EARTH MANUAL

PART 1

THIRD EDITION

U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

EARTH MANUAL

PART 1

Third Edition

Earth Sciences and Research Laboratory
Geotechnical Research
Technical Service Center
Denver, Colorado
1998

U.S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

MISSION STATEMENTS

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.

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PREFACE

Paul Knodel, Chief, Geotechnical Services Branch, Division of Research and Laboratory Services, directed the writing of the *Earth Manual*, Third Edition, Part 1. Richard Young, Technical Specialist, Geomechanics and Research Section, authored chapter 1. Amster Howard, Technical Specialist, Field Operations Team, wrote chapter 3. I drafted chapter 2, along with sections on drilling, excerpted from the *Small Dams Manual* originally written by Robert Hatcher, Division of Geology, and on remote sensing, excerpted from the *Engineering Geology Manual*. H. Kit Fuller of the U.S. Geological Survey, Reston, Virginia updated the information on geologic products available from their agency. The Geology Division (Sam Bartlett, Mark McKeown, and Sandy Kunzer), Construction Division (Pete Aberle), and Embankment Dams Branch (Tom McDaniels) performed Reclamation technical reviews. These technical reviews were essential in improving the quality of the product. Robert T. Sirokman was instrumental in obtaining new photographs of construction from Reclamation Field Offices. Technical editors for this manual were Richard Walters and Lelon A. Lewis. Monica A. Rodriguez and Sharon S. Leffel formatted it.

Part 2 contains standard procedures for investigating and testing soil materials for engineering design, construction, and operation and maintenance of water resources structures. Research and development continue to produce improvements in the knowledge of geotechnical materials and methods in which those materials are evaluated. Therefore, readers benefit from the latest technologies.

After the publication of Part 2 in 1990, efforts began immediately to update Part 1 of the manual. We realized that, with such a broad topic area, we could not afford to go into detail as greatly as we would wish on any one topic. To help alleviate this, a major change to the previous edition is the addition of referenced documents. We have strived to provide the reader the best citations to find further information. We have also referenced other Reclamation documents for more information in the subject areas.

The Bureau of Reclamation has a proud history in the construction of major water resources projects. We have recently broadened our mission to become a water resources management agency. This new direction signals a change from the era of project development. During restructuring in the mid-1990s, the agency witnessed the retirement of hundreds of engineers, technicians, and inspectors. We hope that this manual will stand to document the knowledge gained through the efforts of many of these individuals. The third edition is dedicated to all of those in Reclamation who have participated in these great earthwork endeavors. I know that I, for one, am eternally grateful to my companions in the Bureau of Reclamation, for I have learned so much about geotechnical engineering.

Jeffrey A. Farrar
Earth Sciences Laboratory
Bureau of Reclamation

PREFACE TO THE SECOND EDITION

The purposes of the Second Edition remain essentially the same as those which prompted the First Edition, as described in the latter's Preface. Constantly changing concepts of soil mechanics—as evidenced by new research techniques and ideas, innovations in construction methods and equipment, and computer-generated solutions to previously insurmountable soils-analyses problems—make mandatory this Second Edition. To improve its readability and provide for the new material, the Manual has increased in size as those familiar with the First Edition will recognize.

The contributors to the Manual have held important the need for uniformity in terminology, so that all personnel—field and office alike speak the same language. Much effort has been expended to achieve consistency of terms in the text and the 39 designations or procedures that comprise the appendix. This may be noted especially in the material on soil classification, and methods of logging and reporting; and types and methods of field explorations and investigations, and the tools and equipment required to obtain the desired information.

Although the Manual is primarily geared to the Reclamation organization, engineers and technicians of other governmental agencies, foreign governments, and private firms can, with modifications, utilize the information as a guide to their individual investigations, control of earth construction, and laboratory testing since emphasis is upon practical applications rather than upon complex theory. Users of the Manual should recognize that certain recommendations and values are the result of experience and cannot always be mathematically proved, nor should one attempt to. The Manual has been written as a guide and aid for the construction of a safe and stable structures with utmost concern for the safety of lives.

New material, not covered in the First Edition, includes material on: stabilized soils (soil-cement and asphaltic concrete), more complete information on field investigations and testing equipment in both chapter 2 and designation E-2, an expanded discussion on pipelines, and a newly developed designation, E-39, titled, "Investigations for Rock Sources for Riprap," which describes investigative and reporting procedures. In addition to the conversion factors in the First Edition, conversion curves are included to facilitate the increased utilization of metric units.

Major revisions center on designation E-16, which has been rewritten and retitled, "Measurement of Capillary Pressures in Soils," and designation E-17, "Triaxial Shear of Soils," which has been rewritten to conform to advanced developments in the procedure. Introduced in E-17 is the "Triaxial Shear Test with Zero Lateral Strain" referred to in modern soil mechanics texts as the K_0-test, which now can easily be performed through the use of the electronic computer.

Since the "Rapid Compaction Control" method, designation E-25, is being used extensively in 35 foreign countries as well as the United States, reorientation of the text material has been made for presenting the material in a manner more readily adaptable to both field and office use. More recently (1970), Australia has been granted permission by the Commissioner, Bureau of Reclamation, to incorporate the "Rapid Compaction Control" method in the Australian standards, "Testing Soils for Engineering Purposes." Designation E-12 has similarly been reoriented for ease in performing the relative density test in cohesionless soils.

Designations E-27 through E-35 covering "Instrument Installations" have been revised and updated to reflect changes in equipment and materials, techniques in installation procedures, and to clarify some of the methods of reading and reporting of data. To be commended are those dedicated field personnel who recognize inconsistencies or problems in the field related to "instruments" and who so often resolve the problems on-the-job. Reflected in these designations are many of their recommendations which have been offered unselfishly.

While environmental and ecological problems are major concerns of the Bureau of Reclamation, space and time limitations cause exclusion of discussion of views and policies regarding these highly important design considerations. It still remains the responsibility of each planner, investigator, designer, and constructor to consider these problems in his work.

There are occasional references to proprietary materials or products in this publication. These must not be construed as an endorsement since the Bureau cannot endorse proprietary products or processes of manufacturers or the services of commercial firms for advertising, publicity, sales, or other purposes.

Indicative of the monumental task involved in the preparation of this Second Edition is that some 90 persons—engineers, technicians, and those of other disciplines—from the Bureau of Reclamation at its Engineering and Research Center, Denver, Colo., constructively contributed to the content in some measure. The efforts of these people, some of whom are internationally acknowledged, are greatly appreciated.

Special recognition is given to H. J. Gibbs, Chief, Earth Sciences Branch, Division of General Research, and F. J. Davis, Supervisory Civil Engineer Hydraulic Structures Branch Division of Design and Construction, for authoring much of the technical material, for their technical advice, and for their overall guidance. In addition, recognition is made of engineers C. W. Jones, W. Ellis, R. R. Ledzian, G. DeGroot, and P. C. Knodel, and technician R. C. Hatcher, all of the Division of General Research, and engineer W. W. Daehn of the Division of Design and Construction for their major contributions. Because illustrations are invaluable to a publication, recognition must be made to R. E. Glasco and Mrs. H. Fowler for their patience, guidance, and help in obtaining illustrations of the highest quality.

This Second Edition was edited and coordinated, and supplemental technical information and illustrations provided by H. E. Kisselman, general engineer, Technical Services and Publications Branch.

PREFACE TO THE FIRST EDITION

The need for an up-to-date formal edition of the Bureau of Reclamation's *Earth Manual* became evidenced by the many printings of the June 1951 tentative edition. More than 6,000 copies of that edition were distributed, and a worldwide demand for the publication continues.[1]

The tentative edition was a combination and revision of three early manuals; namely, the *Earth Materials Laboratory Test Procedures*, the *Field Manual for Rolled Earth Dams*, and the *Earth Materials Investigation Manual*. In the present formal edition, the *Earth Manual* has been completely rewritten and contains much material not covered in the earlier edition.

The manual provides current technical information on the field and laboratory investigations and construction control of soils used as foundations and materials for dams, canals, and many other types of structures built for Reclamation projects in the United States of America. It contains both standardized procedures that have been found desirable for securing uniform results throughout the Bureau, and general guidelines intended to assist but not to substitute for engineering judgment.

Chapter I describes the Unified Soil Classification System developed jointly by the Bureau of Reclamation and the Corps of Engineers, Department of the Army, from the system proposed by Professor A. Casagrande of Harvard University, and discusses the various properties of soils relating to engineering uses. Investigations of soils are covered in chapter II which describes the various stages of investigation corresponding to the stages of development of the Bureau projects, and gives technical information necessary for planning and executing explorations and for presenting the results.

Chapter III presents information on the control of construction from the soils standpoint, for both foundation treatment and compaction control of fills. In addition to a general treatment of the subject applicable to all types of earthwork, separate sections are devoted to problems of rolled earth dams, canals, and miscellaneous construction features. For each of these, information on design features and usual specifications provisions are given to provide control personnel with a background to assist in implementing the recommended control techniques.

The appendix contains detailed procedures for sampling, classification, and field and laboratory testing of soils. Instructions for installing and obtaining information from instruments that measure pore-water pressures and displacements within and adjacent to earth embankments are also included. A tabulation of conversion factors commonly used in earth construction is included at the end of the appendix.

The appendix contains laboratory test procedures which are used in both the project laboratories and the central Bureau laboratory, and some of the more complex test procedures which are performed only in the central laboratory. The first group is presented for use by all Bureau laboratories, so that uniformity of test results will be obtained. The second group is presented for the purpose of recording the test procedures used in the central Bureau laboratory, so that the results of these tests, which are often contained in published reports, can be properly understood and interpreted.

Although the basic test procedures are standardized, it is emphasized that soil testing does not lend itself to strict standardization as normally applied to such construction materials as concrete, steel, and asphalt. Tests on soils must be carried out in such a way that natural conditions and operational conditions are fully accounted for during the test. In a simple test as for gradation determinations, the standard procedure may not take into account particle breakdown that may occur during excavation and compaction. Therefore, the test procedure

[1]Since the tentative edition, 28,000 copies of the first formal edition and its revised reprint (first printing) have been printed and distributed.

may require changes to provide "as constructed" gradation data. The compaction and other properties of some soils may be changed by drying and rewetting. In such cases, the field construction conditions must be taken into account during the test.

When variations from standard procedures are required, the variations must be described when reporting results, so that proper interpretations can be made. The exact procedures to be used for consolidation and shear tests normally require considerable judgment to assure that natural and operational prototype conditions are being duplicated as closely as possible. For this reason, the basic procedures presented for these two tests are merely guides and the details of actual test procedures used are covered in individual reports.

There may be instances in which the procedures and instructions in the *Earth Manual* are at variance with contract specifications requirements. In such cases, it should be understood that the specifications take precedence. It is also pointed out that each employee of the Bureau is accountable to his immediate supervisor and should request advice concerning any doubtful procedure.

