the hole. For larger fragments, it is best to drill several holes and distribute the explosive charge rather than place the entire charge in a single hole. All secondary blast holes should be stemmed. Usually, secondary blasts are more violent than primary blasts. Any type of initiation system may be used to initiate a secondary blast and for connecting large numbers of oversize fragments. Detonating cord often is used where noise is not a problem.

Although secondary blasting employs relatively small charges, the potential safety hazards must not be under-estimated. Usually, there is more flyrock, and the flyrock is less predictable than with primary blasting.

Underground Blasting

Underground blast rounds are divided into two basic categories:

(1) Heading, drift, or tunnel rounds, where the only free face is the surface from which the holes are drilled

(2) Bench or stope rounds, where there is one or more free faces in addition to the face where the blast holes are drilled.

In the second category, blast rounds are designed in the same manner as surface blast rounds. Only the blast rounds that are in the first category (those with only one initial free face) are discussed below.

Opening Cuts

The initial and most critical part of a heading round is the opening cut. The essential function of this cut is to provide additional free faces where the rock can be broken. Although there are many specific types of opening cuts and the terminology can be quite confusing, all opening cuts fall into one of two classifications—angled cuts or parallel hole cuts (figure 19-16).

An angled cut, also referred to as a V-cut, draw cut, slab cut, or pyramid cut, breaks out a wedge of rock to create an opening that the remaining burden can move into. Angled cuts are very difficult to drill accurately. The bottoms of each pair of cut holes should be as close together as possible but must not cross. If they cross, the depth of round pulled will be less than designed. If the hole bottoms are more than a foot or so apart, the round may not pull to the proper depth. The angle between the cut holes should be 60 degrees or more to minimize bootlegging. One method to ensure getting a standard angle cut is to supply the drillers with a template with the proper spacing and angles for the angled holes. The selection of the specific type of angled cut is a function of the rock, the type of drilling equipment, and the philosophy of the blasting supervisor. A considerable amount of trial and error usually is involved in determining the best angle cut for a specific site. In small diameter tunnels with narrow headings, it is often impossible to position the drill properly to drill an angled cut. In this case, a parallel hole cut must be used.

Parallel hole cuts, also referred to as Michigan cuts, Cornish cuts, shatter cuts, burn cuts, or Coromant cuts, are basically a series of closely spaced holes, some loaded and some unloaded (figure 19-17) that, when fired, pulverize and eject a cylinder of rock to create an opening where the remaining burden can be broken. Because they require higher powder factors and more drilling per volume of rock blasted, the use of parallel hole cuts usually is restricted to narrow headings where there is not enough room to drill an angled cut.

Parallel hole cuts involve more drilling than angled cuts because the closely spaced holes break relatively small volumes of rock. Parallel cuts are relatively easy to drill because the holes are parallel. Like angled cuts, accurately drilled parallel hole cuts are essential if the blast round is to be effective. Some drill jumbos have automatic controls to ensure that holes are drilled parallel. Drill jumbos of this type are a good investment where parallel hole cuts will be drilled routinely. A template also may

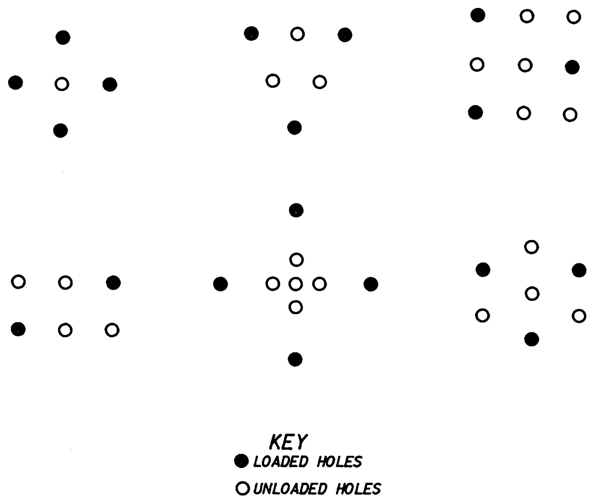


Figure 19-17.—Six designs for parallel hole cuts.

be used in drilling a parallel hole cut. The selection of the type of parallel hole cut also depends on the rock, the type of drilling equipment, and the philosophy of the blasting supervisor. As with angled cuts, trial and error usually is involved in determining the best parallel hole cut for a specific site.

All types of opening cuts must pull to the planned depth because the remainder of the round will not pull more deeply than the opening cut. In blasting with burn cuts, care must be exercised to prevent overloading of the burn holes because overloading may cause the cut to “freeze” or not pull properly. The design of the cut depends on the type of rock and often must be designed and refined by trial and error.

The advantage of a large central hole is that the hole gives a dependable free face where succeeding holes can break. This free face is not always obtained with standard burn cuts. The large central hole ensures a more dependable and deeper pull of the blast round. The disadvantages of a large central hole are the requirements for the proper equipment to drill the large central hole as well as extra time. Intermediate-sized holes, usually 4 to 5 inches (100-130 mm) in diameter, are sometimes drilled using the standard blast hole equipment as a compromise.

In some soft materials, particularly coal, the blasted cut is replaced by a sawed kerf, usually at floor level (figure 19-18). In addition to giving the material a dependable void to break into, the sawed cut ensures that the floor of the opening will be smooth.

Blasting Rounds

Once the opening cut has established the necessary free face, the remainder of the blast holes must be designed so that the burden successively breaks into the void space. The progression of the blast round should provide a proper free face parallel or nearly parallel to the hole at its time of initiation. Figure 19-19 gives the typical nomenclature for blast holes in a heading round.

The holes fired immediately after the cut holes are called the relievers. The burden between these holes must be planned carefully. If the burden is too small, the charges will not pull their share of the round. If the burden is too large, the round may freeze because the rock will have insufficient space to expand. After several relievers have been fired, the opening usually is large enough to permit the remainder of the blast to be designed, as discussed under "Surface Blasting." Where heading rounds are large, the burden and spacing ratio usually is slightly less

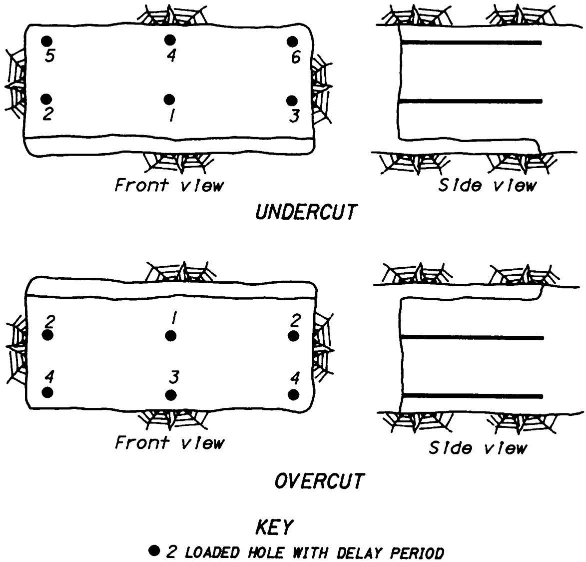


Figure 19-18.—Blast round for soft rock using sawed kerf (numbers on loaded holes show delay in milliseconds.)

than that for surface blasts. In small headings where space is limited, the burden and spacing ratio will be still smaller. Trial and error plays an important part in this type of blast design.

The last holes to be fired in an underground round are the back holes at the top, the rib holes at the sides, and the lifters at the bottom of the heading. Unless controlled blasting is used (discussed below), the spacing between these perimeter holes is about 20 to 25 blast hole diameters. Figure 19-20 shows two typical angled cut blast rounds. After the initial wedge of rock is extracted by the cut, the angles of the subsequent blast holes are

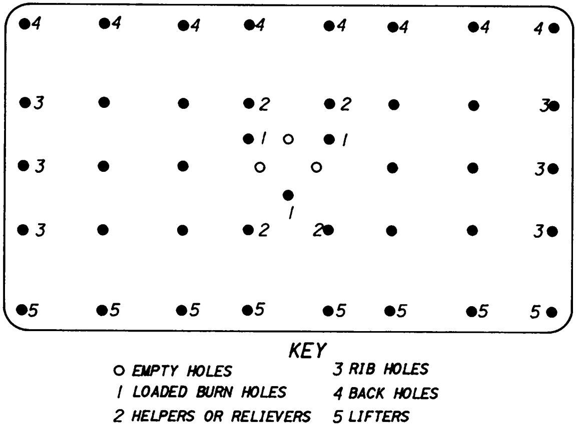


Figure 19-19.—Nomenclature for blast holes in a heading round.

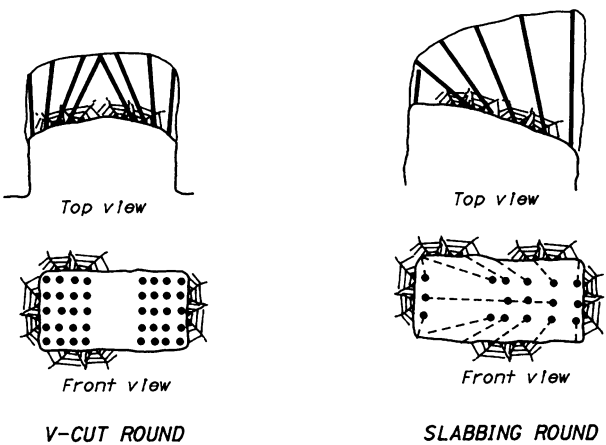


Figure 19-20.—Angled cut blast rounds.

reduced progressively until the perimeter holes are parallel to the heading or looking slightly outward. In designing burden and spacing dimensions for angled cut blast rounds, the location of the bottom of the hole is considered rather than the collar.

Figure 19-21 shows two typical parallel hole cut blast rounds. These rounds are simpler to drill than angled cut rounds. Once the central opening has been established, the round resembles a bench round turned on its side. Figure 19-22 shows a comparison of typical muckpiles obtained from V-cut and burn-cut blast rounds. Burn cuts give more uniform fragmentation and a more compact muckpile than V-cuts. V-cut muckpiles are more spread out and vary in fragmentation. Powder factors and the amount of drilling required are higher for burn cuts.

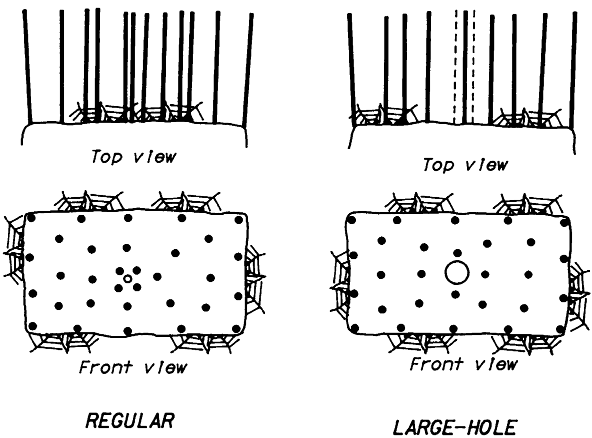


Figure 19-21.—Parallel hole cut blast rounds.

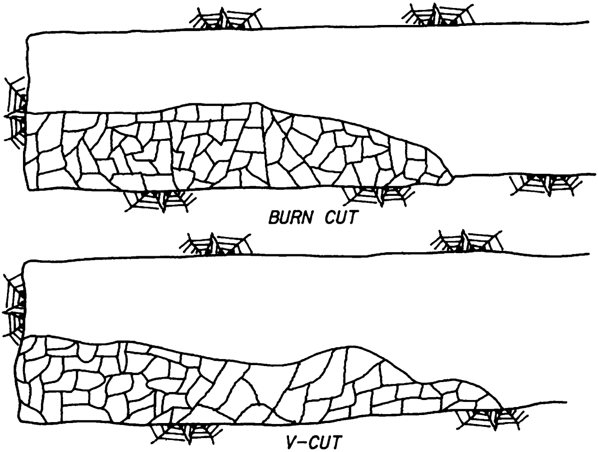


Figure 19-22.—Fragmentation and shape of muckpile as a function of type of cut.

Delays

Two series of delays are available for underground blasting—millisecond delays, the same as those used in surface blasting; and slow, or tunnel delays. The choice of delay depends on the size of the heading being blasted and on the fragmentation and type of muckpile desired. Slow delays give coarser fragmentation and a more compact muckpile. Millisecond delays give finer fragmentation and a looser muckpile (figure19-23). In small headings where space is limited, particularly when using parallel hole cut rounds, slow delays are necessary to ensure that the rock from each blast hole has time to be ejected before the next hole fires. Where delays intermediate between millisecond delays and slow delays is desired, use millisecond delays and skip delay periods.

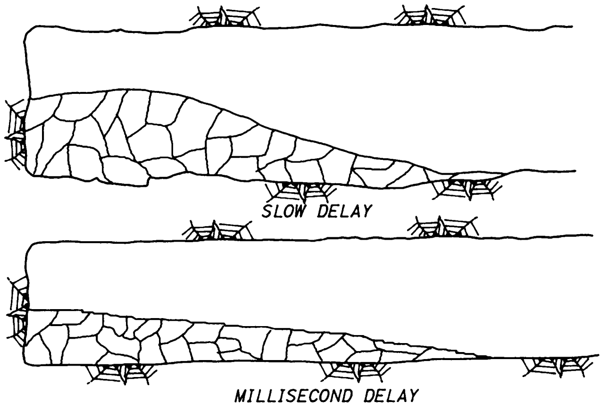


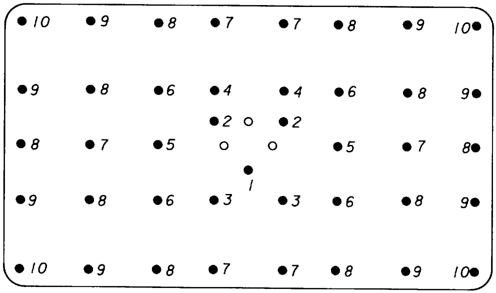
Figure 19-23.—Fragmentation and shape of muckpile as a function of delay.

In an underground blast round, the delay pattern must be designed so that at the time of firing each hole has a good free face where the burden can be displaced. Figure 19-24 shows a typical delay pattern for a burn cut blast round in a heading in hard rock. Figure 19-25 shows a delay pattern for a V-cut blast round.

The shape of the muckpile is affected by the order that the delays are fired (figure19-26). If the blast is designed so that the back holes at the roof are fired last, a cascading effect is obtained, resulting in a compact muckpile. If the lifters are fired last, the muckpile will be displaced away from the face.

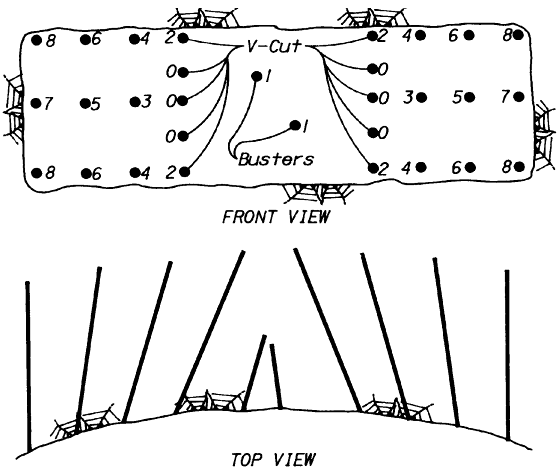
Powder Factor

As with surface blasting, powder factors for underground blasting vary depending on several things. Powder factors for underground blasting may vary from 1.5 to



KEY
O UNLOADED HOLES
●9 LOADED HOLE WITH DELAY PERIOD

Figure 19-24.—Typical burn cut blast round delay pattern (numbers on loaded holes show delay in milliseconds).



KEY
O UNLOADED HOLES
●9 LOADED HOLE WITH DELAY PERIOD

Figure 19-25.—Typical V-cut blast round delay pattern (numbers on loaded holes show delay in milliseconds).

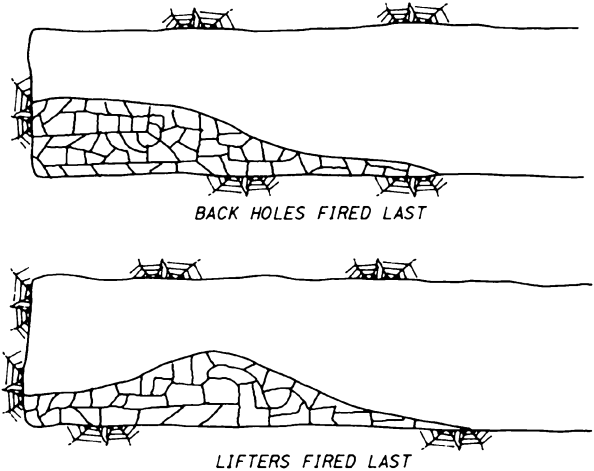


Figure 19-26.—Shape of muckpile as a function of firing order.

12 lb/yd^3 (0.7 to 5.5 kg/m^3). Soft, light weight rock, headings with large cross sections, large blast holes, and angle cut rounds all tend to require lower powder factors than hard, dense rock, small headings, small blast holes, and parallel hole cuts.

Controlled Blasting Techniques

The term, controlled blasting, is used to describe several techniques that limit damage to the rock at the perimeter of the excavation by preventing the force of a blast from continuing into the side walls. Normal blasting propagates cracks into the surrounding rock. These cracks can reduce the stability of the opening. The purpose of controlled blasting is to reduce this perimeter cracking and, thus, increase stability of the final surface. Much of the following discussion on controlled blasting techniques has been determined through years of on-the-job testing and evaluation. The results of controlled blasting are a function of the geology, especially the number and orientation of joint and fracture planes and the quality of the final rock surface that is required.

Line Drilling

Line drilling consists of drilling a row of closely spaced holes along the final excavation limits and not loading the holes with explosive. The line-drilled holes provide a plane of weakness to which the final row of blast holes can break and also reflect a portion of the blast's shock wave. Line drilling is used mostly in small blasting operations and involves small holes in the range of 2 to 3 inches (50-75 mm) in diameter. Line drilling holes are spaced (center to center) two to four diameters apart but are more closely spaced at the corners. The maximum practical depth to which line drilling can be done is governed by how accurately the alignment of the holes can be held at depth (seldom more than 30 feet [10 m]).

To further protect the final perimeter, the blast holes adjacent to the line drill are spaced more closely and loaded more lightly than the rest of the blast, and deck charges are used as necessary. Best results are obtained in a homogeneous rock with few joints or bedding planes or when the holes are aligned with a major joint plane. Line drilling is sometimes used in conjunction with presplitting where the corners are line drilled and the remainder of the perimeter is presplit.

The use of line drilling is limited to jobs where even a light load of explosives in the perimeter holes would cause unacceptable damage. The results of line drilling are often unpredictable, the cost of drilling is high, and the results depend on the accuracy of the drilling.

Presplitting

Presplitting, sometimes called preshearing, is similar to line drilling except that the holes are drilled slightly farther apart and are loaded very lightly. Presplit holes are fired before any of the adjacent main blast holes. The light explosive charges propagate a crack between the holes. In badly fractured rock, unloaded guide holes may be drilled between the loaded holes. The light powder load may be obtained by using specially designed slender cartridges, partial or whole cartridges taped to a detonating cord downline, an explosive cut from a continuous reel, or heavy grain detonating cord. A heavier charge of tamped cartridges is used in the bottom few feet of hole. Figure 19-27 shows three types of blast hole loads used for presplitting. Cartridges ¾ or ⅞ inch by 2 feet (19 or 22 mm by 0.6 m) connected with couplers are available. Manufacturers now produce continuous, small diameter, long-tubular water gel columns for loading presplit holes. The diameter of these continuous tubular columns usually is 1 inch (25 mm), and they come with a built-in downline. They can be loaded easily in rough and inclined holes. The continuous column presplit explosives produce increased loading rates and reduced labor costs.

If possible, stem completely around and between the cartridges in the blast hole; and, although not essential, fire in advance of the main blast. The maximum depth for a single presplit is limited by the accuracy of the drillholes and is usually about 50 feet (15 m). Depths between 20 and 40 feet (6 and 12 m) are recommended. A deviation of greater than 6 inches (152 mm) from the desired plane or shear will give inferior results. Avoid presplitting too

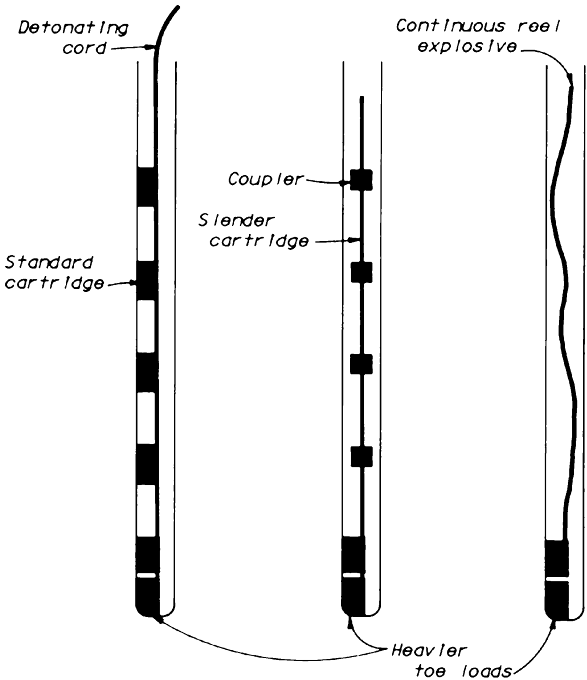


Figure 19-27.—Typical presplit blast hole loading.

far ahead of the production blast. If possible, presplit a short section and dig that section out so that the quality of the presplit can be checked. If the presplit results are unsatisfactory, adjustments can be made in subsequent blasts.

Presplitting is usually done in a separate operation and well in advance of drilling and loading the main blast. The presplit holes can be fired with the main blast by firing the presplit holes on the first delay period. The increased hole spacing compared with line drilling reduces drilling costs. Table 19-4 gives parameters for presplitting.

Smooth Blasting

Smooth blasting, also called contour blasting, perimeter blasting, or sculpture blasting, is the most widely used method of controlling overbreak in underground openings such as drifts and stopes (benches). Smooth blasting is similar to presplitting in that an additional row of holes is drilled at the perimeter of the excavation. These holes contain light loads and are more closely spaced than the other holes in the round (figure19-28). The light powder load usually is continuously "string loaded" (loaded end to end) with slender cartridges or continuous reel explosive is used as shown in figure 19-27. Unlike presplitting, the smooth blast holes are fired after the main blast. Usually, this is done by loading and connecting the entire round and firing the perimeter holes one delay later than the last hole in the round. The burden on the perimeter holes should be approximately 1.5 times the spacing. Table 19-5 gives parameters for smooth blasting.

A compromise for smooth wall blasting is to slightly reduce the spacing of the perimeter holes, compared to a standard design, and "string load" regular cartridges of explosive. The explosive column should be sealed with a tamping plug, clay plug, or other type of stemming to prevent the string-loaded charges from being extracted from the hole by charges on earlier delays. Smooth blasting reduces overbreak and reduces the need for ground support. These advantages usually outweigh the cost of the additional perimeter holes.

Table 19-4.—Parameters for presplitting

Hole diameter		Spacing		Explosive charge	
(inch)	(mm)	(feet)	(m)	(lb/ft)	(kg/m)
1.50-1.75	38-44	1.00-1.50	0.3-0.46	0.08-0.25	0.03-0.1
2.00-2.50	50-64	1.50-2.00	0.46-0.6	.08-0.25	0.03-0.1
3.00-3.50	75-90	1.50-3.00	0.6-1.0	0.13-0.50	0.05-0.23
4.00	100	2.00-4.00	0.6-1.2	0.25-0.75	0.23-0.34

Table 19-5.—Parameters for smooth blasting

Hole diameter		Spacing		Burden		Explosive charge	
(inch)	(mm)	(feet)	(m)	(feet)	(m)	(lb/ft)	(kg/m)
1.50-1.75	38-44	2.00	0.6	3.00	1	0.12-0.25	0.05-0.55
2.0	50	2.50	0.75	3.50	1.06	0.12-0.25	0.05-0.55

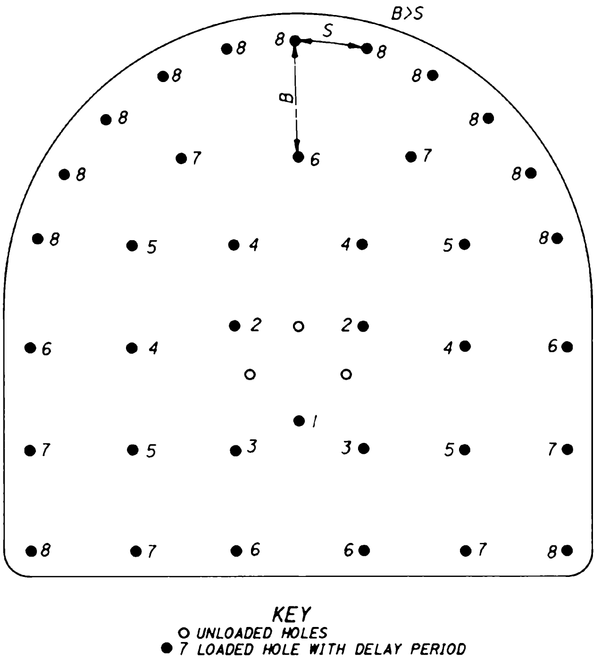


Figure 19-28.—Typical smooth blasting pattern (Burden, B, is larger than spacing, S). Numbers on holes show delay in milliseconds.

Cushion Blasting

Cushion blasting, also called trimming, slabbing, or slashing, is the surface equivalent of smooth blasting. Like other controlled blasting techniques, cushion blasting involves a row of closely spaced, lightly loaded holes at the perimeter of the excavation. Holes up to

6½ inches (165 mm) in diameter have been used in cushion blasting. Drilling accuracy with this larger size borehole permits depths of up to 90 feet (27 m) for cushion blasting. After the explosives have been loaded, stemming is placed in the void space around the charges for the entire length of the column. Stemming "cushions" the shock from the finished wall minimizing the stresses and fractures in the finished wall. The cushion blast holes are fired after the main excavation is removed (figure19-29). A minimum delay between the cushion blast holes is

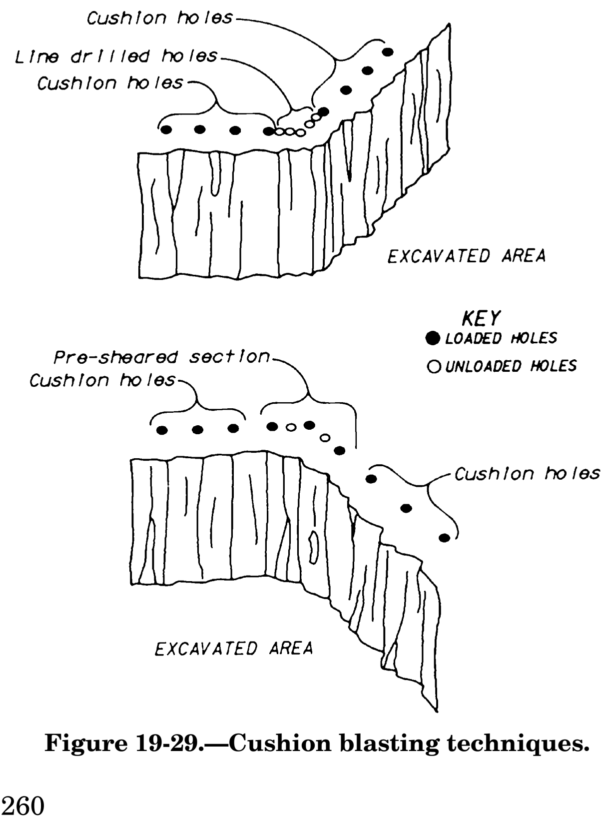


Figure 19-29.—Cushion blasting techniques.

desirable. The same loading techniques that apply to presplitting are used with cushion blasting. The burden on the cushion holes should always be less than the width of the berm being removed.

The large diameter holes associated with cushion blasting result in larger spacings as compared with presplitting reducing drilling costs. Better results can be obtained in poorly lithified and weathered formations than with presplitting, and the larger holes permit better alignment at depth. Table 19-6 gives parameters for cushion blasting.

Riprap Blasting Techniques

Riprap blasting follows the usual rules for blast designs. A few special techniques can aid production of quality riprap.

Powder distribution is the key to satisfactory results. If the above principles of good blast design are followed, satisfactory results should be achieved. The blast design must be monitored continually and changed to produce the best results.

Full column loading is usually the best for initial shots. The starting burden for riprap production should be larger than predicted from table 19-1. If a burden of 7 feet (2.1 m) is predicted, a starting burden of approximately 9 feet (2.7 m) should be tried. Further adjustments can be made to optimize the breakage.

In typical quarry blasting, a spacing of 1.5 times the burden is a good first estimate. The spacing should be about equal to the burden but not smaller than the burden for riprap. Either a square or a staggered drill

Table 19-6.—Parameters for cushion blasting

Hole diameter		Spacing		Burden		Explosive charge	
(inch)	(mm)	(feet)	(m)	(feet)	(m)	(lb/ft)	(kg/m)
2.00-2.50	50-64	3.00	1.0	4.00	1.2	0.08-0.25	0.03-0.1
3.00-3.50	75-90	4.00	1.2	5.00	1.5	0.13-0.5	0.05-0.2
4.00-4.50	100-115	5.00	1.5	6.00	1.8	0.25-0.75	0.1-0.3
5.00-5.50	127-140	6.00	1.8	7.00	2.1	0.75-1.00	0.3-0.45
6.00-6.50	152-165	7.00	2.1	9.00	2.7	1.00-1.50	0.45-0.7

pattern should be used. The square pattern often gives a coarser product but also may give more erratic toe conditions. Where possible, rows of blast holes should be perpendicular to the major vertical plane of weakness such as the primary vertical joint set.

If a reasonably well graded riprap is specified, it will be necessary to perform test or trial blasts. The product should be analyzed for gradation, and further test blasts made and gradations checked until the specifications have been met. If insufficient coarse product is produced, the burden and spacing of subsequent rounds should be increased by 1 foot (0.3 m) per round until the amount of coarse product is adequate. If toe problems occur before the amount of coarse product is adequate, reduce the burden and spacing again and increase the stemming or try deck loading in the upper part of the lift.

If too much coarse product is produced, determine whether the material comes from the top or bottom part of the bench face. If the material comes from the bottom or is generally well-distributed, decrease the burden and spacing in 1-foot (0.3-m) increments until the oversize is controlled. If the oversize comes from the top, try using satellite holes (short, smaller diameter holes between the main blast holes, figure19-4) or less stemming.

It is essential that each blast be laid out accurately and drilled and loaded so that there is a known baseline to adjust from if early results are not good. Keeping good records is essential for later adjustments to the blast design. Single row firing, at least for the early shots, also simplifies blast pattern adjustments.

In dry conditions, ANFO will work well. A good, fast, bottom primer should be used. In wet conditions, an emulsion, water gel, slurry, or gelatin dynamite should be used. The bottom of the blast hole should be primed. Excessive orange smoke from a blast indicates the need for a water resistant product or for a better proportional mix of the ANFO.

Bibliography

Dick, R.A., L.R. Fletcher, and D.V. Andrea, *Explosives and Blasting Procedures Manual.* Bureau of Mines Information Circular 8925, pp. 57-74, 1987.

E. I. du Pont de Nemours & Co., Inc., *Blaster's Handbook.* 16th edition, pp. 494, Wilmington, Delaware, 1978.

Glossary

A

Acoustical impedance - The mathematical expression characterizing a material energy transfer property. The product of a material unit density and sonic velocity.

Adobe charge - See mud cap.

Airblast - An airborne shock wave resulting from the detonation of explosives; may be caused by burden movement or the release of expanding gas into the air. Airblast may or may not be audible.

Airdox - System that uses 10,000 lb/in^2 compressed air to break undercut coal. Airdox will not ignite a gassy or dusty atmosphere.

Aluminum - A metal commonly used as a fuel or sensitizing agent in explosives and blasting agents; normally used in finely divided particle or flake form.

American Table of Distance - A quantity-distance table published by the Institute of Makers of Explosives (IME) as pamphlet No. 2, which specifies safe explosive storage distances from inhabited buildings, public highways, passenger railways, and other stored explosive materials.

Ammonium nitrate (AN) - The most commonly used oxidizer in explosives and blasting agents, NH_4NO_3.

ANFO - An explosive material consisting of a mixture of ammonium nitrate and fuel oil. The most commonly used blasting agent.

ATF - Bureau of Alcohol, Tobacco and Firearms, U.S. Department of the Treasury, which enforces explosives control and security regulations.

Axial priming - A system for priming blasting agents where a core of priming materials extends through most or all of the blasting agent charge length.

B

Back break - Rock broken beyond the limits of the last row of holes.

Back holes - The top holes in a tunnel or drift round.

Base charge - The main explosive charge in a detonator.

Bench - The horizontal ledge in a quarry face along where holes are drilled. Benching is the process of excavating where terraces or ledges are worked in a stepped sequence.

Binary explosive - An explosive based on two nonexplosive ingredients, such as nitromethane and ammonium nitrate. Materials are shipped and stored separately and mixed at the jobsite to form a high explosive.

Black powder - A low energy explosive consisting of sodium or potassium nitrate, carbon, and sulfur. Black powder is seldom used today because of the low energy, poor fume quality, and extreme sensitivity to sparks.

Blast - The detonation of explosives to break rock.

Blast area - The area near a blast within the influence of flying rock or concussion.

Blaster - A qualified person in charge of a blast. Also, a person (blaster-in-charge) who has passed test approved by Office of Surface Mining (OSM) which certifies the blaster's qualifications to supervise blasting activities.

Blasters' galvanometer - Blasters' multimeter; see galvanometer, multimeter.

Blast hole - A hole drilled in rock or other material for the placement of explosives.

Blasting agent - An explosive that meets prescribed criteria for insensitivity to initiation. For storage, any material or mixture consisting of a fuel and oxidizer, intended for blasting, not otherwise defined as an explosive, provided that the finished product, as mixed and packaged for use or shipment, cannot be detonated by means of a No. 8 test blasting cap when unconfined (ATF). For transportation, a material designed for blasting that has been tested in accordance with the Code of Federal Regulations (CFR) Title 49, Section 173.14a, and found to be so insensitive that there is very little probability of accidental initiation to explosion or transition from deflagration to detonation (Department of Transportation [DOT]).

Blasting cap - A detonator that is initiated by safety fuse (Mine Safety and Health Administration [MSHA]). See detonator.

Blasting circuit - The circuit used to fire one or more blasting caps.

Blasting crew - A crew whose job is to load explosive charges.

Blasting machine - Any machine built expressly for the purpose of energizing blasting caps or other types of initiators.

Blasting mat - See mat.

Blasting switch - A switch used to connect a power source to a blasting circuit.

Blistering - See mud cap.

Blockhole - A hole drilled into a boulder to allow the placement of a small charge to break the boulder.

Booster - A unit of explosive or blasting agent used for perpetuating or intensifying an explosive reaction. A booster does not contain an initiating device but is often cap sensitive.

Bootleg - That portion of a borehole that remains relatively intact after having been charged with explosive and fired. A bootleg may contain unfired explosive and may be hazardous.

Borehole (blast hole) - A drilled hole, usually in rock, that is loaded with explosives.

Borehole pressure - The pressure that the hot gases of detonation exert on the borehole wall. Borehole pressure is primarily a function of the density of the explosive and the heat of explosion.

Bridge wire - A very fine filament wire imbedded in the ignition element of an electric blasting cap. An electric current passing through the wire causes a sudden heat rise, causing the ignition element to be ignited.

Brisance - A property of an explosive roughly equivalent to detonation velocity. An explosive with a high detonation velocity has high brisance.

Bubble energy - The expanding gas energy of an explosive, as measured in an underwater test.

Bulk mix - An explosive material prepared for use without packaging.

Bulk strength - The strength of an explosive per unit volume.

Bulldoze - See mud cap.

Burden - The distance to the nearest free or open face from an explosive charge. There may be apparent burden and a true burden, the latter being measured in the direction broken rock will be displaced following firing of the explosive charge. Also, the amount of material to be blasted by a given hole in tons or cubic yards (m^3).

Burn cut - A parallel hole cut employing several closely spaced blast holes. Not all of the holes are loaded with explosive. The cut creates a cylindrical opening by shattering the rock.

Bus wires - The two wires joined to the connecting wire where the leg wires of the electric caps are connected in a parallel circuit. Each leg wire of each cap is connected to a different bus wire. In a series-in-parallel circuit, each end of each series is connected to a different bus wire.

Butt - See bootleg.

C

Cap - See detonator.

Capped fuse - A length of safety fuse with an attached blasting cap.

Capped primer - A package or cartridge of cap-sensitive explosive that is specifically designed to transmit detonation to other explosives and which contains a detonator (MSHA).

Cap sensitivity - The sensitivity of an explosive to initiation, relative to an IME No. 8 test detonator.

Carbon monoxide - A poisonous gas created by detonating explosive materials. Excessive carbon monoxide is caused by an inadequate amount of oxygen in the explosive mixture (excessive fuel).

Cardox - A system that uses a cartridge filled with liquid carbon dioxide, which when initiated by a mixture of potassium perchlorate and charcoal, creates a pressure adequate to break undercut coal.

Cartridge - A rigid or semirigid container of explosive or blasting agent of a specified length or diameter.

Cartridge count - The number of 1¼- by 8-inch (32- by 203-mm) cartridges of explosives per 50-pound (22.7-kg) case.