Unless otherwise noted, the terms and symbols used in the *Earth Manual* conform to those given in ASTM Designation D-653-58T, "Tentative Definitions of Terms and Symbols Relating to Soil Mechanics," ASTM Standards 1958, part 4, page 121 1. These terms are also given in Paper 1826, "Glossary of Terms and Definitions in Soil Mechanics," Journal of the Soil Mechanics and Foundations Divisions, ASCE, October 1958.

Some of the exploratory and test procedures in chapter 11 and in certain of the designations which are applicable to the investigations, design, and construction of small earth structures have also been included in the publication "Design of Small Dams," recently published (1960).

The *Earth Manual* represents the joint efforts of many engineers and technicians over a period of many years covering the preparation of the tentative edition, its three predecessor publications, and this first formal edition. The present edition (including the revised reprint) was prepared by engineers in the Earth Dams Section, Dams Branch, Division of Design, and the Soils Engineering Branch, Division of Research, in Denver, Colo., under the direction of Chief Engineer B. P. Bellport and his predecessors.

Major writing was performed by F. C. Walker, J. W. Hilf, and W. W. Daehn of the Earth Dams Section; and W. G. Holtz and A. A. Wagner of the Soils Engineering Branch. Contributing writers were F. J. Davis, E. E. Esmiol, H. J. Gibbs, and C. W. Jones. The draft was reviewed and helpful suggestions submitted by geologists and engineers in various other branches of the organization. Principal among these were W. H. Irwin, A. F. Johnson, H. G. Curtis (deceased), and C. E. McClaren. The contributions of these individuals, as well as those of the many other engineers and technicians who contributed in various ways to the publication, are gratefully acknowledged. The draft was edited and coordinated by J. W. Hilf, and final review and preparation for publication was performed by E. H. Larson, Head, Manuals and Technical Records Section, Technical and Foreign Services Branch.

There are many references throughout the text to the Assistant Commissioner and Chief Engineer and the Office of Assistant Commissioner and Chief Engineer. For expediency these references were not changed in the reprint to agree with recent organization changes (February 1963), and they should be interpreted as meaning Chief Engineer and Office of Chief Engineer, respectively. References to the Commissioner's Office, Denver, should also be interpreted as meaning the Office of Chief Engineer.

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Chapter 1

PROPERTIES OF SOILS

A. Identification and Classification

1-1. General.—Most soils are a heterogeneous accumulation of mineral grains that are not cemented together. However, the term "soil" or "earth" as used in engineering includes virtually every type of uncemented or partially cemented inorganic and organic material in the ground. Only hard rock, which remains firm after exposure, is wholly excluded. In the design and construction of foundations and earthwork, the physical and engineering properties of soils, such as their density, permeability, shear strength, compressibility, and interaction with water, are of primary importance.

A standard method is used for identifying and classifying soils into categories or groups that have distinct engineering characteristics. This enables a common understanding of soil behavior just by knowing the classification. Written exploration records (logs) as shown on figure 1-1 contain soil classifications and descriptions; they can be used to:

- Make preliminary estimates.
- Determine the extent of additional field investigations needed for final design.
- Plan an economical testing program.
- Extend test results to additional explorations.

For final design of important structures, visual soil classification must be supplemented by laboratory tests to determine soil engineering properties such as permeability, shear strength, and compressibility under expected field conditions. Knowledge of soil classification, including typical engineering properties of various soil groups, is especially valuable when prospecting for earth materials or investigating foundations for structures.

1-2. Unified Soil Classification System.—In 1952, the Bureau of Reclamation (Reclamation) and the U.S. Army Corps of Engineers, with Professor Arthur Casagrande of Harvard University, reached agreement on modifications to his "Airfield Classification System" and named it: *Unified Soil Classification System* (USCS) [1].[1] During World War II, Professor Casagrande developed the "Airfield Classification System" for categorizing soils based on engineering properties related to airfield construction [2]. After the war, Reclamation began using the system, which led to the modifications agreed upon in 1952.

From 1952 to 1986, the system required minor changes—particularly in presenting information in written logs. Most changes were documented in Reclamation's *Earth Manual* (1st and 2d eds.) and in written directives, while some changes were taught in Reclamation training courses without formal documentation.

In 1984, American Society for Testing and Materials (ASTM), with encouragement and participation of several U.S. Government agencies (including Reclamation), reevaluated the Unified Soil Classification System. New data and philosophies [3, 4] resulted in revised ASTM standards, which modified the 1952 system. Reclamation adopted the new standards with only slight modifications.

While the soil classification system has evolved over many years, soil classifications and descriptions recorded at any point in the past must remain valid, because they were based on information available at that time. Therefore, earlier logs and past descriptions must not be changed to conform to current standards.

1-3. Current Classification System.—The Unified Soil Classification System has been through several transitions since it was developed. The current version used by Reclamation went into effect January 1, 1986. The classification system is described in Reclamation's testing procedures USBR 5000 and 5005 [5].[2] These procedures are similar to ASTM D 2487 and D 2488 [6]; the ASTM standards may be used in place of Reclamation procedures. The basics of the system may be mastered by studying and completing exercises in the Geotechnical Branch's Training Manuals Nos. 4 and 5 [7, 8]. They are well-prepared, self-instructing manuals that progress from simple to more difficult topics. Reclamation has standard formats for written logs of test pits and auger holes. Uniformity of data presentation results in consistency and completeness, which enables efficient review of large numbers of logs. Format

[1] Numbers in brackets refer to bibliography.

[2] See bibliography [5] regarding numbered USBR procedures.

7-1336-A (1-86)
Bureau of Reclamation

LOG OF TEST PIT OR AUGER HOLE

HOLE NO. EXAMPLE

FEATURE
AREA DESIGNATION Spillway Foundation
COORDINATES N 1111 E 2222
APPROXIMATE DIMENSIONS 8 by 12 ft
DEPTH WATER ENCOUNTERED 1/ none DATE

PROJECT
GROUND ELEVATION 1234.7 ft
METHOD OF EXPLORATION backhoe
LOGGED BY
DATE(S) LOGGED 1-25 to 1-26, 1986

CLASSIFICATION GROUP SYMBOL (describe sample taken)	CLASSIFICATION AND DESCRIPTION OF MATERIAL SEE USBR 5000, 5005	% PLUS 3 in (BY VOLUME) 3-5 in	5-12 in	PLUS 12 in
CL in-place unit weight three sack samples 4.3 ft	0.0 to 4.3 ft LEAN CLAY: About 90% fines with medium plasticity, high dry strength, medium toughness; about 10% predominantly fine sand; maximum size, coarse sand; no reaction with HCl. IN-PLACE CONDITION: Firm, homogeneous, moist, reddish-brown In-place dry unit weight and moisture from test at 3.0 to 3.7 ft: 112.0 lbf/ft^3, 11.7%. Three 50-lbm sack samples taken for testing from 18-inch-wide sampling trench for entire depth interval on east side of trench. Samples were mixed and quartered.			
SC 8.2 ft	4.3 to 8.2 ft CLAYEY SAND WITH COBBLES: About 55% coarse to fine, hard, subrounded sand; about 35% fines with medium plasticity, medium toughness; about 10% coarse to fine, hard, subrounded gravel; weak reaction with HCl. TOTAL SAMPLE (BY VOLUME): About 5% 3- to 5-inch hard, subrounded cobbles; remainder minus 3 inch; maximum dimension, 100 mm. IN-PLACE CONDITION: homogeneous, moist to 7.4 ft, wet 7.4 to 8.2 ft, gray; some mica present.	5		

REMARKS.

	Date	Depth to water
	01-31-86 8:30	7.4 ft
	01-31-86 1:30	5.2 ft
	01-31-86 5:00	4.7 ft*

* Test pit backfilled before stable water level could be determined

Figure 1-1.—Typical soil exploration log.

guidelines are given in Geotechnical Branch Training Manual No. 6 [9]. They should be adhered to as closely as possible; however, every situation cannot be covered. Therefore, when preparing a log, the object is to present as complete and as clear a description as possible.

The *Soil Classification Handbook* [9] also contains charts, forms, and illustrations useful in soil classification.

1-4. Basis of Unified Soil Classification System.—The USCS is based on engineering properties of a soil; it is most appropriate for earthwork construction. The classification and description requirements are easily associated with actual soils, and the system is flexible enough to be adaptable for both field and laboratory use.

The USCS is a method for describing and categorizing a soil within a group that has distinct engineering properties. Upon recognizing a USCS symbol of a classification group or understanding the description, one can immediately deduce the approximate permeability, shear strength, and volume change potential of a soil and how it may be affected by water, frost, and other physical conditions. Also, using soil classification symbols can assist a contractor in estimating excavation and compaction characteristics, potential dewatering situations, and workability.

The USCS is based on: (a) the distribution (gradation) of various particle sizes, and (b) the plasticity characteristics of very fine particles. Particle-size distribution is determined from a gradation analysis test. In this test, soil is dried and then shaken through a series of sieves having progressively smaller openings. The soil mass retained on each sieve is measured, and the percentage that passes the different openings can be calculated. The gradation analysis test is discussed in part B of this chapter. Plasticity characteristics of the fines (particles that pass a U.S.A. Standard 75-µm [No. 200] sieve and smaller) are determined by performing the Atterberg limits tests. These tests are performed to determine the moisture content of a soil when in a liquid, plastic, semisolid, or solid state. The tests are discussed in part B of this chapter.

Gradation analysis data are used to determine percentages of gravel, sand, and fines in a soil; Atterberg limits are used to determine whether the fines are silt or clay. The four basic soil types are: gravel, sand, silt, or clay. Because soil can contain all of these components, the classification system is used to describe the major components; for example, clayey sand with gravel, sandy silt, or silty clay with sand. The percentages and behavior of each component can be determined from laboratory tests or they can be estimated by visual and manual tests in the field. However, in the USCS system, the method of classification must be stated, whether it is based on visual observations or on laboratory tests.

Unique factors must be realized when comparing the USCS system with other classification methods. First, the USCS is based only on the particles passing a 75-mm (3-in) sieve opening. While particles larger than 75 mm are not used to classify soil, they are of great importance in engineering use of a soil in earthwork. Therefore, cobbles and boulders are included in USCS names, and they are fully described in the narrative portion of field logs.

Second, silt and clay are differentiated by their physical behavior and not by their particle size. Other classification systems define silts and clays as having specific particle sizes with silts being the larger particle size. As a system based on engineering properties, the USCS is concerned only with plasticity characteristics of the fines and not the particle sizes.

1-5. Gravel, Sand, Silt, and Clay.—The relative amounts of gravel, sand, and fines; the plasticity characteristics of the fines (silt, clay, or both); and the presence of cobbles and boulders all affect the soil's use as a construction material. These characteristics also affect selection of laboratory and field tests to be performed on the soil.

Gravel particles are those passing a 4.75-mm (3-in) sieve but retained on a 4.75-mm (No. 4) sieve. Sand particles pass a 4.75-mm sieve and are retained on a 75-µm (No. 200) sieve. Fines are soil particles that pass a 75-µm sieve; they are further characterized as silt or clay, based on their plasticity.

*a. Gravel and Sand.—*Gravel and sand have essentially the same basic engineering properties, differing mainly in degree. The division of gravel and sand sizes by the 4.75-mm sieve is arbitrary and does not correspond to an abrupt change in properties. When devoid of fines, coarse-grained soils are pervious, easy to compact, and little affected by moisture or frost action. Although particle shape, angularity, gradation, and size affect the engineering properties of coarse-grained soils, gravels are generally more pervious, more stable, and less affected by water or frost than are sands containing the same percentage of fines.

As a sand becomes finer and more uniform, it approaches the characteristics of silt, with corresponding decreased permeability and reduced stability in the presence of water. Visually, very fine sands are difficult to distinguish from silts. However, dry sand exhibits no cohesion (does not hold together) and feels gritty in contrast to the very slight cohesion and smooth feel of dried silt.

b. Silt and Clay (Fines).—In soils, small amounts of fines may have important effects on engineering properties. As little as 10 percent of particles smaller than the 75-µm sieve size in sand and gravel may make the soil virtually impervious, especially when the coarse grains are well graded. Also, in well-graded sands and gravels, serious frost heaving may be caused by less than 10-percent fines. The utility of coarse-grained materials for surfacing roads, however, can be improved by adding a small amount of clay to act as a binder for the sand and gravel particles.

Soils containing large quantities of silt and clay are the most troublesome in engineering. These materials exhibit marked changes in physical properties with change of moisture content. For example, a hard, dry clay may be suitable as a foundation for heavy loads as long as it remains dry, but may turn into a quagmire when wet. Many fine-grained soils shrink on drying and expand on wetting; this may adversely affect structures founded upon them or constructed of them. Even when moisture content does not change, the properties of fine-grained soils may vary considerably between their natural condition in the ground and their state after being disturbed. Fine-grained soils, having been subjected to loading in geologic time, frequently exhibit a structure that gives the material unique properties in the undisturbed state. When soil is excavated for use as a construction material or when a natural deposit is disturbed, for example, by driving piles, soil structure is destroyed, and soil properties are radically changed.

Silts differ from clays in many respects, but because of similarity in appearance, often they are mistaken for each other. Dry, powdered silt and clay are indistinguishable from each other visually but are easily identified by their characteristics when wet or moist. Recognition of fines as having either silt or clay behavior is an essential part of the Unified Soil Classification System.

Silts are slightly plastic or nonplastic fines. They are inherently unstable when wet and have a tendency to become "quick" when saturated and unconfined, assuming the character of a viscous fluid, and commonly flow. Silts are fairly impervious, difficult to compact, and highly susceptible to frost heaving. Silt masses undergo change of volume with change of shape (the property of dilatancy), in contrast to clays, which retain their volume with change of shape (the property of plasticity). Silts vary in size and shape of particles; this is reflected mainly in compressibility. For similar conditions of previous geologic loading, the higher the liquid limit of a silt, the more compressible it becomes.

Clays are plastic fines. They have low resistance to deformation when wet but dry to hard, cohesive masses. Clays are virtually impervious, difficult to compact when wet, and impossible to drain by ordinary means. Large expansion and contraction with changes in water content are characteristic of clays. The small size, flat shape, and mineral composition of clay particles combine to produce a material both compressible and plastic. The higher the liquid limit of a clay, the more compressible it will be when compacted. Hence, in the Unified Soil Classification System, the liquid limit is used to distinguish between clays of high compressibility and those of low compressibility. Differences in plasticity of clays are reflected by the plasticity index. At the same liquid limit, the higher the plasticity index, the more cohesive the clay.

c. Organic Matter.—The effect of organic matter upon engineering properties is recognized in the USCS, and organic soils are part of the classification system. Organic matter in the form of fully or partly decomposed vegetation can be found in many soils. Varying amounts of finely divided vegetable matter are found in sediments and often affect sediment properties sufficiently to influence their classification. Thus, the USCS recognizes organic silts having no or low plasticity and organic clays having medium to high plasticity. Even small amounts of organic matter in colloidal form in a clay may result in an appreciable increase in liquid limit (and plastic limit) of the material without increasing its plasticity index.

Generally, organic soils are dark gray or black in color and usually have a characteristic decaying odor. Organic clays feel spongy in the plastic range when compared to inorganic clays. The tendency for soils high in organic content to form voids through decay or to change the physical characteristics of a soil mass through chemical alteration makes them undesirable for engineering use. Soils containing even moderate amounts of organic matter are significantly more compressible and less stable than inorganic soils; hence, they are less desirable for engineering use.

1-6. Cobbles and Boulders.—While particles of rock retained on a 75-mm (3-in) sieve are not considered when classifying a soil according to the USCS, they can have a significant effect on the construction use of a soil and may need to be removed. The presence of cobbles and boulders must *always* be noted in the group name of a soil and in any description of the soil. Cobbles pass a 300-mm

(12-in) square opening but are retained on a 75-mm sieve. Boulders are too large to pass a 300-mm square opening. Note that for engineering use, the definitions of cobbles and boulders refer only to size. Some other definitions consider particle angularity.

For soils that may be used in compacted embankments or compacted backfill, cobbles must be identified as being larger or smaller than 125 mm (5 in) in diameter, as a typical compacted layer of cohesive soil is restricted to about 150 mm (6 in) in thickness. The maximum particle size allowed in the compacted layer is 125 mm. Thus, particles larger than 125 mm must be removed or separated. Logs and soil descriptions must state the percentage of cobbles smaller than and the percentage larger than 125 mm. This topic is discussed in chapter 2, section 17: Logging of Exploratory Holes.

1-7. Shale, Crushed Rock, and Slag.—The USCS can also be used for describing materials such as shale, siltstone, claystone, mudstone, sandstone, crushed rock, slag, cinders, and shells. These materials are not considered natural soils but they can be used as construction materials.

In some cases, materials variously described as shale, claystone, mudstone, or siltstone can be excavated with construction equipment and then broken down using disking or other methods into a material that can be used to construct embankments or other earthworks. In the laboratory, these materials can be processed into a "soil" by grinding or slaking. They can then be classified according to the USCS to evaluate their properties as a construction material. However, these materials should *not* be primarily classified or described as soil because that is not their natural state.

To avoid misinformation, any log, report of laboratory test results, or description(s) must identify the original material as shale, claystone, etc. However, in the description of material properties, a USCS soil classification symbol and name may be used. The classification is secondary, and the symbol and name are to be enclosed in quotation marks to distinguish them from a typical soil classification as shown in the following example:

> SHALE CHUNKS: Retrieved as 50- to 100-mm pieces of shale from power auger hole; dry, brown; no reaction with HCl. After a sample was laboratory processed by slaking in water for 24 hours, the sample was classified as "SANDY LEAN CLAY (CL)" — 61-percent clayey fines, LL = 37, PI = 16; 33-percent fine to medium sand; 6-percent gravel-size pieces of shale.

Other examples are in USBR 5000 and 5005 [5] and training manuals Nos. 4, 5, and 6 [7, 8, 9].

The same format applies to other materials not considered to be natural soils, but which are used in construction such as sandstone, crushed rock, slag, cinders, and shells.

For some materials, the laboratory processing required to test the material as a soil may be significantly different than the processing at the project site using construction equipment. The differences must be considered when evaluating the material for construction. Test sections are often used as a final evaluation of whether to accept or reject these materials.

1-8. Soil Composition.—In addition to soil classification, knowledge of the mineralogy and origin of soils can aid in evaluating the behavior of a soil. While silt particles and sand-size particles are generally equidimensional, clay particles are very small and are flaky, or platelike, as shown on figure 1-2. As clay components of soils become more predominant, the clay mineral characteristics assume great importance. Soil properties such as consistency, shear strength, and moisture content are directly influenced by the mineral constituents of the clay. When soils are fairly moist, clay particles are surrounded by water films. As dehydration takes place, the films become thinner and thinner until adjacent particles are held together by strong cohesive (capillary) forces. As soils are wetted, the films become weaker. Film strength also is related to the fineness and specific surface of the material and to compactness (density) which influences size of void spaces.

The mineralogic clays in soils affect their physical behavior. Two groups of clay minerals are of particular interest—the kaolin group and the montmorillonite group. The kaolin minerals have fixed crystal lattices or layered structure and exhibit only a small degree of hydration and absorptive properties. In contrast, montmorillonite minerals have expanding lattices and exhibit a higher order of hydration and cation absorption.

Montmorillonite soils, with their expanding lattice structure and resulting capacity for wide ranges of water contents, can be particularly troublesome. Settlement from shrinkage, heave from swelling, and loss of strength and stability

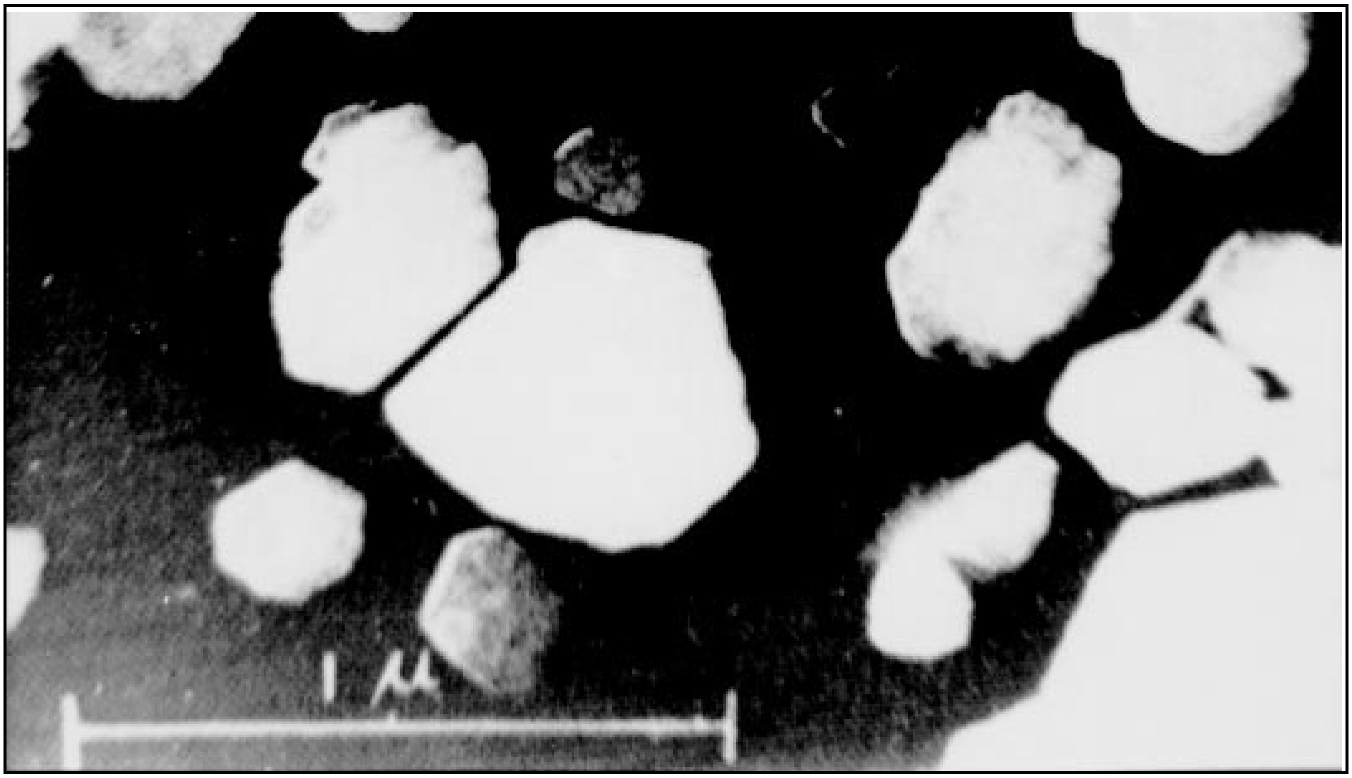

Figure 1-2.—Clay kaolinite particles (1 μ = 0.001 mm).

caused by shrinkage or swelling can create major structural problems. These problems are greatly magnified in the case of hydraulic structures.