Cartridge strength - A rating that compares a given volume of explosive with an equivalent volume of straight nitroglycerin dynamite expressed as a percentage.

Cast primer - A cast unit of explosive, usually pentolite or composition B, commonly used to initiate detonation in a blasting agent.

Chambering - The process of enlarging a portion of blast hole (usually the bottom) by firing a series of small explosive charges. Chambering can also be done by mechanical or thermal methods.

Chapman-Jougeut (C-J) plane - The plane that defines the rear boundary of the primary reaction zone in a detonating explosive column.

Circuit tester - See galvanometer or multimeter.

Class A explosive - Defined by the U.S. Department of Transportation (DOT) as an explosive that possesses detonating or otherwise maximum hazard; such as, but not limited to, dynamite, nitroglycerin, lead azide, black powder, blasting caps, and detonating primers.

Class B explosive - Defined by DOT as an explosive that possesses flammable hazard, such as, but not limited to, propellant explosives, photographic flash powders, and some special fireworks.

Class C explosive - Defined by DOT as an explosive that contains Class A or Class B explosives, or both, as components but in restricted quantities. For example, blasting caps or electric blasting caps in lots of less than 1,000.

Collar - The top or opening of a borehole or shaft. To "collar" in drilling means the act of starting a borehole.

Collar distance - The distance from the top of the powder column to the collar of the blast hole, usually filled with stemming.

Column charge - A long, continuous charge of explosive or blasting agent in a borehole.

Commercial explosives - Explosives designed and used for commercial or industrial, rather than military applications.

Composition B - A mixture of RDX and TNT that has a density of 1.65 g/cm^3 and a velocity of 25,000 feet per second (7,622 m/sec), when cast. It is useful as a primer for blasting agents.

Condenser-discharge blasting machine - A blasting machine that uses batteries or magnets to energize one or more condensers (capacitors) whose stored energy is released into a blasting circuit.

Confined detonation velocity - The detonation velocity of an explosive or blasting agent under confinement such as in a borehole.

Connecting wire - A wire, smaller in gage than the lead wire, used in a blasting circuit to connect the cap circuit with the lead wire or to extend leg wires from one borehole to another. Usually considered expendable.

Connector - See MS connector.

Controlled blasting - Techniques used to control overbreak and produce a competent final excavation wall. See line drilling, presplitting, smooth blasting, and cushion blasting.

Cordeau detonant fuse - A term used to define detonating cord.

Cornish cut - See parallel hole cut.

Coromant cut - See parallel hole cut.

Coupling - The degree that an explosive fills the borehole. Untamped cartridges are decoupled; also capacitive and inductive coupling from power lines to an electric blasting circuit.

Coyote blasting - The practice of driving tunnels horizontally into a rock face at the foot of the shot. Explosives are loaded into these tunnels. Coyote blasting is used where it is impractical to drill vertically.

Critical diameter - The minimum diameter of any explosive for propagation of a stable detonation. Critical diameter is effected by confinement, temperature, and pressure on the explosive.

Crosslinking agent - The final ingredient added to a water gel or slurry, causing the material to change from a liquid to a gel.

Current limiting device - A device used to prevent arcing in electric blasting caps by limiting the amount or duration of current flow. Also used in a blasters' galvanometer or multimeter to ensure a safe current output.

Cushion blasting - A surface blasting technique used to produce competent slopes. The cushion holes, fired after the main charge, have a reduced spacing and employ decoupled charges.

Cushion stick - A cartridge of explosive loaded into a small diameter borehole before the primer. The use of a cushion stick is not generally recommended because of possible bootlegs.

Cut - An arrangement of holes used in underground mining and tunnel blasting providing a free face for the remainder of the round to break.

Cutoffs - A portion of a column of explosives that has failed to detonate owing to bridging or a shifting of the rock formation, often due to an improper delay system; also a cessation of detonation in detonating cord.

D

Dead pressing - Desensitization of an explosive caused by pressurization. Tiny air bubbles required for sensitivity are literally squeezed from the mixture.

Decibel - The unit of sound pressure commonly used to measure airblast from explosives. The decibel scale is logarithmic.

Deck - A small charge or portion of a blast hole loaded with explosives that is separated from other charges by stemming or an air cushion.

Decoupling - The use of cartridged products significantly smaller in diameter than the borehole. Decoupled charges are normally not used except in cushion blasting, smooth blasting, presplitting, and other situations where crushing is undesirable.

Deflagration - A subsonic but extremely rapid explosive reaction accompanied by gas formation and borehole pressure but without shock.

Delay blasting - The use of delay detonators or connectors that cause separate charges to detonate at different times rather than simultaneously.

Delay connector - A nonelectric, short-interval delay device for use in delaying blasts that are initiated by detonating cord.

Delay detonator - An electric or nonelectric detonator with a built-in element that creates a delay between the input of energy and the explosion of the detonator.

Delay electric blasting cap - An electric blasting cap with a built-in delay that delays cap detonation in predetermined time intervals from milliseconds up to a second or more.

Delay element - The portion of a blasting cap that causes a delay between the application of energy to the cap and the time of detonation of the base charge of the cap.

Density - The weight per unit volume of explosive, expressed as cartridge count or grams per cubic centimeter. See loading density.

Department of Transportation (DOT) - A Federal agency that regulates safety in interstate shipping of explosives and other hazardous materials.

Detaline System - A nonelectric system for initiating blasting caps where the energy is transmitted through the circuit by a low-energy detonating cord.

Detonating cord - A plastic covered core of high-velocity explosive, usually PETN, used to detonate explosives. The plastic covering is covered with various combinations of textiles and waterproofing.

Detonation - A supersonic explosive reaction where a shock wave propagates through the explosive accompanied by a chemical reaction that furnishes energy to sustain stable shock wave propagation. Detonation creates both a detonation pressure and a borehole pressure.

Detonation pressure - The head-on pressure created by the detonation proceeding down the explosive column. Detonation pressure is a function of the explosive density and the square of the explosive velocity.

Detonation velocity - See velocity.

Detonator - Any device containing a detonating charge that is used to initiate an explosive. Includes, but is not limited to, blasting caps, electric blasting caps, and nonelectric instantaneous or delay blasting caps.

Ditch blasting - See propagation blasting.

DOT - See Department of Transportation.

Downline - The line of detonating cord in the borehole that transmits energy from the trunkline down the hole to the primer.

Drilling pattern - See pattern.

Drop ball - Known also as a headache ball. An iron or steel weight held on a wire rope that is dropped from a height onto large boulders to break them into smaller fragments.

Dynamite - The high explosive invented by Alfred Nobel. Any high explosive where the sensitizer is nitroglycerin or a similar explosive oil.

E

Echelon pattern - A delay pattern that causes the true burden at the time of detonation to be at an oblique angle from the original free face.

Electric blasting cap - A blasting cap designed to be initiated by an electric current.

Electric storm - An atmospheric disturbance of intense electrical activity presenting a hazard in all blasting activities.

Emulsion - An explosive material containing substantial amounts of oxidizers dissolved in water droplets surrounded by an immiscible fuel. Similar to a slurry.

Exploding bridge wire (EBW) - A wire that explodes upon application of current. The wire takes the place of the primary explosive in an electric detonator. An exploding bridge wire detonator is an electric detonator that employs an exploding bridge wire rather than a primary explosive. An exploding bridge wire detonator is instantaneous.

Explosion - A thermochemical process where mixtures of gases, solids, or liquids react with the almost instantaneous formation of gas pressures and sudden heat release.

Explosion pressure - See borehole pressure.

Explosive - Any chemical mixture that reacts at high velocity to liberate gas and heat causing very high pressures. ATF classifications include high explosives and low explosives. Also, any substance classified as an explosive by DOT.

Explosive materials - A term that includes, but is not necessarily limited to, dynamite and other high explosives, slurries, water gels, emulsions, blasting agents, black powder, pellet powder, initiating explosives, detonators, safety fuses, squibs, detonating cord, igniter cord, and igniters.

Extra dynamite - Also called ammonia dynamite, a dynamite that derives the major portion of its energy from ammonium nitrate.

Extraneous electricity - Electrical energy other than actual firing current that may be a hazard with electric blasting caps. Includes stray current, static electricity, lightning, radio frequency energy, and capacitive or inductive coupling.

F

Face - A rock surface exposed to air. Also called a free face, a face provides the rock with room to expand upon fragmentation.

Firing current - Electric current purposely introduced into a blasting circuit for initiation. Also, the amount of current required to activate an electric blasting cap.

Firing line - A line, often permanent, extending from the firing location to the electric blasting cap circuit. Also called lead wire.

Flash over - Sympathetic detonation between explosive charges or between charged blast holes.

Flyrock - Rock that is propelled through the air from a blast. Excessive flyrock may be caused by poor blast design or unexpected weak zones in the rock.

Fracturing - The breaking of rock with or without movement of the broken pieces.

Fragmentation - The extent that a rock is broken into pieces by blasting. Also the act of breaking rock.

Fuel - An ingredient in an explosive that reacts with an oxidizer to form gaseous products of detonation.

Fuel oil - The fuel in ANFO, usually No. 2 diesel.

Fume Classification - An IME quantification of the amount of fumes generated by an explosive or blasting agent.

Fume quality - A measure of the toxic fumes to be expected when a specific explosive is properly detonated. See fumes.

Fumes - Noxious or poisonous gases liberated from a blast. May be due to a low fume quality explosive or inefficient detonation.

Fuse - See safety fuse.

Fuse lighter - A pyrotechnic device for rapid and dependable lighting of safety fuse.

G

Galvanometer - (More properly called blasters' galvanometer.) A measuring instrument containing a silver chloride cell and/or a current limiting device that is used to measure resistance in an electric blasting circuit. Only a device specifically identified as a blasting galvanometer or blasting multimeter should be used for this purpose.

Gap sensitivity - The gap distance an explosive can propagate across. The gap may be air or a defined solid material. Gap sensitivity is a measure of the likelihood of sympathetic propagation.

Gas detonation system - A system for initiating caps where the energy is transmitted through the circuit by a gas detonation inside a hollow plastic tube.

Gelatin - An explosive or blasting agent that has a gelatinous consistency. The term is usually applied to a gelatin dynamite but may also be a water gel.

Gelatin dynamite - A highly water-resistant dynamite with a gelatinous consistency.

Generator blasting machine - A blasting machine operated by vigorously pushing down a rack bar or twisting a handle. Now largely replaced by condensor discharge blasting machines.

Grains - A weight measurement unit where 7,000 grains equal 1 pound.

Ground vibration - Ground shaking caused by the elastic wave emanating from a blast. Excessive vibrations may cause damage to structures.

H

Hangfire - The detonation of an explosive charge after the designed firing time. A source of serious accidents.

Heading - The working face or end of an excavation driven in an underground mine.

Hercudet - See gas detonation system.

Hertz - A term used to express the frequency of ground vibrations and airblast. One hertz is one cycle per second.

High explosive - Any product used in blasting that is sensitive to a No. 8 test blasting cap and reacts at a speed faster than that of sound in the explosive medium. A classification used by ATF for explosive storage.

Highwall - The bench, bluff, or ledge on the edge of a surface excavation. This term is most commonly used in coal strip mining.

I

Ignitacord – A cordlike fuse that burns progressively along its length with an external flame at the zone of burning and is used for lighting a series of safety fuses in sequence. Burns with a spitting flame similar to a Fourth-of-July sparkler.

IME - The Institute of Makers of Explosives. A trade organization dealing with the use of explosives and concerned with safety in manufacture, transportation, storage, handling, and use. The IME publishes a series of blasting safety pamphlets.

Initiation - The act of detonating a high explosive by means of a cap, mechanical device, or other means. Also the act of detonating the initiator.

Instantaneous detonator - A detonator that contains no delay element.

J

Jet 1oader - A system for loading ANFO into small blastholes where the ANFO is sucked from a container and blown into the hole at high velocity through a loading hose.

Jumbo - A machine designed to mount two or more drilling units that may or may not be operated independently.

K

Kerf - A slot cut in a coal or soft rock face by a mechanical cutter to provide a free face for blasting.

L

Lead wire - The wire connecting the electrical power source with the leg wires or connecting wires of a blasting circuit. Also called a firing line.

LEDC - Low-energy detonating cord used to initiate nonelectric blasting caps.

Leg wires - Wires connected to the bridge wire of an electric blasting cap and extending from the waterproof plug. The opposite ends are used to connect the cap into a circuit.

Lifters - The bottom holes in a tunnel or drift round.

Line drilling - An overbreak control method where a series of very closely spaced holes are drilled at the perimeter of the excavation. These holes are not loaded with explosive.

Liquid oxygen explosive - A high explosive made by soaking cartridges of carbonaceous materials in liquid oxygen. This explosive is rarely used today.

Loading density - The pounds of explosive per foot of charge of a specific diameter.

Loading factor - See powder factor.

Loading pole - A pole made of nonsparking material used to push explosive cartridges into a borehole and to break and tightly pack the explosive cartridges into the hole.

Low explosive - An explosive where the speed of reaction is slower than the speed of sound, such as black powder. A classification used by ATF for explosive storage.

LOX - See liquid oxygen explosive.

M

Magazine - A building, structure, or container specially constructed for storing explosives, blasting agents, detonators, or other explosive materials.

Mat - A covering placed over a shot to hold down flying material. The mat is usually made of woven wire cable, rope, or scrap tires.

Maximum firing current - The highest current (amperage) recommended for the safe and effective initiation of an electric blasting cap.

Metallized - Sensitized or energized with finely divided metal flakes, powders, or granules, usually aluminum.

Michigan cut - See parallel hole cut.

Microballoons - Tiny hollow spheres of glass or plastic that are added to explosive materials to enhance sensitivity by assuring adequate entrapped air content.

Millisecond (ms) - Short delay intervals equal to 1/1,000 of a second.

Millisecond delay caps - Delay detonators that have built-in time delays of various lengths. The interval between the delays at the lower end of the series is usually 25 ms. The interval between delays at the upper end of the series may be 100 to 300 ms.

Minimum firing current - The lowest current (amperage) that will initiate an electric blasting cap within a specified short interval of time.

Misfire - A charge or part of a charge that has failed to fire as planned. All misfires are dangerous.

Monomethylaminenitrate - A compound used to sensitize some water gels.

MS connector - A device used as a delay in a detonating cord circuit connecting one hole in the circuit with another or one row of holes to other rows of holes.

MSHA - The Mine Safety and Health Administration. An agency under the Department of Labor that enforces health and safety regulations in the mining industry.

Muckpile - A pile of broken rock or dirt that is to be loaded for removal.

Mud cap - Referred to also as adobe, bulldoze, blistering, or plaster shot. A charge of explosive fired in contact with the surface of a rock usually covered with a quantity of mud, wet earth, or similar substance. No borehole is used.

Multimeter - A multipurpose test instrument used to check line voltages, firing currents, current leakage, stray currents, and other measurements pertinent to electric blasting. (More properly called blasters' multimeter.) Only a meter specifically designated as a blasters' multimeter or blasters' galvanometer should be used to test electric blasting circuits.

N

National Fire Protection Association (NFPA) - An industry/government association that publishes standards for explosive material and ammonium nitrate.

Nitrocarbonitrate - A shipping classification once given to a blasting agent by DOT.

Nitrogen oxides - Poisonous gases created by detonating explosive materials. Excessive nitrogen oxides may be caused by an excessive amount of oxygen in the explosive mixture (excessive oxidizer) or by inefficient detonation.

Nitroglycerin (NG) - The explosive oil originally used as the sensitizer in dynamites, $C_3H_5(ONO_2)_3$.

Nitromethane - A liquid compound used as a fuel in two-component (binary) explosives and as rocket and dragster fuel.

Nitropropane - A liquid fuel that can be combined with pulverized ammonium nitrate prills to make a dense blasting mixture.

Nitrostarch - A solid explosive similar to nitroglycerin used as the base of "nonheadache" powders.

Nonel - See shock tube system.

Nonelectric delay blasting cap - A detonator with a delay element capable of being initiated nonelectrically. See shock tube system; gas detonation system; Detaline System.

No. 8 test blasting cap - See test blasting cap No. 8.

O

OSHA - The Occupational Safety and Health Administration. An agency under the Department of Labor that enforces health and safety regulations in the construction industry, including blasting.

OSM - The Office of Surface Mining Reclamation and Enforcement. An agency under the Department of the Interior that enforces surface environmental regulations in the coal mining industry.

Overbreak - Excessive breakage of rock beyond the desired excavation limit.

Overburden - Worthless material lying on top of a deposit of useful materials. Overburden often refers to dirt or gravel but can be rock, such as shale over limestone or shale and limestone over coal.

Overdrive - Inducing a velocity higher than the steady state velocity in a powder column by the use of a powerful primer. Overdrive is a temporary phenomenon, and the powder quickly assumes its steady state velocity.

Oxides of nitrogen - See nitrogen oxides.

Oxidizer - An ingredient in an explosive or blasting agent that supplies oxygen to combine with the fuel to form gaseous or solid detonation products. Ammonium nitrate is the most common oxidizer used in commercial explosives.

Oxygen balance - A mixture of fuels and oxidizers where the gaseous products of detonation are predominantly carbon dioxide, water vapor (steam), and free nitrogen. A mixture containing excess oxygen has a negative oxygen balance.

P

Parallel circuit - A circuit where two wires, called bus wires, extend from the lead wire. One leg wire from each cap in the circuit is hooked to each of the bus wires.

Parallel hole cut - A group of parallel holes, some of which are loaded with explosives, used to establish a free face in tunnel or heading blasting. One or more of the unloaded holes may be larger than the blast holes; also called Coromant, Cornish, burn, shatter, or Michigan cut.

Parallel series circuit - Similar to a parallel circuit but involving two or more series of electric blasting caps.

One end of each series of caps is connected to each of the bus wires; sometimes called series-in-parallel circuit.

Particle velocity - A measure of ground vibration. Describes the velocity of particle vibration when excited by a seismic wave.

Pattern - A drill hole plan laid out on a face or bench to be drilled for blasting.

Pellet powder - Black powder pressed into 2-inch-long, 1¼- to 2-inch diameter cylindrical pellets.

Pentaerythritoltetranitrate (PETN) - A military explosive compound used as the core load of detonating cord and the base charge of blasting caps.

Pentolite - A mixture of PETN and TNT used as a cast primer.

Permissible - A machine, material, apparatus, or device that has been investigated, tested, and approved by the Bureau of Mines or MSHA maintained in permissible condition.

Permissible blasting - Blasting according to MSHA regulations for underground coal mines or other gassy underground mines.

Permissible explosives - Explosives that have been approved by MSHA for use in underground coal mines or other gassy mines.

PETN - See pentaerythritoltetranitrate.

Plaster shot - See mud cap.

Pneumatic 1oader - One of a variety of machines powered by compressed air used to load bulk blasting agents or cartridged water gels.

Powder - Any solid explosive.

Powder chest - A strong, nonconductive portable container equipped with a lid used at blasting sites for temporary explosive storage.

Powder factor - A ratio between the amount of powder loaded and the amount of rock broken, usually expressed as pounds per ton or pounds per cubic yard. In some cases, the reciprocals of these terms are used.

Preblast survey - A documentation of the existing condition of a structure. The survey is used to determine whether subsequent blasting causes damage to the structure.

Premature - A charge that detonates before intended.

Preshearing - See presplitting.

Presplitting - Controlled blasting where decoupled charges are fired in closely spaced holes at the perimeter of the excavation. A presplit blast is fired before the main blast. Also called preshearing.

Pressure vessel - A system for loading ANFO into small diameter blast holes. The ANFO is contained in a sealed vessel where air pressure forces the ANFO through a hose and into the blast hole; also known as pressure pot.

Prill - A small porous sphere of ammonium nitrate capable of absorbing more than 6 percent by weight of fuel oil. Blasting prills have a bulk density of 0.80 to 0.85 g/cm^3.

Primary blast - The main blast executed to sustain production.

Primary explosive - An explosive or explosive mixture sensitive to spark, flame, impact, or friction used in a detonator to initiate the explosion.

Primer - A unit, package, or cartridge of cap-sensitive explosive used to initiate other explosives or blasting agents and that contains a detonator.

Propagation - The detonation of explosive charges by an impulse from a nearby explosive charge.

Propagation blasting - The use of closely spaced, sensitive charges. The shock from the first charge propagates through the ground, setting off the adjacent charge and so on. Only one detonator is required; primarily used for ditching in damp ground.

Propellant explosive - An explosive that normally deflagrates and is used for propulsion.

Pull - The quantity of rock or length of advance excavated by a blast round.

R

Radio frequency energy - Electrical energy traveling through the air as radio or electromagnetic waves. Under ideal conditions this energy can fire an electric blasting cap. IME Pamphlet No. 20 recommends safe distances from transmitters to electric blasting caps.

Radio frequency transmitter - An electric device, such as a stationary or mobile radio transmitting station, that transmits a radio frequency wave.

RDX - Cyclotrimethylenetrinitramine, an explosive substance used in the manufacture of compositions B, C-3, and C-4. Composition B is useful as a cast primer.

Relievers - In a heading round, holes adjacent to the cut holes, used to expand the opening made by the cut holes.

Rib holes - The holes at the sides of a tunnel or drift round that determine the width of the opening.

Riprap - Coarse rocks used for river bank or dam stabilization to reduce erosion by water flow.

Rotational firing - A delay blasting system where each charge successively displaces its burden into a void created by an explosive detonated on an earlier delay period.

Round - A group or set of blast holes used to produce a unit of advance in underground headings or tunnels.

S

Safety fuse - A core of potassium nitrate black powder, enclosed in a covering of textile and waterproofing, used to initiate a blasting cap or a black powder charge. Safety fuse burns at a continuous, uniform rate.

Scaled distance - A ratio used to predict ground vibrations. Scaled distance equals the distance from the blast to the point of concern in feet divided by the square root of the charge weight of explosive per delay in pounds. When using the equation, the delay period must be at least 9 ms.

Secondary blasting - Using explosives to break boulders or high bottom resulting from the primary blast.

Seismograph - An instrument that measures and may supply a permanent record of earthborne vibrations induced by earthquakes or blasting.

Semiconductive hose - A hose used for pneumatic loading of ANFO that has a minimum electrical resistance of 1,000 ohms per foot, 10,000 ohms total resistance, and a maximum total resistance of 2,000,000 ohms.

Sensitiveness - A measure of an explosive's ability to propagate a detonation.

Sensitivity - A measure of an explosive's susceptibility to detonation by an external impulse such as impact, shock, flame, or friction.

Sensitizer - An ingredient used in explosive compounds to promote greater ease in initiation or propagation of the detonation reaction.

Sequential blasting machine - A series of condenser discharge blasting machines in a single unit that can be activated at various accurately timed intervals following the application of electrical current.

Series circuit - A circuit of electric blasting caps where each leg wire of a cap is connected to a leg wire from the adjacent caps so that the electrical current follows a single path through the entire circuit.

Series-in-parallel circuit - See parallel series circuit.

Shatter cut - See parallel hole cut.

Shock energy - The shattering force of an explosive caused by the detonation wave.

Shock tube system - A system for initiating caps where the energy is transmitted to the cap by a shock wave inside a hollow plastic tube.

Shock wave - A pressure pulse that propagates at supersonic velocity.

Shot - See blast.

Shot firer - Also referred to as the shooter. The person who actually fires a blast. A powderman may charge or load blast holes with explosives but may not fire the blast.

Shunt - A piece of metal or metal foil that short circuits the ends of cap leg wires to prevent stray currents from causing accidental detonation of the cap.

Silver chloride cell - A low-current cell used in a blasting galvanometer and other devices to measure continuity in electric blasting caps and circuits.

Slurry - An aqueous solution of ammonium nitrate, sensitized with a fuel, thickened, and crosslinked to provide a gelatinous consistency. Sometimes called a water gel. DOT may classify a slurry as a Class A explosive, a Class B explosive, or a blasting agent. An explosive or blasting agent containing substantial portions of water (MSHA). See emulsion; water gel.

Smooth blasting - Controlled blasting used underground where a series of closely spaced holes is drilled at the perimeter, loaded with decoupled charges, and fired on the highest delay period of the blast round.

Snake hole - A borehole drilled slightly downward from horizontal into the floor of a quarry face. Also, a hole drilled under a boulder.

Sodium nitrate - An oxidizer used in dynamites and sometimes in blasting agents.

Spacing - The distance between boreholes or charges in a row measured perpendicular to the burden and parallel to the free face of expected rock movement.

Specific gravity - The ratio of the weight of a given volume of any substance to the weight of an equal volume of water.

Spitter cord - See Ignitacord.

Springing - See chambering.

Square pattern - A blast hole pattern where the holes in succeeding rows are drilled directly behind the holes in the front row. In a truly square pattern, the burden and spacing are equal.

Squib - A firing device that burns with a flash. Used to ignite black powder or pellet powder.

Stability - The ability of an explosive material to maintain its physical and chemical properties over a period of time in storage.

Staggered pattern - A blast hole pattern where the holes in each row are drilled between the holes in the preceding row.

Static electricity - Electrical energy stored on a person or object in a manner similar to that of a capacitor. Static electricity may be discharged into electrical initiators, detonating them.

Steady state velocity - The characteristic velocity at which a specific explosive, under specific conditions in a given charge diameter, will detonate.

Stemming - The inert material such as drill cuttings used in the collar portion (or elsewhere) of a blast hole to confine the gaseous products of detonation; also, the length of blast hole left uncharged.

Stick count - See cartridge count.

Stray current - Current flowing outside the desired conductor. Stray current may come from electrical equipment, electrified fences, electric railways, or similar sources. Flow is facilitated by conductive paths such as pipelines and wet ground or other wet materials. Galvanic action of two dissimilar metals in contact or connected by a conductor may cause stray current.

Strength - An explosive property described variously as cartridge or weight strength, seismic strength, shock or bubble energy, crater strength, or ballistic mortar strength. Not a well-defined property. Used to express an explosive's capacity to do work.

String 1oading - Loading cartridges end-to-end in a borehole without deforming them. Used mainly in controlled blasting and permissible blasting.

Subdrill - To drill blast holes beyond the planned grade lines or below floor level to insure breakage to the planned grade or floor level.

Subsonic - Slower than the speed of sound.

Supersonic - Faster than the speed of sound.

Swell factor - The ratio of the volume of a material in an undisturbed state to that when broken. May also be expressed as the reciprocal of this number.

Sympathetic propagation (sympathetic detonation) - Detonation of an explosive material by an impulse from another detonation through air, earth, or water.

T

Tamping - Compressing the stemming or explosive in a blast hole. Sometimes used synonymously with stemming.

Tamping bag - A cylindrical bag containing stemming material used to confine explosive charges in boreholes.

Tamping pole - See loading pole.

Test blasting cap No. 8 - A detonator containing 0.40 to 0.45 g of PETN base charge at a specific gravity of 1.4 g/cm^3 primed with standard weights of primer, depending on the manufacturer.

Toe - The burden or distance between the bottom of a borehole and the vertical free face of a bench in an excavation. Also the rock left unbroken at the foot of a quarry blast.

Transient velocity - A velocity different from the steady state velocity that a primer imparts to a column of powder. The powder column quickly attains steady state velocity.

Trinitrotoluene (TNT) - A military explosive compound used industrially as a sensitizer for slurries and as an ingredient in pentolite and composition B. Once used as a free-running pelletized powder.

Trunkline -A detonating cord line used to connect the downlines of other detonating cord lines in a blast pattern. Usually runs along each row of blast holes.

Tunnel - A basically horizontal underground passage.

Two-component explosive - See binary explosive.

U

Unconfined detonation velocity - The detonation velocity of an explosive material not confined by a borehole or other confining medium.

V

V-cut - A cut employing several pairs of angled holes meeting at the bottoms used to create free faces for the rest of the blast round.

Velocity - The rate that the detonation wave travels through an explosive. May be measured confined or unconfined. Manufacturer's data are sometimes measured with explosives confined in a steel pipe.

Venturi 1oader - See jet loader.

Volume strength - See cartridge strength or bulk strength.

W

Water gel - An aqueous solution of ammonium nitrate, sensitized with a fuel, thickened, and crosslinked to provide a gelatinous consistency; also called a slurry; may be an explosive or a blasting agent.

Water stemming bags - Plastic bags containing a self-sealing device that are filled with water. Classified as a permissible stemming device by MSHA.

Weight strength - A rating that compares the strength of a given weight of explosive with an equivalent weight of straight nitroglycerin dynamite or other explosive standard, expressed as a percentage.

Chapter 20

WATER CONTROL

Introduction

This chapter discusses site conditions that need dewatering and the extent and basic requirements for construction dewatering. A more comprehensive discussion of technical requirements and methods for constructing dewatering facilities is available in the Reclamation *Ground Water Manual*. The subjects that are discussed include:

- Site review
- Site investigations
- Data collection
- Data interpretation, evaluation, and presentation
- Specifications paragraphs
- Construction considerations
- Supervision and oversight
- Documentation of results (final construction report)

Water control is lumped into two categories—dewatering and unwatering. Water control is the removal or control of groundwater or seepage from below the surface (dewatering) or the removal or control of ponded or flowing surface water by ditches, surface drains, or sumps (unwatering). Excavation of materials or construction near or below the water table or near surface water bodies usually requires control of the groundwater or seepage. Control may involve isolation with cutoffs, stabilization by freezing, grouting, or other methods, or by a combination of methods. Control of groundwater and seepage usually involves installation and operation of wells or drains. A key operation in most water control in unstable materials is the removal of water from below the ground surface in advance of excavation and maintaining the water level at a suitable depth below the working surface. These steps permit construction "in the dry," unhampered by the adverse effects of water.

Understanding what factors control water is critical for ensuring short-term stability during construction of any facility and for the long-term stability of any structure. This chapter discusses what should be considered. If water is considered a potential problem, any field investigation must obtain information on the site's various potential water sources including seasonal precipitation and runoff periods and the identification and understanding of the water bearing zones within a site. This means not only identifying and determining the depths and extent of the various water bearing zones and their water surfaces but also understanding the formation hydraulic parameters (porosity, permeability, and, when necessary, storage).

Figure 20-1 illustrates the importance of understanding the site hydraulic parameters in selecting the appropriate dewatering method. Using the wrong method for the site materials can result in a totally ineffective dewatering system. Figure 20-1 can be used to assess what is the appropriate dewatering method based on the site materials.

In excavations in rock, cemented granular materials, clays, and other stable materials, water removal or unwatering may be by draining to sumps and using surface pumps concurrently with or following completion of the excavation. Subsurface cutoffs such as sheet piling and slurry walls in soil or soft rock and grouting in rock are also used to control ground and surface water and for ground support. Cutoffs are seldom 100-percent effective, and supplementary dewatering or unwatering is usually required. Soils are frozen for short periods such as for a

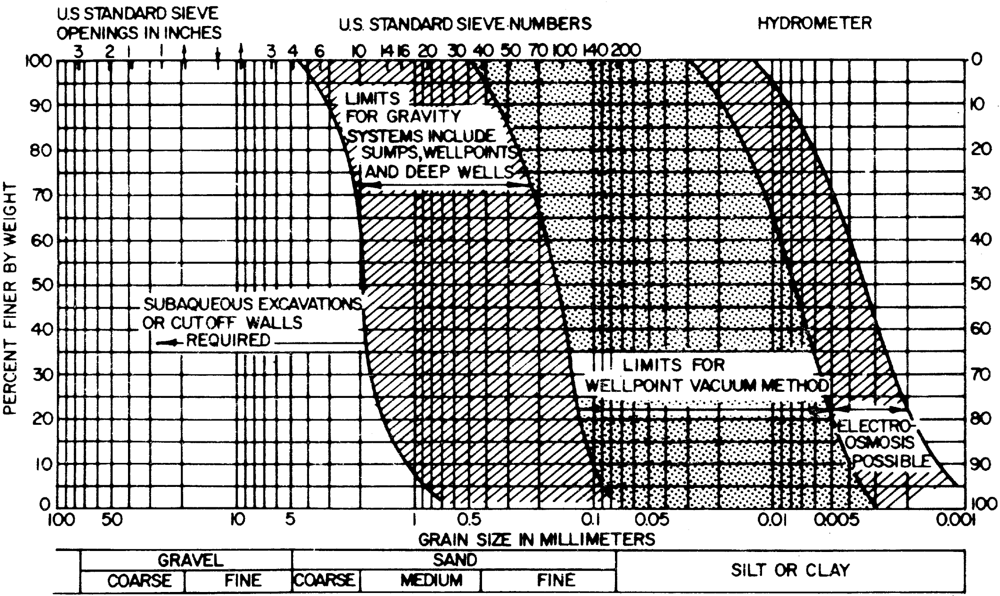


Figure 20-1.—Limits of dewatering methods for different materials.

temporary excavation, especially in clayey or silty soils that have low permeability and are difficult to drain.

Unwatering methods are commonly used in soils that have high porosity but low permeabilities (clayey or silty soils), bedrock that has solution cavities, and lava tubes that carry large volumes of water in isolated areas. Unwatering methods are commonly used to control surface water.

Unwatering usually is performed in conjunction with dewatering to ensure control of surface water and to permit dewatering to proceed unaffected by recharge or flooding from nearby surface water. Failure to properly remove or control water during unwatering or dewatering may result in:

- Unstable natural or excavated slopes
- Unstable, unworkable, or unsuitable subgrade
- Boils, springs, blowouts, or seeps on slopes or in the subgrade
- Flooding of excavations or structures
- Uplift of constructed features such as concrete slabs
- Dilution, corrosion, or other adverse effects on concrete, metals, or other construction materials
- Instability of nearby structures
- Draining surrounding surface water and groundwater

- Instability of cutoff facilities such as cofferdams
- Loss of fines from the foundation
- Safety problems
- Delays in construction
- Increased construction costs

Water control may vary widely in scope, magnitude, and difficulty. Some controlling factors related to the constructed feature include footprint, foundation depth, and construction time. Factors related to site conditions are:

- Subsurface geology including general material types; bedding, attitudes and lateral extent of bedding; and attitudes, continuity, and apertures of fractures
- Subsurface hydraulic conditions including permeabilities and thicknesses of different materials, groundwater occurrence, and levels
- Recharge conditions including proximity to surface water bodies and precipitation
- Other facilities including cofferdams and site access

Conditions that may indicate the need for dewatering and the possibility for difficult dewatering include:

- Site location adjacent to a large body of surface water, a stream, a marshy area, or an area subject to flooding
- An excavation significantly below the water table

- Complex foundation geology
- An artesian zone immediately below grade
- Existing structures or facilities
- Existing use of groundwater
- Poor quality groundwater
- Presence of hazardous materials
- Thick zones of saturated, low-strength materials such as silt or soft clay, especially under artesian conditions
- Presence of cofferdams or other similar features for which dewatering is needed for stability
- Conditions where failure of dewatering facilities could result in catastrophic failure of protective or other features and a hazard to life or property

Sites requiring groundwater or seepage control for construction or proper operation of a facility should be identified as early in the planning or design process as possible.

Exploration Program

Investigations for water control should be part of the general exploration and design data collection program. Advantages of conducting investigations together include:

- Maximizing the use of design data collection and lowering exploration costs
- Providing dewatering data early in the program
- Using field personnel efficiently

Water control investigations generally cannot be established in detail until some reconnaissance level drilling has been done and general subsurface geologic and hydrologic conditions are determined. General surface conditions including topography and surface hydrology should be known. A specialist should be consulted as early in the program as possible to maximize the benefits of the obtained data.

Design Data Requirements, Responsibilities, and Methods of Collection and Presentation

Adequate surface and subsurface data are essential to the proper design, installation, and operation of water control facilities. The amount of data required for water control facilities design may equal or exceed the foundation data required for design of the structure. Water control facilities may be designed in-house or by the contractor; but investigations may be extensive, complex, time consuming, and beyond the capability of a contractor to accomplish in the bidding period. An in-house design is generally better because the time is usually available to do the job right, the designer has control over the design data, responsibility for the water control design is clear to the owner and the contractor, and the contractor can bid the water control installation more accurately.

Design data for water control facilities should be obtained regardless of who is ultimately responsible for the design.

The extent and level of data should be appropriate for the anticipated water control requirements and facilities. All subsurface investigations should be sufficiently detailed to determine the groundwater conditions, including the depth of the vadose zone and the various potentiometric water surfaces; and the investigations should provide at least enough data to estimate the permeability of the soil or rock. Crude soil permeabilities can be estimated from blow counts and the visual or laboratory soil classification. Permeability tests should be considered in any exploration program. If aquifer tests are required, the test wells should approximate the size and capacity of anticipated dewatering wells. Facilities should be preserved and made available to prospective contractors for their testing or use, if appropriate.

If the contractor is responsible for the dewatering design, all field design data should be included in the construction specifications. Data include details of drilling and completion of exploratory drill holes, wells, piezometers, and other installations as well as test data. Data should be as concise as possible and clearly show the history, sequence, and location of all exploration. Design data for water control facilities should be obtained concurrently with feature design data if possible. Specialists should be consulted when preparing design data programs, especially programs for foundation drilling and aquifer testing. Water control data are an essential part of the design data package and must be given the required priority in funds, time, and personnel to minimize problems such as construction delays and claims. Where dewatering may have impacts on existing adjacent structures, wells, facilities, or water resources, a study of the area surrounding the site may be necessary to determine and document impacts. In addition to data involving the constructed feature such as a structure layout, excavation depth, and time to construct, other information is required on site conditions such as:

Surface Data

- Site and surrounding topography at an appropriate scale and contour interval
- Cultural features on and off site
- Site and surrounding surface geology with descriptions of materials
- Site and adjacent surface water features such as lakes, streams, swamps, bogs, and marshes
- Plan map of all data points, including locations of drill holes, test holes, piezometers, observation wells, test wells, and overlays on the plan of the proposed feature

Surface information should include data on soil erosion or resistance or how erosion relates to runoff or the potential recharge of the groundwater system. Soil infiltration data from Natural Resource Conservation Service mapping should be included if available.