The clay mineral halloysite, illustrated on figure 1-3, can cause construction problems. Typically, it is a hollow, rod-shaped particle and decreases soil density. The soil changes engineering behavior as its structure changes because of particle breakdown during drying or construction and the irreversible process of water being removed from inside the tubal particles.

The presence of mica in soils causes a highly compressible material. The thin, flaky particles act like springs that separate other soil particles, thus creating low densities and also deforming under load. This occurrence is especially important in sands containing mica.

Many low-density deposits are found in the arid and semiarid parts of the United States. These soils are unsaturated deposits of loose, wind-deposited loess and loess-like soils or colluvial and alluvial soils deposited by flash runoffs, often in the form of mudslides. In none of these situations have the soils been completely wetted or worked to allow breakdown and consolidation of the loose structure. Generally, these soils have high dry strengths created by a well dispersed clay binder. Major wind-blown loess deposits are found in the Plains States of Kansas and Nebraska and can be found in other areas. Loessial soils form high vertical faces that are stable as long as the water content is low. However, upon wetting, strength is largely lost and slope failures occur. Similarly, loessial soils support heavy structural loads on footings or pilings when dry; however, they lose their bearing capacity and resistance to compression when their loose structure collapses upon becoming wet.

Certain alluvial fan soil deposits such as those adjacent to the southwestern foothills of the Central Valley of California have similar characteristics. When dealing with hydraulic engineering works, where the subsoils will eventually become saturated, it is imperative to recognize these soils' characteristics and to take precautionary measures to improve them before constructing structures on them. Many engineering problems associated with the effects of mineralogy and origin are discussed later in more detail in this manual.

Figure 1-3.—Halloysite particles (1μ = 0.001 mm).

B. Index Properties of Soils

1-9. General.—Engineers are continually searching for simplified tests that will increase their knowledge of soils beyond that which can be gained from visual examination without having to resort to the expense, detail, and precision required for engineering properties tests. These simplified *index tests* or *physical properties tests* provide indirect information about engineering properties of soils; the most widely used are gradation analyses and Atterberg limits tests. Both tests define the limits of the various groups of soils when using the USCS.

Moisture content and density relationships are commonly used as index properties in evaluating foundations and for construction control. Other index tests include:

- Laboratory penetration resistance needle tests on compacted specimens
- Field penetration resistance tests with split-tube drive samplers
- Unconfined compression tests on undisturbed or compacted samples of fine-grained cohesive soil

Data secured from index tests, together with descriptions of visual observations, are often sufficient for design purposes for minor structures. This information is used also in making preliminary designs for determining probable cost of a major structure and to limit the amount of detailed testing. An assumption is that construction materials within a limited area, having similar index properties, will exhibit similar engineering properties; however, correlations between index properties and engineering properties are not perfect. If index properties are used in design, a liberal factor of safety should be included.

1-10. Terms and Units of Measure.—The terms and units of measure used here follow.

a. Mass.—Mass, the amount of matter an object contains, remains constant even if temperature, shape, or other physical attributes of the object change. The object's mass does not depend on local gravitational attraction and is independent of the object's location in the universe. The output or reading from *any* type of balance or scale is *mass*—not weight. Typical units of mass are gram (g) or kilogram (kg) and pound mass (lbm).

b. Density.—Density is mass per unit volume. The symbol for density is the lowercase Greek letter rho, ρ. Units of density are megagram per cubic meter (Mg/m^3) and pound mass per cubic foot (lbm/ft^3).

c. Weight.—Weight is the gravitational force that causes a downward acceleration of an object. Units of weight are the same as units of force: newton (N) and pound force (lbf).

d. Unit Weight.—*Unit weight* is weight per unit volume. The symbol for unit weight is the lowercase Greek letter gamma, γ. Units of unit weight are kilonewton per cubic meter (kN/m³) and pound force per cubic foot (lbf/ft³).

e. Percentage and Decimal Use.—Certain terms pertaining to soil properties are customarily expressed as a percentage, whereas others are usually expressed as decimals. For example, degree of saturation (S), moisture content (w), and porosity (n) are commonly written in percentage as:

$$S = 85.6\%,\ w = 16.2\%,\ \text{and}\ n = 34.3\%$$

Conversely, void ratio (e) and specific gravity (G_s) are expressed in decimals as:

$$e = 3.52,\ \text{and}\ G_s = 2.76$$

To avoid confusion in computations involving these quantities, a simple rule should be followed: Always express quantities as a decimal in all computations. The answers can be given as percentages—provided the percent sign is used, as shown in the following example.

Given:

$\gamma_d = 113.2$ lbf/ft^3, $w = 16.2\%$, $G_s = 2.76$, and $n = 34.3\%$.

Find the degree of saturation, S

$$S = \frac{\dfrac{wG_s}{n}}{1 - n} = \frac{\dfrac{0.162 \times 2.76}{0.343}}{1 - 0.343} = 0.856 \text{ or } 85\% \qquad \textbf{1-1}$$

1-11. Gradation.—Gradation is a descriptive term that refers to distribution and size of grains in a soil. It is determined by the gradation analysis of soils,[3] and is presented in the form of a cumulative, grain-size curve in which particle sizes are plotted logarithmically with respect to percentage (by dry mass) of the total specimen plotted to a linear scale (fig. 1-4).

Certain terms and expressions are used when referring to a gradation curve. A particular diameter of particle is indicated by D with a numeric subscript that corresponds to a point on the *curve* equivalent to the percentage of particles passing. On figure 1-4, D_{10} equals 0.075 mm. This means 10 percent, by dry mass of soil, is composed of particles smaller than 0.075 mm. The 10-percent size (D_{10} size) is also called the "effective" grain size. This term was introduced by Hazen [10] in connection with his work on sanitary filters. Hazen found that sizes smaller than the effective grain size affected the functioning of filters as much or more than did the remaining 90 percent of sizes. Other sizes, such as D_{15}, D_{50}, and D_{85}, are also used in filter design. The sizes D_{10}, D_{30}, and D_{60} are used in defining the gradation characteristics of a soil. The gradation curve is used to designate various soil components by grain size.

A soil is said to be *well graded* when a good representation of all particle sizes exists—from the largest to the smallest. A soil is considered to be *poorly graded* if an excess or a deficiency of certain particle sizes occurs within the limits of the minimum and maximum sizes or if the range of predominant sizes falls within three or less consecutive sieve-size intervals on the gradation curve. A poorly graded soil is called *uniform* if all the particles are about the same size. When there is an absence of one or more intermediate sizes, the material is said to have a *gap* or *skip* gradation.

To determine whether a material is well graded or poorly graded, coefficients describing the extent and shape of the gradation curve have been defined as follows and illustrated on figure 1-4:

$$\text{Coefficient of uniformity, } C_u = \frac{D_{60}}{D_{10}} \qquad \textbf{1-2}$$

$$\text{Coefficient of curvature, } C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} \qquad \textbf{1-3}$$

To be well graded, a material must have a coefficient of curvature between 1.0 and 3.0; in addition, the coefficient of uniformity must be greater than 4.0 for gravels and greater than 6.0 for sands. If one or both of these criteria are *not* satisfied, the soil is poorly graded. A poorly graded soil having a coefficient of uniformity of 2.0 or less is uniform [11].

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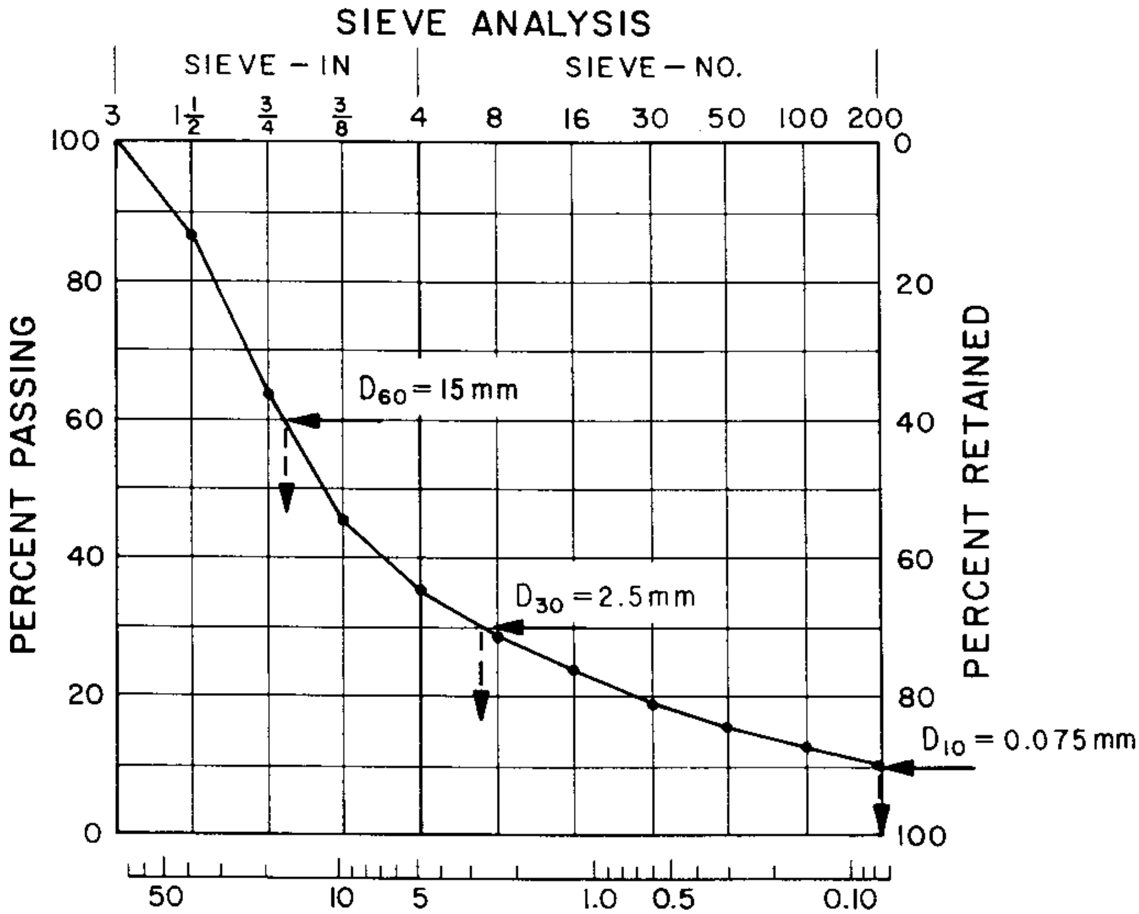
[3] 5325: Performing Gradation Analysis of Gravel Size Fraction of Soils.

5330: Performing Gradation Analysis of Fines and Sand Size Fraction of Soils, Including Hydrometer Analysis.

5335: Performing Gradation Analysis of Soils Without Hydrometer—Wet Sieve.

5345: Performing Gradation Analysis of Soils by the Bottom Withdrawal Test Method.

(Numbered footnotes refer to groups of related soil testing procedures, as described in Part 2 of the *Earth Manual*. Each footnote lists the soil testing procedures in a group by their USBR designations.)



$$Cu = \frac{D_{60}}{D_{10}} = \frac{15}{0.075} = 200 \qquad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(2.5)^2}{0.075 \times 15} = 5.6$$

Figure 1-4.—Gradation plot.

A soil sample may be divided into parts—called *fractions*—based on grain size. For example, in construction control of earthwork, soil is frequently divided into two fractions using the U.S.A. Standard 4.75-mm sieve (No. 4). The fraction finer than the 4.75-mm sieve (the minus 4.75-mm fraction) is called the *control fraction*, and the fraction coarser than the 4.75-mm sieve (the plus 4.75-mm fraction) is called the *oversize*.

A number of different upper-size limits are frequently used in appraising soils. The workability of a soil is often affected appreciably by the largest soil particles present. Important upper-size limits for various applications are:

U.S.A. Standard Series Sieve Size		*Application*
300 mm	12 inch	Sand, gravel, and cobble fill; division between cobbles and boulders
125 mm	5 inch	Impervious rolled fill
75 mm	3 inch	Soil classification, large-scale laboratory testing, division between gravel and cobbles
4.75 mm	No. 4 sieve	Most laboratory testing, division between sand and gravel
425 μm	No. 40 sieve	Atterberg limits test
75 μm	No. 200 sieve	Division between fines and sand

Use of gradation in determining engineering properties is limited to coarse-grained materials. In poorly graded uniform materials, permeability increases approximately as the square of the effective grain size (D_{10}) [10]. For such materials, compressibility is normally small except for in very fine sands. Shear strength consists almost wholly of internal friction and is more or less independent of grain size. Uniform materials are generally workable; that is, they are easily excavated and compacted. As the range in sizes of coarse-grained soils increases:

- permeability decreases,
- compressibility decreases, and
- shear strength increases.

Workability of well-graded soil is good.

Amount and plasticity of fines influence the properties of coarse-grained materials. Permeability is reduced with increasing quantity of fines—rapidly with a small quantity and at a slower rate with a large quantity. Compressibility and shear strength are affected only slightly by small percentages of fines in coarse-grained soils, but the effects increase with increase in fines.

1-12. Atterberg Limits.—The physical properties of most fine-grained soils, and particularly clayey soils, are greatly affected by moisture content. The consistency of a clay may be very soft, that is, a viscous liquid; or it may be very hard, having the properties of a solid—depending on its moisture content. In between these extremes, clay may be molded and formed without cracking or rupturing the soil mass. In this condition, it is referred to as being plastic.

Plasticity is an outstanding characteristic of clays and is used to identify and classify clayey soils.

In 1911, a Swedish soil scientist, A. Atterberg, developed a series of hand-performed tests for determining the clay activity or plasticity of soil [12]. These tests are known as the *Atterberg limits* tests. The series of tests is common to engineering classification of soil.

The consistency of a soil can go through four stages:

1. liquid,
2. plastic,
3. semisolid, and
4. solid.

The stages are related to moisture content. Although the transition between stages is gradual, test conditions have been arbitrarily established to delineate the moisture content as a precise point in the transitions between the four stages. These moisture contents, which are determined by oven-drying procedures, are called the:

- liquid limit,
- plastic limit, and
- shrinkage limit.

As the tests are performed only on the soil fraction that passes the U.S.A. Standard 425-µm (No. 40) sieve, the relation of this fraction to total material must be considered when determining the state of the total material from these tests.

The significance of the limits and their relation to the phases of a soil-water system can be explained by referring to figures 1-5 and 1-6. As a very wet, fine-grained soil dries, it passes progressively through different phases. In a very wet condition, the mass acts like a viscous liquid, which is referred to as the liquid state. As the soil dries, a reduction in volume of the mass takes place that is nearly proportional to the loss of water. When moisture in a soil is at a value equivalent to the liquid limit, the mass becomes plastic.

Liquid limit, LL, is that moisture content (expressed as a percentage of the dry mass of soil) when the soil first shows a small but definite shear strength as moisture content is reduced. Conversely, with increasing moisture, liquid limit is that moisture content when the soil mass just begins to

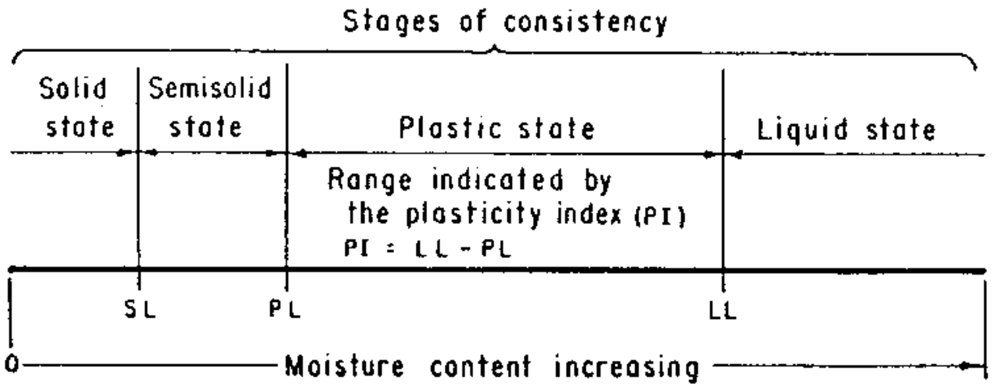


Figure 1-5.—Consistency limits.

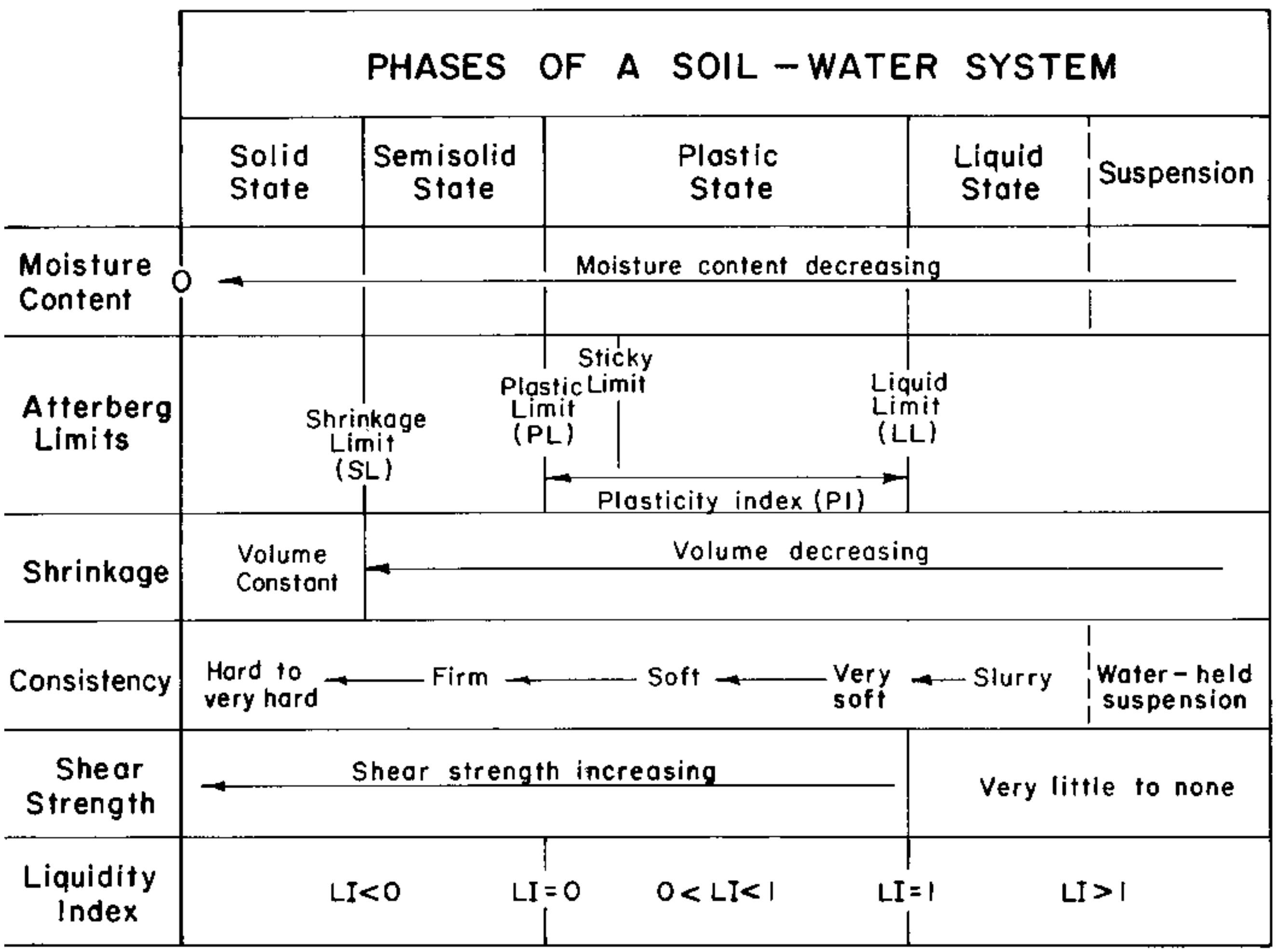


Figure 1-6.—Phases of a soil-water system.

become fluid under the influence of a series of standard blows, as outlined in USBR 5350 and 5355.[4]

As moisture content is reduced below the liquid limit, the soil mass becomes stiffer and will no longer flow as a liquid. However, it will continue to be deformable, or plastic, without cracking until the plastic limit is reached.