Subsurface Data

Subsurface data should include representative permeabilities, a real distribution of permeabilities, location and potential recharge sources or barriers, and anticipated seasonal changes in the groundwater system. When a project has a relatively high soil or rock permeability and the permeable formation extends laterally over a large area, storage (storativity, effective porosity) of the aquifer requires evaluation. Specific data that may be necessary include:

- General geology of the site and surrounding area including geologic cross sections that show vertical and lateral variations in materials

- Logs of drill holes, test holes, and piezometers showing depths and thicknesses of materials, descriptions of materials, and results of testing

- Results of material sampling including depths, descriptions, mechanical analyses, and hydrometer analyses

- Geophysical logs

- Aquifer or permeability test results including yields and drawdown with time and static water levels

- Layout of test holes and depths and designs of wells and piezometers

- Water quality analyses

Other Data

- Climatic data for the nearest station including daily temperature and precipitation and details on the occurrence of severe storms

- Streamflow and elevation, lake or reservoir depth, elevation, and other similar data

- Groundwater levels for monitored observation wells, piezometers, test wells, drill holes, and pits

- If a nearby surface water body and the groundwater are connected, continuous monitoring of both features for a typical hydrologic cycle

Presentation of Data

The presentation of dewatering data may differ from the presentation of conventional geologic data because water control data are subject to a variety of quantitative interpretations and because water levels, flow, and water quality vary with time.

Because of the potential for different interpretations, most dewatering data such as those from aquifer tests and packer tests are presented as observed field data and as interpretations. A complete description of the site, subsurface conditions, and test facilities should be given along with the data (figures 20-2 and 20-3).

Where time related data are presented, the information should be in a form that will ensure maximum recognition and proper interpretation. Hydrographs that plot time versus water levels mean a lot more than a table of readings.

Monitoring

Water control activities must be monitored during construction and, in some cases, up to a year or more before and following completion of the facility. Details of the monitoring program including design and layout of the system, the responsibilities for installing the system, and monitoring and maintaining records must be included in the specifications and construction considerations to ensure adequate reliable data and inform construction personnel of monitoring requirements.

PUMPED WELL WELL NO. 269			OBSERVATION WELL DH-270 RADIUS 30 FEET		OBSERVATION WELL DH-271 RADIUS 99 FEET	
ELAPSED TIME (MINUTES)	DRAWDOWN (FEET)	DISCHARGE (GAL./MIN.)	ELAPSED TIME (MINUTES)	DRAWDOWN (FEET)	ELAPSED TIME (MINUTES)	DRAWDOWN (FEET)
0	0	0	0	0	0	0
0.17	0.93	-	1	0.07	1	0
0.50	1.02	-	1.5	0.09	2	0.05
0.67	1.03	-	4	0.10	3	0.05
0.83	1.07	-	5	0.07	4	0.05
1	1.09	-	8	0.11	5	0.05
1.17	1.12	-	10	0.11	7	0.05
1.33	1.13	170	14	0.11	9	0.05
1.67	1.05	-	21	0.11+	11	0.09
1.83	1.01	-	31	0.12	13	0.09
2	1.03	-	40	0.11+	15	0.09
2.33	1.11	-	48	0.13	20	0.09
2.67	1.10	-	53	0.13	25	0.09
3	1.08	-	58	0.14	30	0.09
3.50	1.09	-	73	0.14+	35	0.09
4	1.11	-	83	0.15	55	0.15
5	1.10	170	97	0.16	65	0.10
6	1.10	-	103	0.17	75	0.15
7	1.11	-	118	0.19	85	0.15
8	1.11	-	138	0.20	95	0.15
9	1.11	-	168	0.18	105	0.15
10	1.11	175+	171	0.17	123	0.15
12	1.12	-	183	0.18	138	0.15
14	1.11	-	231	0.19	153	0.15
16	1.15	-	250	0.19	168	0.15
18	1.15	-	265	0.16	183	0.15
20	1.14	-	318	0.16	198	0.15
26	1.15	-	378	0.21	213	0.15
30	1.14	175+	453	0.21	228	0.15
35	1.15	-	498	0.22	243	0.15
40	1.13	-	618	0.24	258	0.15
45	1.14	-	738	0.24	318	0.16
50	1.15	-	858	0.25	378	0.10
60	1.15	-	978	0.23	438	0.15
70	1.17	-	1103	0.23	498	0.23
80	1.18	-	1118	0.19	741	0.25
90	1.17	-			853	0.25
100	1.15	178			980	0.25
120	1.15	180			1100	0.28
135	1.15	-			1126	0.29

OBSERVATION WELL DH-100 RADIUS 100 FEET				OBSERVATION WELL DH-102 RADIUS 87 FEET			
DEEP PIEZOMETER *		SHALLOW PIEZOMETER		DEEP PIEZOMETER		SHALLOW PIEZOMETER	
ELAPSED TIME (MINUTES)	DRAWDOWN (FEET)	ELAPSED TIME (MINUTES)	DRAWDOWN (FEET)	ELAPSED TIME (MINUTES)	DRAWDOWN (FEET)	ELAPSED TIME (MINUTES)	DRAWDOWN (FEET)
0	0	0	0	0	0	0	0
41	+0.57	41	0.13	36	0.03	36	0.25
63	+1.09	315	0.11	83	0.03	84	0.22
151	+1.36	379	0.18	146	0.04	147	0.24
194	+1.37	441	0.19	201	0.05	202	0.24
314	+1.42	496	0.16	311	0.10	312	0.38
378	+1.33	619	0.20	377	0.04	377	0.38
441	+1.36	731	0.27	402	0.06	402	0.30
496	+1.38	859	0.19	439	0.08	439	0.15
620	+1.33	*976	0.21	491	0.09	491	0.36
731	+1.34			617	0.16	617	0.37
858	+1.34			738	0.17	739	0.38
976	+1.43			855	0.11	856	0.36
1088	+1.58			979	0.12	979	0.36
		(Obstruction in pipe at 5.36 feet)		1098	0.15	1099	0.35

* Apparently influenced by discharge water. One small part of flow ponds about 10 feet west of well, and then flows within 8 feet of the well as it flows past on the way to the lake.

* May have been responding. However readings believed to be unreliable, due to obstruction in pipe and behavior of the deep piezometer. No further readings taken.

NOTE: Well located about 30 feet from lake shore.

Figure 20-3.—Aquifer test data.

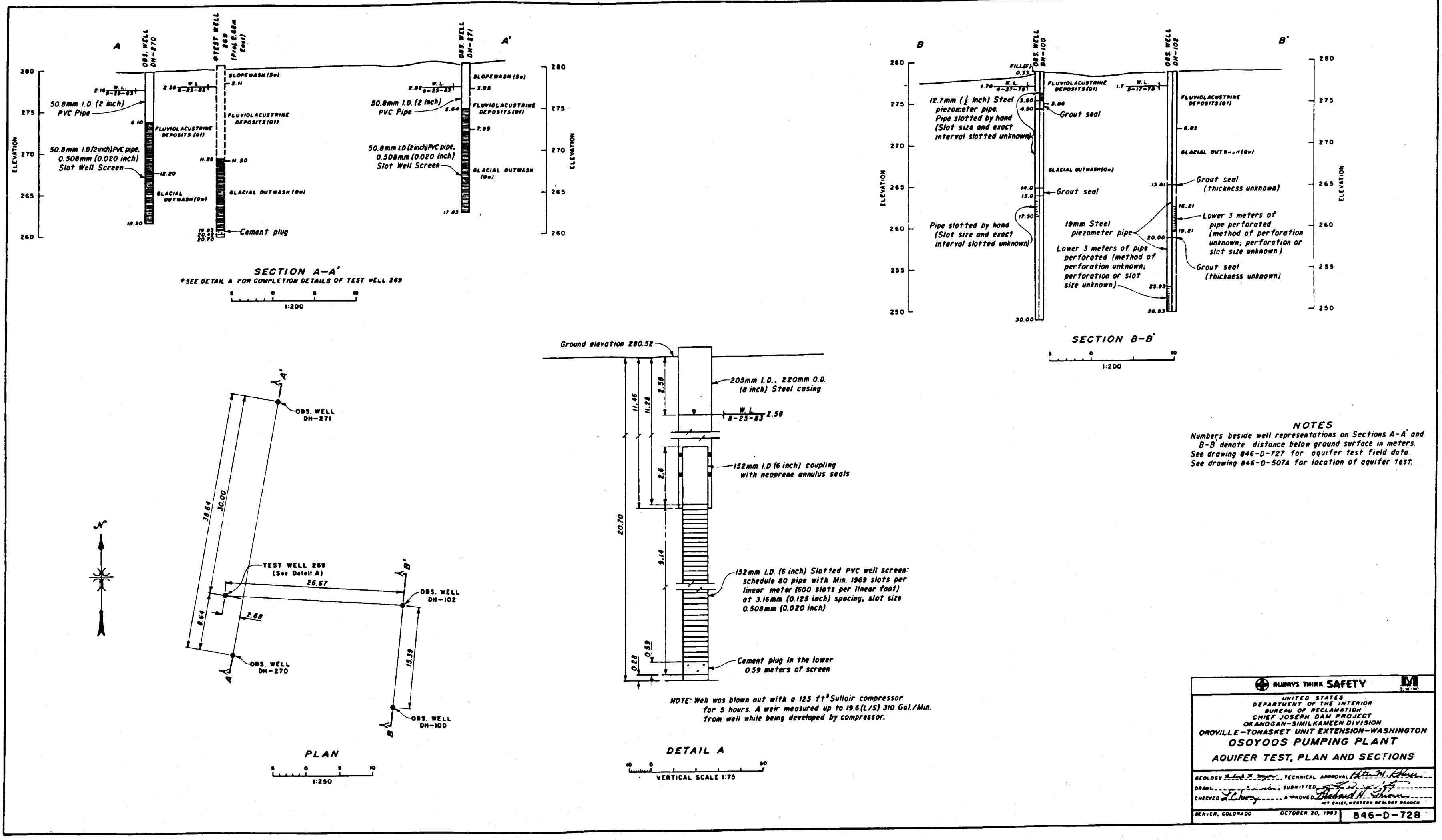


Figure 20-2.—Aquifer test, plan, and sections.

Monitoring is intended to:

- Provide data on base level conditions
- Confirm that the specifications requirements are being met
- Maintain a general data base on conditions and impacts resulting from the dewatering
- Alert personnel to unsuitable, hazardous, or potentially hazardous conditions
- Document conditions in the event of claims or litigation
- Provide background data for a followup analysis in the event of a slope, foundation, or structural failure

Monitoring of water control parameters, activities, and features should include:

- Groundwater levels
- Discharges
- Sediment content of discharges
- Chemical and biologic quality of water discharged
- Horizontal and vertical control on constructed features and natural and excavated slopes
- Levels and sizes of nearby surface water bodies
- Stability of nearby structures

The monitoring facilities may range from a few observation wells to a complex system involving sophisticated equipment and continuous and possibly remote monitoring. The extent, complexity, and capability of water control facilities depend on the size and complexity of the facilities and the hydrologic and geologic conditions and excavation depths.

Groundwater Monitoring

Monitoring during construction should include groundwater levels in excavations, areas surrounding the excavations, and off-site locations. Excavations of any substantial width (other than trenches) and a depth of more than a few feet below the anticipated water level may need groundwater monitoring instrumentation installed directly in the excavation. Excavations underlain at shallow depths by artesian conditions may need to be monitored because of the potential for blowout.

Groundwater Monitoring Locations

Groundwater monitoring instrumentation located within an excavation may interfere with the contractor during construction, but monitoring groundwater levels within an excavation generally justifies inconveniences. Because the most difficult area to dewater is usually the center of the excavation, groundwater monitoring instruments should be located near the center. Special provisions may be necessary to ensure continued groundwater monitoring of the facilities during all stages of the excavation. In some cases, groundwater monitoring instrumentation may be incorporated into the structure. This may require that the monitoring instrument be embedded in concrete such as a wall or pier to permit continuous monitoring and simplify backfill operations.

Water level monitoring instrumentation should be located in areas surrounding the excavation to monitor representative areas and specific problem areas. Groundwater levels in off-site locations should be monitored to maintain a general record of conditions and to document dewatering. Off-site monitoring is especially important in areas where groundwater is widely used, where groundwater levels are crucial to existing activities, or where there might be subsidence or groundwater level decline.

Locations of individual water level monitoring instruments must be based on conditions encountered at the site, construction activities, and the dewatering facilities. Monitoring instrumentation locations generally will have to be selected after other features such as dewatering systems and roads have been located to avoid conflicts and to ensure representative and reliable monitoring. Instrumentation should be installed and functioning long before construction to obtain trends and base level conditions. Existing instrumentation may be used for monitoring; but unless the instrumentation construction and other details are known, instrumentation designed specifically for the conditions of the site should be installed as a part of construction.

Groundwater Monitoring Instrumentation

Instrumentation for monitoring groundwater levels usually consists of several observation wells or piezometers. The type of instrumentation, depth, and riser and hole diameter depends primarily on subsurface conditions, desired operating life, and type of monitoring. The design of the instrumentation should be tailored to the subsurface and data requirements so that measurements are a true indication of in place conditions.

Observation wells intended to monitor general groundwater levels can be used in areas where the foundation material is relatively uniform in depth, there is little or no layering, and groundwater levels do not vary appreciably with depth. Piezometers should be used where layering exists or perching, artesian, or complex conditions are expected.

Observation wells and piezometers usually consist of a section of well screen, perforated pipe, or porous tube isolated in the zone to be monitored and connected to a length of standpipe, riser, or casing extending to the surface. The section of well screen, perforations, or porous tube must be isolated with a watertight grout or bentonite seal, and a watertight riser must be used. The diameter of the screen and standpipe should conform to ASTM D-5092. Water levels are measured directly in the well or piezometer by use of tape or electric sounder (M-scope). Float-type recorders can be used to continuously record water level fluctuations but may require a minimum 4- to 6-inch (10- to 15-cm) diameter casing or standpipe. A wide range of electronic and pneumatic instruments is available for monitoring and recording groundwater levels.

Special types of monitoring wells or piezometers may be necessary if hazardous materials are present.

Monitoring Discharges From Dewatering Systems

The discharge from dewatering facilities such as wells, well point systems, drains, and sump pumps should be monitored to provide a record of the dewatering quantities. Data should include starting and stopping times, instantaneous rates of discharge, changes in rates, combined daily volumes, and, in some cases, water chemistry, turbidity, and biologic content.

Discharge rates can be monitored by flow meters (propeller, pitot tube, and acoustic), free discharge orifices, weirs, flumes, and volumetric (tank-stopwatch) methods. Meters must be calibrated using a volumetric test before flow testing. Measuring devices generally must be accurate within 10 percent.

Sediment content of dewatering facilities including wells, well point systems, and drains should be monitored. Sediment can damage pumping equipment, cause deterioration of water quality in a receiving water body, and create voids in the foundation that result in well collapse and foundation settlement.

Sediment content usually is measured in parts per million by volume of water or in nephelometric turbidity units (NTU) in water taken directly from the discharge. Measurement requires special equipment. If the limits are 50 parts per million or less, a special centrifugal measuring device is required. If the limits are more than 50 parts per million, an Imhoff cone can be used. A turbidity meter typically measures values less than 2,000 NTUs. Values less than 200 NTUs are generally acceptable for discharge. If sediment yield increases rapidly, the facility may need to be shut down to avoid serious damage or contamination.

The chemical and biologic content of water discharged from dewatering systems should be monitored by periodic collection and analysis of samples taken directly from the system discharge. A single representative sample is adequate if there are a number of discharges from the same source. If there is pumping from different sources, multiple samples may be needed. The initial samples should be taken shortly after startup of the dewatering system (or during any test pumping done during exploration) and periodically during operation. Each sample may need biologic and chemical analyses for heavy metals, organics, and pesticides. More frequent sampling and analyses may be made to determine total dissolved solids, conductivity, pH, and sediment content or turbidity.

Monitoring Water and Ground Surfaces and Structures

Well or Piezometer Design.—Most groundwater monitoring for water control investigations is to collect water level information, but some monitoring is to obtain water quality information. All monitoring wells should be constructed according to ASTM D-5092 and constructed for site conditions and purposes. Geologic conditions, available drilling equipment, depth to water, and the frequency of the readings should be factored into the design of a monitoring well. If a monitoring well requires daily or weekly readings, the well may require installation of an automated water level monitoring system. The type and diameter of riser should be based on the intended use of the monitoring well. If monitoring is to continue into construction, protecting the well should be part of the design. Too often, a piezometer or observation well is constructed without any serious attempt to design the well. Typically, monitoring wells are constructed using 1-inch (2.5-cm) diameter PVC risers with a screen slot size of 0.010 inch (0.25 mm) or with or without a 10/20 sand pack. This “low permeability” monitoring well design is appropriate if the well is monitored only for water levels and the adjacent sources of recharge due to precipitation, dewatering, unwatering activities, or other water bodies is not changing rapidly.

All monitoring wells should be completed using the appropriate seals, risers, and openings to allow the water level in the riser or piezometer to respond to the formation rather than to the water within the riser, the screen, or the sand pack. Improper or inadequate removal of cuttings and drill fluids, or too fine a slot or sand pack, can delay the response of groundwater within a riser. Too large openings can result in clogging of the monitoring well and impact the chemistry of the groundwater. A water quality monitoring well in soils or decomposed to intensely weathered bedrock should have a minimum of 2-inch (5-cm) risers with an annular space of at least 2 inches (5 cm). All wells with screen or slots should be developed, especially if the well is being used to obtain water quality samples. Clays and humic materials of colloidal sizes can skew water quality tests.

Ground Surface Monitoring.—The ground surface and other pertinent points should be monitored for settlement during dewatering activities. Use standard surveying methods (at least third order) and temporary benchmarks or other facilities to maintain survey control. Horizontal control is necessary in large or deep excavations that may be susceptible to slope failure. Special instrumentation with alarm warning capability may be necessary if ground failure might endanger life or property.

Water Surface Monitoring.—Nearby surface water bodies, swamps, drains, and other similar features should be monitored, if appropriate, for elevation and discharge changes and environmental changes.

Structural Monitoring.—Nearby structures that may be influenced by settlement or horizontal displacement caused by groundwater withdrawal should be monitored. Both horizontal and vertical measurements should be acquired to detect movement.

Performance Evaluation During Construction

The performance of the water control facilities should be evaluated and documented periodically during construction. This will ensure that the specifications are being met and that unusual or unexpected conditions are properly accommodated. Charts and diagrams such as hydrographs or plots of well or system discharge rates should be prepared at the start of dewatering operations and updated throughout the construction period. An evaluation, along with tables and graphs, should be included in monthly reports to provide essential data for final reports. The evaluation should be coordinated with the design staff to ensure complete understanding of conditions.

Final Reporting

The final construction report should include a section on water control. This section should include a chronology of dewatering and an evaluation of the performance of the facilities as well as contractor compliance with the specifications. Problem areas and unusual events such as pump or slope failures should be documented. Monitoring results, including groundwater levels and discharge rates, should be presented in the form of hydrographs and other similar plots or tabulated data.

Bibliography

Bureau of Reclamation, U.S. Department of the Interior, *Ground Water Manual*, 2nd Edition, 1995.

Cedergren, Harry R., *Seepage, Drainage and Flow Nets*, John Wiley and Sons, Inc., New York, New York, 1967.

Driscoll, Fletcher, G., *Ground Water and Wells*, Johnson Division, St. Paul, Minnesota, 1986.

Joint AEG-ASCE Symposium, *Practical Construction Dewatering*, Baltimore, Maryland, May 16, 1975.

Leonards, G.A., Editor, *Foundation Engineering,* McGraw-Hill Book Co., Inc., New York, New York, 1962.

Powers, J. Patrick, *Construction Dewatering,* John Wiley and Sons, New York, New York, 1981.

U.S. Departments of the Army, the Navy, and the Air Force, *Dewatering and Groundwater Control for Deep Excavations*, TM5-818-5, NAVFAC P-418, AFM 88-5, Chapter 6, April 1971.

Chapter 21

FOUNDATION PREPARATION, TREATMENT, AND CLEANUP

This chapter discusses foundation preparation for and the placement of the first several layers of earthfill and concrete. Preparation includes excavating overburden; shaping the foundation surface with dental concrete; filling surface irregularities with slush grout (usually a cement/water mixture poured in cracks) or dental concrete (conventional concrete used to shape surfaces, fill irregularities, and protect poor rock); treating faults, shears, or weak zones; and cleanup.

Earthfill Dams

Shaping

Shape the foundation to ensure proper compaction of fill and to prevent stress anomalies in the overlying embankment. Fractures and resultant seepage problems in embankment dams may be caused by irregularities in the foundation such as stepped surfaces, abrupt changes in slope, and excessively steep surfaces. Embankment zones may differentially settle adjacent to these areas, resulting in cracks. Arching occurs near stepped surfaces resulting in a zone of low horizontal stress adjacent to the steep surface. Preventing arching is more important in the upper 1/6 to 1/4 of a dam or in low dams where stresses are low and tension zones develop along steep or diverging abutments. Tension zones or areas of low stresses are susceptible to hydraulic fracturing. The foundation surface should be shaped by excavating or by using concrete to obtain a smooth, continuous surface that minimizes crack potential. When an inclined core is used in a rockfill dam, the core derives support from the underlying filter and transition material. The zone beneath the filter and transition should be shaped using core contact criteria.

Foundation outside the core contact must be shaped to facilitate fill placement and compaction, and unsuitable weak or compressible materials must be removed. Impervious materials beneath drainage features which would prevent drainage features from properly functioning must also be removed. Materials may move into the foundation, and erodible foundation materials may move into the embankment. Appropriately graded filters between the drainage materials and foundation may be necessary.

The minimum treatment of any foundation consists of stripping or removing organic material such as roots and stumps, sod, topsoil, wood, trash, and other unsuitable materials. Cobbles and boulders may also require removal depending on the type of embankment material to be placed.

The weak points in earthfill dams are generally within the foundation and especially at the contact of the foundation with the embankment. Foundation seepage control and stability features must be carefully supervised by the inspection force during construction to ensure conformance with the design, specifications, and good practice. Water control methods used in connection with excavating cutoff trenches or stabilizing the foundations should ensure that fine material is not washed out of the foundation because of improper screening of wells and that the water level is far enough below the foundation surface to permit construction “in the dry.” Whenever possible, locate well points and sumps outside the area to be excavated to avoid loosening soil or creating a "quick" bottom caused by the upward flow of water or equipment vibration. Avoid locating sumps and associated drainage trenches within the impervious zone because of the difficulty in properly grouting them after fill placement and the danger of damaging the impervious zone/foundation contact.

Found cutoff walls or concrete grout caps in the best rock available. Prohibit or strictly control blasting for the excavation of these structures to avoid damaging the foundation. If the material cannot be excavated with a hydraulic excavator fitted with a rock bucket, a grout cap or cutoff wall is probably not needed and grout nipples can be set directly in the foundation. A highly weathered zone can be grouted effectively by:

- Leaving the foundation high.

- Setting grout nipples through the unsuitable material. Long grout nipples may be necessary in poor rock.

- Excavating to final foundation grade after grouting.

In hard, sound rock, neither a grout cap nor a high foundation is necessary.

When overburden is stripped to rock, carefully clean the rock surface and all pockets or depressions of soil and rock fragments before the embankment is placed. This may require compressed air or water cleaning and handwork. Rock surfaces that slake or disintegrate rapidly on exposure must be protected or covered immediately with embankment material or concrete. Foundation rock should be shaped to remove overhangs and steep surfaces (figure 21-1). High rock surfaces must be stable during construction and should be cut back to maintain a

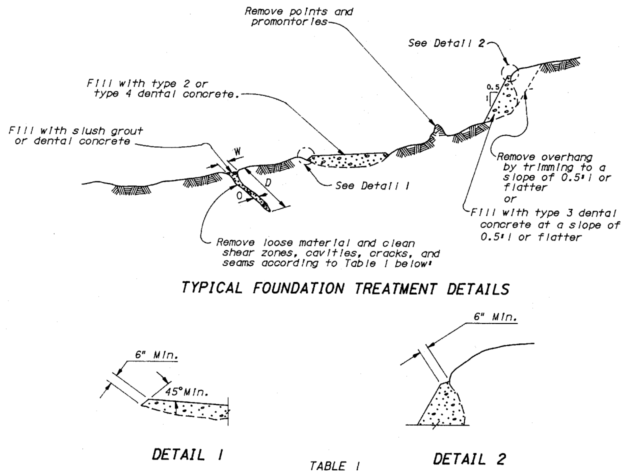


TABLE I

CASE	WIDTH OF FEATURE (W)	DEPTH OF FEATURE (D)	TREATMENT
1.	2 Inches or less	3 Times the width	Slush grout
2.	2 Inches to 5 feet	3 Times the width or to a depth where opening (O) is 0.5 inch or less	Type 4 dental concrete
3.	Greater than 5 feet	3 Times the width or as directed by Contracting Officer	Type 1 dental concrete

Figure 21-1.—Example foundation treatment details from specifications.

smooth, continuous profile to minimize differential settlement and stress concentrations within the embankment. Final slopes should be 0.5:1 (horizontal to vertical [H:V]) or flatter. Beneath the impervious zone, all overhangs should be removed; stepped surfaces steeper than 0.5:1 and higher than 0.5 foot (15 cm) should be excavated or treated with dental concrete to a slope of 0.5:1 or flatter. Outside the impervious zone, all overhangs should be removed, and stepped surfaces steeper than 0.5:1 and higher than 5 ft (1.5 m) (should be excavated or treated with dental concrete to a slope of 0.5:1 or flatter.

Slush grout or joint mortar should be used to fill narrow cracks in the foundation (figure 21-1). However, they should not be used to cover exposed areas of the foundation. Slush grout and joint mortar are composed of Portland cement and water or, in some cases, Portland cement, sand, and water. The slush grout is preferably used just before fill placement to eliminate any tendency for hardened grout to crack under load. Dental concrete should be used to fill potholes and grooves created by bedding planes and other irregularities such as previously cleaned shear zones and large joints or channels in rock surfaces. Formed dental concrete can be used to fillet steep slopes and fill overhangs.

Care should be used during all blasting to excavate or to shape rock surfaces. Controlled blasting techniques, such as line drilling and smooth blasting, should be used.

Soil Foundations

When the foundation is soil, all organic or other unsuitable materials, such as stumps, brush, sod, and large roots, should be stripped and wasted. Stripping should be performed carefully to ensure the removal of all material that may be unstable because of saturation, shaking, or decomposition; all material that may interfere with the creation of a proper bond between the foundation and the embankment; and all pockets of soil significantly more compressible than the average foundation material. Stripping of pervious materials under the pervious or semipervious zones of an embankment should be limited to the removal of surface debris and roots unless material removal is required for seismic stability. Test pits should be excavated if the stripping operations indicate the presence of unstable or otherwise unsuitable material.

Before placing the first embankment layer (lift) on an earth foundation, moistening and compacting the surface by rolling with a tamping roller is necessary to obtain proper bond. An earth foundation surface sometimes requires scarification by disks or harrows to ensure proper bonding. No additional scarification is usually necessary if the material is penetrated by tamping rollers.

All irregularities, ruts, and washouts should be removed to provide a satisfactory foundation. Cut slopes should be flat enough to prevent sloughing, and not steeper than 1:1. Material that has been loosened to a depth of less than 6 in (15 cm) may be treated by compaction. Loosened material deeper than 6 in (15 cm) cannot be adequately compacted and should be removed.

Foundation materials at the core/foundation contact must be compacted to a density compatible with the overlying fill material. A fine-grained foundation should be compacted and disked to obtain good mixing and bond between the foundation and the first lift of core material.

Fine-grained foundations should be compacted with a roller. If the foundation is too firm for the tamping feet to penetrate, the foundation surface should be disked to a depth of 6 in and moistened before compaction. Smooth surfaces created by construction traffic on a previously compacted foundation surface should be disked to a depth of 2 in (5 cm).

Coarse-grained foundations should be compacted by rubber-tired or vibratory rollers. Vibratory compactors create a more uniform surface for placement of the first earthfill and are the preferred method of compaction.

Cemented and highly overconsolidated soils that break into hard chunks should not be reworked or disked to mix foundation and core material. The first lift of embankment material should be placed in a manner similar to that required for rock foundations as described above.

Soil foundation compaction requirements beneath filter and shell zones should be the same as those outlined above, except bonding the foundation to the overlying fill is not required.

The moisture content of the upper 6 in (15 cm) of a fine-grained soil foundation should be within 2 percent dry and 1 percent wet of the Proctor optimum moisture content for adequate compaction. Coarse-grained foundation materials should be just wet enough to permit compaction to the specified relative density, but saturation is not permitted. Dry materials must be disked and moistened to provide a homogeneous moisture content within the specified limits in the upper 6 in (15 cm) of the foundation. Wet materials must be dried by disking to bring the upper 6 in of foundation materia1 within the specified moisture content limits. Wet foundations should be unwatered or dewatered sufficiently to prevent saturation of the upper 6 in (15 cm) of foundation material due to capillary rise or pumping caused by construction equipment travel.

All embankment materials should be protected from eroding into coarser soil zones in the foundation by transitions satisfying filter criteria or by select zones of highly plastic, nonerodible material. Transition zones or filters on the downstream face of the cutoff trench and beneath the downstream zones should prevent movement of fine material in the foundation into the embankment. Dispersive embankment materials must be protected from piping into coarse material in the foundation by placing select zones of nondispersive material between the embankment and foundation. Lime-treated or naturally nondispersive earthfill is appropriate for the first several lifts of fill material or filters. Except for areas where an impervious seal between the embankment and foundation is required, filter zone(s) are the preferred method.

Rock Foundations

Rock foundation surfaces should be moistened, but no standing water should be permitted when the first lift is placed. The use of very wet soil for the first lift against the foundation should generally be avoided, but having the soil slightly wet of optimum moisture content is better. Any material more plastic than what is typically available for embankment construction is commonly used on the first few lifts. The foundation should be properly moistened to prevent drying of the soil. On steep, irregular rock abutments, material slightly wetter than optimum may be necessary to obtain good workability and a suitable bond. Be careful when special compaction is used to ensure that suitable bonds are created between successive layers of material. This usually requires light scarification between lifts of compacted material. Where a rock foundation would be damaged by penetration of the tamping roller feet, the first compacted lift can be thicker than that specified. The first lift thickness should never exceed 15 in (40 cm) loose for 9-in-long (23-cm) tamper feet, and additional roller passes are probably required to ensure proper compaction. Special compaction methods, such as hand tamping, should be used in pockets that cannot be compacted by roller instead of permitting an unusually thick initial lift to obtain a uniform surface for compaction. Irregular rock surfaces may prevent proper compaction by rollers, and hand compaction may be necessary. However, where foundation surfaces permit, a pneumatic-tire roller or pneumatic-tire equipment should be used near foundation contact surfaces. An alternative to using thick lifts is using a pneumatic-tire roller or loader with a full bucket and disking or scarifying the lift surfaces to obtain a bond between lifts. The tamping roller can be used when the fill is sufficiently thick and regular to protect the foundation from the tamping feet. Unit weight and moisture should be carefully monitored in the foundation contact zone, and placing and compacting operations should be carefully inspected.

On steep surfaces, ramping the fill aids compaction; about a 6:1 slope should be used for ramping the fill. The surfaces of structures should be sloped (battered) at about 1:10 to facilitate compaction.

The basic objectives of foundation surface treatment within the impervious core are:

- Obtain a good bond between the impervious core materials and the foundation. The foundation surface must be shaped by excavation or concrete placement to provide a surface suitable for earthfill compaction. Compaction techniques used for initial earthfill placement should result in adequately compacted embankment material in intimate contact with and tightly bonded to the foundation without damaging the foundation during placement of the first lifts. A plastic material is preferred next to the foundation, and special requirements on the plasticity, gradation, and moisture content are commonly specified.

- Defend against erosion of embankment materials into the foundation by filling or covering surface cracks in the foundation, using blanket grouting, protective filters, and nonerodible embankment materials (natural or manufactured) at the foundation contact.

- Remove erodible, weak, unstable, compressible, loose, or pervious materials to ensure a foundation of adequate strength and appropriate permeability. When in doubt, take it out. In rock foundations, defects such as faults, fractures, erosion channels, or solution cavities or channels sometimes cannot be completely removed. Material defects in the rock mass include fault gouge, rock fragments, soft or pervious soil, or solutioned rock. These materials require removal to an adequate depth and replacement with slush grout, dental concrete, or specially compacted earthfill.

How the exposed rock surface is treated after removal of unsuitable overlying materials depends on the type of rock and the irregularities present. Construction activities such as using tracked equipment on soft rock surfaces, using rippers near foundation grade, or nearby blasting may loosen rock or open joints in originally satisfactory rock. This type of damage should and can be avoided to limit excavation and cleanup. The configuration of exposed rock surfaces is controlled largely by bedding, joints, other discontinuities, and excavation methods. Depending on discontinuity orientations, these features can result in vertical surfaces, benches, overhangs, or sawteeth. Features such as potholes, buried river channels, solution cavities, and shear zones can create additional irregularities requiring treatment. Unsuitable material must be removed from the irregularities, and the foundation surface must be shaped to provide a sufficiently regular surface that earthfill can be placed without differential settlement. If the irregularities are small enough and discontinuous both horizontally and vertically, overexcavation can be appropriate. Generally, the foundation surface can be shaped adequately by conventional excavation or smooth blasting. When smoothing of irregularities requires excessively large quantities of excavation or requires blasting that may damage the foundation, shaping with dental concrete may be appropriate.

High rock surfaces must be stable during construction and must be laid back to maintain a smooth, continuous profile to minimize differential settlement and stress concentrations. Slopes should be 0.5:1 (H:V) or flatter, depending on the fill material.

Remove overhangs. Stepped surfaces that are steeper than 0.5:1 and higher than 1 foot (30 cm) should be excavated or treated with dental concrete to a resultant slope of 0.5:1 or flatter, depending on the fill material (figure 21-1). In the lower portions of a high dam, this requirement may be relaxed. For example, a vertical surface less than 20 ft (6.1 m) long and 5 ft (1.5 m) high might be tolerated if the surface is well within the impervious zone.

Remove all overhangs under the outer zones of earthfill dams and under transitions and filters of rockfill dams. Stepped surfaces that are steeper than 0.5:1 and higher than 5 ft (1.5 m) should be flattened or treated with dental concrete to a slope of 0.5:1 or flatter. Beneath the outer rockfill zones of rockfill dams, nearly vertical abutment contact slopes have been permitted in high, steep-walled canyons. Overhangs should be trimmed or the undercut below the overhang filled with concrete.

If shaping requires blasting, proper blasting procedures are essential to ensure that the permeability and strength of the rock is not adversely affected and that the rock can stand on the slopes and handle the imposed loads. Existing fractures and joints in a rock mass, as well as poor blasting, often result in unacceptable excavated surfaces. Prior competent review, approval, and enforcement of the Contractor's blasting plan, control of blasting details, requirements for acceptabi1ity of the excavated surface, and control of vibration levels can help obtain the desired excavation surface.

All loose or objectionable material should be removed by handwork, barring, picking, brooming, water jetting, or air jetting. Remove accumulated water from cleaning operations. When the rock surface softens or slakes by water washing, compressed-air jetting or jetting with a small amount of water added to the air should be used. Loose or unsuitable material in cavities, shear zones, cracks, or seams should be treated as follows (figure 21-1):

- Openings narrower than 2 in (5 cm) should be cleaned to a depth of three times the width of the opening and treated.

- Openings wider than 2 in (5 cm) and narrower than 5 ft (1.5 m) should be cleaned to a depth of three times the width of the opening or to a depth where the opening is 0.5 in (12mm) wide or less, but not to a depth exceeding 5 ft (5 cm) and treated.

- Openings wider than 5 ft (1.5 m) are a special case where the required depth of cleaning and treatment is determined in the field.

Special cleanup procedures are required for foundation materials that deteriorate when exposed to air or water (slake). The foundation must be kept moist if deterioration is caused by exposure to air and kept dry if deterioration is caused by exposure to water. Spray coating with material (similar to concrete curing compound) designed to reduce slaking may (but probably will not) be effective. Cleanup and placing a lean concrete “mud slab” approximately 4 in (15 cm) thick may be effective. Usually, removing the last few inches of material and doing final cleanup just before first placement of fill is the best approach. A maximum time interval may also be specified between the time of exposure of the final grade and the time that the foundation is protected with earthfill or a suitable protective coating.

Cleanup outside the core is normally less critical. Loose material should be removed so that the embankment is in direct contact with suitable rock. If defects are contained within the foundation area, they may not require cleaning and refill. If a defect crosses the entire foundation area, it may require cleaning similar to the foundation beneath the core.

Dental concrete is used to fill or shape holes, grooves, extensive areas of vertical surfaces, and sawteeth created by bedding planes, joints, and other irregularities such as previously cleaned out solution features, shear zones, large joints, or buried channels. Formed dental concrete can be used to fillet steep slopes and fill overhangs. Placing a concrete mat over a zone of closely spaced irregularities is appropriate in local areas that are not large in relation to the core dimensions. Dental concrete shaping can be used instead of excavation by blasting or when excessive amounts of excavation would otherwise be required.