Plastic limit, PL, is that moisture content (expressed as a percentage of the dry mass of soil) when the soil mass ceases to be plastic and becomes brittle, as determined by a procedure for rolling the soil mass into threads 3 mm (1/8 in) in diameter. Plastic limit is always determined by reducing the moisture content of the soil mass.

As the moisture content of soil is reduced below the plastic limit, the soil becomes a semisolid; that is, it can be deformed, but considerable force is required and the soil cracks. This condition is referred to as a *semisolid state*. As further drying takes place, the soil mass will eventually reach a *solid* state when further volume change (shrinkage) will not occur. Moisture content under this condition is called *shrinkage limit*, SL. This is the moisture content at which a reduction in moisture will not cause a decrease in the volume of the soil mass. Below the shrinkage limit, soil is considered to be a solid; that is, most particles are in very close contact and are very nearly in an arrangement which will result in the most dense condition.

In most fine-grained plastic soils, plastic limit will be appreciably greater than shrinkage limit. However, for coarser fine-grained soils (soils containing coarse silt and fine sand sizes), the shrinkage limit will be near the plastic limit. The shrinkage limit, together with other indexes, is useful in identifying expansive soils [13].

Plasticity index, PI, is the difference between the liquid and plastic limits, and represents the range of moisture content over which soil is plastic. (See figs. 1-5 and 1-6 and the plasticity chart on fig. 1-7.) Silts have low or no plasticity indexes, whereas clays have higher indexes. Plasticity index, in combination with liquid limit, indicates how sensitive a soil is to change in moisture content.

Liquidity index, LI, is the ratio of the difference between natural moisture content and plastic limit, w_n – PL, to the plasticity index, PI:

$$\mathrm{LI} = \frac{w_n - \mathrm{PL}}{\mathrm{PI}} \qquad \textbf{1-4}$$

Liquidity index is a useful indicator of the behavior of a fine-grained soil when sheared. If liquidity index is less than zero (negative), soil at natural moisture will exhibit brittle stress-strain behavior if sheared. If liquidity index is between zero and one, soil at natural moisture will behave like a plastic. If liquidity index is greater than one, soil at natural moisture will act like a viscous liquid when sheared [14].

Relative consistency, C_r, is the ratio of the difference between liquid limit and natural moisture content to the plasticity index:

$$C_r = \frac{\mathrm{LL} - w_n}{\mathrm{PI}} \qquad \textbf{1-5}$$

If moisture content of a soil in its natural state, or in place, is greater than the liquid limit (relative consistency less than zero), any process of remolding will transform the soil into a thick, viscous slurry. If natural moisture content is less than the plastic limit (relative consistency greater than one), the soil cannot be remolded [15].

Soils may be grouped according to their liquid limits and plasticity indexes on a plasticity chart as shown on figure 1-7. Such plots can be useful in predicting properties of soils by comparing with similar plots for tested soils (see fig. 1-8).

As an index of engineering properties, soil plasticity applies only to fine-grained soils. The variation of engineering properties is generally related to the four zones on the plasticity chart (fig. 1-7), which determines the soil classification group. For soils plotting above the "A" line, permeability is very low. Compressibility increases with increasing liquid limit. For the same liquid limit, the greater the plasticity index, the greater will be the shear strength at the plastic limit.

1-13. Porosity and Void Ratio.—In evaluating a soil, either the amount of solids contained in a given volume or the remaining voids can be considered. Many of the computations in soil mechanics are simplified by considering the voids rather than the solids. Two expressions, *porosity* and *void ratio*, are used to define the void space. Porosity, *n*, is the ratio (expressed as a percentage) of space in the soil mass not occupied by solids (volume of voids) with respect to total volume. Void ratio, *e*, is the ratio of space

[4] 5350: Determining the Liquid Limit of Soils by the One-Point Method.
5355: Determining the Liquid Limit of Soils by the Three-Point Method.

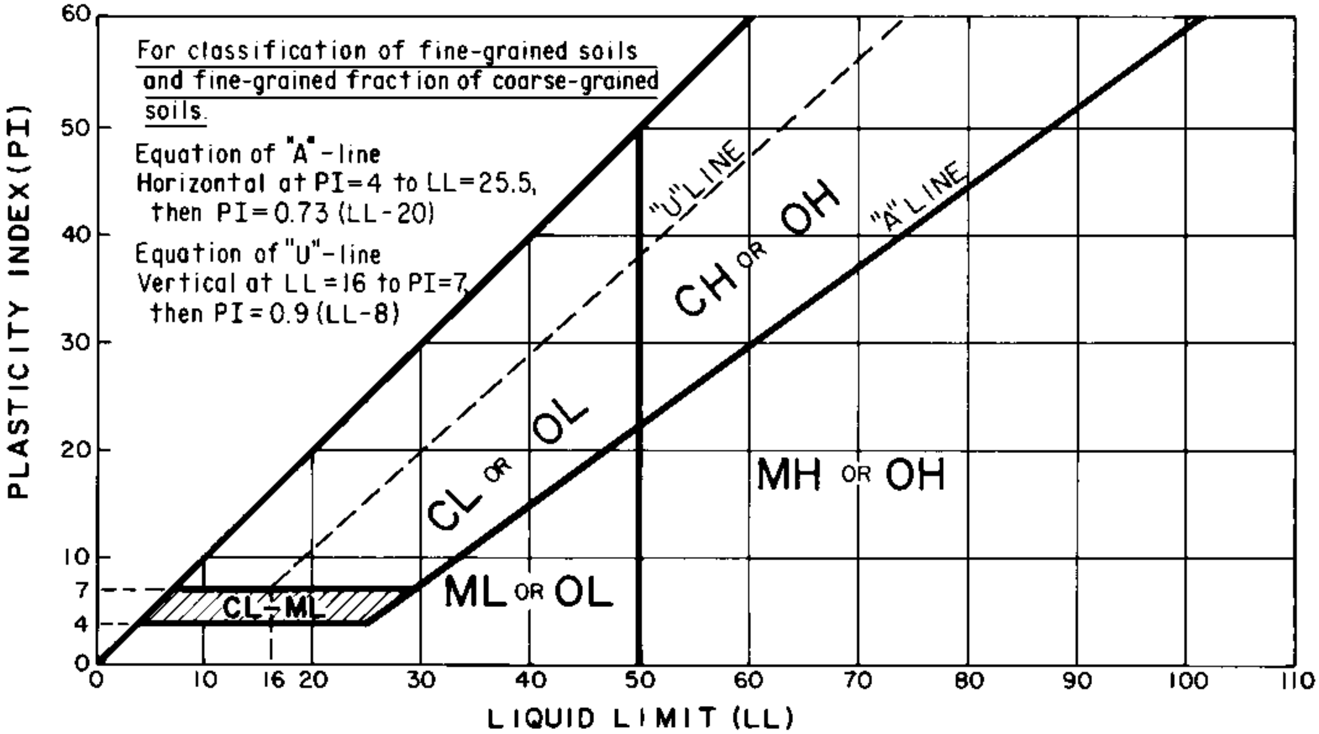


Figure 1-7.—Plasticity chart.

not occupied by solid particles (volume of voids) to the volume of solid particles in a given soil mass. These relationships are expressed as follows:

$$n = \frac{V_v}{V_t} = \frac{e}{1 + e} \qquad \textbf{1-6}$$

$$e = \frac{V_v}{V_s} = \frac{n}{1 - n} \qquad \textbf{1-7}$$

where:

- n = porosity expressed as a percentage
- e = volume of voids to volume of solid particles
- V_t = total volume
- V_v = volume of voids
- V_s = volume of solid particles

Porosity and void ratio are measures of the state or condition of a soil structure. As porosity and void ratio decrease, engineering properties of a given soil become more dependable with decreases in permeability and compressibility and an increase in strength. As porosity decreases, and, consequently, the void ratio decreases, excavating the material becomes more difficult. At a given moisture content, the compactive effort must be increased to obtain a decrease in porosity. However, similar properties may be obtained in different soils at widely different conditions of porosity. Engineering properties of a soil do not vary directly with its porosity; the relationship is complex.

1-14. Specific Gravity.—In investigating a soil, the most easily visualized condition involves the volume occupied by soil solids, V_s , the volume occupied by water, V_w, and the volume occupied by air, V_a, in the soil mass (fig. 1-9). However, most measurements are more readily obtained by mass. To correlate mass and volume, the *specific gravity* factor is required. Specific gravity is defined as the ratio between the density of a substance and the density of water at 4 °C. Several different types of specific gravity are in common use. Reclamation uses apparent specific gravity and several types of bulk specific gravities. These values are obtained using methods outlined in USBR 5320: *Determining Specific Gravity of Soils*[5]

[5] 5320: Determining Specific Gravity of Soils.

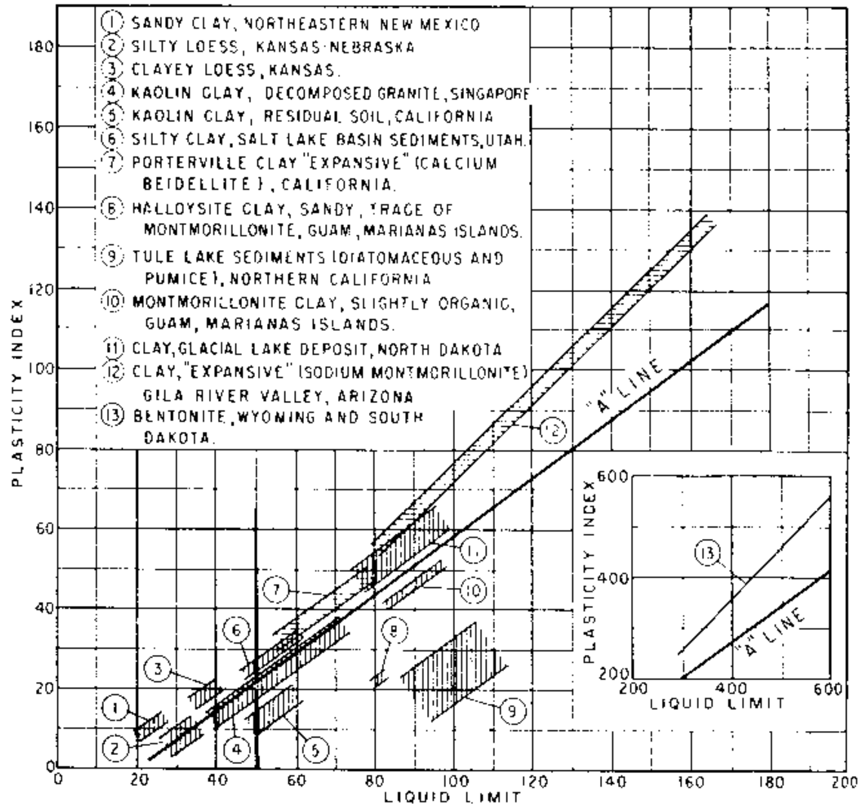


Figure 1-8.—Typical relationships between the liquid limit and the plasticity index for various soils.

Apparent specific gravity, G_a, is determined on soil particles as they occur naturally. Voids that exist within grains and cannot be filled with water are referred to as "impermeable voids." Apparent specific gravity ranges from 2.50 to 2.80 for most soils, with a majority of soils having an apparent specific gravity near 2.65. Unless specifically stated to the contrary, "specific gravity" (in this manual) is assumed to mean the "apparent specific gravity." It is used to compute many important soil properties involving volume determinations such as porosity, void ratio, and degree of saturation as shown in the following equations:

$$n = 1 - \frac{V_s}{V_t} = 1 - \frac{\gamma_d}{\gamma_w \, G_a} \qquad \textbf{1-8}$$

$$e = \frac{V_t}{V_s} - 1 = \frac{\gamma_w \, G_a}{\gamma_d} - 1 \qquad \textbf{1-9}$$

$$S = \frac{V_w}{V_v} = \frac{w \, G_a}{e} \qquad \textbf{1-10}$$

where:

e = volume of voids to volume of solid particles
n = porosity expressed as a percentage
s = degree of saturation
w = moisture content
V_v = volume of voids
V_s = volume of solid particles
V_t = total volume
V_w = volume of water
γ_d = dry density
γ_w = density of water at 4 °C

Bulk specific gravity (specific mass gravity), G_m, is the specific gravity having the permeable or surface voids of the particles filled with water. This value is smaller than the value determined for apparent specific gravity unless there are no permeable voids within the particles. The bulk specific gravity, saturated surface dry, of aggregate (sand and gravel) is used for concrete mix design and also for quality tests for riprap and rockfill materials. The bulk

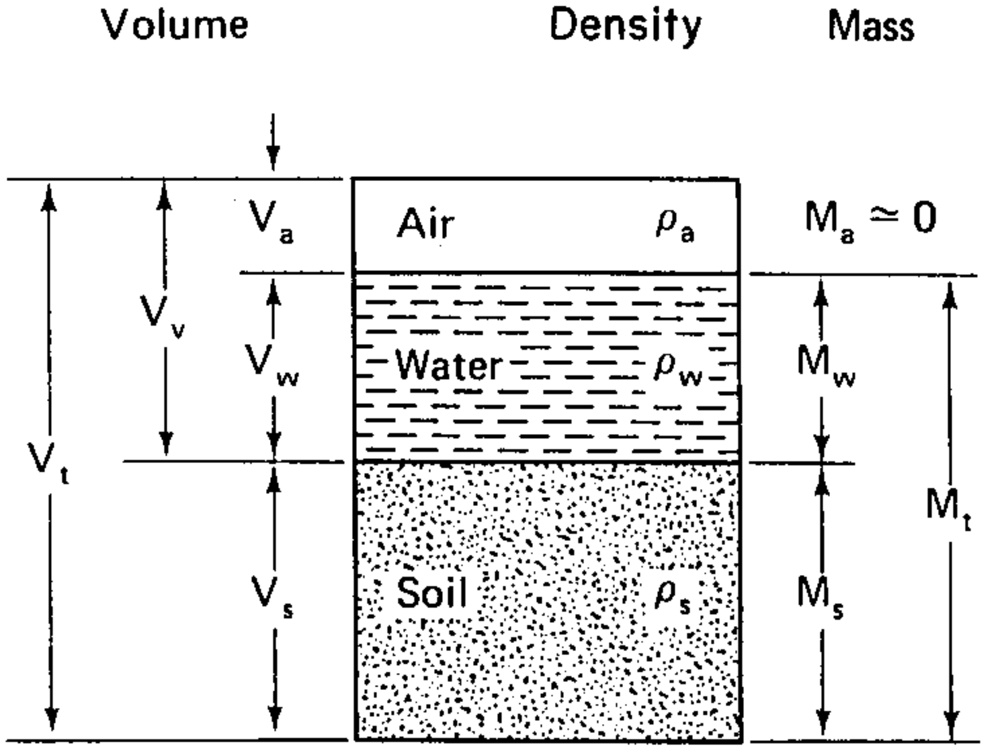


Figure 1-9.—Relationship between air, water, and solids in a soil mass.

specific gravity of gravel particles may be determined on a wet, surface-dry basis, where permeable voids may not be entirely filled with water. Bulk specific gravity (wet surface dry) is used in all *in situ* density testing procedures. Bulk specific gravity (ovendry) is, in effect, the minimum dry density of all particles distributed throughout the entire or effective volume of particles; that is, both impermeable and permeable voids associated with individual particles in the ovendry condition are included in the volume determination. It is the smallest value of the different specific gravities.

As an index test, specific gravity is somewhat indicative of the durability of a material. Materials having low specific gravity are likely to break down and change properties with time whereas, high specific gravity materials normally do not deteriorate rapidly. The test is applicable primarily in evaluating coarse-grained materials and riprap or rockfill materials.

1-15. Moisture Content.—Moisture content, *w*, is defined as the ratio (expressed as a percentage) of mass of water to mass of soil solids. Moisture (water) is the most influential factor affecting soil properties. Also, it is the principal factor subject to change either from natural causes or at the discretion of the engineer. In soil used as construction material or as encountered in foundations, the control of moisture often represents an important part of the structure's cost and may considerably influence the construction procedures used. Therefore, it is essential that the moisture content of a soil be determined, recorded, and reported in conjunction with all investigations, tests, and construction control work. Moisture content is an important factor in formulating designs, and the requirements for moisture are delineated in most specifications for construction. Regarding materials or foundation investigations, descriptive terms such as dry, moist, and wet may provide sufficient information to decide whether to use a given material. In all tests and in construction control procedures, a quantitative determination of moisture content is required. Moisture content is determined by a particular testing procedure.[6]

[6] 5300: Determining Moisture Content of Soil and Rock by the Oven Method.
5305: Determining Moisture Content of Soils Using the Moisture Teller Device.
5310: Determining Moisture Content of Soils Using the Calcium Carbide Reaction Device.
5315: Determining Moisture Content of Soils by the Microwave Oven Method.

Note that moisture content is expressed as a percentage of the dry mass of soil. Because moisture content is measured in terms of mass, it is independent of the volume occupied by the soil mass; that is, moisture content does not change whether material is in a loose state or in a dense state. Drying a soil specimen to a constant dry mass at a temperature of 110 °C (see USBR 5300) is accepted as a standard method for determining moisture content. Occasionally, however, soils are encountered for which this procedure is not valid; then, special methods are required. Soils containing organic matter, an appreciable amount of soluble solids, or unusual clay minerals such as halloysite or allophane require special treatment. Sometimes, ovendrying causes irreversible changes in soil properties; therefore, ovendried materials should not be used in laboratory tests unless specifically required.

a. Optimum Moisture Content.—In 1933, R.R. Proctor [16, 17, 18, 19] showed that the dry density of a soil obtained by a given compactive effort depends on the amount of water the soil contains during compaction. He pointed out that, for a given soil and a given compactive effort, there is one moisture content that results in a maximum dry density of the soil, and that moisture contents both greater and smaller than this optimum value will result in dry densities less than the maximum (see fig. 1-10c). As early as 1934, moisture content at maximum dry density was termed *optimum moisture content*, w_o, by other investigators of soil compaction [20]. As defined for Reclamation work, optimum moisture content is based on the compactive effort for the standard laboratory compaction test described in USBR 5500.[7] Figure 1-11 shows the variation of optimum moisture content when compactive effort is varied from the standard [21].

b. Absorbed Moisture.—Absorbed moisture is that on saturated surface-dry particles. Depending upon moisture conditions in a soil containing gravel-size particles, sufficient moisture may not always be available for the gravel to take up the maximum amount. The actual amount of absorbed moisture is determined by separating the gravel from the other soil particles and determining the wet and dry mass of the gravel.[8]

If a soil has been separated into fractions by sieving, average moisture content of the total material, w, (in percent) may be calculated by:

$$w = P_1 \left(\frac{w_1}{100} \right) + P_2 \left(\frac{w_2}{100} \right) + \ . \ . \ . \ . \ + P_n \left(\frac{w_n}{100} \right) \qquad \textbf{1-11}$$

where:

P_1 , P_2 , P_n = percent of each size fraction in total material, %
w_1 , w_2 , w_n = moisture content of each size fraction, %
100 = constant representing the total materials, %

Engineering properties change so much with moisture content that it is used primarily to assist in interpreting other index properties. Primarily, moisture content is used to evaluate soils in their natural state, both as foundations and as construction materials sources.

1-16. Density and Unit Weight.—Mass or weight of a unit volume of soil is an easily determined property. Consequently, density and unit weight are basic parameters to which all other performance characteristics are related. Relationships between density, unit weight, and other soil properties—as a rule—are complex, but in engineering practice, simple relationships are assumed to exist. A large number of qualified expressions are in common use for density or unit weight. To avoid confusion, the kind of density or unit weight reported must be clearly delineated. Generally, density is used in laboratory testing and construction control; unit weight is used in engineering analysis and design.

a. In Situ Unit Weight.—The soil's unit weight as it exists in a natural deposit, at any particular time, is the in-place or natural unit weight. It can be determined either by measuring the mass and volume of an undisturbed sample of material or by measuring the mass of material removed from a hole of known volume. In-place density is calculated from mass and volume and then converted to unit weight. In-place unit weight is used in engineering computations such as:

- slope stability,
- bearing capacity,
- settlement analyses, and
- determining earth pressures.

In some soil types, sufficient correlations exist between in-place unit weight and compression characteristics so that foundations for simple structures may be designed based only on those data and a knowledge of in-place unit weight

[7] 5500: Performing Laboratory Compaction of Soils—5.5-lbm Rammer and 18-in Drop.

[8] Determining Unit Weight of Soils In-Place by the Sand Replacement Method in a Test Pit.