Slabs of dental concrete should have a minimum thickness of 6 in (15 cm) if the foundation is weak enough to allow cracking of the concrete under load (figure 21-1).

Thin areas of dental concrete over rock projections on a jagged rock surface are likely places for concrete cracking and should be avoided by using a sufficient thickness of dental concrete or by avoiding continuous slabs of concrete over areas containing numerous irregularities on weak foundations. Feathering at the end of concrete slabs on weak foundations should not be permitted, and the edges of slabs should be sloped no flatter than 45 degrees. Formed dental concrete should not be placed on slopes greater than 0.5:1 (H:V). When dental concrete fillets are placed against vertical or nearly vertical surfaces in weak rock, feathering should not be permitted, and a beveled surface with a minimum thickness of 6 in (15 cm) is required at the top of the fillet.

Concrete mix proportions should provide a 28-day strength of 3,000 pounds per square inch (21 MPa). The maximum aggregate size should be less than one-third the depth of slabs or one-fifth the narrowest dimension between the side of a form and the rock surface. Cement type will depend on the concentration of sulfates in the foundation materials and groundwater. Low-alkali cement is required for alkali-reactive aggregates. Cement type should be the same as used in structural concrete on the job. Aggregate and water quality should be equal to that required in structural concrete.

The rock surface should be thoroughly cleaned, as described below, and moistened before concrete placement to obtain a good bond between the concrete and the rock surface. When overhangs are filled with dental concrete, the concrete must be well bonded to the upper surface of the overhang. The overhang should be shaped to allow air to escape during concrete placement to prevent air pockets between the concrete and the upper surface of the overhang. The concrete must be formed and placed so that the head of the concrete is higher than the upper surface of the overhang. If this is impractical, grout pipes should be installed in the dental concrete for later filling of the air voids. If grouting through dental concrete is done, pressures should be closely controlled to prevent jacking the concrete or fracturing the fill.

Finished horizontal dental concrete slabs should have a roughened, broomed finish for satisfactory bonding of fill to concrete. Dental concrete should be cured by water or an approved curing compound for 7 days or covered by earthfill. Earthfill placement may not be permitted over dental concrete for a minimum of 72 hours or more after concrete placement to allow concrete time to develop sufficient strength to withstand stress caused by placing earthfill. Inadequate curing may result in the concrete cracking.

Slush grout is a neat cement grout or a sand-cement slurry that is applied to cracks in the foundation. Cracks or joints are filled with grout rather than spreading grout on the surface (figure 21-1). Slush grout should be used to fill narrow surface cracks and not used to cover areas of the foundation. Slush grout may consist of cement and water, or sand, cement, and water. To ensure adequate penetration of the crack, the maximum particle size in the slush grout mixture should be no greater than one-third the crack width. Generally, neat cement grout is used. The consistency of the slush grout mix may vary from a very thin mix to mortar as required to penetrate the crack. The grout preferably should be mixed with a mechanical or centrifugal mixer, and the grout should be used within 30 minutes after mixing.

The type of cement required will depend on the concentration of sulfates in the foundation materials and groundwater. Low-alkali cement is required for alkali sensitive aggregates. Cement type should be that specified for structural concrete. Sand and water quality should be equal to that required for structural concrete.

Clean out cracks as described above. All cracks should be wetted before placing slush grout. Slush grout may be applied by brooming over surfaces containing closely spaced cracks or by troweling, pouring, rodding, or funneling into individual cracks. Slush grout is best applied just before material placement so cracking will not occur during compaction.

Shotcrete is concrete or mortar that is sprayed in place. The quality of the shotcrete depends on the skill and experience of the crew, particularly regarding the amount of rebound, thickness, feather edges, and ensuring adequate thickness over protrusions on irregular surfaces. Untreated areas can be inadvertently covered because of the ease and rapidity of placement. Shotcrete should be used beneath impervious zones only when site conditions preclude using dental concrete. If shotcrete is used, close inspection and caution are necessary. Shotcrete is an acceptable alternative to dental concrete outside the core contact area.

The fill compaction method used depends on the steepness of the surface, the nature of the irregularities in the foundation surface, and the soil material.

A hand tamper may be used to compact earthfill in or against irregular surfaces on abutments, in potholes and depressions, and against structures not accessible to heavy compaction equipment. Hand-tamped, specially compacted earthfill is typically placed in 4-in (10-cm) maximum compacted lifts with scarification between lifts.

Site-specific conditions determine whether hand-compacted earthfill or filling with dental concrete is the best solution.

The feet of the roller must not penetrate the first layer of earthfill and damage the foundation. Penetration can be prevented by using a rubber-tired roller or loader to compact the first few lifts above the foundation surface with scarification between lifts. Earthfill specially compacted by pneumatic-tired equipment is typically placed in 6-in (15-cm) maximum compacted lifts. Placement of horizontal lifts against mildly sloping rock surfaces can result in feathering of the earthfill lift near the rock contact. Placement of the initial lift parallel to the foundation surface (as opposed to a horizontal lift) for foundation surfaces flatter than 10:1 (H:V) is acceptable if the compactor climbing up the slope does not loosen or disturb the previously compacted earthfill.

Core material compacted against steep surfaces is typically placed in 6-in (15-cm) compacted lifts with scarification between lifts. Earthfill 8 to 10 ft (2.4 to 3 m) from a steep surface should be ramped toward the steep surface at a slope of 6:1 to 10:1 so that a component of the compactive force acts toward the steep surface.

Earthfill placed against irregular surfaces should be plastic and deformable so that the material is forced (squished) into all irregularities on the foundation surface by compaction or subsequent loading. The first layer soil moisture contents should range from 0 to 2 percent wet of optimum. Select material with a required plasticity range is commonly specified. A soil plasticity index ranging from 16 to 30 is preferred although not absolutely necessary.

Core materials that are erodible include low plasticity or nonplastic, silty materials and dispersive clays. Preventing erosion of embankment materials into the foundation includes sealing cracks in the foundation with slush grout and dental concrete and using filter zone(s) between the fine-grained material and the foundation. Sealing cracks is not totally reliable because concrete and mortar can crack due to shrinkage or loading. Using natural or manufactured nonerodible material for the first several lifts of embankment at the core-foundation contact is good practice.

If erosion-resistant plastic materials are available, these materials should be used for the first several lifts along the foundation contact to avoid placing erodible nonplastic materials directly against the rock surface. If plastic materials are not available, the natural soil can be mixed with sodium bentonite to produce core material to be placed against the foundation. Laboratory testing should establish the amount of sodium bentonite required to give the soil the characteristics of a lean clay.

If nondispersive material is available, it should be used instead of dispersive material, at least in critical locations such as along the core-foundation contact. In deposits containing dispersive material, the dispersion potential generally varies greatly over short distances. Selectively excavating nondispersive material from a deposit containing dispersive materials is frequently difficult and unreliable. Lime can be added to dispersive materials to reduce or convert the soil to a nondispersive material. The amount of lime required to treat the dispersive soil should be established by performing dispersivity tests on samples of soils treated with varying percentages of lime. Adding lime to a soil results in reduced plasticity and a more brittle soil. The lime content should be the minimum required to treat the soil. Do not treat material that has naturally low plasticity with lime if it is not necessary.

Concrete Arch Dams

The entire area to be occupied by the base of a concrete arch dam should be excavated to material capable of withstanding the loads imposed by the dam, reservoir, and appurtenant structures. Blasting operations must not damage the rock foundation. All excavations should conform to the lines and dimensions shown on the construction drawings, where practical, but it may be necessary to vary dimensions or excavation slopes because of local conditions.

Foundations containing seams of shale, siltstone, chalk, or mudstone may require protection against air and water slaking or, in some environments, against freezing. Excavations can be protected by leaving a temporary cover of unexcavated material, by immediately covering the exposed surfaces with a minimum of 12 in (30 cm) of concrete, or by any other method that will prevent damage to the foundation.

Shaping

Although not considered essential, a symmetrical or nearly symmetrical profile is desirable for an arch dam for stress distribution. However, asymmetrical canyons frequently have to be chosen as arch dam sites. The asymmetry may introduce stress problems, but these can be overcome by proper design. Abutment pads between the dam and foundation may be used to overcome some of the detrimental effects of asymmetry or foundation irregularities. Thrust blocks are sometimes used at asymmetrical sites. The primary use of a thrust block is not to provide symmetry but to establish an artificial abutment where a natural one does not exist. Overexcavation of a site to achieve symmetry is not recommended. In all cases, the foundation should be excavated in such a way as to eliminate sharp breaks in the excavated profile because these may cause stress concentrations in both the foundation rock and the dam. The foundation should also be excavated to about radial or part radial lines.

Dental Treatment

Exploratory drilling or final excavation often uncovers faults, seams, or shattered or inferior rock extending to depths that are impracticable to remove. Geologic discontinuities can affect both the stability and the deformation modulus of the foundation. In reality, the foundation modulus need not be known accurately if the ratio of the foundation modulus E_f to the concrete modulus E_c of the dam is known to be greater than 1:4. Canyons with extensive zones of highly deformable materials and, consequently, E_f / E_c ratios less than 1:4 should still be considered potential arch dam sites. The deformation modulus of weak zones can be improved by removing sheared material, gouge, and inferior rock and replacing the material with backfill concrete.

Analytical methods can provide a way to combine the physical properties of different rock types and geologic discontinuities such as faults, shears, and joint sets into a value representative of the stress and deformation in a given segment of the foundation. This also permits substitution of backfill concrete in faults, shears, and zones of weak rock and evaluates the benefit contributed by dental treatment.

Data required for analysis are the dimensions and composition of the lithologic bodies and geologic discontinuities, deformation moduli for each of the elements incorporated into the study, and the loading pattern imposed by the dam and reservoir.

Dental treatment concrete may also be required to improve the stability of a rock mass. By using data related to the shear strength of faults, shears, joints, intact rock, pore water pressures induced by the reservoir and/or groundwater, the weight of the rock mass, and the driving force induced by the dam and reservoir, a safety factor for a particular rock mass can be calculated.

Frequently, relatively homogeneous rock foundations with only nominal faulting or shearing do not require the sophisticated analytical procedures described above. The following approximate formulas for determining the depth of dental treatment can be used:

$$d = 0.002\ bH + 5 \text{ for } H \geq 150 \text{ ft}$$

$$d = 0.3b+5 \text{ for } H<150 \text{ ft}$$

where:

- H = height of dam above general foundation level in ft
- b = width of weak zone in ft
- d = depth of excavation of the weak zone below the surface of adjoining sound rock in ft. In clay gouge seams, d should not be less than 0.1 H.

These rules provide an estimate of how much should be excavated, but final decisions must be made in the field during actual excavation.

Protection Against Piping

The methods described in the preceding paragraphs will satisfy the stress, deformation, and stability requirements for a foundation, but the methods may not provide suitable protection against piping. Faults, shears, and seams may contain pipable material. To protect against piping, upstream and downstream cutoff shafts may be necessary in each seam, shear, or fault, and the shafts backfilled with concrete. The dimension of the shaft perpendicular to the seam should be equal to the width of the weak zone plus a minimum of 1 foot on each end to key the concrete backfill into sound rock. The shaft dimension parallel to the seam should be at least one-half the other dimension. In any instance a minimum shaft dimension of 5 ft (1.6 m) each way should be used to provide working space. The depth of cutoff shafts may be computed by constructing flow nets and computing the cutoff depths required to eliminate piping.

Other adverse foundation conditions may be caused by horizontally bedded clay and shale seams, caverns, or springs. Procedures for treating these conditions will vary and will depend on the characteristics of the particular condition to be remedied.

Foundation Irregularities

Although analyses and designs assume relatively uniform foundation and abutment excavations, the final excavation may vary widely from that which was assumed. Faults or crush zones are often uncovered during excavation, and the excavation of the unsound rock leaves depressions or holes which must be filled with concrete. Unless this backfill concrete has undergone most of its volumetric shrinkage at the time overlying concrete is placed, cracks can occur in the overlying concrete near the boundaries of the backfill concrete as loss of support occurs because of continuing shrinkage of the backfill concrete. Where dental work is extensive, the backfill concrete should be placed and cooled before additional concrete is placed over the area.

Similar conditions exist where the foundation has abrupt changes in slope. At the break of slope, cracks often develop because of the differential movement that takes place between concrete held in place by rock and concrete held in place by previously placed concrete that has not undergone its full volumetric shrinkage. A forced cooling of the concrete adjacent to and below the break in slope and a delay in placement of concrete over the break in slope can minimize cracking at these locations. If economical, the elimination of these points of high stress concentration is worthwhile. Cracks in lifts near the abutments very often develop leakage and lead to spalling and deterioration of the concrete.

Concrete Gravity Dams

The entire base area of a concrete gravity dam should be excavated to material capable of withstanding the loads imposed by the dam, reservoir, and appurtenant structures. Blasting should not shatter, loosen, or otherwise adversely affect the suitability of the foundation rock. All excavations should conform to the lines and dimensions shown on the construction drawings, where practicable. Dimensions or excavation slopes may vary because of local conditions.

Foundations such as shale, chalk, mudstone, and siltstone may require protection against air and water slaking or, in some environments, against freezing. These excavations can be protected by leaving a temporary cover of unexcavated material, by immediately applying a minimum of 12 in (30 cm) of mortar to the exposed surfaces, or by any other method that will prevent damage to the foundation.

Shaping

If the canyon profile for a damsite is relatively narrow and the walls are steeply sloping, each vertical section of the dam from the center towards the abutments is shorter in height than the preceding one. Consequently, sections closer to the abutments will be deflected less by the reservoir load, and sections toward the center of the canyon will be deflected more. Since most gravity dams are keyed at the contraction joints, a torsional effect on the dam is transmitted to the foundation rock.

A sharp break in the excavated profile of the canyon will result in an abrupt change in the height of the dam. The effect of the irregularity of the foundation rock causes a marked change in stresses in the dam and foundation and in stability factors. For this reason, the foundation should be shaped so that a uniform profile without sharp offsets and breaks is obtained.

Generally, a foundation surface will appear as horizontal in the transverse (upstream-downstream) direction. However, where an increased resistance to sliding is desired, particularly for structures founded on sedimentary rock foundations, the surface can be sloped upward from heel to toe of the dam.

Dental Treatment

Faults, seams, or shattered or inferior rock extending to depths that are impractical to remove require special treatment by removing the weak material and backfilling the resulting excavations with dental concrete. General rules for how deep transverse seams should be excavated have been formulated based on actual foundation conditions and stresses in dams. Approximate formulas for determining the depth of dental treatment are:

$$d = 0.002\,bH + 5 \text{ for } H \geq 150 \text{ ft}$$

$$d = 0.3\,b + 5 \text{ for } H < 150 \text{ ft}$$

where:

H = height of dam above general foundation level in feet

b = width of weak zone in feet

d = depth of excavation of weak zone below surface of adjoining sound rock in feet. In clay gouge seams, d should not be less than 0.1 H.

These rules provide guidance for how much should be excavated, but actual quantities should be determined in the field during excavation.

Although the preceding rules are suitable for foundations with a relatively homogeneous rock foundation and nominal faulting, some damsites may have several distinct rock types interspersed with numerous faults and shears. The effect on the overall strength and stability of the foundation of rock differences complicated by large zones of faulting may require extensive analysis. Data required for analysis include the dimensions and composition of the rock mass and geologic discontinuities, deformation moduli for each of the elements incorporated into the study, and the loading pattern imposed on the foundation by the dam and reservoir.

Dental treatment may also be required to improve the stability of the rock mass. A safety factor for a particular rock mass can be calculated using data related to the strength of faults, shear zones, joints, intact rock, pore water pressures induced by the reservoir and/or groundwater, the weight of the rock mass, and the driving forces induced by the dam and reservoir.

Protection Against Piping

The methods discussed above can satisfy the stress, deformation, and stability requirements for a foundation, but they may not provide suitable protection against piping. Faults and seams may contain pipeable material, and concrete backfilled cutoff shafts may be required in each fault or seam. The dimension of the shaft perpendicular to the seam should be equal to the width of the weak zone plus a minimum of 1 foot (0.3 m) on each end to key the concrete backfill into sound rock. The shaft dimension parallel to the seam should be at least one-half the other dimension. A minimum shaft dimension of 5 ft (1.6 m) each way should be provided for working space. The depth of cutoff shafts may be computed by constructing flow nets and calculating the cutoff depths required to eliminate piping.

Other adverse foundation conditions may be caused by horizontally bedded clay and shale seams, caverns, or springs. Procedures for treating these conditions will vary and will depend on field studies of the characteristics of the particular condition to be remedied.

Foundation Irregularities

Although analyses and designs assume relatively uniform foundation and abutment excavations, the final excavation may vary widely from that which was assumed.

Faults or crush zones are often uncovered during excavation, and the excavation of the unsound rock leaves depressions or holes that must be filled with concrete. Unless this backfill concrete has undergone most of its volumetric shrinkage at the time overlying concrete is placed, cracks can occur in the overlying concrete near the boundaries of the backfill concrete. Shrinkage of the backfill concrete results in loss of support. Where dental work is extensive, the backfill concrete should be placed and cooled before additional concrete is placed over the area.

Similar conditions exist where the foundation has abrupt changes in slope. At the break of slope, cracks often develop because of the differential movement that takes place between concrete held in place by rock and concrete held in place by previously placed concrete that has not undergone its full volumetric shrinkage. A forced cooling of the concrete adjacent to and below the break in slope and a delay in placement of concrete over the break in slope can be employed to minimize cracking at these locations. If economical, the elimination of these points of high stress concentration is worthwhile. Cracks in lifts near the abutments very often leak and lead to spalling and deterioration of the concrete.

Cleanup

Proper cleaning and water control on a foundation before placing fill or concrete allows the structure and soil or rock contact to perform as designed. Good cleanup allows the contact area to have the compressive and shear strength and the permeability anticipated in the design. Poor cleanup reduces the compressive and shear strength resulting in a weak zone under the structure and providing a highly permeable path for seepage.

Cleaning

Foundation cleanup is labor intensive and costly, so it is routinely neglected. The result is substandard foundations that do not meet design requirements. Rock foundations should be cleaned by:

- Barring and prying loose all drummy rock
- Using an air/water jet to remove as much loose material and fluff as possible
- Removing by hand loose material that an air/water jet misses

Soil foundations should be cleaned by removing material missed by machine stripping that will not be suitable foundation after compaction.

Foundations of weak rock or firm soil can be cleaned by placing a steel plate (butter bar) across the teeth of a backhoe or hydraulic excavator and "shaving" or "peeling" objectionable material off the surface, leaving a clean foundation requiring very little hand cleaning.

Water Removal

Water in small quantities can be removed by vacuuming (with a shop vac or air-powered venturi pipe) or blotting with soil and wasting the wet material just before fill placement. Larger water quantities from seeps can be isolated in gravel sumps and pumped. Grout pipes should be installed; the sumps covered with fabric or plastic; fill placed over the fabric; and after the fill is a few feet above the sump, the sump should be cement grouted by gravity pressure. Seeps in concrete foundations can be isolated and gravel sumps constructed and subsequently grouted as described above or if the seeps are not too large, the concrete can be used to displace the water out of the foundation during placement.

Bibliography

Abutment and Foundation Treatment for High Embankment Dams on Rock, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM10, pp.1017-1032, October 1972.

Current Practice in Abutment and Foundation Treatment, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM10, pp. 1033-1052, October 1972.

Earth and Rockfill Dams, New York, John Wiley and Sons, Inc., 1963.

Foundations and Abutments - Bennett and Mica Dams, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM10, pp. 1053-1072, October 1972.

Foundation Practices for Talbingo Dam, Australia, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM10, pp. 1081-1098, October 1972.

Foundation Treatment for Embankment Dams on Rock, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM10, pp. 1073-1080, October 1972.

Foundation Treatment for Rockfill Dams, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM10, pp. 995-1016, October 1972.

Preparation of Rock Foundations for Embankment Dams, Embankment Dams Engineering, New York, John Wiley and Sons, Inc., pp. 355-363, 1973.

Treatment of High Embankment Dam Foundations, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM10, pp. 1099-1113, October 1972.

Chapter 22

PENETRATION TESTING

Introduction

This chapter discusses the Standard Penetration Test (SPT), Becker Penetration Test (BPT), and Cone Penetration Test (CPT). Penetration tests are used to determine foundation strength and to evaluate the liquefaction potential of a material. SPTs for liquefaction evaluations are stressed in the discussion. The significant aspects of the tests and the potential problems that can occur are included.

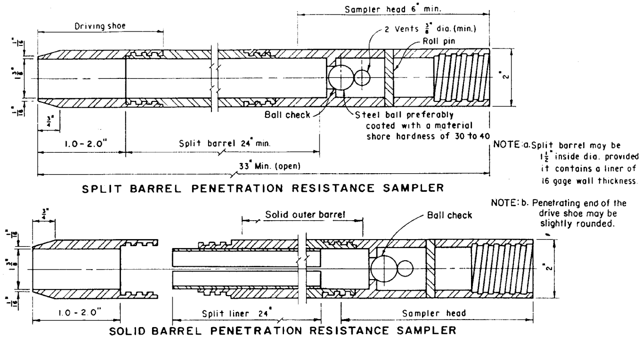
History

Penetration resistance testing and sampling with an open ended pipe was started in the early 1900s. The Raymond Concrete Pile Company developed the Standard Penetration Test with the split barrel sampler in 1927. Since then, the SPT has been performed worldwide. The SPT or variations of the test are the primary means of collecting geotechnical design data in the United States. An estimated 80-90 percent of geotechnical investigations consist of SPTs.

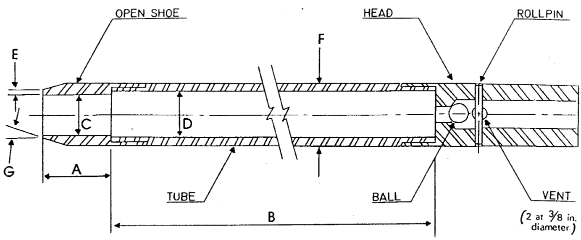
Standard Penetration Testing

Equipment and Procedures

The SPT consists of driving a 2-inch (5-cm) outside diameter (OD) "split barrel" sampler (figure 22-1) at the bottom of an open borehole with a 140-pound (63.6-kg) hammer dropped 30 inches (75 cm). The "N" value is the number of blows to drive the sampler the last 1 foot (30 cm), expressed in blows per foot. After the penetration



D 1586



A = 1.0 to 2.0 in. (25 to 50 mm)
B = 18.0 to 30.0 in. (0.457 to 0.762 m)
C = 1.375 ± 0.005 in. (34.93 ± 0.13 mm)
D = 1.50 ± 0.05 − 0.00 in. (38.1 ± 1.3 − 0.0 mm)
E = 0.10 ± 0.02 in. (2.54 ± 0.25 mm)
F = 2.00 ± 0.05 − 0.00 in. (50.8 ± 1.3 − 0.0 mm)
G = 16.0° to 23.0°

The 1½ in. (38 mm) inside diameter split barrel may be used with a 16-gage wall thickness split liner. The penetrating end of the drive shoe may be slightly rounded. Metal or plastic retainers may be used to retain soil samples.

Split-Barrel Sampler

Figure 22-1.—ASTM and Reclamation SPT sampler requirements.

test is completed, the sampler is retrieved from the hole. The split barrel is opened, the soil is classified, and a moisture specimen is obtained. After the test, the borehole is extended to the next test depth and the process is repeated. SPT soil samples are disturbed during the driving process and cannot be used as undisturbed specimens for laboratory testing.

The American Society of Testing and Materials standardized the test in the 1950s. The procedure required a free falling hammer, but the shape and drop method were not standardized. Many hammer systems can be used to perform the test, and many do not really free fall. The predominant hammer system used in the United States is the safety hammer (figure 22-2) that is lifted and dropped with the a rope and cat head. Donut hammers (figure 22-3) are operated by rope and cat head or mechanical tripping. Donut hammers are not recommended because the hammers are more dangerous to operate and are less efficient than safety hammers. Automatic hammer systems are used frequently and are preferred because the hammers are safer and offer close to true free fall conditions, and the results are more repeatable.

The SPT should not be confused with other thick-wall drive sampling methods such as described in ASTM Standard D 3550 which covers larger ring-lined split barrel samplers with up to 3-inch (7.6-cm) OD. These samplers are also know as "California" or "Dames & Moore" samplers. These drive samplers do not meet SPT requirements because they use bigger barrels, different hammers, and different drop heights to advance the sampler.

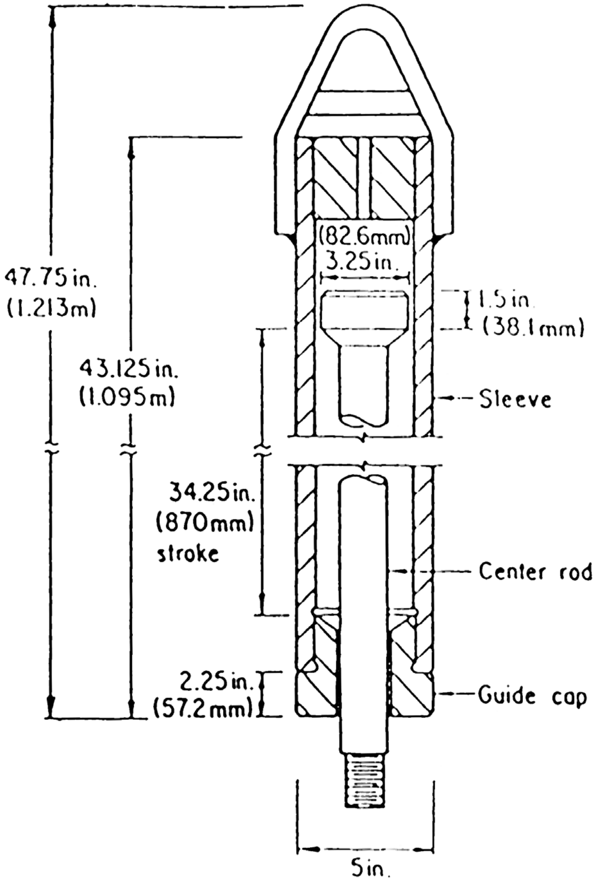


Figure 22-2.—Safety hammer.

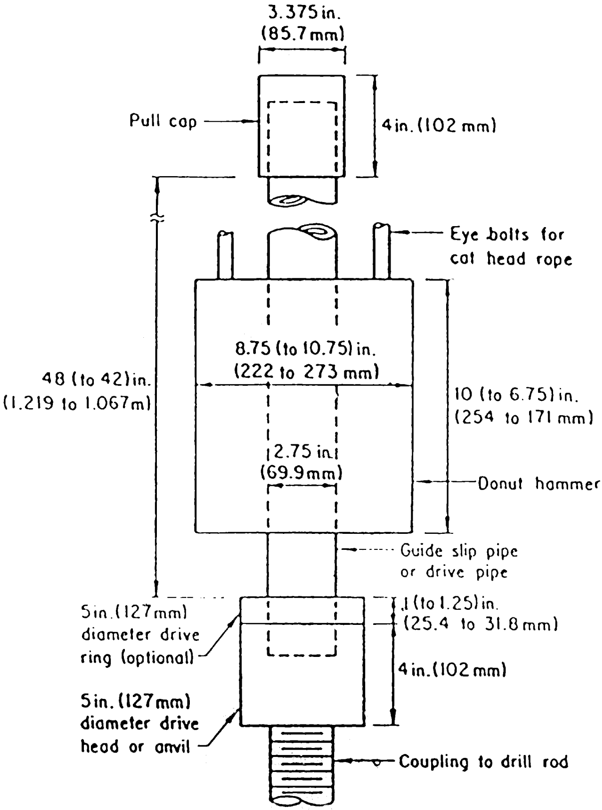


Figure 22-3.—Donut hammer.

The energy delivered to the sampler can vary widely because of the wide variety of acceptable hammer systems. Numerous studies of SPT driving systems indicate that the energy varies from 40 to 95 percent of the theoretical maximum energy. The "N" value is inversely proportional to the energy supplied to the sampler, and the energy delivered to the sampler is critical. Because of energy losses in the impact anvil, energy from the hammer should be measured on the drill rod below the impact surface. Drill rod energy ratio is determined by measuring the force-time history in the drill string. Both acceleration and force-time history can be measured and are important in determining the normalized penetration resistance of sands for liquefaction resistance evaluations (ASTM D 6066). Common practice is to normalize the SPT N value to a 60-percent drill rod energy ratio. Adjustment factors can be as large as 20 to 30 percent.

The largest cause of error in the SPT is drilling disturbance of the material to be tested. This is especially true when testing loose sands below the water table. Field studies have shown that "sanding in" can be prevented by using rotary drilling with drill mud and upward-deflected-discharge bits and by maintaining the fluid level in the drill hole at all times. Hollow-stem augers are especially popular for drilling in the impervious zones in dams but can cause problems when loose sand is encountered below the water table. Many other drilling methods are available for performing SPTs, and each should be evaluated relative to potential problems and how the data will be used.

Information Obtainable by SPT

The SPT does provide a soil sample. Sampling is not continuous because the closest recommended test interval is 2.5 feet (75 cm). Typical sampling is at 5-foot (1.5-m) intervals or at changes in materials. The test recovers a disturbed soil sample that can be classified on site, or the sample can be sent to the laboratory for physical properties tests.

SPT N values have been correlated to numerous soil properties. In cohesionless soils (sands), the SPT can be used to predict the relative density of sands (i.e., very loose, loose, medium, etc.) (table 22-1).

Table 22-1.—Penetration resistance and soil properties based on the SPT (Peck, et al.)

Sands (Fairly reliable)		Clays (Rather reliable)	
Number of blows per foot (30 m), N	Relative density	Number of blows per foot (30 cm), N	Consistency
		Below 2	Very soft
0-4	Very loose	2-4	Soft
4-10	Loose	4-8	Medium
10-30	Medium	8-15	Stiff
30-50	Dense	15-30	Very stiff
Over 50	Very dense	Over 30	Hard

The SPT has been widely used to predict the allowable bearing capacity of footings on sand. There are several empirical methods that are based either on case histories or on drained modulus of deformation predictions. The application of these predictions should be tempered by local experience. There are many proposed methods for estimating bearing capacity. The methods are probably slightly conservative and should be applied carefully.

SPT N values must be corrected for overburden pressures and the location of the water table.

For clays, the SPT is less reliable for predicting strength and compressibility, especially for weaker clays. The SPT is commonly used to assess the consistency of clays by grouping clays as very soft, soft, medium, etc. Predictions of undrained strengths should be used with extreme caution, especially in weak clays, because the SPT barrel remolds the clay, and the penetration resistance is more a measure of remolded strength. For evaluating undrained strength in clays, vane shear, unconfined compression, or CPTs are better than SPTs. SPT data should not be used to estimate the compressibility of clays. To evaluate compression behavior of clays, use either empirical factors based on water content and atterberg limits or obtain undisturbed samples for laboratory consolidation testing.

SPT data routinely have been used for predicting liquefaction triggered by earthquake loading. If liquefaction is predicted, the SPT data can be used to estimate the post-earthquake shear strengths. Extensive case history data have been collected to evaluate liquefaction; however, the data are subject to drilling disturbance errors and the energy delivered by the hammer system must be known. If drilling disturbance is evident or suspected, the CPT is an alternative because the soil can be tested in place. Procedures for evaluating liquefaction from SPTs are given in Reclamation's *Design Standards No. 13, Embankment Dams*, "Chapter 13, Seismic Design and Analysis." SPT N data can be used to estimate the shear modulus of clean sands, but the method is approximate. If the shear modulus is needed, directly measuring the shear wave velocity is preferred.

Liquefaction occurs when water pressure builds up in granular soils during an earthquake. Soils mostly susceptible to liquefaction are "cohesionless" soils, primarily clean sands and gravels (GP, SP, GW, SW, GP-GM, SP-SM) and silty sands and gravels (SM, GM). The term, "sands," in the following discussion refers to all these soils. The water pressure buildup results in strength loss and possibly deformation, slippage, and failure. Data collected at liquefaction sites have been used to assess whether a deposit is liquefiable.

Testing Cohesionless Soils

Earthquake induced liquefaction is commonly associated with sands below the water table. Good drilling technique is critical to ensuring that the sands are undisturbed prior to the SPT. Unfortunately, loose sand is one of the most difficult materials to drill.

If disturbed sands are present, take measures to avoid continued disturbance. Perform depth checks to assess the sand depth at the bottom of the drill hole. These depth checks are made by seeing exactly where the sampler rests before testing. Depth checks that can be made during drilling will be discussed below. Do not drill at excessive rates. Signs of disturbance are excessive slough in the SPT barrel, drill fluid in the sample, and failure of the sampler to rest at the proper cleanout depth. Slough is the disturbed material in the drill hole that caves from the sidewalls but can include disturbed sand that heaves or flows upward into the drill hole. Slough can also consist of cuttings which settle from the drill fluid before testing.

The SPT sampler must rest at the intended depth. This depth is to the end of the cleanout bit or the end of the pilot bit in hollow-stem augers. If the sampler rests at an elevation that is 0.4 foot (12 cm) different from the cleanout depth, disturbance of the soil may be occurring, and the hole must be recleaned.

There are a number of advantages to the SPT:

(1) The test is widely used, and often local experience is well developed.

(2) The test is simple, and many drillers can perform the test.

(3) The SPT equipment is rugged, and the test can be performed in a wide range of soil conditions.

(4) There are numerous correlations for predicting engineering properties with a good degree of confidence.

(5) The SPT is the only in place test that collects a soil sample.

Although the SPT is commonly used and is a flexible in place test, there are significant disadvantages. The test does not provide continuous samples. Different soils in the SPT interval tend to be logged as one soil, especially if the soil core is combined into one laboratory test specimen and laboratory data are used in the logs. Hollow-stem augers can give disturbed samples between test intervals, and the intervals between tests can be logged. The greatest disadvantage to SPTs is the lack of reproducibility of the test results. Drilling disturbance, mechanical variability, and operator variability all can cause a significant variation in test results. The SPT should not be used unless the testing is observed and logged in detail. Old data where drilling and test procedures are not documented should be used with extreme caution. Another disadvantage to SPTs is that progress is slower than other in place tests because of incremental drilling, testing, and sample retrieval, and SPTs may be more expensive than other in place tests. The SPT is influenced by more than just overburden stress and soil density. The soil type, particle size, soil age, and stress history of the soil deposit all influence SPT results.

Drilling Methods

Fluid Rotary Drilling

Rotary drilling with clear water results in N values that are much lower than N values that are obtained when drilling mud is used. Two factors are involved: (1) the water from drilling can jet into the test interval disturbing the sand, and (2) the water level in the borehole can drop and the sand can heave up the borehole when the cleanout string is removed. These two factors must be minimized as much as is practical.

The best way to drill loose, saturated sands is to use bentonite or polymer-enhanced drill fluid and drill bits that minimize jetting disturbance. Also when drilling with fluid, use a **pump bypass line** to keep the hole full of fluid as the cleanout string is removed from the drill hole. **The lack of fluid in the hole is one of the most frequent causes of disturbed sands**. If the soils are fine-grained, use a fishtail-type drag bit with baffles that deflect the fluid upwards. A tricone rockbit is acceptable if gravels or harder materials are present, but adjust the flow rates to minimize jetting.

Casing can help keep the borehole stable, but keep the casing back from the test interval a minimum of 2.5 feet

(75 cm) or more if the hole remains stable. Using a bypass line to keep the hole full of fluid is even more important with casing because the chance of sand heave up into the casing is increased if the water in the casing drops below natural groundwater level. The imbalance is focused at the bottom, open end of the casing. In extreme cases, the casing will need to be kept close to the test interval. Under these conditions, set the casing at the base of the previously tested interval before drilling to the next test interval. Intervals of 2.5 feet (75 cm) are recommended as the closest spacing for SPTs.

Use drilling mud when the SPT is performed for liquefaction evaluation when rotary drilling. A bentonite-based drilling mud has the maximum stabilizing benefit of mud. Bentonite provides the maximum weight, density, and wall caking properties needed to keep the drill hole stable. When mixing mud, use enough bentonite for the mud to be effective. There are two ways to test drill mud density or viscosity—a Marsh Funnel or a mud balance. A mud sample is poured through a Marsh Funnel, and the time needed to pass through the funnel is a function of the viscosity. Water has a Marsh Funnel time of 26 seconds. Fine-grained soils require mud with Marsh Funnel times of 35 to 50 seconds. Coarser materials such as gravels may require funnel times of 65 to 85 seconds to carry the cuttings to the surface. If using a mud balance, typical drill mud should weigh 10-11 pounds per gallon (lbs/gal) (1-1.1 kilograms per liter [kg/L]). Water weighs about 8 pounds per gallon (0.8 kg/L).

Exploration holes are often completed as piezometers. Revertible drilling fluids have been improved, and there are synthetic polymers that break down more reliably. If necessary, specific "breaker" compounds can be used to break down the mud and clean the borehole. If the borehole cannot be kept stable with polymer fluid, bentonite mud should be used and a second hole drilled for the piezometer installation. Do not combine drill hole purposes if the data from SPTs or piezometers are compromised.

Drilling sands with clear water is possible, but only if the driller is very experienced. As long as drilling is carefully performed, drilling with water can result in SPT N values close to those obtained using mud. Disturbance can be avoided; but without drill mud, jetting disturbance, cave, and sand heave caused by fluid imbalance are likely.

If the water level in the sand layer is higher than the ground surface, sand heave is really going to be a problem. Under these conditions, heavy bentonite mud (80 to100 sec on the Marsh Funnel) is required. A fluid bypass to keep the hole full of mud is required, and an elevated casing or drill pad to hold down the sand can be used. Some successful mud improvement is possible with Barite or Ilmenite additives. Mud can be weighted to about 15 lb/gal with these additives. Sodium or calcium chloride can be used to give polymer fluid better gel strength. In artesian conditions, it may not be possible to keep the sand stable. In these cases, other tests such as the CPT can be used to evaluate the sand.

When using fluid rotary drilling, circulate the drill fluid to remove the cuttings. Pull back the cutting bit several feet, cut fluid circulation, and then slowly and gently lower the bit to rest on the bottom of the hole. Check to see if the depth is within 0.4 foot (10 cm) of the cleanout depth. This check determines if there is cuttings settlement, wall cave, or jetting disturbance.

The bottom of the borehole normally heaves when the cleanout drill string is pulled back creating suction. Fluid should be added to the drill hole as the cleanout string is removed to help avoid problems. Once the sampler is placed, check the sampler depth and compare it to the cleanout depth. A difference of 0.4 foot (10 cm) is unsatisfactory. If sands or silty sands heave up into the borehole, the SPT sampler will often sink through most of the slough. The only way to check for this situation is to carefully inspect the top of the sampler and the ball check housing for slough or cuttings. If the ball check area is plugged with cuttings, the SPT N value may have been affected. A thin plastic cover is sometimes used to keep the slough out of the sampler. The cover is either sheared off at the first blow or it is shoved up into the sampler.

The fluid rotary method is probably the best method for determining SPT N values in saturated sands. In the following sections, two other acceptable drilling methods are discussed. If these methods do not work, use the fluid rotary method.

Hollow-Stem Augers

Hollow-stem augers (HSA) have been used successfully to do SPTs in loose saturated sands. With the proper precautions, hollow-stems can be used reliably in sands, but there are some problems with HSAs. The primary problem with the HSA in loose sands is sand heaving into the augers. This occurs when the pilot bit or the HSA sampler barrel is removed in preparation for the SPT. Sometimes, sand can heave 5 to 10 feet (1.5 to 3 m) up inside the augers. SPT N values taken with this amount of disturbance are unacceptable. These problems can be overcome in most cases by using water-filled augers and removing the pilot bit or HSA sampler **slowly** to avoid the suction. Drilling mud is not usually required and can cause sealing problems.

There are two types of HSA systems shown in figure 22-4—wireline and rod type. With either type of system, removal of the pilot bit or HSA sampler barrel can result in sand heaving into the augers. The rod type system is best at preventing sample barrel rotation during soil sampling. In sanding conditions, the wireline system is sometimes harder to operate because the withdrawal rate of the bit or HSA sampler is harder to control. Sanding-in also prevents re-latching of the wireline barrel. Rod type systems are recommended when drilling in heaving sands. If sand heaves a considerable height into the augers, the auger will need to be cleaned or retracted in order to continue drilling using either system. If the augers have to be pulled up 3 feet (1 m) to re-latch a pilot bit or sampler barrel, tremendous suction occurs at the base of the boring, which can disturb the next SPT test interval.

When using HSAs below the water table, the hole must be kept full of fluid, just like it must when using fluid rotary methods. **A water or mud source and a bypass line are required**. Some successful techniques for hollow-stem drilling in flowing sands are:

- When approaching the test interval, slow the auger rotation to just enough to cut the soil; do not continue to rotate without advancement near the test interval. In flowing sands, continued rotation near the test interval will create a large void around the hole annulus and increase the chance of caving and disturbance of the test interval. If high down pressure is used with wireline systems, the pressure should be relaxed; and the augers should be slightly

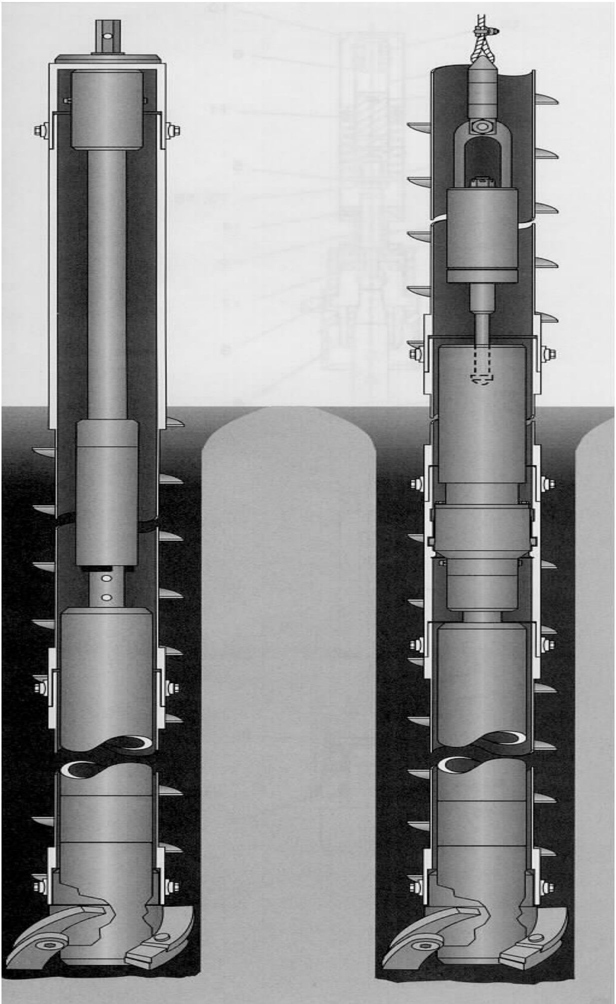


Figure 22-4.—Example of rod-type and wireline-type hollow-stem augers.

retracted ½ inch (1 cm) or so to re-latch bits or barrels. There is no need to release down pressure or retract the augers with rod-type systems.

- Add water to a level higher than the surrounding groundwater level before pulling the pilot bit or sampling barrel. In most cases, water can be added to the top of the augers without concern for disturbance. Add water by removing the drive cap using a hose from the bypass line. When removing the drive cap on rod-type systems, be careful to disconnect the drive cap bearing from the inner rods, or the pilot bit or sampler will be pulled prematurely before adding water. When using a wireline system, the latching device can be sent down the hole and latched before adding water.

The water level is not always maintained at the top of the column, especially if there is a thick layer of unsaturated soil above the test zone. Water can leak through the auger joints, and it may be necessary to add a lot of water.

Pulling the Sampler Barrel.—The sample barrel assembly is generally 5 feet (1.5 m) long. This barrel does not have much clearance with the inside of the augers, especially in the bushing at the base of the augers. With the augers full of water, reconnect the drive cap to the inner rods. Pull the barrel slowly up 0.1 to 0.3 foot (3 to 10 cm) and observe the water level in the augers. If water flows upward, out of the augers, there is a seal between the augers and the sampler, and the sampler barrel is acting like a syringe. If water flows from the top with rod type systems, rotate the barrel or work the barrel slightly down and up to try to break the seal and vent. For wireline systems, release the pulling force and re-apply. Pull slowly and attempt to break the seal. Once the seal is broken, remove the sampler **slowly**. Remember, with rapid withdrawal rates, suction can be created anywhere in the auger column. For rod systems, add water during pulling to account for water level drop. The same rule applies for wireline systems, but less water is needed.

Pulling the Pilot Bit.—Most pilot bits are seated flush in a brass bushing in the end (crown) of the augers. The pilot bit cutting teeth should be set to a lead distance the same as the outer cutting teeth, so that the body of the pilot bit sits correctly in the bushing. Do not drill with the pilot bit in advance of the outer cutting teeth. A useful procedure in heaving sands is to use a pilot bit one size smaller than the augers being used. For example, if a 4.25-inch (11-cm) inside diameter (ID) HSA is used, a 3.75-inch (9.5-cm) ID HSA pilot bit can be used to reduce vacuum and suction effects.

When drilling with the pilot bit, pull the bit back slowly about 0.1 to 0.2 foot (3 to 6 cm) to allow any seal in the bushing to vent. If the bit is withdrawn quickly, suction will likely occur. If water flows out the top of the augers, suction is occurring. If suction is occurring, rotate the pilot bit and work it down and up to try to break the seal. Once the bit clears the bushing, the tendency to bind is reduced. Withdraw the pilot bit **slowly** and add water, to account for water level drop as the rods are removed. Remember, with rapid withdraw rates, suction effects can be created anywhere in the auger column.

If sanding-in cannot be controlled with fluid or slow pulling, there are special flap valves that can be placed in the pilot bit seat. Drill without the pilot bit with flap valves.

Once the sampler has been inserted to the base of the boring, determine the depth to the sampler tip as a quality check. If there is more than 0.4 foot (12 cm) of slough or heave, the test may not be acceptable. This guideline is arbitrary, and it is possible to get a reliable test with as much as 0.5 foot (15 cm) or more slough as long as the vent and ball check of the sampler are not plugged. If the SPT barrel is used to test the bottom of the hole, the sampler will often penetrate loose slough or heave. Checks with a weighted tape may be more accurate in determining the depth to the slough. When using the HSA sampler barrel to core before testing, sand falling out of the barrel could be the cause of slough inside the auger. To avoid this problem, use catcher baskets in the HSA sampler barrel.

When testing at close intervals of 2.5 feet (75 cm) or less, it may be necessary to add water to the augers as the SPT sampling string is removed to avoid water level imbalance and possible heave.

It's a good idea to combine the continuous sampler of the HSA with SPT operations. If SPTs are at 2.5-foot (75-cm) intervals, perform the SPT and then sample the 2.5-foot (75-cm) and over-sample the 1.5-foot (45-cm) test interval. This adds some time, but allows continuous sampling. This sampling method provides a look at the soils between the test intervals. It is also helpful if recovery is low.

Rotary Casing Advancers

Rotary casing advancers can provide good SPT N values in sands. The casing advancer method uses drilling fluid (bentonite and water) as a circulation medium and is a fluid rotary drilling method. This method is successful because the large diameter outer rods remain filled with drill fluid and keep the sand down. The casing advancer normally has a diamond bit but can be equipped with tungsten carbide drag bits on the outside edge to over-cut soil. Typically, an HQ- or HW-size casing advancer is used with or without a pilot bit. The pilot bit can be a tricone bit removed via wire line. Suction is possible when a pilot bit is removed. If suction occurs, drilling without a pilot bit should be tried. An advantage of drilling with a wireline is that when the pilot bit is removed, the line takes up little volume and results in a minor drop in fluid level inside the rod column. Since a good fluid column remains in the rods, a fluid bypass is not needed. The only problem is that whenever adding rods to the SPT drill string, fluid flows out of the advancer.

The casing advancer must be operated very carefully to avoid sand disturbance. Fluid is pumped down the casing and up a narrow annulus along the exterior of the casing. A casing advancer, especially without a pilot bit, is equivalent to a bottom discharge bit. If excessive fluid pressures are used or if circulation is lost, jetting or hydraulic fracturing the material in the SPT test interval is possible. Drilling the material with a slow advance rate and with low pressure while maintaining circulation is necessary to drill successfully with this system. If circulation return stops, blockage may be occurring; and if pump pressures increase, hydraulic fracturing could occur. If the advance rate is too fast, circulation will be blocked. Water is not an acceptable drill fluid with this method, and drill mud must be used.

Summary of Drilling Effects

Table 22-2 illustrates the effects of different drilling and mechanical variables on the SPT N value (items 1 through 5). A typical N value in clean quartz sand is 20 blows per foot (30 cm). The possible range of N for the material is shown if the material is subject to errors in testing.

Table 22-2 shows that drilling disturbance can have drastic effects on the N value. In fact, zero blows can be obtained. Zero blows may not be realistic because, in many cases, loosened sand settles back to the bottom of the hole. Also, very loose sand normally does not allow the sampler to settle under the weight of the assembly. Drilling disturbance usually results in a low N value. Low blow counts indicate loose, weak soils, and a weak foundation may be assumed. Erroneous low disturbed N values can result in costly over design of structures. The most important aspect of SPT testing is the way the hole is drilled.

Procedure Variables

The recommended 2.5-foot (75-cm) interval is to ensure that the next interval is not disturbed. If material that only has a few thin layers of sand is drilled, continuous sampling is possible, but difficult, and should not be attempted unless necessary.

Hammer Blow Rate

The blow count rate is important when soil drainage needs to be considered. Most test standards request SPT blows at a rate of 20 to 40 blows per minute (bpm). Blows at 55 bpm are not likely to have an effect on clean sand; but at some fines content, blows will be reduced by the lack of drainage. Blows should be between 20 and 40 bpm if a hammer with a controllable rate is used. Some hammer systems are designed to deliver blows at a faster rate. The automatic hammer is designed to deliver blows at a rate of 50 to 55 bpm. The hammer can be set to run at 40 bpm by adding a spacer ring to the impact anvil. If a hammer rate differs from 50 bpm, clearly note it on the drill logs.

Table 22-2.—Estimated variability of SPT N Values

Cause: Basic	Cause: Description	Typical raw SPT value in clean sand N = 20	Typical raw SPT value in clay N=10
Drilling method	1. Using drilling mud and fluid bypass	20	10
	2. Using drill mud and no fluid bypass	0-20	8-10?
	3. Using clear water with or without bypass	0-20	8-10?
	4. Using hollow-stem augers with or without fluid	0-20	8-10?
	5. 8-inch (20-cm) diameter hole compared to 4 inches (10 cm)	17	8-10?
Sampler	6. Using a larger ID barrel, without the liners	17	9
	7. Using a 3-inch (7.6-cm) OD barrel versus a 2-inch (5-cm) barrel	[e]25-30	10
Procedure	8. Using a blow count rate of 55 blows per minute (bpm) as opposed to 30 bpm	[e1]20	[e1]10

Table 22-2.—Estimated variability of SPT N Values (continued)

Cause: Basic	Cause: Description	Typical raw SPT value in clean sand N = 20	Typical raw SPT value in clay N=10
Energy Transmission Factors			
Drill rods	9. AW rod versus NW rod	[e2]18-22	[e2]8-10
	10. SPT at 200 feet (60 m) as opposed to 50 feet (30 m)	[4]22	[e3]5
	11. SPT at less than 10 feet (3 m) as opposed to 50 feet (30 m) with AW rods	30	15
	12. SPT at less than 10 feet (3 m) as opposed to 50 feet (30 m) with NW rods	25	12
Hammer operation	13. Three wraps versus two wraps around the cathead	22	11
	14. Using new rope as opposed to old rope	19	9
	15. Free fall string cut drops versus two wrap on cathead	16	8
	16. Using high-efficiency automatic hammer versus two wrap safety hammer	14	7

Table 22-2.—Estimated variability of SPT N Values (continued)

Cause: Basic	Cause: Description	Typical raw SPT value in clean sand N = 20	Typical raw SPT value in clay N=10
Energy Transmission Factors (continued)			
Hammer Operation	17. Using a donut hammer with large anvil as opposed to safety hammer	24	12
	18. Failure to obtain 30-inch (75-cm) drop height (28 inches [70 cm])	22	11
	19. Failure to obtain 30-inch (75-cm) drop height (32 inches [80 cm])	18	9
	20. Back tapping of safety hammer during testing	25	12

e = Estimated value.

1 = Difference occurs in dirty sands only.

2 = It is not known whether small drill holes are less or more efficient; with larger rods, N may be less in clay because of the weight.

3 = N in clay may be lower because of the weight of the rods.

4 = Actual N value will be much higher because of higher confining pressure at great depth. The difference shown here is from energy only and confining pressure was not considered.

Limiting Blow Counts

The Reclamation test procedure calls for stopping the test at 50 blows per foot (30 cm). Other agencies sometimes go to 100 blows per foot (30 cm) because the ASTM test standard D 1586 sets a 100-blow limit. The Reclamation standard is lower to reduce equipment wear.

Using the soil liquefaction criteria for sand at a depth of 100 feet (30 m), 50 blows would not be considered liquefiable. SPT data are corrected to a stress level of 1 ton per square foot (ton/ft^2). In a typical ground mass, a 1 ton/ft^2 stress level occurs at a depth of 20 to 30 feet (6 to 9 m), depending on the location of the groundwater table. Blow counts in a sand of constant density increase with depth. A correction factor is used to adjust for this overburden effect. In earthquake liquefaction clean sand N160 values greater 30 blows per foot (bpf) are not liquefiable. A blow count of 50 bpf at 100 feet (30 m) corrects to about 30 bpf at 1 ton/ft^2. Higher blow counts would not be considered liquefiable. If testing is deeper than 100 feet (30 m) it will be necessary to increase the limiting blow counts to 100. The refusal rule still applies; if there is no successive advance after 10 blows, the test can be stopped.

SPT N values in gravels generally are much higher than in sands. Liquefaction criteria for sands are not reliable criteria for gravels.

Penetration per Blow or Blows per 0.1 Foot (3 cm)

Penetration for each blow should be recorded when drilling in gravelly soils. If penetration per blow is recorded, sand layers can be resolved, and the N value of the sand can be estimated. The blow count in sand can be estimated from a graph of penetration per blow. The extrapolation is generally reliable if the blows start in sand. If the interval starts with gravel and then penetrates into sand, the extrapolation is less reliable because the sampler could be plugged by gravel.

The number of blows for 0.1 feet (3 cm) is the minimum penetration rate data that should be collected. If three people are present, it is very easy to record "penetration per blow," and these data are preferred over the coarser blows per 0.1 feet (3 cm). To record penetration per blow, make a form with three columns. In one column, list the blows 1 through 100. Mark the drill rods in 0.1-foot (3-cm) intervals or use a tape starting at zero from the edge of a reference point. In the second column, record the total penetration as the test is performed. This will require a reader to call off the total penetration. The reader can interpolate between the 0.1-foot (3-cm) increments, or the penetration can be read directly from a tape. After the test is done the incremental penetration can be calculated from the cumulative penetration data and recorded in the third column.

Equipment and Mechanical Variables

Sampler Barrel

The standard sampler barrel is 2 inches (5.1 cm) in OD and is the barrel that should be used. In private industry, 2.5- (6.4-cm) and 3-inch (7.6-cm) OD barrels are occasionally used. If sample recovery in coarse materials is poor, it is acceptable to re-sample with a 3-inch (7.6-cm) barrel equipped with a catcher.

Gravelly soils generally do not provide reliable SPT data for liquefaction evaluations that are based on sands. Other methods use larger samplers and hammers to evaluate the liquefaction potential of gravelly soils. The BPT is used at gravel sites. Often, the BPT is used at gravel sites after a first round of SPT testing shows considerable gravels present.

Sampler Shoe

The dimensions of the sampler shoe should meet ASTM D 1586 requirements. Some drill equipment catalogs claim to have special "heavy duty" sample barrels and shoes. The "Terzaghi" style does not meet the ASTM and Reclamation requirements. When buying shoes, check their dimensions to be sure they meet test requirements. Figure 22-1 shows both Reclamation and ASTM sampler requirements.

Shoe ruggedness can be improved by "carburizing" the metal. This is a process where the shoe is heated in a carbon gas to improve the surface hardness of the steel. This makes the shoe more rugged but also more brittle. Most drill manufacturers supply untreated low carbon steel such as 1040 alloy. Generally, a local machine shop can "carburize" the shoe, an inexpensive process.

Sample Retainers

A sample retainer should not be used for liquefaction studies except in desperation because the effects are unknown. If the sample cannot be retained, a sample may be taken with a large diameter split barrel sampler with a retainer re-driven through the test interval. The over coring procedure discussed earlier using HSAs could also be used.

There are several types of retainers available and some types are better than others. There is a flap valve device that actually looks like a toilet seat (a small one) that places a large constriction inside the barrel. This device is the least desirable of the retainers if the N value is important. The basket type catcher is made of curved fingers of steel, brass, or plastic. This type of retainer is only a minor constriction because the holding ring fits into the recessed area between the shoe and the barrel. The problem with this catcher is that the fingers may not always fall back into position to hold the core. A better variation of this catcher is the "Ladd" type retainer that combines the finger basket with a plastic sleeve. This retainer is the most successful at retaining flowing sand because the bag adds extra retaining capability.

Sampler Liners

Most of the SPT samplers in the USA accept liners, but the liner is usually omitted. To determine if the sampler will accept a liner, feel for an offset (increased diameter) inside the shoe. If an offset is present, the barrel is 1½-inch (3.8-cm) ID. Log whether a constant diameter or an enlarged diameter barrel is used because the sample type can effect recovery. For liquefaction evaluation, a constant ID barrel is recommended.

A sampler used without liners is actually better for recovery. Average recovery of a constant ID barrel is about 60 percent, and the average for the barrel without liners is about 80 percent. The difference in N value between constant and enlarged diameter barrels is not known, but an increase in blows in the range of 1 to 4 is likely with a constant ID barrel.

Sampler Length

A 24-inch- (61-cm-) long split barrel can normally accommodate any slough in the drill hole without plugging the ball check device.

Sampler Vent Ports

The required vent ports for the sampler top subassembly in ASTM and Reclamation test procedures are inadequate when drilling with mud. The ASTM standard requires two ⅜-inch (1-cm) diameter vents above the ball check. When drilling with mud, the fluid gets loaded with sand and can easily plug these ports. The sampler and rods fill with mud as they are lowered into the drill hole. A big column of drill mud may try to push the sample out if the ball check does not seat. Drill larger vent ports in the top subassembly to avoid this problem. Some drillers use a 0.5- to 1-foot (15- to 30-cm) drill rod sub just above the sampler with extra holes drilled in it to easily drain drill fluid from the rod column.

Hammers, Anvils, Rods, and Energy Effects

The variables in energy transmission are hammer type, hammer drop height, hammer drop friction, energy losses in impact anvil(s), and energy losses in rods. The energy in the drill rods is called the "Drill Rod Energy Ratio" or ERi.

Some hammers, especially donut (casing type) hammers with large anvils, deliver approximately 50 percent of the total potential energy of a 140-lb (63.6-kg) hammer dropping 30 inches (75 cm). The N value is proportional to the energy delivered, and the N values can be adjusted to a common energy delivery level. The current practice is to adjust SPT N values to 60-percent drill rod energy.

Safety Hammers

There are many kinds of SPT hammers. Pin-guided and donut type hammers were common in the past, but these hammers have generally been replaced by the "safety" hammer which has an enclosed anvil (figure 22-2). There are also new automatic hammers that improve the repeatability of delivered hammer energy to the sampler.

The safety hammer provides an economical and safe method of performing the SPT. The enclosed anvil removes hazards from flying metal chips, and operators cannot get their hands in the impact surface. Due to their inherent geometry, safety hammer energy transmission can vary only by about 20 percent as long as the hammers are operated correctly and consistently.

Safety hammers should be designed with a total stroke of about 32 inches (80 cm), and there should be a mark on the guide rod so the operator can see the 30-inch (75-cm) drop. The hammer weight should be 140 pounds (63.6 kg). These characteristics should be verified on the hammer. An easy way to weigh the hammer is to place the total assembly on a platform scale, get the total weight, then lift the outer hammer off the anvil, and weigh the guide rod and anvil. The difference in the two weights is the hammer weight. The hammer weight should be 140 +/- 2 lb (63.6 kg +/- 0.9 kg). Hammers should be stamped with an ID number. It is best to keep a given hammer for a specific drill, especially if the energy transmission of the drill has been measured in the past.

The assumption is that safety hammers deliver 60-percent drill rod energy with two wraps of rope around the cathead. Actually, the hammers deliver about 60 to 75 percent depending on their construction. The guide rod is one factor that affects the energy transmission.

Some safety hammers come with a solid steel guide rod, and others use a hollow AW drill rod. The solid guide rod absorbs energy, and the solid steel guide rod safety hammer will deliver lower energy than the hollow guide rod safety hammer. These differences are not enough to recommend one design over another. Another variable with safety hammers is a vent. Some hammers have vents near the top of the hammer. A vent allows some air to escape as the anvil moves toward the impact surface. These vents allow the best free fall possible.

Donut Hammers

These hammers are not recommended except in special cases such as when clearance is a problem. If the testing is for liquefaction evaluation, it may be necessary to measure the energy of the donut hammer used. The donut hammer is supposed to be inefficient, but if the hammer has a small anvil, efficiencies may be similar to the safety hammer. The larger anvil absorbs part of the hammer energy.

Rope and Cathead Operations

Most SPTs are performed using the rope and cathead method. In this method, the hammer is lifted by a cathead rope that goes over the crown sheaves. ASTM and Reclamation standards require two wraps of rope around the cathead. After the hammer is lifted to the 30-inch (75-cm) drop height, the rope is thrown toward the cathead, allowing the hammer to drop as freely as possible.

Three wraps will reduce the drill rod energy by about 10 percent and will result in a higher N value. As the rope gets old, burned, and dirty, there is more friction on the cathead and across the crown sheaves. New rope is stiffer and is likely to have higher friction than a rope that has been broken in. A wet rope may have less friction, but the energy differences are small enough that it is not necessary to stop testing in the rain. Rain should be noted on the drill report and log. Frozen rope may have considerably more friction. Under wet and freezing conditions, exercise the rope and warm it up prior to testing.

Consistent rope and cathead operations depend on having well maintained crown sheaves on the mast. Crown sheaves should be cleaned and lubricated periodically to ensure that they spin freely.

Automatic Hammers

Automatic hammers are generally safer and provide good repeatability. Central Mine Equipment (CME) made one of the first automatic hammers commercially available in the United States. This hammer uses a chain cam to lift a hammer that is enclosed in a guide tube. The chain cam is driven with a hydraulic motor. The drop height of this hammer depends on the chain cam speed and the anvil length. Problems with this hammer system primarily result from the speed not being correctly adjusted. The hammer should be run at 50 to 55 bpm to obtain a 30-inch (75-cm) drop. There are blow control adjustments on the hammer, and there is a slot on the side of the hammer casing to observe the hammer drop height. Be sure the hammer is providing a 30-inch (75-cm) drop by adjusting the blow control.

The CME automatic hammer is designed to exert a down force on the rods. This down force from the assembly is about 500 lbs (227 kg). A safety hammer assembly weighs from 170 to 230 pounds (77 to 104 kg). In very soft clays, the sampler will more easily sink under the weight of the assembly, and with the automatic hammer, the blow counts will be lower.

The Foremost Mobile Drilling Company hammer “floats” on a wireline system. The drop mechanism does not depend on rate. Energy transfer is about 60 to 70 percent.

Energy transfer of some automatic hammers is significantly higher than rope and cathead operated hammers. The CME hammer can deliver up to 95 percent energy. This could result in very low blow counts in sands. Energy corrections are usually required for automatic hammers. The Mobile Drilling Company hammer is less efficient because of a large two-piece anvil.

If an automatic hammer is used, report detailed information on the hammer use. Report make, model, blow count rates, and any other specific adjustments on the drilling log. In liquefaction investigations, the energy transfer must be known. For some hammer systems, such as the CME and Mobile Drilling Company, the energy transfer is known if the hammers are operated correctly, but for some systems, energy measurements may be required.

Spooling Winch Hammers

Mobile Drilling Company developed a hammer called the “Safety Driver.” This hammer system used a steel wireline cable connected to an automated spooling winch with magnetic trip contacts. The contacts sensed when the hammer was lifted 30 inches (75 cm), and the hammer then dropped with the spool unrolling at the correct rate for the dropping hammer.

Energy measurements of this hammer system show some extreme energy variations. Apparently, the contacts and spooling systems require continual adjustment to operate correctly. This type of hammer system is not recommended because of energy transmission problems.

Drill Rods

Any rod from AW to NW size is acceptable for testing. There is some concern about whipping or buckling of smaller AW rods at depths greater than 75 feet (23 m). In these cases, use BW rods or larger. There is not much difference in energy transfer between AW and NW rods. The type of rod changes a blow count in sand only by about two blows and maybe less.

SPT drill rods should be relatively tight during testing. Energy measurements on differing locations of the drill rods do not show significant energy loss on joints that are loose. There has to be a real gap on the shoulders to cause significant energy loss. This is because when the rod is resting in the hole, the shoulders of the joints are in contact. There is no need to wrench tighten joints unless rod joints are really loosening during testing. Be sure to firmly hand tighten each joint.

Drill Rod Length

When using very short rods, energy input to the sampler is attenuated early because of a reflected shock wave. The driller can usually hear this because there is a second hammer tap. The early termination of energy is a problem to depths of 30 feet (9 m), but the correction is small and is often ignored. The energy termination is also a function of the size of the drill rods. There is some energy loss for drill rod strings longer than 100 feet (30 m), and a correction is necessary. A constant density sand will have an increasingly higher penetration resistance as depths increase. This is because the confining pressure increases in the ground mass with depth.

Summary

How Good is the SPT Test

Figure 22-5 is a summary graph of a study performed in Seattle by the American Society of Civil Engineers (ASCE). In this study, several private geotechnical firms and agencies drilled SPTs at the same site. Six drills were used. Some had safety hammers, and others had automatic hammers. One drill was equipped with a 300-lb (136-kg) safety hammer.

The graph shows a wide variation in raw N value versus depth. The soil conditions at the site are not well documented. Some gravel layers are present. Note that the spooling winch system resulted in unreliably high SPT N values.

The variability of SPT drilling can be reduced if drillers are aware of the problems inherent to the SPT. Interpretation of the data improves if all unusual occurrences during SPTs are reported. **Drill logs should clearly describe in detail the equipment used.**

Liquefaction studies are done in loose sands below the water table. Unfortunately, this material is the hardest to drill without disturbance. Fluid rotary drilling is the preferred approach for keeping the sand stable. HSAs and casing advancer systems have also been successfully used.

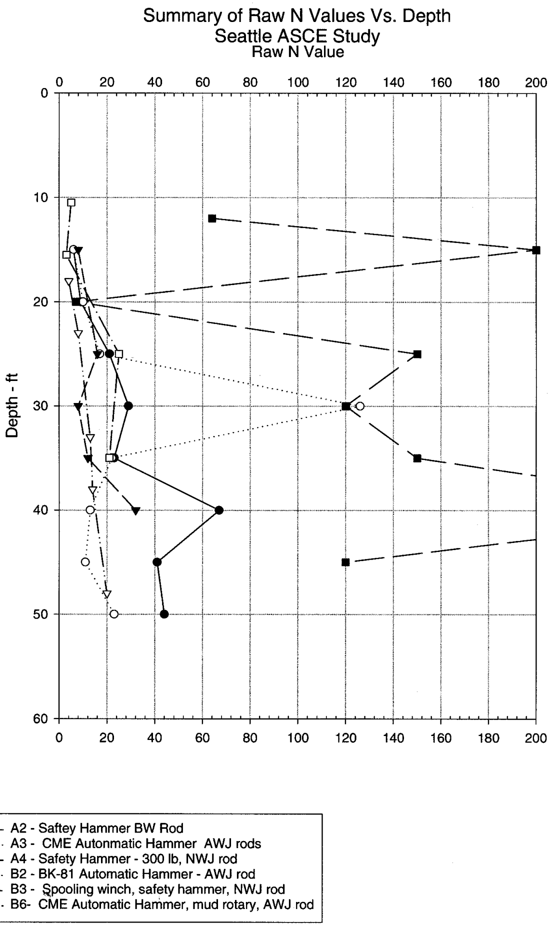


Figure 22-5.—Results of SPT with six different drills—ASCE Seattle study.

The drilling part of SPTs is the most important. Generally, disturbance from improper drilling technique results in lower N values.

Energy transfer effects can be important, especially if highly efficient automatic hammers are used.

Becker-Hammer Penetration Testing for Gravelly Soils

Introduction

The BPT is used to test the density of materials that are too coarse for the SPT or the CPT. Gravel can cause misleading results in the SPT and CPT. Because the diameter of the BPT penetrometer tip is much larger than that of the SPT sampler or the cone penetrometer, gravel-sized particles do not seriously affect the BPT.

The BPT consists of driving a plugged steel casing into the ground using a diesel pile-driving hammer. The blows per foot (30 cm) of penetration are recorded and adjusted for driving conditions. An empirical correlation is then used to estimate equivalent SPT values. The BPT is performed with a Becker Drills, Ltd. model AP-1000 or B-180 drill rig, equipped with an International Construction Equipment (ICE) model 180 closed-end diesel hammer. The standard configuration uses 6.6-inch (16.8-cm) OD double-wall casing and a plugged "crowd-out" bit. Some ICE 180 hammers are marked "Linkbelt."

The BPT is rapid and economical to perform. Production can reach 500 feet (150 m) per day. A disadvantage is that no sample is retrieved with the BPT, so other sampling, such as SPT or coring, is also required. Another disadvantage is the uncertainty in interpretation of the data. Since the BPT is generally used to estimate equivalent SPT blow counts, significant uncertainty is introduced by that step, in addition to the uncertainty that exists in predictions of soil behavior from N values.

The penetration resistance of soils is influenced by a large number of factors, including soil type (grain-size distribution, plasticity, particle sizes, particle shapes), density, confining stress, energy delivered to the penetrometer, size and shape of the penetrometer, and friction on the sides of the penetrometer. The BPT differs from the SPT test in many ways, and correlation between BPT and SPT data is not consistent. The BPT is not performed in an open hole with a diameter greater than the rod diameter, and the penetrometer tip is not open like a SPT tip, so there is substantial friction on the drill string. This greatly complicates the analysis. Like the SPT, the BPT may give misleading results in soils containing boulders, cobbles, or even large amounts of gravel coarser than about 1½ inches (4 cm).

The effect of fines in the relationship between Becker penetration resistance and liquefaction potential has not been established by experiment or field performance. The effect of fines is generally assumed to be similar to what occurs with the SPT. Since the BPT does not return a sample, it is often necessary to estimate the fines content from nearby drill holes or to neglect the potential benefit of fines.

Role of BPT in Exploration

In soils containing gravel, measured SPT or CPT resistance may be misleadingly high, and there is potential for damage to CPT equipment. CPT equipment generally cannot be advanced through thick gravel layers with more than about 30 percent gravel, depending on the size of the gravel and the density of the soil. Results may be misleading with smaller gravel contents. BPTs are rarely performed at the start of an investigation and are generally done after SPTs or CPTs have been attempted and found to be inappropriate because of too much gravel. BPT testing generally should not be relied on as the sole basis for liquefaction evaluation without site-specific verification of the SPT-BPT correlation, corroboration by shear-wave velocities, or other liquefaction resistance predictors.

A Becker drill can also be used for other tasks such as installation of instrumentation or holes for geophysical testing. Some soil is compacted around each Becker hole, and the holes may be more prone to deviate from vertical than holes drilled by conventional methods. The extent of densification is not known, so if the holes are to be used for geophysical measurements (such as shear-wave velocity), vary the spacings to evaluate the effect of compaction around the hole. Rotary drilling can also be done inside the double-wall Becker casing to socket installations such as inclinometers into bedrock. This is more expensive than standard Becker testing because of delays and the need for a second rig. Becker rigs do not have rotary drilling capability.

Equipment

Becker drills can be operated with a variety of equipment configurations, but for penetration testing, the standard testing setup is as follows:

- Drill rig: Becker Drills, Ltd. model AP-1000 rig
- Hammer: Supercharged ICE model 180 diesel hammer

- Casing (rods): 168-mm (6.6-in) OD, double-wall
- Drive bit: Crowd-out plugged bit

The correlation between BPT and SPT data proposed by Harder and Seed relies on the use of the standard equipment configuration. The method proposed by Sy requires that at least the last two conditions be met. All four conditions should be met because analyses by the Sy method would probably be duplicated by the Harder-Seed method for preliminary calculations and/or verification. Harder and Seed determined that open-bit tests were inconsistent and erroneously low relative to the closed-bit standard. The older model B-180 and HAV-180 rigs, equipped with the same hammer, transfer about 50 percent more of the energy to the drill string than do AP-1000 rigs. This factor has been tentatively confirmed by energy measurements, but it is preferable to avoid the issue by specifying the use of AP-1000 rigs only.

The diesel hammer does not provide consistent energy to the drill string. This is because the energy depends on combustion conditions, which are affected by fuel condition, air mixture, ambient pressure, driving resistance, and throttle control. The closed-end diesel pile hammer is equipped with a "bounce chamber" where air is compressed by the rising ram after each blow; the air acts as a spring to push the ram back down for the next blow (unlike the more common open-ended diesel hammer that uses gravity alone to return the ram). Measuring the bounce-chamber pressure provides an indirect measure of combustion energy.

Harder-Seed Method of BPT Interpretation

The Harder-Seed method of interpreting the BPT uses measurements of bounce-chamber pressure as an indication of the energy imparted to the rods by each blow. The bounce-chamber pressure is used to adjust the blow count for the actual combustion condition to that produced by a hypothetical constant combustion condition. The measured bounce-chamber pressure must be adjusted at altitudes above 1000 feet (300 m). The throttle should be kept wide open and the supercharger should be operated any time data are being recorded. Some drillers prefer to use a smaller throttle opening or no supercharger at the beginning of driving when the blowcounts are smaller, producing high blow counts. If the blowcounts required for analysis are near the surface, the driller should be instructed to keep the throttle wide open. Instances where full throttle and supercharger are not used should be recorded in the field notes.

The bounce chamber pressure needs to be monitored continuously during testing. An electronic recording system is available to monitor the bounce chamber. The pressure gauge provided by the hammer manufacturer can be used to record the data manually, but the gauge reading is sensitive to the length of hose used to connect the gauge to the hammer.

If a B-180 or HAV-180 rig is used, the data can be adjusted by multiplying by the factor 1.5 to account for the difference in energy transmitted to the rods. This factor is supported by few data and is considered approximate. An AP-1000 rig is preferred.

Testing for the Harder-Seed Method of Interpretation

The Harder-Seed method requires that the number of blows to drive BPT rods each foot (30 cm) of depth and bounce-chamber pressure during that interval be recorded. Record the driving conditions and note if the drillers pull the rods back to loosen them up to reduce the driving friction.

Sy Method of BPT Interpretation

The method proposed by Sy and Campanella is more rigorous, but more costly and time-consuming. Friction on the sides of the rods may contribute a substantial portion of the driving resistance. A pile-driving analyzer (PDA) is used to record acceleration and rod force during individual blows of the hammer. The PDA also measures the driving energy for each blow. The force and acceleration histories are then analyzed to separate the resistance to driving contributed by the bearing capacity of the tip and by the side friction using a computer program called CAPWAP. PDA operation and CAPWAP analyses are usually done by the contractor.

The PDA measurement eliminates concern about the performance of the hammer, effects of altitude, or loss of energy between the hammer and the rods. At least in theory, analyses should eliminate the effects of varying amounts of side friction on the blow count. The primary drawback is the need for PDA measurements and special analyses. These substantially increase the cost of the testing program and slow the process of testing and interpretation.

The side friction can also be measured directly by pullback tests, where the force required to pull the rods back a few inches is measured by a load cell. This measurement can be substituted for some of the CAPWAP data, but it is not recommended that CAPWAP calculations be completely eliminated. CAPWAP data is the standard from which the method was developed.

Testing for the Sy Method of Interpretation

The Sy method requires:

Using the PDA, record rod force, acceleration, and transmitted energy. Record the number of blows for each 1-foot (30-cm) interval of BPT driving. Record driving conditions, and note if the drillers pull the rods back to loosen them up to reduce the driving friction.

Discussion of Methods

For routine investigations of typical alluvial materials that do not have dense material overlying them, a PDA is generally not necessary, and the Harder-Seed approach should usually be sufficient. In cases where drill rod friction is likely to be a problem (penetration through compacted fill or deep deposits), the Sy method may be better. BPTs can be done after pre-drilling and casing or after pre-driving the BPT with an open bit through compacted fill overlying the tested layers. This reduces the friction but does not necessarily provide valid predictions of SPT N_{60} with the Harder-Seed method and may cause them to be low.

With either method, the field notes should mention any time that the drillers pull back the rods to reduce the friction. There is no way to explicitly account for this in the Harder-Seed method. When using the Sy method, the locations for calculations should be selected with the pullbacks in mind. Ideally, pullbacks should be done only before and after critical layers are penetrated. This way, the rod friction can be interpolated between analyzed zones with no pullbacks between them to invalidate the interpolation. Zones to be tested and pullbacks should be discussed with the drillers prior to each hole. Substantial uncertainties exist both in the correlations to estimate the equivalent SPT N_{60} and in the correlations to estimate soil behavior from the SPT blow count.

Contracting for Becker Drilling Services

In addition to the usual specifications requirements, the work statement for BPT should address:

- Work requirements — explain general work requirements.

- Purpose and scope — state which portions of the work are for liquefaction assessment and which are for instrumentation or other purposes.

- Local conditions and geology — describe anticipated drilling conditions and potential problem areas.

- Equipment and personnel to be furnished by the contractor — specify complete details on the equipment: rig model numbers, hammers, superchargers, and double wall pipe for rods. See above for details.

- Drilling requirements — list special considerations such as staking, calibration requirements, and refusal criteria.

- Hole completion — describe all hole completion or abandonment procedures.

- Driller's logs — list requirements for the driller's report, including forms to be used.

- Field measurement — specify method of measurement of depths for payment.

In the contract for PDA work, specify the following:

- Purpose and scope of testing.
- Estimated number of feet of driving to be monitored by PDA.

Cone Penetration Test

Test History

The CPT was introduced in northern Europe in the 1930s to facilitate the design of driven pile foundations in soft ground. Early devices were mechanical penetrometers that incrementally measured the cone tip resistance. In the 1960s, mechanical cones, known as Begemann friction cones, were developed. This penetrometer measured both the tip resistance and the side resistance along a sleeve above the cone tip (figure 22-6). At about this same time, the CPT was introduced in North America. Using technology from the rapidly advancing electronics industry, an electric cone penetrometer was developed that used electrical transducers to measure the tip and side resistance (figure 22-7). Most of the work today is performed with electronic cone penetrometers, and the manual does not discuss mechanical systems. The use of electronics allows the incorporation of additional sensors in the cone system, including those for pore water stress, temperature, inclination, acoustic emissions, down-hole seismic, and resistivity/conductivity. In the 1990s, sensors such as laser or other energy-induced fluorescence spectroscopy sensors, membrane interface probes, and even video cameras have been added to detect groundwater contamination. Penetrometers capable of measuring dynamic or static pore water pressures are called piezometric cones or piezocones (CPTU). CPT

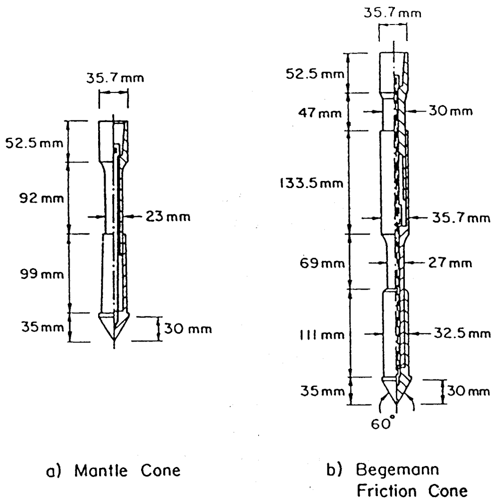


Figure 22-6.—Mechanical cone penetrometers.

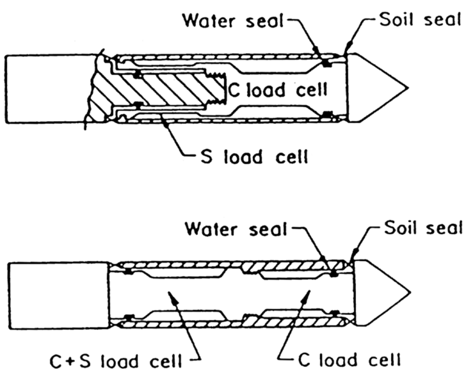


Figure 22-7.—Typical electrical cone penetrometers.

has continued to gain wide acceptance as an effective site investigation tool in North America.

Test Procedure

The procedures for performing CPTs are standardized in Reclamation procedures USBR 7020 and 7021 and ASTM D-5778 and D-6067. The test is highly reproducible as opposed to SPTs. Test standards call for a cone tip 35.7 mm in diameter with a 10-square-centimeter (cm^2) projected area and an apex angle of 60 degrees. The friction sleeve is 150 cm^2. Larger diameter penetrometers of 15-cm^2 projected area are sometimes used in very soft soils. Smaller diameter penetrometers are sometimes used for laboratory studies of soils.

The cone is advanced at a constant rate of 20 mm per second. Since the penetration resistance depends significantly on the advance rate, the push rate must be checked in the field. The basic equipment required to advance any cone penetrometer is a hydraulic jacking system. Trucks or vehicles built for CPT are typically used; but, in some cases, the hydraulics of rotary drill rigs are used. Semi-portable equipment has been developed for remote site testing. Rigs can be mounted on trucks, tracked vehicles, trailers, barges, or diving bells, depending on accessability. The capacity of cone rigs varies from 100 to 200 kilonewtons (kN) (11.2 to 22.4 tons). The upper bound is the maximum allowable thrust on the cone penetration rods.

Electronic cone penetrometers have built in load cells to measure the tip and side resistance simultaneously (Figure 22-7). Bonded strain gauges typically are used in the load cells because of their simplicity and ruggedness. The load cells commonly have a range of 90 kN (10 tons) for tip resistance and 9 kN (1 ton) for side resistance. The load cell capacity can be varied, depending on the strength of the soils to be penetrated. The load cells are usually connected by an electric cable passing through the drill rods to a data acquisition system at the surface. Cordless models are also available that transmit sonically and "Memo" cones that store the data internally until retrieved at the surface. Data are recorded digitally, which greatly enhances the use of CPT results in engineering applications. The data can be sent in daily by e-mail to the engineer and geologist.

Nearly all electronic cone penetrometers are equipped with a pore pressure element. This pore pressure sensor is typically located between the tip and the friction sleeve. The element can record dynamic water pressure as the cone is being pushed, as well as static water pressures during pauses in testing. The typical capacity of the water pressure transducer is 2.2 kN (500 lb/in^2), and the accuracy of water pressure head is about ± 3 cm (0.1 foot). Cones are almost always equipped with inclinometers. The inclinometers are used to monitor rod bending during push and are an essential part of protecting the cone from damage. The inclinometer can be monitored by computer, and pushing can be stopped if bending is excessive. Cone rods can bend as much as 10 to 20 degrees. If the cone is used to detect bedrock or hard layers, this error can be significant. The inclinometer is not directional, so the error from bending can only be estimated.

Advantages and Disadvantages

The CPT has several advantages over other routine in place tests. The tests are rapid and inexpensive compared to other geotechnical profiling techniques. Penetration rates of 3 feet (1 m) per minute are common in many soils. Penetration is stopped only to add sections of push rods, except when pore water stress dissipation measurements are made with the piezocone. With electrical equipment, continuous profiles are recorded and plotted as penetration progresses, and operator effects are minimized. As discussed below, the test results have been correlated to a variety of soil properties. Digital data acquisition with electrical cones enhances interpretation and provides continuous profiles of soil property estimates.

Although the tests are applicable to a wide range of soil conditions, penetration is limited in certain ground conditions. Well-cemented soils, very stiff clays, and soils containing gravel and cobbles may cause damage to the penetrometer tips.

The CPT can be used at nearly any site because portable devices are available. Portable hydraulic jacking systems can be used for soft soils in locations not accessible to standard rigs.

The CPT has several disadvantages. The test does not provide soil samples. The test is unsuited for well-cemented, very dense and gravelly soils because these soils may damage the relatively expensive penetrometer tips.

Local experience with this test is less than that with the SPT. Although the test is rapidly gaining acceptance in the United States, some drilling contractors do not have the equipment or experience necessary to perform the test. The equipment is expensive and may not be available in some locations. Maintaining the electronics for the CPT and CPTU equipment may be a problem in some test locations.

Data Obtainable

The CPT is primarily a logging tool and provides some of the most detailed stratigraphic information of any penetration test. With electronic cones, data are typically recorded at 5-cm-depth intervals, but data can be recorded at closer spacings. Layers as thin as 10 mm can be detected using the CPT, but the tip resistance can be influenced by softer or harder material in the layer below the cone. Full tip resistance of an equivalent thicker layer may not be achieved. The penetration resistance of the soil is a function of the drainage conditions during penetration. In sands that are drained, the penetration resistance is high, but in clays that are undrained, the penetration resistance is low.

A typical CPT data plot is shown in figure 22-8. CPT plots should show all recorded data (i.e., Tip Resistance, q_c , Sleeve Resistance, f_s , Pore pressure, u, and for this example, cone inclination and temperature). CPT data should be plotted to consistent scales on a given project so that the plots can be more easily evaluated.

The CPT does not obtain a soil sample. However, the soils may be classified by comparing the tip resistance to the ratio of tip to sleeve resistance which is known as the friction ratio, F_r . Friction ratio should also be shown on the summary plots. Figures 22-9 and 10 show commonly used relationships to estimate the "soil behavior type." Clay soils have low tip resistance and high friction ratio, while sands have high tip resistance and low friction ratio. Mixed soils fall in zones 4 through 7. There are also classification methods that incorporate the dynamic pore water pressure generation. The CPT cannot exactly classify soil according to the Unified Soil Classification System. Experience at many sites shows that soils give consistent signatures; and even though the soil behavior

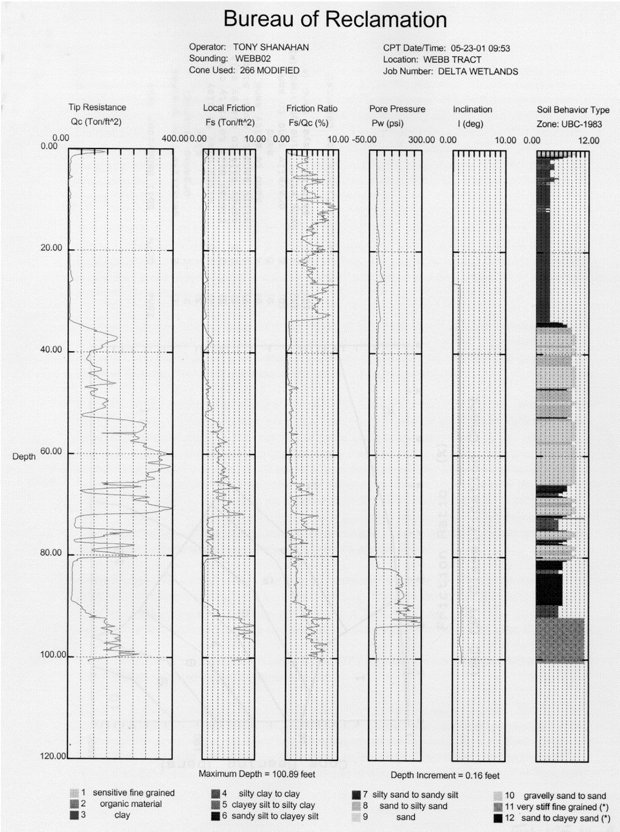
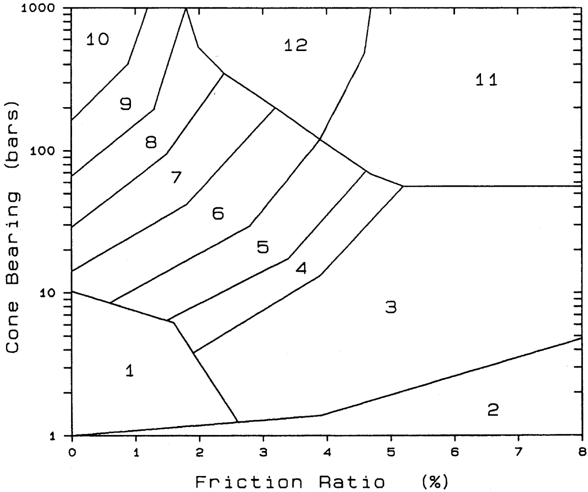


Figure 22-8.—Example CPT data plot.



UBC CPT Classification Chart

Zone	Qc/N	Soil Behaviour Type
1)	2	sensitive fine grained
2)	1	organic material
3)	1	clay
4)	1.5	silty clay to clay
5)	2	clayey silt to silty clay
6)	2.5	sandy silt to clayey silt
7)	3	silty sand to sandy silt
8)	4	sand to silty sand
9)	5	sand
10)	6	gravelly sand to sand
11)	1	very stiff fine grained (*)
12)	2	sand to clayey sand (*)

(*) overconsolidated or cemented

Figure 22-9.—Chart for estimating the soil behavior type.

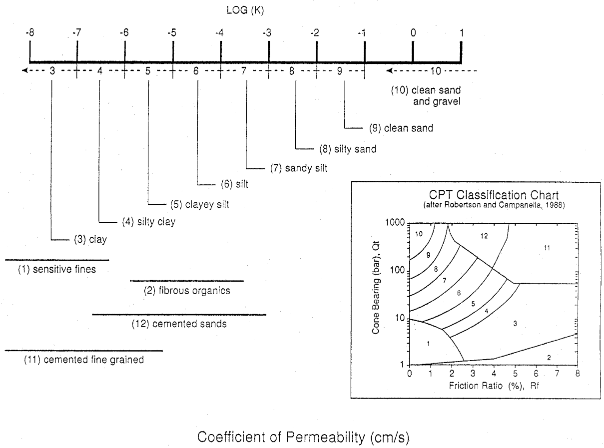


Figure 22-10.—Chart for estimating the soil behavior type and the coefficient of permeability.

type is generally correct, the soil types should be confirmed with a sample boring. Soil behavior type prediction in the unsaturated zone is less reliable but often still useful. The summary plot in figure 22-8 also shows the soil behavior group on the right side bar.

Soil permeability can be estimated from CPT because the tip resistance is a function of drainage during penetration. The permeability estimate is generally within an order of magnitude, which is suitable for most groundwater and seepage studies (figure 22-10).

Numerous correlations of CPT data to strength and compressibility of soils have been developed. These correlations are based primarily on tip resistance but are also supplemented by sleeve friction and dynamic pore water pressure data.

CPTs in clean sands have been performed in large calibration chambers where the density and confining pressure have been controlled. Based on the chamber data, the relative density and friction angle of sand can be estimated using relationships such as those shown in figure 22-11. The tip resistance at a constant relative density increases with increasing confining pressure. Once the relative density is estimated, the friction angle

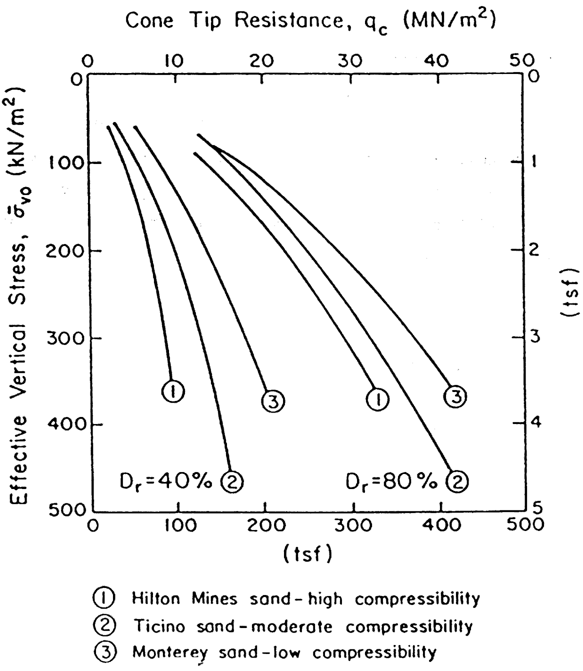


Figure 22-11.—Relationships between cone tip resistance, relative density, and effective vertical stress.

can be estimated. The compressibility of the sand depends on the mineralogy of the sand particles. Highly compressible sands may contain soft particles. If mica is present in the sand at percentages as low as 5 percent, the compressibility will increase. Samples of the sand to determine mineralogy may be necessary. These estimates for sands are not applicable to sands containing more than 10 percent fines.

The CPT can be used to estimate the undrained strength, S_u, for clays because the CPT is like a cone bearing test in rapid, undrained loading. Figure 22-12 shows that the cone factor, N_k, must be estimated for clay. Typically, a factor of 12 to 15 is used. The factor can be refined by cross correlating with sampling and unconfined compression testing or by vane shear testing.

Compressibility of soils can be estimated by the CPT test, but the consolidation behavior should be confirmed by sampling and laboratory testing.

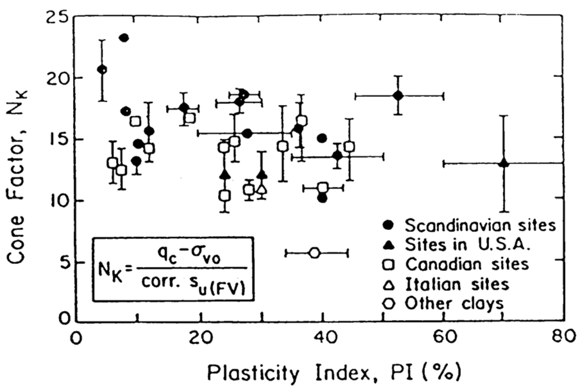


Figure 22-12.—Empirical cone factor, N_k, for clays.

The CPT is the best method for estimating the liquefaction resistance of sandy soils. The SPT is also used but has many problems in drilling and with equipment. The CPT tests the sand in place without disturbance, and the test is highly repeatable. Figure 22-13 shows the chart used to estimate liquefaction triggering. The chart is based on "clean sands," but the method includes conversion of dirty sands for evaluation. If the CPT can be used for liquefaction evaluation, it should definitely be considered in the exploration plan. SPT should still be performed at a few sites, but the CPT can be used to rapidly and economically

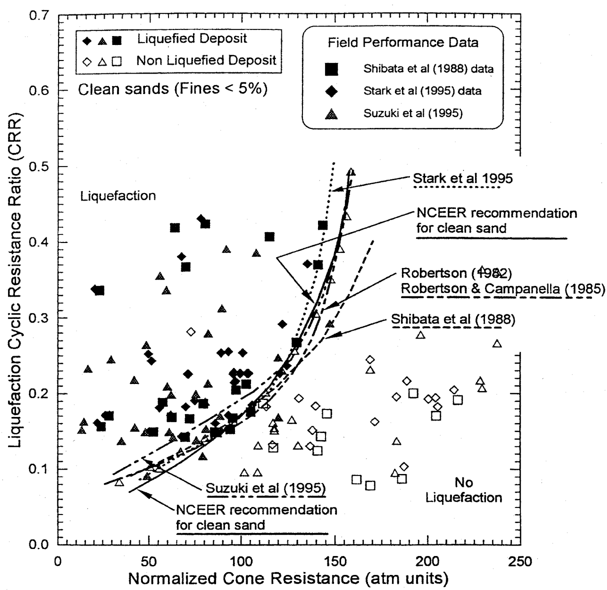


Figure 22-13.—Comparison of various cyclic resistance ratio (CRR) curves and field data.

map the extent of liquefiable strata. CPT is also used extensively for evaluating ground improvement of liquefiable deposits.

The cone is like a miniature pile and is used for evaluating pile capacity. CPT tests are often performed at the abutments of bridges for pile design. Numerous methods exist for estimating pile capacity.

Economics

Equipment costs for CPT range from low for mechanical devices to high for piezocones, and generally two technicians are required to perform CPTs. These personnel should have a working knowledge of the equipment, but highly trained technicians are not required. The equipment mobilization is similar to that required for the SPT, but portable devices can be used for remote locations. Unit costs are difficult to estimate because the tests provide continuous or nearly continuous measurements. Rig costs are comparable to costs for the SPT, with an added capital cost to convert a conventional drilling rig for CPT testing. However, 200 feet (60 m) of penetration per day is typical; and in some cases, maximum production of 400 feet (120 m) per day is possible. This cost is the lowest of any geotechnical drilling, sampling, and logging method.

Bibliography

Harder, L.F., *Application of the Becker Penetration Test for Evaluating the Liquefaction Potential of Gravelly Soils*, NCEER Workshop on Evaluation of Liquefaction Resistance, held in Salt Lake City, Utah, 1997.

Harder, L.F. and Seed, H.B., *Determination of Penetration Resistance for Coarse-Grained Soils Using the Becker Hammer Drill*, Report No. UC/EERC-86-06, NTIS PB87-124210, Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley, California, May 1986.

Peck, Ralph B., *Foundation Engineering*, Second Edition, John Wiley & Sons, 1973.

Rausche, F., Goble, G.G., and Likins, Jr., G.E., *Dynamic Determination of Pile Capacity*, Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 3, pp. 367-383, 1985.

Sy, A. and Campanella, R.G., *Becker and Standard Penetration Tests (BPT-SPT) Correlations with Consideration of Casing Friction*, Canadian Geotechnical Journal, 31(3): 343-356, 1994.

Sy, A., Campanella, R.G., and Stewart, R.A., *BPT-SPT Correlations for Evaluation of Liquefaction Resistance of Gravelly Soils, Static and Dynamic Properties of Gravelly Soils*, ASCE Special Publication 56, M.D. Evans and R.J. Fragazy, Editors, American Society of Civil Engineers, New York, New York, 1995.

Chapter 23

HANDLING AND TRANSPORTING ROCK AND SOIL SAMPLES

Introduction

Subsurface sampling is expensive and time consuming, so it makes sense to obtain all information possible from the investigative process and the retrieved samples. After expending the money and effort to obtain subsurface information, the samples should not be subjected to unacceptable temperature, rough handling, shoddy packaging, or harsh transportation methods. Any type of mishandling of a sample may make the sample useless for testing or logging, and the investigation may have to be repeated.

There are certain procedures in the handling and transporting of samples that should be a part of every investigation. However, some programs might require more demanding techniques in the handling of samples than others, depending on the types and ultimate uses of the samples. Proper handling results in lower overall costs when the cost of repeating an investigation is considered.

The care and handling required for samples collected for hazardous or toxic material investigations can vary considerably, and details for this type of sampling are not provided in this chapter. Information on required volumes of material, types of storage containers, methods of collection and preservation, and maximum holding periods should be obtained from the appropriate regulatory agency.

Poor samples are sometimes the result of poor drilling practices rather than mishandling. Compression fracturing of core, "spun" core, or uneven or wavy core surfaces may indicate that drilling techniques are inappropriate for the conditions. Problems should be noted in the logs and corrections or improvements should be made immediately, if practicable, to the drilling procedures.

When samples requiring extreme care are obtained, personnel involved in the investigation must be totally familiar with the following procedures, and they must have a clear understanding of which geologic features are to be sampled and why they are to be sampled.

The geologic characteristics and the intended use of the rock and soil samples determine the extent and type of care required. If engineering properties are to be determined, the sample must be handled and preserved so that the properties are not significantly influenced by mechanical damage, changes in chemistry, and environmental conditions such as moisture and temperature.

Design requirements for samples range from geologic logging to complex and critical testing in the laboratory. Priorities for multiple uses or different types of tests must sometimes be established when available samples are limited and when a use or test precludes another test. When the required level of protection is unknown, overprotection is better than underprotection. If the sample is intended for geologic logging only, the care and preservation may be entirely different from that which would be needed if testing in the laboratory is required. Even though the sample has been geologically logged, other studies might be necessary, and continuous care in handling and proper storage may be required.

Samples can range from highly disturbed to relatively undisturbed. The methods of obtaining soil samples vary and may range from "grab" samples to samples obtained by highly refined drilling and sampling techniques. Soil can be difficult to sample whether by drilling or test pits, and obtaining an "undisturbed" sample of soil can be an ordeal. To obtain good "undisturbed" (minimally-disturbed) soil samples, extensive knowledge and experience with numerous types of soil sampling equipment, drilling methods, and drilling fluid additives is necessary.

Specific procedures for the disposal of rock core and samples depend on several factors; see chapter 24 for guidelines.

Sample Protection

The following describes recommended procedures for handling rock and soil samples (samples). The sample groupings are based on the type of testing to be performed, rather than any physical condition of the samples, and generally conform with ASTM Standards. Groups 1 through 4 deal with rock core, but note that the change to soil-like characteristics is transitional rather than abrupt. Table 23-1 summarizes the procedures.

Drilled core not requiring special preservation should be removed from the core barrel and placed directly into the core box. Core requiring special preservation should be placed in the core box as soon as possible after completion of preservation. Any required special moisture preservation procedures should be done immediately. Fragile core must be immediately protected by wrapping and sealing. Preliminary logging should take place in the field, but protective measures take precedence over time-consuming detailed logging. Any core that is to be wrapped or otherwise protected should be photographed first. With extremely sensitive core, protective measures take precedence over photography.

Table 23-1.—Rock- and soil-sample categories for handling and transportation

			Group							
			Rock				Soil			
			1	2	3	4	A	B	C	D
Characteristics	Rock	Non-sensitive, non-fragile	●							
		Moisture sensitive		●						
		Fragile, moisture or temperature sensitive			●					
		Soil-like rock				●				
	Soil	Visual identification, gradation					●			
		Percent moisture, Proctor, density						●		
		Intact, special testing [2]							●	
		Intact; fragile or highly sensitive for special testing								●
Testing Requirements	Logging		●	●	●	●	●	●	●	●
	Percent Moisture			●	●			●	●	●
	Special Testing [2]			●	●			●	●	●
Handling and Transporting	Rock	Core boxes	●	●	●					
		Film, foil, and wax		●	●			●	●	●
		Shock or vibration protection			●	[1]			●	●
		Heat or cold protection			●				●	●
	Soil	Avoid contamination					●			
		Any type container					●			
		Sealed, moisture-proof container			●			●	●	●
		Keep in natural position							●	●

[1] See Groups A through D requirements.

[2] Special testing can include density and swell determinations, consolidation, permeability, shear and compressive strength, and other tests as required.

The core should be photographed to provide images as shown in figure 23-1. The boxes should be oriented relative to the camera to minimize distortion and lighting should be appropriate to maximize resolution. A photography frame, as shown in figure 23-2, provides consistent orientation and image quality.

Group 1: Nonsensitive, nonfragile samples for which only geologic logging is necessary belongs to this group.

For rock cored in 5- to 10-foot (1.5- to 3-m) runs, samples are sufficiently protected if placed in strong wood or plastic core boxes. Cardboard boxes are unacceptable because the boxes deform when wet, rot, and are readily destroyed by insects. If very long solid cores have been recovered and need to be preserved intact, place each core in a stiff tube or two half-rounds of tubing of equal or slightly greater length than the core and secure both ends. The inside diameter of the tube should be slightly larger than the core diameter and the tube walls must be rigid enough to prevent core breakage because of bending.

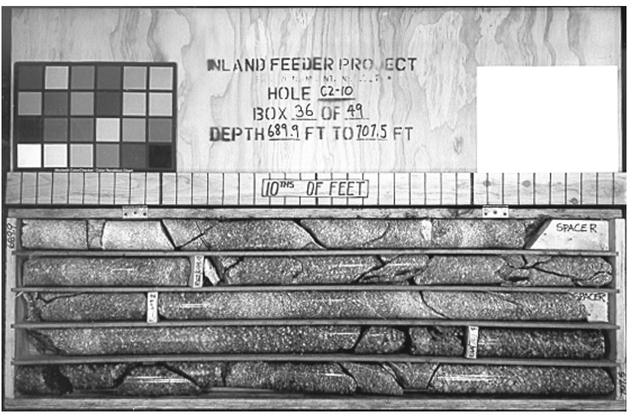


Figure 23-1.—Properly boxed and labeled core.

Figure 23-2.—Core storage area with boxes neatly arranged and separated by spacers for ventilation. Note core photography frame in center of photograph.

The tubing should be impermeable, such as Lucite or PVC, to prevent loss of stiffness of the tubing in moist conditions.

Group 2: Rock core that is subject to moisture loss or gain and must be tested later belongs in this group. Requirements for this level of protection include requirements for Group 1.

The moisture condition of some rocks and even the moisture history of rocks such as shales affect the rock properties. If tests are to be performed on the core and if a change in moisture conditions may influence the test results, the core must be sealed to prevent moisture loss. This same procedure applies to any samples when it is important to maintain fluids other than water, such as hydrocarbons.

The term, “wax,” refers to a “plastic” microcrystalline mixture composed of one part microcrystalline wax for strength and adhesion and one part paraffin or beeswax for plasticity. This mixture optimizes adherence and minimizes brittleness and cooling cracks in the wax. This material should be applied at no more than 18 degrees Fahrenheit (° F) (10 degrees Centigrade (° C) above its melting point, especially if the wax is applied in direct contact with the sample material. Wax that is too hot can penetrate sample pores and cracks. If the wax bonds to the sample, the wax is too hot.

Sealing samples to preserve moisture should consist of a tightly fitting wrapping of a plastic film, such as vinylidene chloride (Saran wrap), covered by a tight wrapping of aluminum foil. Each of these wrappings should be applied so that as little air as possible is trapped beneath the wrappings. Lap the ends of the wrappings over the ends of the sample and fold over to seal the ends. Finally, apply a minimum of ⅛ inch (3 mm) of wax over the entire surface of the sample. A layer of cheese cloth applied on the foil before waxing is a good idea. This thickness of wax should consist of at least two coatings and preferably more. For long periods of storage, apply a minimum of ¼ inch (6 mm) of wax. Wax may be applied by brushing it on or by dipping the wrapped sample in a container of melted wax. After waxing, the sample may be placed and transported in a core box.

In some cases, aluminum foil wrapping might react chemically with either the sample or the sample’s fluids. If a reaction is possible, aluminum foil should be replaced with some other metal foil that is nonreactive. A less preferable method is to omit the metal foil and increase the thickness of the sealing wax. If the metal foil is not used, the thickness of the sealing wax should be increased to ¼ inch (6 mm) when the required storage time is short (up to 1 month). For a longer period, a thickness of ⅜ inch (9 mm) is adequate. A layer of cheese cloth should be incorporated into the wax.

Group 3: These are samples that are fragile or moisture or temperature sensitive. This protection level includes the requirements for Groups 1 and 2.

If shock, vibration, or variation in temperature may subject samples to unacceptable conditions during transport, the samples should be placed in core boxes that provide cushioning or thermal insulation. Fragile samples may require packing in a cushioning material, such as sawdust, rubber, Styrofoam, (poly)urethane foam, bubble wrap, or similar material. The cushioning between the samples and walls of the core boxes should have a minimum thickness of 1 inch (25 mm) and should have a minimum thickness of 2 inches (50 mm) on the core box bottom and lid. The samples must fit snugly into the boxes.

Samples that are temperature sensitive should be insulated by placing the core container (box or tube) inside another container that is designed specifically to provide thermal insulation. These containers are generally constructed of double or triple layers of insulating material and are usually relatively airtight.

Group 4: These materials are so poorly indurated that soil sampling procedures must be used to obtain intact pieces of core. Certain shale or highly weathered rock that contains these materials as interbeds belong to this group. Group 4 samples are more soil-like than rock-like and should be treated according to the appropriate instructions for Groups A through D below.

Group A: Consists of soil samples for which only visual identifications or gradations are necessary. The main concern in dealing with this group is to avoid contamination with other soils. Group A samples may be transported in any type of container by any transportation method. If transported commercially, the container need only meet the requirements of the transporting agency and any other requirements necessary to prevent sample loss.

Group B: These are samples for which only water content and classification tests are required or Proctor compaction, relative density, or profile logging are required. Bulk samples that will be remolded or compacted into specimens for tests such as swell pressure, percent swell, consolidation, permeability, and shear testing are also included.

In all cases for Groups B, C, and D soils, a sample should be obtained for determination of in place moisture content; and the determination should be performed as soon as possible.

Group B samples should be preserved and transported in sealed, moisture-proof containers. Containers should be thick enough and strong enough to protect against breakage and moisture loss. The container types include: waterproof plastic bags or pails, glass or plastic jars, thin-walled tubes, and liners. Cylindrical and cube samples should be wrapped in suitable plastic film or cheese cloth, or both, and should be coated with several layers of wax according to the instructions for Group 2 samples.

If plastic bags or wrapping are used, the bags should be placed as tightly as possible around the sample, squeezing or sucking out as much air as possible. The plastic should be three mil or thicker to prevent leakage. If glass or plastic jars are used that do not close tightly, the lids should be sealed with wax. If plastic pails are used and the lids are not air tight, the lids should be sealed with tape and wax.

Thin-wall push tubes can be sealed with expandable packers, waxed wood disks, or cheesecloth and wax. The preferred (although expensive) method for sealing thin-wall tubes is with plastic or metal expandable packers, seated on the soil surface. These devices, tightly abutting the soil, will keep the soil firmly fixed in position in the tube. If waxed wood disks are used, a disk slightly smaller than the inside diameter of the tube should be inserted in each end of the tube, be in snug contact with the soil and then be sealed with wax. Several thin layers of wax are preferred over one thick layer. The final thickness should be at least 0.4 inch (10 mm).

Other spacers or packing materials extending from the waxed-wood disk or waxed-soil surface to the tube end are not recommended because these can allow the samples to move in the tubes. Any packing material must be nonabsorbent and must support the samples throughout shipment and storage.

Metal, rubber, or plastic end caps should be sealed with tape. For long-term storage (longer than 1 month), the taped end caps should be dipped in wax, applying two or more layers. End closures solely of cheesecloth and wax should consist of alternating layers of (a minimum of two each) cheesecloth and wax.

Cylindrical, cubical, or other samples wrapped in plastic or foil should be protected with a minimum of three coats of wax and cheesecloth. Cylindrical and cube samples wrapped in cheesecloth and wax should be sealed with a minimum of three alternating layers. Cylindrical samples and small cube samples placed in cartons must be positioned so that wax can be poured completely around the sample. The wax should fill the void between the sample and container wall. To facilitate handling when placed in cartons, large cube samples, such as waxed "block" samples, should be encapsulated in damp sawdust rather than wax. Generally, waxed samples should be wrapped in plastic or foil before being surrounded by wax in a carton.

These samples may be transported by any available transportation. The samples should be shipped as prepared or placed in larger containers such as bags, cardboard or wood boxes, or barrels. Appropriate packing should be provided to prevent breakage or puncturing of the individual sample wrapping or container.

Group C: These are intact, natural or field compacted samples for density determinations or for swell pressure, percent swell, consolidation, permeability testing, and shear testing with or without stress-strain and volume-change measurements.

These samples are preserved and transported in sealed, moisture-proof containers. Containers should be thick enough and strong enough to prevent breakage and moisture loss. The container types include: plastic bags or pails, glass or waterproof plastic jars for disturbed samples, and thin-walled tubes and liners. Cylindrical and cube samples should be wrapped in suitable plastic film or aluminum foil and coated with several layers of wax and cheesecloth according to the instructions for Group 2 samples. Some soils may corrode aluminum foil; and direct contact with aluminum foil should be avoided where sample composition might cause adverse effects. Temperature sensitive Group C samples should be insulated similar to Group 3.

Samples transported on seats of vehicles can be placed in cardboard boxes or similar containers, and samples need to be packed to prevent bumping, rolling, and dropping.

If the samples are not transported on vehicle seats, the individual samples should be placed in wood, metal, or other types of suitable shipping containers that provide cushioning or insulation for each sample. The cushioning material should completely encase each sample. The cushioning between the samples and walls of the shipping containers should have a minimum thickness of 1 inch (25 mm). A minimum thickness of 2 inches (50 mm) should be provided on the container bottom. When required, the samples should be kept in the same position in which they were sampled from the time they leave the ground until testing is completed. Special conditions should be provided, such as freezing, controlled drainage, or sufficient confinement to maintain sample integrity.

Group D: Samples that are fragile or highly sensitive and that require tests in Group C are assigned to Group D. The requirements for Group C must be met in addition to the following requirements: Samples should be handled and stored, including during transportation, in the same orientation in which they were sampled.

Storage Containers

Containers should be designed to include:

For core

- Core boxes must be constructed rigidly enough to prevent flexing of the core when the box is picked up by its ends. Wood is preferred and should be ¾ inch (nominal) (19 mm) thick. Partitions between core rows should be firmly fixed in place to increase the stiffness of the box. The lid should have strong hinges and hasps or fastened with screws. Do not drive nails in the lid. An example of core box construction is shown in figure 23-3.

- The core box should be designed for the anticipated diameter of core, including any packing and cushioning materials. If the core box is too large for the core, spacers or packing material should be placed in the box to support the core and prevent the core from moving in the box. If the core box is too small for the core, the core should not be hammered into the box.

For soils in Groups C and D and for rock in Groups 3 and 4, if required:

- The container should be constructed so that the samples can be maintained in the same position as when sampled or packed.

- The container should include sufficient packing material to cushion or isolate the samples from vibration and shock.

- The container should include sufficient insulating material to prevent freezing.

Shipping Containers

The following features should be included in the design of shipping containers:

- Plywood (preferably marine plywood) ½ or ¾ inch (13 to 19 mm) thick may be used for shipping containers. The top (cover) should be hinged and latched or fastened with screws. The entire shipping container should be lined with insulation a minimum of 2 inches (50 mm) thick for protection against freezing or temperature fluctuation.

- Metal shipping containers should have cushioning and insulating material similar to wood shipping containers, although slightly greater thicknesses are appropriate. Cushioning with a spring suspension system or any other means that provide similar protection is acceptable.

- Bulk Styrofoam with slots or pockets in the shape of the sample tube or liner should be enclosed in a protective outer box of plywood or cardboard.

- Properly lined containers constructed of laminated fiberboard, plastic, or reinforced cardboard outer walls are acceptable.

Core Handling

The following procedures are necessary to ensure that rock core is firmly fixed in the core box and is appropriately labeled for inspection and logging:

- Orient the core box with the slots horizontal. Place the core in the core box starting with the shallowest depth at the upper left hand corner and progressing across and then downward, as in reading a book, with the deepest depth at the lower right hand corner.

- Place core blocks at the ends of each run with the depth clearly written on them.

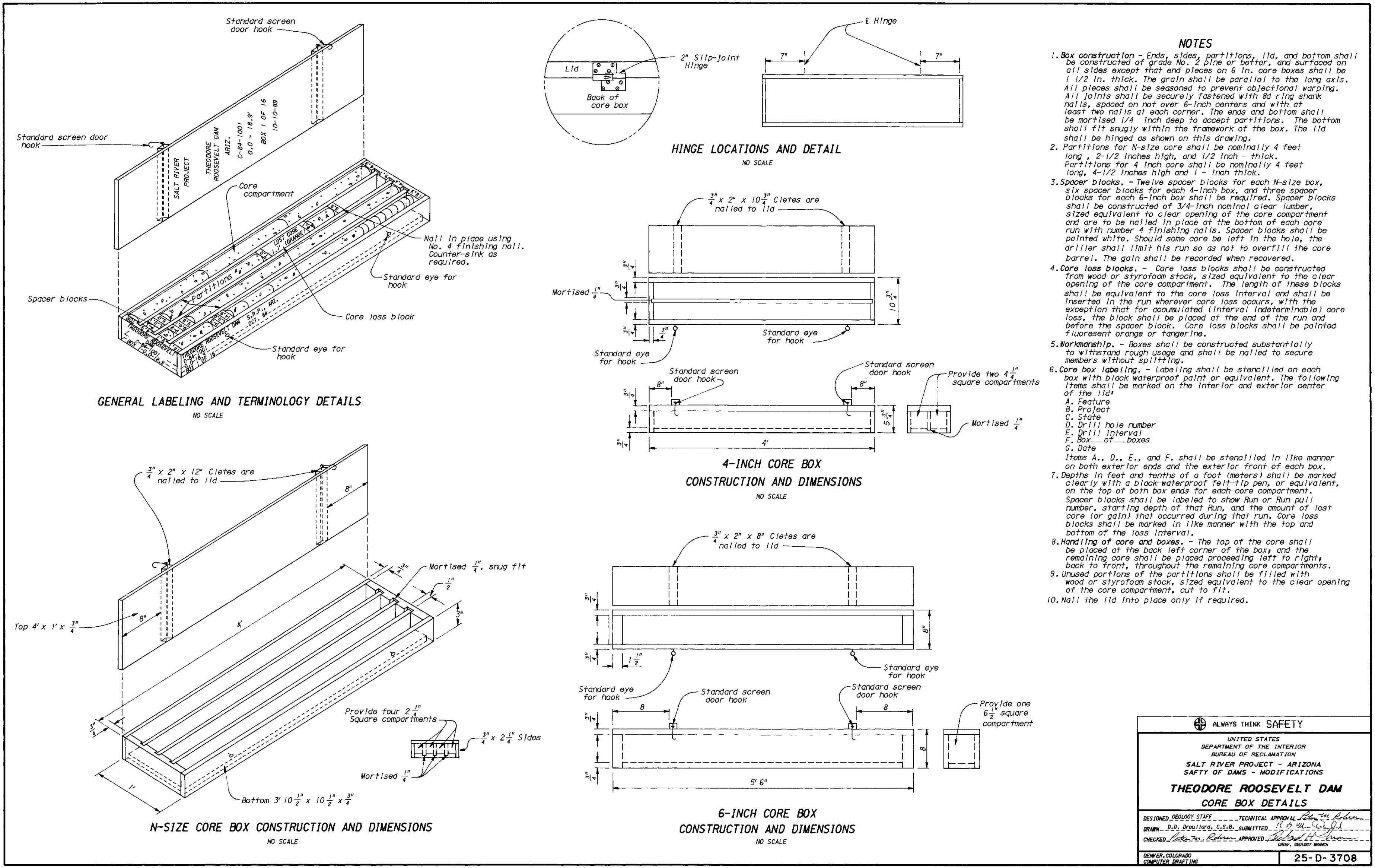


Figure 23-3.—Example core box construction.

- Runs of less than 100-percent core recovery should be shown by spacer blocks placed in the core box at the suspected missing interval and clearly identified.

- After photographing and after samples are prepared, samples removed from the core should be replaced by a spacer block equal in length to the core removed. The block should be labeled “sample,” and depths should be noted and placed in the core run.

- Breaking the core should be avoided because it may reduce the number of available test specimens. If it is necessary to make core fit in the core box, note on both the daily drill report and the geologic log and mark on the core to indicate mechanical breakage.

- If needed, add sufficient spacer blocks to prevent the core from shifting during subsequent box handling.

Figure 23-1 is an example of properly boxed and labeled core.

Identification of Samples

All samples, whether rock or soil, must be properly identified, and the following information should be written on the sample or container:

- Project name
- Feature name
- Hole, pit, or trench number
- Top and bottom depths of sample interval
- Sampling date
- Sample number

In addition to the above:

- Clearly and prominently mark samples containing suspected or known hazardous materials.

- Attach sample tracking or "chain-of-custody" form, if required.

- Clearly mark samples that must be handled and stored in the same orientation as sampled, stating that requirement and the required orientation.

Transportation Requirements and Procedures

For all modes of transportation, the loading, transporting, and unloading of shipment containers should be supervised as much as possible. The following requirements and procedures should be considered when transporting core:

- Remove the sample from the drill site before it has a chance to freeze, heat up, or be damaged by activities at the drill site.

- Handle samples gently during loading and unloading. Never drop boxes or tubes; slide them gently into position. If a box or tube is dropped, record this fact.

- Transport by an appropriate vehicle to prevent damage by mechanical vibration, shock, freezing, and high temperatures along the entire transport route. In some cases, it may be necessary to remove the core from the vehicle and place it in a motel room or suitable storage area at night unless the vehicle can be left idling to keep the samples at an appropriate temperature.

- If rough terrain must be crossed, protect samples in the Group 3 and Group D categories by padding around the sides, bottom, and top of the core containers.

- Transportation by passenger car or air freight, rather than van or truck, may be more appropriate for fragile core. However, if air freight is used, ensure that the samples are transported in a heated, pressurized compartment. If the core is to be transported in the bed of a truck, there should be no heavy objects that could slide or roll into the sample containers.

- The driver of the vehicle carrying the samples should be thoroughly instructed in the required care of the samples.

- The vehicle carrying the samples should be in good mechanical condition. A breakdown on the transport route could leave the samples exposed to extreme weather conditions.

- Commercial carriers should be used only when samples are not sensitive to shipping damage and in-house forces and equipment are unavailable. Commercial carriers should be used only if the carrier is capable of delivering the samples in the desired condition. The sample custodian should ensure that all sample care requirements are clearly indicated on the lading documents.

Upright Handling and Shipping of Samples

For some types of studies or testing, such as those performed on liquefiable sands and sensitive clays, the samples must never leave the vertical or upright position. If samples should never leave the vertical or upright position, the following should be considered:

- Samples that must remain in the vertical position should be obtained with a thin-wall push-tube sampler. These samples are the easiest to handle once they are out of the ground. The ends can be easily sealed with expandable packers.

- If thin-wall push tube samples are not taken, sample lengths and diameters should be kept to a minimum. Six-inch- (15-cm-) diameter core that is 2 feet (0.6 m) long is about the maximum size that should be obtained. Samples larger than this are very difficult to handle in any position.

- Samples should never be removed from the liner or sampling tube.

- Sample tubes should be marked so that top and bottom are easily discernable.

- Racks that will hold the samples in an upright position must be designed and built for the size of sample being taken.

- Samples that must be transported commercially should be packed in a wood box. They should be insulated against temperature changes and protected against movement within the box. The packing box should be clearly marked to indicate which side is to remain up, that the contents are fragile, and that the contents require protection from freezing.

Storage Environment

All samples should be grouped according to storage requirements and stored so that air can circulate around the sample or sample container (figure 23-2). Samples that are sensitive to moisture loss should be stored at room temperature (73 °F, 22 °C) and at approximately 60-percent relative humidity. Higher levels of relative humidity can cause fungal growth. Samples in metal tubes should be processed as soon as practical to minimize sample changes that might occur because of interaction between the sample and the metal of the tube.

Wood boxes containing samples should be protected from repeated wetting and drying. This could cause warping and delaminating of the wood. The boxes should be protected against damage by insects and rodents.

Recommended Equipment

Following is a list of equipment necessary to process, handle, and transport either rock or soil samples:

- Vinylidene chloride (Saran wrap) film, aluminum foil, wax, down spout, PVC, or similar tubing
- Stove to melt wax
- Lucite or PVC tubing
- Sawdust, rubber, Styrofoam, or material of similar resiliency to cushion the core
- Miscellaneous equipment, such as adhesive tape and waterproof felt-tip markers

- Wood disks, prewaxed, 1 inch (25 mm) thick, with a diameter slightly less than the inside diameter of the thin wall tube
- Tape, either waterproof plastic or duct tape
- Cheesecloth
- Caps, either plastic, rubber, or metal, to be placed over the end of thin wall tubes
- Plastic or metal expandable end caps to seal the ends of samples within thin wall tubes by mechanically expanding an "O" ring against the tube wall
- Plastic or glass jars, wide-mouthed, with rubber-ringed lids or lids lined with a coated paper seal, commonly ½ pint (250 milliliter [mL]) and quart (1,000 mL)
- Bags, either plastic or burlap, with waterproof liner
- Plastic pails with air-tight lids
- Packing material to protect against vibration and shock
- Insulation to resist temperature change of samples to prevent freezing
- Sample cube boxes for transporting cube (block) samples, constructed with ½- to ¾-inch (13- to 19-mm) plywood (marine type)
- Cylindrical sample containers somewhat larger than the thin-wall tubes or liner samples

- Shipping containers, either box or cylindrical type, of proper construction to protect against vibration, shock, and weather; the length, girth, and weight of the containers must be considered when using commercial transportation

- Spacer-block material to replace samples taken

Chapter 24

CARE, RETENTION, AND DISPOSAL OF DRILL CORE, SOIL, AND ROCK SAMPLES

General

These guidelines for the care, retention, and disposal of drill core, soil, and rock samples apply to (1) foundation exploration for structures, such as dams, canals, tunnels, powerplants and pumping plants, (2) construction materials investigations including riprap and borrow, and (3) safety of existing structures investigations. Sample retention and disposal guidelines for concrete aggregate, pozzolan, land classification, drainage (except for drainage structures), and hazardous waste studies are excluded.

These guidelines are intended to maximize care of samples and cores, minimize storage costs, meet minimum technical requirements, and avoid unnecessarily long storage periods. Special circumstances or needs may require modification of the guidelines.

These guidelines apply to rock and soil core and samples taken without special sampling sleeves, tubes, or containers. When undisturbed samples taken in sleeves or containers or undisturbed hand-cut samples are specifically intended for laboratory examination or testing to supply design data, the samples should be taken, prepared, cared for, and handled in accordance with USBR 7100 [1] and USBR 7105 [2]. Undisturbed samples should be shipped immediately to avoid deterioration and changes in physical properties. Chapter 23 describes proper procedures for protecting and shipping samples. Appropriate logs, sample data sheets, or other pertinent information should be submitted with the samples.

Location of Storage Facilities

The preparation of an adequate log or description of the core or samples should be done before storing and subsequently disposing of samples. If the sampled material, such as slaking shales, breaks down upon drying and loses in-place characteristics during storage, it may be better to drill new holes than to store large quantities of core that will seldom be looked at or will lead to wrong interpretations if inspected or tested after years of storage.

The following are the general requirements for storage of core and samples:

1. Provide minimum storage conditions. (See the next section, "Conditions of Storage.")

2. Eliminate unsupervised storage. The storage facility should be at or near the site of a project or investigation.

3. Minimize transferring core and samples from one storage facility to another because work on a structure site or project progresses through successive stages of investigation and construction.

4. Provide accessibility so that minimum travel is required for examination of the core and samples. Frequent inspection may be necessary for design purposes and for prospective bidders on construction specifications.

5. Survey existing unused space before constructing or acquiring new space or use space initially provided for other purposes but adaptable to core and sample storage. For example, the lower

floors and galleries of many powerplants and some pumping plants or buildings erected for temporary use during construction have storage space.

Storage During Investigations

Core and samples should be kept at the nearest existing storage facility. A new storage facility should be established only if required by the location, the amount of core and number of samples anticipated, and the duration of investigation or access convenience. In some instances during periods when actual drilling and sampling operations are in progress, temporary storage at the site may be required to facilitate logging, selection of samples, or ready reference while geologic, soils, or other field studies are in progress. Temporary onsite storage should be limited to short periods of actual need and should prevent damage or deterioration of the core and samples.

When a field office maintaining core storage facilities is closed following preliminary investigations, core and samples no longer required should be discarded (see section entitled Retention of Rock Core and Samples), and the remaining core and samples should be transferred to an appropriate location. When exploration is resumed, the core should be examined to determine whether the samples are still useful or whether redrilling is necessary.

Storage During Construction

Core and samples should be stored locally during construction. The core and samples provide foundation and materials data to geologists, designers, and construction personnel during construction and claim processing.

At the close of construction, special samples should be selected for possible future testing (See the section on "Retention of Rock Core and Samples"). Sample selection is better when personnel familiar with the core, samples, and foundation conditions are available.

At the close of construction, arrangements should be made for continued storage of core and samples until applicable core and sample retention criteria have been met. The core and samples should remain at the local storage facility, if practical.

Storage During Operation and Maintenance

Any transfer of a structure from one organization to another should include arrangements for core and sample storage until the applicable standard core and sample retention periods are met.

Conditions of Storage

The minimum conditions for core and sample storage are:

1. Protection against wetting and drying but not against temperature changes. Cores that might be damaged by drying or freezing should be appropriately protected.

2. Protection against unauthorized access, damage, vandalism, and snake or rodent infestation.

3. Grouping core boxes and samples by drill hole number, investigation site, and project and stacking to permit reasonably easy access and examination. The core and samples should be accessible without excessive unstacking and moving of adjacent core boxes and samples. The

safe height of core box or sample stacks depends on the strength of the boxes and reasonable ease of handling. Core box racks should be provided only when repeated reexamination and study are anticipated and when the time saved by having the boxes separated in units of four or five boxes would be cost effective.

4. The office responsible for storage should maintain adequate records including copies of final logs, photographs, inventory, and location map of the core and samples stored at each core and sample storage location. The records should include the name of the feature or investigation site, drill hole number, number of boxes and samples, and whether the cores or samples have been tested.

Length of Storage

Proposed Structures or Projects

All core and samples collected during site investigations should be kept until the proposed structure, unit, or project is built and has performed satisfactorily for a period of time or is abandoned. The following are exceptions.

Appraisal.—Core and samples obtained in investigations for dams, canals, etc., should be disposed of at the end of appraisal stage investigations, such as the abandonment of a proposed structure site or completion of reconnaissance designs and estimates, except in special cases.

Feasibility.—Disturbed soil samples from borrow pit investigations obtained during feasibility stage investigations and not used for testing should be discarded at the end of the investigations unless interest in the project is sufficient to ensure continuing development. If, at the end of 1 year, further development work has not materialized, the samples should be discarded on the assumption that deterioration makes tests on the samples unreliable.

Undisturbed soil samples can be discarded after 1 year.

Design Investigations and Completed Structures or Projects

Requirements for length of storage for various types of samples and core are as follows:

Borrow Pit Investigations for Design.—Borrow pit soil samples collected for design investigations should be retained until construction is complete and all claims are settled except as the samples are consumed in routine testing.

Borrow Investigations for Construction Control.— Soil samples collected for construction control investigations and field laboratory testing should be discarded after completion of the field laboratory testing and all claims are settled.

Record Samples from Fill.—Samples collected for record tests and not consumed should be retained unless the number of samples is more than necessary to represent the materials used for construction.

Foundation Core and Samples.—Foundation core and samples obtained during design and construction investigations and not consumed in testing may be disposed of when the following requirements are met:

1. Canals and distribution systems including pipelines, minor or small engineering structures, and low dikes or embankments are trouble free for 1 year.

2. Large or major engineering structures such as dams, powerplants or pumping plants, main feeder canals, and main power conduits are trouble free for 5 years. The 5-year time period should begin after operating level reservoir filling for large dams. Representative core and samples from a few representative drill holes should be kept for the life of the structure.

3. Contractual negotiations and settlement of claims which might involve reinspection or testing of the core or samples are complete.

4. Initial safety of dams analysis, modification, and construction activities are complete.

Disposal of Core and Samples

Photographs and an adequate log or description of the core and samples should be available before disposal. When the minimum retention periods and other requirements outlined above have been met, concurrence of the design and operation and maintenance (O&M) offices should be obtained by the office responsible for the core or samples before any core or samples are discarded.

The responsible office should consider giving the State Geologist's Office, technical schools, colleges, museums, or similar groups the opportunity to select samples or retain the core.

The recommended disposal procedure is to place permanent markers in the core boxes, wrap the boxes in plastic, and bury the boxes in a marked location.

Retention of Rock Core and Samples

If selected cores or samples are retained as representative of the foundation and dam, appropriate assistance in selecting the samples is necessary. Assistance can usually be obtained from the design office.

To facilitate storage, handling, and inspection, standard core box sizes and labeling procedures should be used. Core boxes should withstand a reasonable amount of careful handling, stacking, and shipping when filled with rock core.

To help identify the boxes when stacked, information should be shown on one end and the top of each core box. Temporary labeling of core boxes used while drilling a hole should be replaced with permanent labeling of core boxes which will remain legible after handling, stacking, or storing for long periods. A map or an index file of drill hole core should be maintained which shows the location of drill hole core if the boxes are stored in a large or congested storage area.

Bibliography

Bureau of Reclamation, U.S. Department of the Interior, USBR-7100, "Obtaining Undisturbed Block Samples by the Hand and Chain Saw Methods," *Earth Manual*, Part II, 3rd edition, 1990.

Bureau of Reclamation, U.S. Department of the Interior, USBR-7105, "Performing Undisturbed Soil Sampling by Mechanical Drilling Methods)," *Earth Manual*, Part II, 3rd edition, 1990.

Chapter 25

GLOBAL POSITIONING SYSTEM

System Description

The Navigation Satellite Time and Ranging (NAVSTAR) Global Positioning System (GPS) is a space-based satellite radio navigation system developed by the U. S. Department of Defense (DoD). GPS receivers provide land, marine, and airborne users with continuous three-dimensional (3D) position, velocity, and time data (PVT). This information is available free of charge to an unlimited number of users. The system operates under all weather conditions, 24 hours a day, anywhere on Earth. The Union of Soviet Socialist Republics developed a similar system that is generally not used because it is a duplication of the NAVSTAR function.

GPS System Design

The GPS system consists of (1) the space segment, (2) the control segment, and (3) the user segment.

Space Segment.—The space segment consists of a nominal constellation of 24 operational satellites (including 3 spares) that have been placed in 6 orbital planes 10,900 miles (20,200 kilometers [km]) above the Earth's surface. The satellites are in circular orbits with a 12-hour orbital period and an inclination angle of 55 degrees. This orientation provides a minimum of five satellites in view at any time anywhere on Earth. Each satellite continuously broadcasts two low-power, spread-spectrum, RF Link signals (L1 and L2). The L1 signal is centered at 1575.42 megahertz (MHz), and the L2 signal is centered at 1227.6 MHz.

Control Segment.—The control segment consists of a Master Control Station (in Colorado Springs) and a number of monitor stations at various locations around the world. Each monitor station tracks all the GPS satellites in view and passes the signal measurement data back to the Master Control Station. Computations are performed at the Master Control Station to determine a precise satellite ephemeris and satellite clock errors. These data are then uplinked to the individual satellites and, subsequently, rebroadcast by the satellite as part of a navigation data message.

User Segment.—The user segment is all GPS receivers and their application support equipment such as antennas and processors. This equipment allows users to receive, decode, and process the information necessary to obtain accurate position, velocity, and timing measurements. These data are used by the receiver's support equipment for specific application requirements.

GPS Basic Operating Concepts

Satellite Signals.—The satellites transmit their signals using spread-spectrum techniques that employ two different spreading functions: a 1.023-MHZ coarse/acquisition (C/A) code on L1 only and a 10.23-MHz precision (P) code on both L1 and L2. The two spreading techniques provide two levels of GPS service: Precise Positioning Service (PPS) and Standard Positioning Service (SPS). SPS uses C/A code to derive position, while PPS uses the more precise P-code (Y-code).

C/A Code.—The C/A code is a 1,023 bit nonrepeating code sequence with a clock rate of 1.023 MHZ. The satellite repeats the code once every millisecond. Each satellite is assigned a unique C/A code that is chosen from a set of codes known as Gold Codes.

P(Y)-Code.—The P-code consists of a 2.36e+14 bit nonrepeating code sequence with a clock rate of 10.23 MHZ. The entire code would take 267 days before a repetition occurs; however, each satellite is assigned a unique 1-week segment of this code that restarts every Saturday/Sunday midnight.

The P-code has a number of advantages over C/A code. (1) The P-code rate is 10 times faster; therefore, the wavelength is 1/10th as long, giving the P-code a much higher resolution. (2) The higher rate spreads the signal over a wider frequency range (see figure 25-1). This frequency spreading makes the P-code much more difficult to jam. (3) By encrypting the P-code (creating Y-code), the receiver is not susceptible to spoofing (false GPS signals intended to deceive the receiver).

The drawback of P-code is that it is relatively difficult to acquire because of its length and high speed. For this reason, many PPS receivers first acquire C/A code, then switch over to the P(Y)-code. Y-code is an encrypted

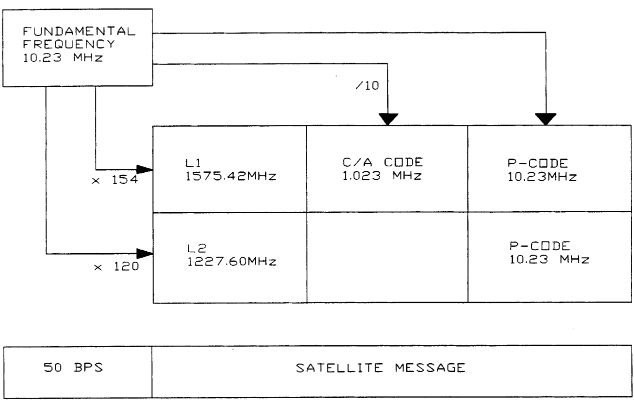


Figure 25-1.—Satellite signal structure.

version of P-code that is used for anti-spoofing (A-S). Because of the similarity of these two codes, they are referred to collectively as P(Y)-code.

Navigation (NAV) Message

Superimposed on both the P-code and the C/A code is a navigation (NAV) message containing satellite ephemeris data, atmospheric propagation correction data, satellite clock-bias information, and almanac information for all satellites in the constellation.

The navigation message consists of 25 1,500-bit frames and is broadcast at 50 bits per second. It takes 30 seconds to receive a data frame or 12.5 minutes to receive all 25 data frames. Each satellite repeats its own ephemeris data and clock bias every frame along with a portion of the almanac. The receiver will receive the critical acquisition information within 30 seconds, but a full almanac will require 12.5 minutes to download.

Signal Acquisition.—The GPS satellites use Bi-Phase Shift Keyed (BPSK) modulation to transmit the C/A and P(Y)-codes. The BPSK technique involves reversal of the carrier phase whenever the C/A or P(Y)-code transitions from 0 to 1 or from 1 to 0.

To the casual observer, the very long sequence of ones and zeros that make up the C/A and P-codes appears random and blends into the background noise. For this reason, the codes are known as pseudo-random noise (PRN).

In actuality, the C/A and P-codes generated are precisely predictable to the start time of the code sequence and can be duplicated by the GPS receiver. The amount the receiver must offset its code generator to match the

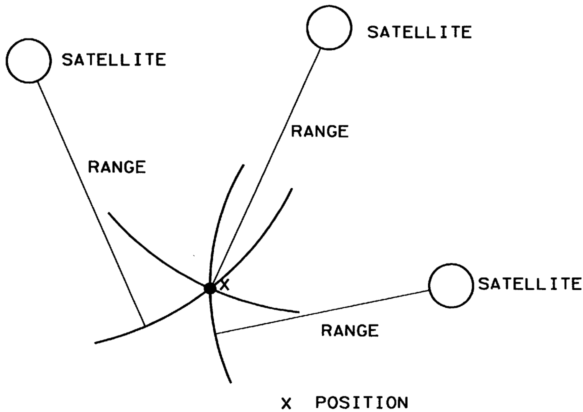


Figure 25-2.—Satellite ranging intersections.

incoming code from the satellite is directly proportional to the range between the GPS receiver antenna and the satellite.

By the time the spread-spectrum signal arrives at the GPS receiver, the signal power is well below the thermal noise level. To recover the signal, the receiver uses a correlation method to compare the incoming signals with its own generated C/A or P(Y) codes. The receiver shifts its generated code until the two codes are correlated.

Satellite Ranging.—The receiver continuously determines its geographic position by measuring the ranges (the distance between a satellite with known coordinates in space and the receiver's antenna) of several satellites and computes the geometric intersection of these ranges. To determine a range, the receiver measures the time required for the GPS signal to travel from the satellite to the receiver antenna. The resulting time shift is multiplied by the speed of light, arriving at the range measurement.

Because the resulting range measurement contains propagation delays caused by atmospheric effects as well as satellite and receiver clock errors, the measurement is referred to as a "pseudorange." A minimum of four pseudorange measurements are required by the receiver to mathematically determine time and the three components of position (latitude, longitude, and elevation). The solution of these equations may be visualized as the geometric intersections of ranges from known satellite locations (see figure 25-2). If one of the variables, such as elevation, is known, only three satellite pseudorange measurements are required for a PVT solution, and only three satellites would need to be tracked.

GPS Accuracy

GPS accuracy has a statistical distribution that depends on a number of important factors, including dilution of precision (DOP) satellite position and clock errors, atmospheric delay of the satellite signals, selective availability (SA), signal obstruction, and multipath errors.

Dilution of Precision

Geometric Dilution of Precision (GDOP) is a measure of the amount of error due to the geometry of the satellites. The errors in the range measurements that are used to solve for position may be magnified by poor geometry. The least error results when the lines of sight have the greatest angular separation between satellites.

There are four other DOP components that indicate how the geometry specifically affects errors in horizontal position (HDOP), vertical position (VDOP), position (PDOP), and time (TDOP). DOPs are computed based on the spatial relationships between the satellites and the user and vary constantly due to the motion of the satellites. Lower DOPs mean more accurate estimates. A PDOP less than six is necessary for accurate GPS positioning.

Satellite Position and Clock Errors

Each satellite follows a known orbit around the earth and contains a precise atomic clock. The monitor stations closely track each satellite to detect any errors in the orbit or the clock. Corrections for errors are sent to each satellite as ephemeris and almanac data. The ephemeris data contain specific position and clock correction data for each satellite while the almanac contains satellite position data for all satellites. The NAV set receives the ephemeris and almanac data from the satellites and uses these data to compensate for the position and clock errors when calculating the NAV data.

Atmospheric Delay of Satellite Signals

Electromagnetic signals (such as GPS signals) travel at the speed of light, which is always a constant in a vacuum but not in the atmosphere. There are two layers of the Earth's atmosphere that affect satellite signals, the ionosphere and the troposphere.

The ionosphere is a 90-mile (150-km) thick layer of the upper atmosphere in which ultraviolet radiation from the sun has ionized a fraction of the gas molecules, thereby releasing free electrons (ions). The shape of the ionosphere and its electron density varies with latitude, time of day, time of year, number of sunspots, solar flares, and other cosmic activity. The magnitude of the error caused by ionospheric effects can translate to a position error as large as 130 ft (40 m).

The troposphere is the dense, humid layer of atmosphere near the surface of the Earth. This layer refracts the satellite signals in proportion to the humidity and density of the air. The magnitude of the tropospheric error can be as small as 8 ft (2.3 m) at the zenith, or as large as 65 ft (20 m) at 10 degrees above the horizon.

There are two ways to compensate for the atmospheric delays: modeling and direct measurement. The ionospheric and tropospheric delays are inversely proportional to the square of the frequency. If a receiver can receive L1 and L2 frequencies, it can measure the difference between the two signals and calculate the exact atmospheric delay.

Most receivers currently use mathematical models to approximate the atmospheric delay. The tropospheric effects are fairly static and predictable, and a model has been developed that effectively removes 92-95 percent of the error. This reduces the total two-dimensional (2D) position error caused by the troposphere to around 8 ft (2.0 m).

The ionosphere is more difficult to model because of its unusual shape and the number of factors that affect it. A model has been developed that requires eight variable coefficients. Every day, the control segment calculates the coefficients for the ionospheric model and uplinks them to the satellites. The data are then rebroadcast in the NAV messages of the C/A and P(Y)-codes. This model can effectively remove 55 percent of the ionospheric delay, reducing the total 2D position error caused by the ionosphere to around 25 ft (7.5 m).

Selective Availability and Anti-Spoofing

GPS satellites provide two levels of navigation service: Standard Position Service and Precise Position Service.

SPS receivers use GPS information that is broadcast in the clear and is available to anyone in the world. This information may contain built-in errors that limit the accuracy of the receiver. This is a security technique called Selective Availability (SA). These SA errors are variable. In normal conditions, the U.S. Government guarantees that these errors do not exceed 100 m horizontal, 140 m vertical, and time accuracy of 340 nanoseconds, 95 percent of the time. There are times when an SPS receiver error exceeds these limits. SPS receivers are for civil use, and SA may or may not be on.

PPS receivers use the same information as SPS receivers. PPS receivers also read encoded information that contains the corrections to remove the intentional SA errors. Only users who have crypto keys to decode this information get the PPS accuracy. U.S. Government agencies and some Allies are authorized to have these crypto keys. A receiver with valid crypto keys loaded and verified is a PPS receiver.

To protect authorized users from hostile attempts to imitate the GPS signals, a security technique called anti-spoofing is also used. This is an encrypted signal from the satellites that can be read only by PPS receivers. SPS receivers are not capable of using anti-spoofing. A receiver with valid crypto keys loaded and verified reads this encrypted signal and operates in a spoofing environment.

GPS Signal Obstruction

Normal operation of the GPS receiver requires undisturbed reception of signals from as few as four satellites (in normal 3D mode) or three satellites in fixed-elevation mode. The signals propagating from the satellites cannot penetrate water, soil, walls, or other similar obstacles.

The antenna and the satellites are required to be in "line-of-sight" with each other. GPS cannot be used for underground positioning in tunnels, mines, or subsurface marine navigation. In surface navigation, the signal can be obscured by buildings, bridges, trees, and other matter that might block an antenna's line-of-sight from the GPS satellites. In airborne applications, the signal can be shaded by the aircraft's body during high banking angles. For moving users, signal shading or temporary outages are generally transitory and should not degrade the overall positioning solution.

The GPS receiver uses five channels to minimize the effects of obstruction. During normal operation, four channels track the four primary GPS satellites while the fifth channel tracks the remaining visible satellites and recovers the ephemeris data for each satellite. If one or more of the primary satellites are obscured, the receiver contains the data to support rapid acquisition of alternative satellites.

If only three satellites can be tracked, receivers may feature an "Elevation Hold" mode and continue to navigate as previously noted. The accuracy of the PVT in this mode will not be greatly affected unless the elevation changes. The receiver will use either the last computed elevation or an elevation provided by the user or host system. The accuracy of the "Elevation Hold" mode depends on the accuracy of the elevation provided to the receiver.

Multipath Interference

Multipath errors result from the combination of data from more than one propagation path. These errors distort the signal characteristics of the range measurements and result in pseudorange errors. These errors depend on the nature and location of a reflective surface peculiar to each user location. The effects are less detrimental for a moving user because small antenna movements can completely change the multipath characteristics.

The receiver is designed to reject multipath signals. First, the active patch antennas are designed to have a sharp gain roll-off near the horizon while providing nominal gain for the primary satellite signal. Since most multipath signals are reflected from ground structures, they tend to be attenuated. Second, the antenna is right-hand polarized. When a right-hand polarized GPS signal is reflected off a conductive surface, the signal becomes left-hand polarized and rejected by the antenna. The receiver also has hardware and software designed to reduce the effects of any multipath interference errors.

Differential GPS

Differential GPS (DGPS) may be used to eliminate the effects of SA and correct certain bias-like errors in the GPS signals. A Reference or Base Station Receiver measures ranges from all visible satellites to its surveyed position. Differences between the measured and estimated ranges are computed and saved for post processing or transmitted by radio or other signals to differential equipped receivers. Sub-meter to centimeter position accuracy is possible using local base station differential GPS.

Quality of Measurement

The accuracy with which positions are determined using GPS depends on two factors—satellite geometry and user measurement accuracy.

Satellite Geometry

Satellite geometry effects on specific position solutions can be expressed as dilution of precision. DOP is used to describe the contribution of satellite geometry to total positioning accuracy. An ideal satellite constellation would not dilute the positioning accuracy.

Ideal Satellite Constellation

The term, "current satellite constellation," refers to the satellites being used in the current position solution. When the current satellite constellation geometry is ideal, the DOP value is 1. This means the current satellite constellation contributes a "dilution factor" of 1 to positioning accuracy. Most of the time, the satellite geometry is not ideal, which means the current satellite constellation geometry "dilutes the precision" of the positioning accuracy. As the satellite constellation becomes less ideal, the adverse effect on positioning accuracy is greater. The dilution will change over time as satellites travel along their orbits, as different satellites are used to solve for GPS position, and as the user moves.

DOP Effects

The effect of satellite geometry is not equally distributed in all three positional vectors (x, y, and z in Earth-centered, Earth-fixed coordinates). Several DOP related values are commonly used. Table 25-1 shows expected values for HDOP, VDOP, and others. The range given for

Table 25-1.—Expected values of dilution of precision

DOP	Description	68-percent probability of obtaining	95-percent probability of obtaining
HDOP	Horizontal DOP is the satellite geometry factor in the 2D horizontal position solution.	1.50	1.78
VDOP	Vertical DOP is the satellite geometry factor in the vertical position (elevation) solution.	2.08	2.70
PDOP	Positional DOP is the satellite geometry factor in the 3D position solution.	2.56	3.23
$\mathrm{PDOP} = \sqrt{\left(\mathrm{HDOP}^2 + \mathrm{VDOP}^2\right)}$			
TDOP	Time DOP is the satellite geometry factor in the time solution.	N/A	N/A
HTDOP	Horizontal and time DOP is the satellite geometry factor in the 2D horizontal position and time solutions. HTDOP = SQRT ($\mathrm{HDOP}^2 + \mathrm{TDOP}^2$)	N/A	N/A
GDOP	Geometrical DOP is the satellite geometry factor in the 3D position and time solutions. GDOP = SQRT ($\mathrm{HDOP}^2 + \mathrm{VDOP}^2 + \mathrm{TDOP}^2$)	2.79	3.61

VDOP, HDOP, GDOP, and PDOP assumes a 5-degree antenna mask (signals 5 degrees or more above the horizon) and a 24 satellite constellation.

Unique user conditions, such as more than 5 degrees of antenna masking (e.g., in deep canyons), satellite outages, and poor constellation choice may result in a degraded DOP.

Quality Indicators

The indicators of solution quality are computed based on several things. These include uncertainties in receiver measurement processes, current satellite constellation geometry, and errors as reported by the satellites, such as the User Range Accuracy (URA) index. There are different measures of quality. A quality indicator must be selected based on the requirements of the application and how meaningful that measure will be. Some measures of quality indicators are shown below.

Circular Error Probable.—Circular Error Probable (CEP) is an estimate of horizontal accuracy that expresses the radius of a circle in the horizontal plane that will contain at least 50 percent of the GPS 2D positional solutions. CEP is generally expressed in meters; a smaller value indicates a higher quality 2D position solution. CEP is not desirable as a quality measure for navigation systems because a 50-percent probability is too small. This measure can be useful in some applications, for example, a bombing run where a better than 50-percent chance of hitting a target is required. CEP is not available from all GPS units. However, CEP may be estimated by the following equation.

$$CEP = 0.8323 * EHE$$

EHE is the expected horizontal error.

R95.—R95 is similar to CEP except that the circle contains at least 95 percent of the GPS 2D positional solutions. R95 is generally expressed in meters, and a smaller value indicates a higher quality 2D position. R95 is better than CEP as a measure for navigation systems because a 95-percent probability is more commonly associated with navigation requirements. R95 is not available from all GPS units. However, it may be estimated by the following equation.

$$R95 = 1.731 * EHE$$

Spherical Error Probable.—Spherical error probable (SEP) is an estimate of 3D accuracy that is the radius of a sphere that will contain at least 50 percent of the GPS 3D positional solutions. SEP is generally expressed in meters, and a smaller value indicates a higher quality 3D position solution. SEP is not desirable as a quality measure for navigation systems for the same reason that CEP is not; a 50-percent probability is too small. A spherical model of errors is not well suited to the GPS model because error probabilities in the horizontal plane are of a smaller scale than those in the vertical plane (compare HDOP to VDOP). This leads to more of a football shaped error distribution than a spherical distribution. SEP is not available from all GPS units.

Expected Position Error.—Expected position error (EPE) is a 1-sigma calculation of 3D position solution estimate of error. EPE is expressed in meters, and a smaller EPE indicates a higher quality position solution. Similarly computed values are often referred to, including:

EHE, Expected Horizontal Error
(1-sigma estimate of 2D error)

EVE, Expected Vertical Error
(1-sigma estimate of one-dimensional error)

$$\boldsymbol{EPE} = \sqrt{\left(\boldsymbol{EHE}^2 + \boldsymbol{EVE}^2\right)}$$

EPE is one of the best available indicators of quality for navigation systems.

EPE can also be output as Figure of Merit (FoM). (See below.)

Root Mean Square.—Root Mean Square (RMS) is a 1-sigma calculation of error in position. Horizontal RMS is equivalent to EHE, vertical RMS is equivalent to EVE, and 3D RMS is equivalent to EPE.

The above is true only if the mean values of the error components are zero or the biases of the errors can be independently determined and removed.

Figure of Merit.—Figure of Merit (FOM) is an expression of EPE, as shown in table 25-2.

From a historical perspective, FOM was created at the inception of GPS to provide users with a simple and quick method of indicating the solution accuracy; it is still widely used in many military applications today. It does not have the granularity to take advantage of the significant improvements in accuracies that may be achieved today. FOM is available from some GPS units on the display.

Table 25-2.—FOM related to EPE

FOM	EPE (meters)
1	< 25
2	< 50
3	< 75
4	< 100
5	< 200
6	< 500
7	< 1,000
8	< 5,000
9	≥ 5,000

Expected Time Error.—Expected Time Error (ETE) is a calculation of time error that is computed similarly to EPE. ETE is a 1-sigma estimate of time solution error. ETE is expressed in seconds, and a lower ETE indicates a higher quality time solution. Some GPS units calculate and output EPE in the form of Time Figure of Merit.

Time Figure of Merit.—Time Figure of Merit (TFOM) is an expression of ETE as shown in table 25-3.

Table 25-3.—TFOM related to ETE

TFOM	ETE
1	< 1 nanosec
2	< 10 nanosec
3	< 100 nanosec
4	< 1 microsec
5	< 10 microsec
6	< 100 microsec
7	< 1 millisec
8	< 10 millisec
9	≥ 10 millisec

User Measurement Accuracy

Several factors are involved in the receiver's ability to accurately measure the distance to the satellites. The most significant factors in user measurement accuracy are space and control segment errors, atmospherics, and user equipment (UE) errors. These errors are generally grouped together and referred to as User Equivalent Range Error (UERE).

User Equivalent Range Error

User Equivalent Range Error represents the combined effect of space and control segment errors (satellite vehicle position [ephemeris] and clock errors), atmospherics (ionospheric and tropospheric), and user equipment errors (receiver measurement uncertainties). UERE cannot be broadcast by the satellites and can be estimated only at the receiver. The point that sets UERE apart from other measures of error is that UERE does not take into account any satellite geometry effects, such as HDOP or VDOP. Table 25-4 shows a typical error budget for a P(Y)-code GPS receiver.

Table 25-4.—Typical GPS receiver error budget

Accuracy factor	Error
Space and control segment	4.0 m
Ionospheric	5.0 m
Tropospheric	2.0 m
Receiver noise	1.5 m
Multipath	1.2 m
Miscellaneous	0.5 m
$\mathbf{UERE} = \sqrt{\left(4^2 + 5^2 + 2^2 + 1.5^2 + 1.2^2 + 0.5^2\right)}$	7.0 m

Space and Control Segment Errors

Satellites provide an estimate of their own satellite position and clock errors. Each satellite transmits an indication of these errors in the form of User Range Accuracy (URA).

URA is a value transmitted by each GPS satellite that is a statistical indicator (1-sigma estimate) of ranging accuracies obtainable from that satellite. URA includes all errors that the space and control segments are responsible for—for example, satellite clock error and SA error. URA has very coarse granularity, however.

The URA value is received in the form of an index related to URA as shown in table 25-5.

The URA index broadcast by each satellite will change over time. In practice, the control segment will upload correction data to each satellite at least once every 24 hours. When a satellite first receives its upload, it should have a very low URA index. The amount of error will increase over time, because of things such as satellite clock drift, and the URA index for that satellite will grow. Some satellites' URA index will grow more rapidly than others.

In practice, authorized PPS users should expect to see URA index values in the range of 0 to 5. More commonly, values of 2-4 should be expected. The values depend on the length of time since the last control segment upload to that satellite.

Table 25-5.—URA index and values

URA index	URA (meters)
1	0.00 < URA <= 2.40
2	2.40 < URA <= 3.40
3	3.40 < URA <= 4.85
4	6.85 < URA <= 9.65
5	9.65 < URA <= 13.65
6	13.65 < URA <= 24.00
7	24.00 < URA <= 48.00
8	48.00 < URA <= 96.00
9	96.00 < URA <= 192.00
10	192.00 < URA <= 384.00
11	384.00 < URA <= 768.00
12	768.00 < URA <= 1,536.00
13	1,536 < URA <= 3,072.00
14	3,072.00 < URA <= 6,144
15	6,144 < URA (or no accuracy prediction available)

Wide Area GPS Enhancement

Wide Area GPS Enhancement (WAGE) is a feature available in Precise Positioning Service PLGR+96 receivers. The WAGE feature uses encrypted satellite data to reduce some space and control segment errors.

Each satellite broadcasts WAGE data that are valid for 6 hours after that satellite receives a data upload. These data may be used to correct satellite clock errors on other satellites that have not received an upload recently. These clock corrections are used to reduce the error caused by satellite clock biases when a period of time has passed since the last upload to those satellites. It takes approximately 12.5 minutes to download a complete WAGE data set. Only WAGE data from the most recently updated satellite are used in PLGR.

Atmospheric Errors

Atmospheric errors are those caused by the satellite RF signal passing through the earth's atmosphere. These errors include ionospheric and tropospheric delays.

Ionospheric delay affects the GPS signal as it passes through the earth's ionosphere. The ionospheric delay to GPS signals is very dynamic and depends on the time of day, the elevation angle of the satellites, and solar flare activity. Single frequency receivers use a modeled estimate of the range error induced by GPS navigation signals passing through the Earth's ionosphere. It is very difficult, however, to estimate the error in this model.

Dual frequency receivers measure this delay by tracking both the GPS L1 and L2 signals. The magnitude of the delay is frequency dependent, and the absolute delay on either frequency can be scaled from the differential delay between L1 and L2.

Tropospheric delay is the expected measure of range error induced by GPS navigation signals passing through the Earth's troposphere. This measure is based on satellite elevation. Unlike the ionospheric delay, the tropospheric delay is very predictable, and there is very little error in the compensation.

User Equipment Errors

User equipment errors are those uncertainties in the measurements that are inherent in the receiver's collection and computation of measurements. Several factors influence these uncertainties, including the GPS hardware, the design of the tracking loops, the code type being tracked (C/A or P(Y)), and the strength of the satellite signals. These uncertainties are taken into account in the Kalman filter processing done in a PLGR when computing EPE and ETE.

Multipath effects are caused by a signal arriving at the receiver site over two or more different paths. The difference between the path lengths can cause them to interfere with one another in the receiver. Buildings, parking lots, or other large objects may reflect a signal from a satellite, causing multipath effects. Signal averaging can be used to minimize the effects of multipath signals.

Error Source Summary

There are many factors that influence the accuracy of GPS measurements. Many of the factors do not contribute large errors but cumulatively are important.

In practice, the three major sources of error are (1) performing the survey when satellite geometry is poor, (2) using equipment not designed to provide the desired accuracy, and (3) mixing datums and coordinate systems.

Satellite Geometry

The most useful error measurement available is position dilution of precision (PDOP). PDOP is a number derived from the geometry of the visible satellites and changes as the satellites move in their orbits. The smaller the PDOP number, the better the satellite geometry. The PDOP value should be six, or preferably less, for accurate GPS work. Some GPS units display the PDOP or an accuracy estimate in real time. Software is available that calculates the PDOP for any planned GPS survey and should be used before conducting the survey to determine whether accurate GPS work is possible at the planned location and time. The software also can be used to determine satellite positions and whether terrain or other obstructions will affect the survey. Presurvey planning, especially relative to PDOP, can make the difference between a mediocre or failed GPS survey and a successful survey.

Equipment

Using inappropriate equipment is another major factor in GPS survey problems. The accuracy of the equipment must be known, and the equipment must be used properly. A small, inexpensive hand-held GPS unit can provide an accuracy of 10s of feet when selective availability is off and approximately 100 feet (30 m) when SA is on. A PLGR (military GPS) can provide similar accuracy when SA is on. A mapping grade GPS unit can provide 1- to 2-ft (0.5-m) accuracy if the data are post-processed and 3-ft (1-m) real-time accuracy if a satellite differential correction signal is available. Centimeter or survey-grade accuracy is possible with a local differential base station providing real-time or post-processing correction of rover data. Do not expect more accuracy than the equipment can deliver. Manufacturer estimates of accuracy are generally better than what is commonly encountered in the field.

The equipment must be well maintained. Many apparently satellite related problems are caused by equipment malfunctions, poorly charged batteries, or faulty cable connections.

Datums and Coordinate Systems

An apparent GPS problem is introduced by collecting or comparing position data based on different datums or coordinate systems. Many old topographic maps and surveys use the 1927 North American Datum (NAD27), and most new survey data are based on the 1983 North American Datum (NAD83). Also, some sites use a local reference system that is not related to a regional datum or coordinate system. Systematic errors are a good indication of a datum problem. Software is available that very accurately converts between datums and coordinate systems and is very useful for diagnosing and correcting problems. Chapter 6 of Volume 1 contains a discussion of datums, map projections, and coordinate systems.

APPENDIX A

ABBREVIATIONS AND ACRONYMS COMMONLY USED IN BUREAU OF RECLAMATION ENGINEERING GEOLOGY

AGC	automatic gain control
AN	ammonium nitrate
A-S	anti-spoofing
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing Materials
ATF	Bureau of Alcohol, Tobacco, and Firearms
AVIRIS	Airborne Visible and Infrared Imaging Spectrometer
BIPS	Borehole Image Processing System
bpf	blow per foot
bpm	blow per minute
BPSK	bi-phase shift keyed
BPT	Becker Penetration Test
C/A	coarse/acquisition
CDP	common depth point
CFR	Code of Federal Regulations
cm	centimeter
cm^2	centimeter squared
CME	Central Mine Equipment
cm/sec	centimeter per second
CPE	Circular Probable Error
CPT	Cone Penetration Test
CPTU	piezometric cones or piezocones
CRR	cyclic resistance ratio
DGPS	differential GPS

DoD	U.S. Department of Defense
DOP	dilution of precision
DOT	Department of Transportation
DS13	*U.S. Bureau of Reclamation Design Standard No. 13 for Embankment Dams*
EBW	exploding bridge wire
EHE	expected horizontal error
EM	electromagnetic
EPE	expected position error
ERi	drill rod energy ratio
ETE	expected time error
EVE	expected vertical error
FoM	Figure of Merit
F_r	friction ratio
f_s	sleeve resistance
ft	feet
ft/sec	feet per second
ft^3/min	cubic feet per minute
ft^3/sec	cubic feet per second
gal/min	gallons per minute
GDOP	geometric dilution of precision
g/cm^3	grams per cubic centimeter
GPR	ground penetrating radar
GPS	Global Positioning System
HDOP	horizontal dilution of precision
HSA	hollow-stem auger
ICE	International Construction Equipment
ID	inside diameter
IME	Institute of Makers of Explosives
in	inch

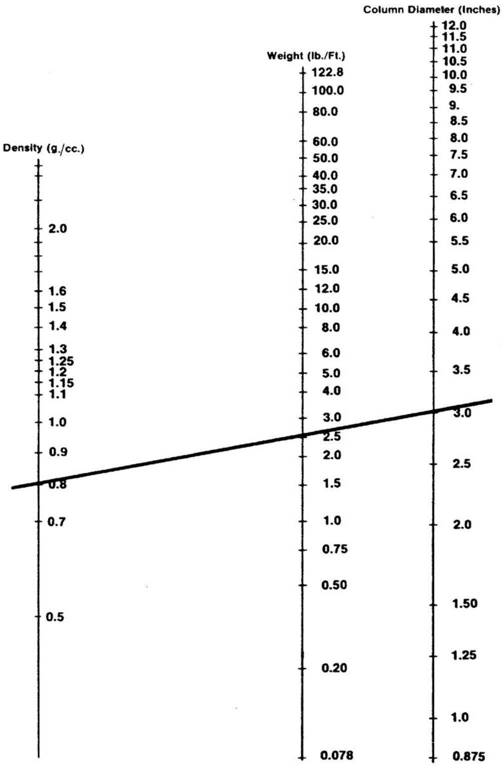
in/sec	inch per second
JPL	Jet Propulsion Laboratory
kg	kilogram
$kg/cm^2/m$	kilograms per square centimeter per meter
kg/L	kilogram per liter
kg/m^3	kilogram per cubic meter
kHz	kilohertz
km	kilometer
kPa	kilopascal
L	liter
lb/ft^3	pounds per cubic foot
lb/in^2	pounds per square inch
lb/gal	pounds per gallon
lb/yd^3	pounds per cubic yard
LEDC	low-energy detonating cord
L/min	liter per minute
L/min/m	liter per minute per meter
lu	Lugeon
LVL	low-velocity layer
m	meter
m^2	square meters
MHz	megahertz
mL	milliliter
mm	millimeter
mm/sec	millimeters per second
m/sec	meters per second
m^3/sec	cubic meters per second
ms	millisecond
MSHA	Mine Safety and Health Administration

MSS	multispectral scanner
NAD27	1927 North American Datum
NAD83	1983 North American Datum
NASA	National Aeronautics and Space Administration
NAV	navigation
NAVSTAR	Navigation Satellite Time and Ranging
NFPA	National Fire Protection Association
NG	nitroglycerin
NIST	National Institute of Standards and Technology
NTU	nephelometric turbidity unit
OD	outside diameter
O&M	operation and maintenance
OSHA	Occupational Safety and Health Administration
OSM	Office of Surface Mining
P	compressional wave
PDA	pile-driving analyzer
PDOP	position dilution of precision
PETN	pentaerythritoltetranitrate
PMT	photomultiplier tube
PPS	Precise Positioning Service
PRN	pseudo-random noise
psi	pounds per square inch
psi/ft	pounds per square inch per foot
PVC	polyvinyl chloride
PVT	position, velocity, and time data
q_c	tip resistance
RMS	root mean square

rps	revolutions per second
S	shear wave
SA	selective availability
SEP	spherical error probable
SLAR	side-looking airborne radar
SP	self-potential, spontanious potential
SPS	Standard Positioning Service
SPT	Standard Penetration Test
SSS	side scan sonar
SSSG	surface saturated dry specific gravity
TDOP	time dilution of precision
TFOM	Time Figure of Merit
TM	thematic mapper
TNT	trinitrotoluene
ton/ft^2	ton per square foot
UE	user equipment
UERE	User Equivalent Range Error
URA	User Range Accuracy
USGS	U.S. Geological Survey
VDOP	vertical position dilution of precision
WAGE	Wide Area GPS Enhancement
° C	degrees Centigrade
° F	degrees Fahrenheit
3D	three-dimensional
2D	two-dimensional

APPENDIX B

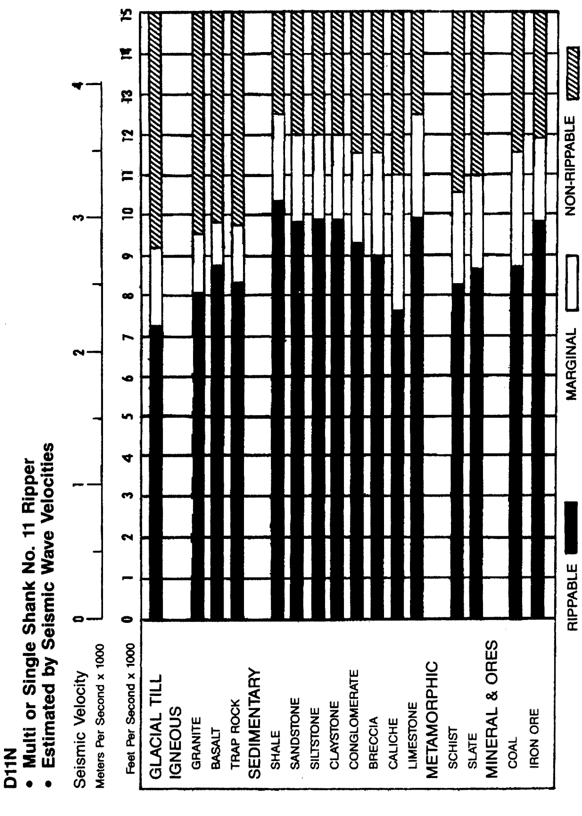
NOMOGRAPH RELATING THE DENSITY OF AN EXPLOSIVE IN G/CC, THE DIAMETER OF THE EXPLOSIVE IN INCHES, AND THE POUNDS OF EXPLOSIVE PER LINEAL FOOT



"Column Diameter" equals hole diameter for poured and pumped blasting products. (For cartridged products, estimate the explosives column average diameter, based upon cartridge diameter and amount of tamping or slump.) For example: a line drawn through the 0.8 gm/cc point (for ANFO) and the 3-inch column diameter point intersects the "weight" line at about 2.5 lb/ft.

APPENDIX C

CHART SHOWING RIPABILITY VERSUS SEISMIC VELOCITY FOR A D11N BULLDOZER



APPENDIX D

CHARTS SHOWING WEIGHT OF MATERIALS REQUIRED FOR TYPICAL LABORATORY TESTS

Weight of material required for aggregate soundness test

Designation No.	Nominal description of test	Maximum particle size	Minimum particle size	Practical field sample[1]	Recommended minimum test sample[1]	Required minimum test specimen	Comments
USBR 4088 ASTM C 88-83 (modified)	Soundness using $NaSO_4$	No. 30 (600 µm)	No. 50 (300 µm)	1.2 lb, 520 g*	0.3 lb, 130 g*	100 g*	*If present in the source material in amounts of 5 percent or more ** For each fraction, with a 1-inch (25-mm) spread in sieve size.
		No. 16 (1.18 mm)	No. 30 (600 µm)	1.2 lb, 520 g*	0.3 lb, 130 g*	100 g*	
		No. 8 (2.36 mm)	No. 16 (1.18 mm)	1.2 lb, 520 g*	0.3 lb, 130 g*	100 g*	
		No. 4 (4.75 mm)	No. 8 (2.36 mm)	1.2 lb, 520 g*	0.3 lb, 130 g*	100 g*	
		⅜ in (9.5 mm)	No. 4 (4.75 mm)	1.2 lb, 520 g*	Same as test specimen	100 g*	
		¾ in (19 mm)	⅜ in (9.5 mm)	5 lb, 2 kg*		500 g*	
		1½ in (37.5 mm)	¾ in (19 mm)	18 lb, 8 kg*		5 lb, 2 kg*	
		2½ in (63 mm)	1½ in (37.5 mm)	44 lb, 20 kg*		11 lb, 5 kg*	
		Larger sizes		**		15.4 lb, 7 kg*,**	

[1] To permit quartering, the field sample should be ≥four times the required weight of the test sample. The test sample is sized to yield the required number of prepared test specimens.

Weight of material required for aggregate durability test

Designation No.	Nominal description of test	Maximum particle size	Minimum particle size	Practical field sample[1]	Recommended minimum test sample[1]	Required minimum test specimen	Comments
USBR 4131 ASTM C 131-81 (modified)	Resistance to degradation of small-size coarse aggregate	No. 4 (4.75 mm)	No. 8 (2.36 mm)	50 lb, 23 kg	Same as test specimen	11.02 lb, 5,004 g	
		¼ in (6.3 mm)	No. 4 (4.75 mm)	22 lb, 10 kg		5.51 lb, 2,502 g	
		⅜ in (9.5 mm)	¼ in (6.3 mm)	22 lb, 10 kg		5.51 lb, 2,502 g	
		½ in (12.5 mm)	⅜ in (9.5 mm)	22 lb, 10 kg		5.51 lb, 2,502 g	
		¾ in (19 mm)	½ in (12.5 mm)	22 lb, 10 kg		5.51 lb, 2,502 g	
		1 in (25 mm)	¾ in (19 mm)	11 lb, 5 kg		2.76 lb, 1,253 g	
		1½ in (37.5 mm)	1 in (25 mm)	11 lb, 5 kg		2.76 lb, 1,253 g	
USBR 4535 ASTM C 535-81 (modified)	Resistance to degradation of large-size coarse aggregate	1 in (25 mm)	¾ in (19 mm)	50 lb, 23 kg	Same as test specimen	11.02 lb, 5,004 g	
		1½ in (37.5 mm)	1 in (25 mm)	50 lb, 23 kg		11.02 lb, 5,004 g	
		2 in (50 mm)	1½ in (37.5 mm)	50 lb, 23 kg		11.02 lb, 5,004 g	
		2½ in (63 mm)	2 in (50 mm)	22 lb, 10 kg		5.51 lb, 2,502 g	
		3 in (75 mm)	2½ in (63 mm)	22 lb, 10 kg		5.51 lb, 2,502 g	

[1] To permit quartering, the field sample should be ≥4 times the required weight of the test sample. The test sample is sized to yield the required number of prepared test specimens.

Weight of material required for individual soil tests - sheet 1 of 7
(footnotes are at the end of the table)

Designation No.	Nominal description of test	Maximum particle size[1,2,3]	Practical field sample[4] English	Practical field sample[4] Metric	Recommended minimum test sample[4] English	Recommended minimum test sample[4] Metric	Required minimum test specimen	Comments
USBR 5300 ASTM D 2216 USBR 5305 ASTM D 4959 USBR 5310 ASTM D 4944 USBR 5315 ASTM D 4643	Moisture content	No. 40 (425 µm)	0.1 lb	40 g	Same as test specimen		10 g	
		No. 4	2 lb	800 g			200 g	
		⅜ in	9 lb	4 kg			2.2 lb, 1 kg	
		¾ in	18 lb	8 kg			4.4 lb, 2 kg	
		1½ in	26 lb	12 kg			6.6 lb, 3 kg	
		3 in	26 lb	12 kg			6.6 lb, 3 kg*	*Or more to obtain a representative sample
USBR 5320 Method A ASTM D 854	Specific gravity minus No. 4	No. 4	5 lb	2.3 kg	1.1 lb	500 g	100 g oven dried	

Weight of material required for individual soil tests - sheet 2 of 7
(footnotes at end of table)

Designation No.	Nominal description of test	Maximum particle size[1,2,3]	Practical field sample[4]		Recommended minimum test sample[4]		Required minimum test specimen	Comments
			English	Metric	English	Metric		
USBR 5320 Method B or C ASTM C 127	Specific gravity and absorption plus No. 4	⅜ in	20 lb**	9.1 kg**	Same as test specimen	4.4 lb, 2 kg**	**Required weight of plus No. 4 particles, air dried	
		¾ in	30 lb**	14 kg**		6.6 lb, 3 kg**		
		1½ in	50 lb**	23 kg**		11 lb, 5 kg**		
		3 in	200 lb**	91 kg**		40 lb, 18 kg**		
USBR 5330 or USBR 5335 ASTM D 2487	Gradation: minus No. 4	No. 4	5 lb	2.3 kg	1.1 lb	500 g	100 g oven dried	*10 lb recommended for large mechanical sieve shakers **Air dried
USBR 5325 ASTM D 2487	Gradation: gravel sizes	⅜ in	5 lb*	2.3 kg*	If not air dried, need test specimen weight plus amount equivalent to weight of moisture content		200 g*, **	
		¾ in	15 lb	6.8 kg			2.4 lb, 1.1 kg*, **	
		1½ in	100 lb	45 kg			20 lb, 9.1 kg**	
		3 in	200 lb [4]n/a	91 kg			150 lb, 68 kg**	

Weight of material required for individual soil tests - sheet 3 of 7
(footnotes at end of table)

Designation No.	Nominal description of test	Maximum particle size[1,2,3]	Practical field sample[4]		Recommended minimum test sample[4]		Required minimum test specimen	Comments
			English	Metric	English	Metric		
USBR 5350 ASTM D 4318	LL - 1 point	No. 4	5 lb*	2 kg*	1.1 lb*, **	500 g*, **	100 g**	*Or more, as required, to have 120 g of air-dried soil (150 g if shrinkage limit is included) **Air dried
USBR 5355 ASTM (none)	LL - 3 point						100 g**	
USBR 5360 ASTM D 4318	Plastic limit						20 g**	Two 8-g wet-weight specimens
USBR 5365 ASTM D 427	Shrinkage limit						30 g**	

Weight of material required for individual soil tests - sheet 4 of 7
(footnotes at end of table)

Designation No.	Nominal description of test	Maximum particle size[1,2,3]	Practical field sample[4]		Recommended minimum test sample[4]		Required minimum test specimen	Comments
			English	Metric	English	Metric		
USBR 5400 ASTM (none) USBR 5405 ASTM D 4647 USBR 5410 ASTM D 4221	Dispersivity (crumb, pinhole, double hydrometer)	No. 4	5 lb	2.3 kg	1.1 lb	500 g	500 g	
USBR 5525 and/or USBR 5530 ASTM D 4564	Minimum and/or maximum index unit weight (wet or dry maximum)	No. 4, ⅜ or ¾ in	100 lb	45 kg	25 lb	12 kg	25 lb, 12 kg	
		1½ or 3 in	150 lb	68kg	75 lb	34 kg	75 lb, 34 kg	
	(Wet and dry maximum)	No. 4, ⅜ or ¾ in	100 lb	45 kg	50 lb	23 kg	50 lb, 23 kg	
		1½ or 3 in	200 lb	91 kg	150 lb	68 kg	150 lb, 68 kg	

Weight of material required for individual soil tests - sheet 5 of 7
(footnotes at end of table)

Designation No.	Nominal description of test	Maximum particle size[1,2,3]	Practical field sample[4]		Recommended minimum test sample[4]		Required minimum test specimen	Comments
			English	Metric	English	Metric		
USBR 5500 ASTM D 2937	Laboratory compaction: minus No. 4	No. 4	200 lb	91 kg	50 lb**	23 kg**	50 lb, 23 kg**	**Air dried
USBR 5515 ASTM (none)	Laboratory compaction: gravelly soils (one specimen)	3 in	350 lb*	159 kg*	350 lb*	159 kg*	225 lb, 102 kg	*Or more as needed to have at least 50 lb of minus No. 4 and at least 50 lb of plus No. 4
	3 specimens		900 lb	409 kg	900 lb	409 kg	900 lb, 409 kg	
USBR 5600	Permeability minus No. 4	No. 4	50 lb	23 kg	15 lb	7 kg	15 lb, 7 kg	
ASTM D 2434	Permeability gravelly soils	3 in	350 lb*	159 kg*	350 lb*	159 kg*	225 lb, 102 kg	

Weight of material required for individual soil tests - sheet 6 of 7
(footnotes at end of table)

Designation No.	Nominal description of test	Maximum particle size[1,2,3]	Practical field sample[4]		Recommended minimum test sample[4]		Required minimum test specimen	Comments
			English	Metric	English	Metric		
USBR 5700 ASTM D 2435	One-dimensional consolidation,							
USBR 5705 ASTM D 3877	Expansion or	No. 4	15 lb	7 kg	3 lb	1.4 kg	1.5 lb, 0.7 kg	
USBR 5715 ASTM D 3877	Uplift							

Weight of material required for individual soil tests - sheet 7 of 7

Designation No.	Nominal description of test	Maximum particle size[1,2,3]	Practical field sample[4]		Recommended minimum test sample[4]		Required minimum test specimen	Comments
			English	Metric	English	Metric		
USBR 5740 ASTM D 2850	Triaxial K_0 (usually performed in conjunction with other triaxial tests)	No. 4	20 lb	9.1 kg	4 lb	1.8 kg	2 lb, 0.9 kg	One 2-in-dia specimen
		¾ in	100 lb	45 kg	50 lb	23 kg	45 lb, 20 kg	One 6-in-dia specimen
		1½ in	200 lb	91 kg	170 lb	77 kg	150 lb, 68 kg	One 9-in-dia specimen
USBR 5745 ASTM D 2850	UU	No. 4	65 lb	30 kg	16 lb	7.3 kg	8 lb, 3.6 kg	Four 2-in-dia specimens
USBR 5740 ASTM D 2850	CU or	¾in	200 lb	91 kg	200 lb	91 kg	180 lb, 82 kg	Four 6-in-dia specimens
USBR 5755 ASTM (none)	CD	1½ in	700 lb	318 kg	700 lb	318 kg	600 lb, 273 kg	Four 9-in dia specimens

[1] Maximum particle size present in original sample.

[2] No. 4 fraction means either maximum particle in original sample was No. 4 or smaller or a representative portion of the minus No. 4 fraction of the original sample is required.

[3] Metric equivalents are: No. 4 sieve - 4.75 mm, ⅜ inch - 9.5 mm, ¾ inch - 19.0 mm, 1½ inch - 37.5 mm, 3 inch - 75 mm and 5 inch - 125 mm.

[4] To permit quartering, the field sample should be ≥4 times the required weight of the test sample. The test sample is sized to yield the required number of prepared test specimens.

APPENDIX D

Weight of material required for combinations of soil tests - sheet 1 of 1

Test combinations[1]	Maximum particle size[2,3]	Minimum sample required	
		English	Metric
Laboratory classification (gradation, Atterberg limits)	No. 4	50 lb	23 kg
	⅜ in		
	¾ in		
	1½ in	100 lb	46 kg
	3 in	200 lb	91 kg
Physical properties - Cohesive soil (gradation, Atterberg limits, specific gravity, laboratory compaction	No. 4	50 lb	23 kg
	⅜ in	100 lb[4]	46 kg[4]
	¾ in		
	1½ in		
	3 in	200 lb[4]	91kg[4]
Physical properties - Cohesionless soil (gradation, Atterberg limits, specific gravity, maximum and minimum index unit weight)	No. 4	100 lb	46 kg
	⅜ in		
	¾ in	200 lb	91 kg
	1½ in		
	3 in		
Soil-cement (for 3 cement contents)	1½ in	2,000 lb	908 kg

[1] Assumes field sample will be used for more than one test. If sample is divided to provide separate test samples, add up values for individual soil tests.

[2] Maximum particle size present in original sample.

[3] Metric equivalents are: No. 4 sieve - 4.75 mm, ⅜ inch - 9.5 mm, ¾ inch - 19.0 mm, 1½ inches - 37.5 mm, 3 inches - 75 mm, and 5 inch - 125 mm.

[4] Or more, as needed to have 50 lb of minus No. 4.

In place density test requirements - test apparatus and minimum excavation

Cohesive materials		
Maximum particle size (in)	Minimum required volume (ft^3)	Apparatus and template opening
¾	0.25	8-in sand cone
1½	0.5	12-in sand cone
3	1	20-in sand cone or 24-in-square frame
5	2	30-in square frame
8	8	4-ft-diameter ring
12	27	6-ft-diameter ring
18	90	9-ft-diameter ring
>18	Determine on a case-by-case basis	
Cohesionless materials		
¾	0.25	20-in sand cone
1½	0.5	24-in-square frame
3	1	33-in-square frame
5	2	40-in-diameter ring
8	8	62-in-diameter ring
>8	Determine on a case-by-case basis	

Weight of material required for slope protection tests

Reclamation designation No.	Minimum required sample		Comments
	English	Metric	
6025 Design Standard No. 13	600 lb	275 kg	The minimum dimension of individual rock fragments selected should be at least 0.5 feet (15 cm). If source material quality is variable, the sample should include rock fragments representing the quality range of the source material.

APPENDIX E

USEFUL CONVERSION FACTORS METRIC AND ENGLISH UNITS (INCH-POUND)

To convert units in column 1 to units in column 4, multiply column 1 by the factor in column 2.
To convert units in column 4 to units in column 1, multiply column 4 by the factor in column 3.

Column 1	Column 2	Column 3	Column 4
	Length		
inch (in)	2.540×10^{1}	3.937×10^{-2}	millimeter (mm)
hundredths of feet	3.048×10^{2}	3.281×10^{-3}	millimeter (mm)
foot (ft)	3.048×10^{-1}	3.281	meter (m)
mile (mi)	1.6093	6.2137×10^{-1}	kilometer (km)
	Area		
square inch (in^2)	6.4516×10^{-4}	1.550×10^{-3}	square meter (m^2)
square foot (ft^2)	9.2903×10^{-2}	1.0764×10^{1}	square meter (m^2)
acre	4.0469×10^{-1}	2.4711	hectare
square mile (mi^2)	0.386×10^{-2}	259.0	hectares
	Volume		
cubic inch (in^3)	1.6387×10^{-2}	6.102×10^{-2}	cubic centimeter (cm^2)
cubic feet (ft^3)	2.8317×10^{-2}	3.5315×10^{1}	cubic meter ($m^{3)}$
cubic yard (yd^3)	7.6455×10^{1}	1.3079	cubic meter (ms)
cubic feet (ft^3)	7.4805	1.3368×10^{-1}	gallon (gal)
gallon (gal)	3.7854	2.6417×10^{-1}	liter (L)
acre-feet (acre-ft)	1.2335×10^{3}	8.1071×10^{-4}	cubic meter (m^3)
	Flow		
gallon per minute (gal/min)	6.309×10^{-2}	1.5850×10^{1}	liter per second (L/s)
cubic foot per second (ft^3/s)	4.4883×10^{2}	2.228×10^{-3}	gallons per minute (gal/min)
	1.9835	5.0417×10^{-1}	acre-feet per day (acre-ft/d)
cubic foot per second (ft^3/s)	7.2398×10^{2}	1.3813×10^{-3}	acre-feet per year (acre-ft/yr)
	2.8317×10^{-2}	3.531×10^{1}	cubic meters per second (m^3/s)
	8.93×10^{5}	1.119×10^{-6}	cubic meters per year (m^3/yr)
	Permeability		
k, feet/year	9.651×10^{-7}	1.035×10^{6}	*k*, centimeter per second (cm/sec)
	Density		
pound-mass per cubic foot (lb/ft^3)	1.6018×10^{1}	6.2429×10^{-2}	kilogram per cubic meter (kg/m^3)
	Unit Weight		
pound force per cubic foot (lb/ft^3)	0.157	6.366	kilonewton per cubic meter (kN/m^3)
	Pressure		
pounds per square inch (psi)	7.03×10^{-2}	1.4223×10^{1}	kilogram per square centimeter (kg/cm^3)
	6.8948	0.145	kiloPascal (kPa)
	Force		
ton	8.89644	1.12405×10^{-1}	kilonewton (kN)
pound-force	4.4482×10^{-3}	224.8096	kilonewton (kN)
	Temperature		
	°C = 5/9 (°F - 32 °)		°F = (9/5 °C) + 32 °
	Grouting		
Metric bag cement per meter	3.0	0.33	U.S. bag cement per foot
Water:cement ratio by volume	0.7	1.4	water:cement ratio by weight
pounds per square inch per foot	0.2296	4.3554	kilogram per square centimeter per meter ($kg/cm^2/m$)
k, feet/year	0.1	10	Lugeon

Index

A

B

C

D

E

F

G

H

I

J

K

L

M

N

O

P

R

S

T

U

V

W

Y