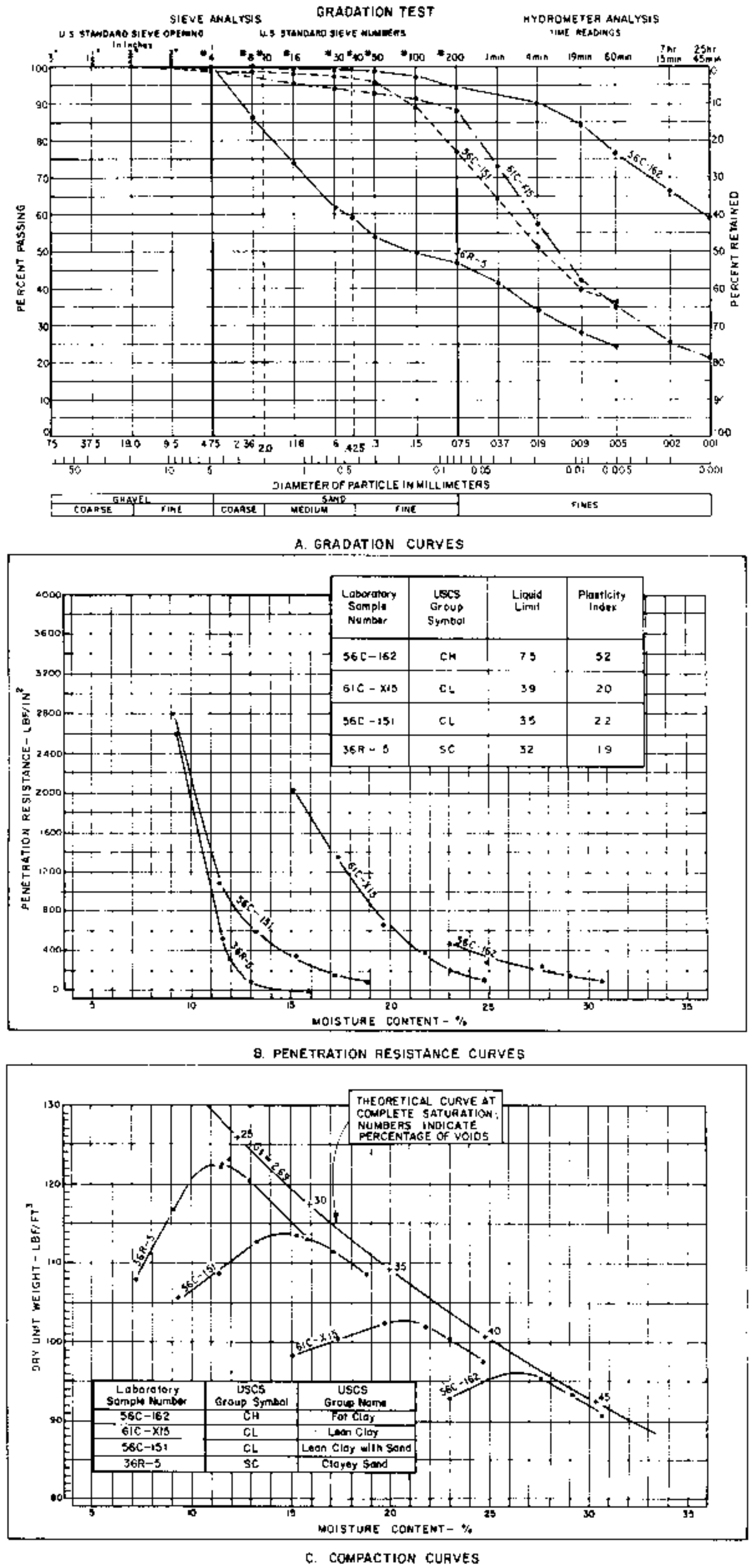


Figure 1-10.—Gradation, penetration, and compaction curves for various soils.

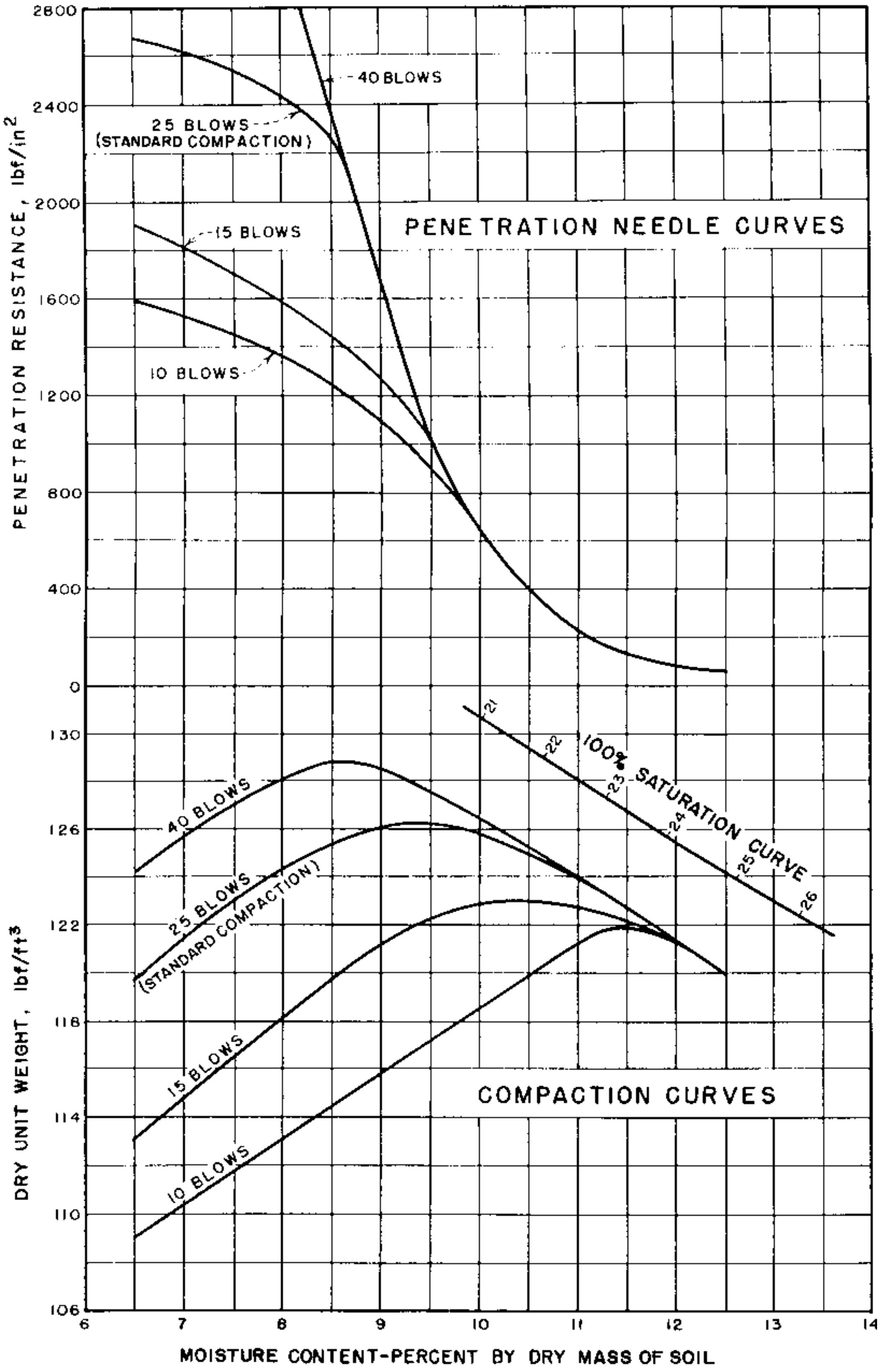


Figure 1-11.—Effect of compactive effort on the compaction and penetration-resistance curves.

in the foundation under consideration. The in-place or natural unit weight can be expressed as a wet or dry unit weight.

b. Wet Unit Weight.—The unit weight of the solid particles and the contained water is called wet unit weight, γ_{wet}. It includes—but is not restricted to—in-place unit weight. The method for its determination and its application is the same as that previously described in "In situ *unit weight*."

c. Dry Unit Weight.—Dry unit weight is the normal expression for unit weight of soil. It is regarded as a fixed quantity independent of moisture content unless compactive effort is applied to the soil to change it. Dry unit weight, γ_d, is computed from wet unit weight, γ_{wet}, and moisture content using the following:

$$\gamma_d = \frac{\gamma_{wet}}{1 + \frac{w}{100}} \qquad \textbf{1-12}$$

where:

- w = moisture content
- γ_d = dry unit weight
- γ_{wet} = wet unit weight of solid particles plus contained water

d. Laboratory Maximum Dry Unit Weight.—Dry unit weight—produced with Reclamation's standard compactive effort at optimum moisture content—is called the laboratory maximum dry unit weight. Test procedures are performed on the soil fraction that passes the U.S.A. Standard 4.75-mm (No. 4) sieve, according to procedures outlined in USBR 5500, which provides a compactive effort of 600 kN-m/m^3 (12,375 ft-lbf/ft^3). Figure 1-10c shows the variation in laboratory maximum dry unit weight for several soil types. Figure 1-11 shows typical compaction curves of various compactive efforts expressed in terms of number of standard blows per layer of soil. The maximum density and the optimum moisture content vary with the compactive effort. When the first laboratory compaction test procedure was developed, the compactive effect used was similar to the compactive effort of the construction equipment used to compact soils in the field. As construction equipment got heavier, additional procedures using various compactive efforts were developed. Reclamation has maintained a single standard. However, Reclamation may specify percentages of the maximum density and deviations from optimum moisture for various types of earthwork construction.

e. Relative Density.—Soils consisting almost exclusively of coarse-grained particles (e.g., sands and gravels), when compacted by impact according to procedures outlined for determining laboratory maximum dry unit weight, have unit weight-moisture content relationships that correlate poorly with other properties. Furthermore, compaction curves are erratic; often they do not produce a definable maximum unit weight. To evaluate such soils, relative density (D_d) procedures consisting of three independent laboratory and field determinations have been developed.[9]

When shear strength of coarse-grained soils is correlated with unit weight in the range between minimum and maximum index unit weights, a fairly reliable relationship exists. With a reasonable amount of construction control, a given type and amount of compactive effort can be expected to produce a related relative density. In practice, a relative density of 70 percent or greater is satisfactory for most conditions. However, special analyses of sands and gravels may be required to assess liquefaction potential in zones of high earthquake probability. While several different relationships for expressing the relative density exist, the original and the one now generally used is to express relative density (D_d) in terms of void ratio using the following:

$$D_d = \left(\frac{e_{max} - e}{e_{max} - e_{min}}\right) 100 \qquad \textbf{1-13}$$

where:

- D_d = relative density expressed as a percentage
- e = void ratio in place
- e_{max} = void ratio in loosest state
- e_{min} = void ratio in densest state

Relative density may also be expressed in terms of dry unit weight as follows:

$$D_d = \frac{\gamma_{dmax}}{\gamma_d}\left(\frac{\gamma_d - \gamma_{dmin}}{\gamma_{dmax} - \gamma_{dmin}}\right) \qquad \textbf{1-14}$$

[9] 5525: Determining the Minimum Index Unit Weight of Cohesionless Soils.
5530: Determining the Maximum Index Unit Weight of Cohesionless Soils.
7250: Determination of Percent Relative Density.

where:

γ_d = dry unit weight in place
γ_{dmax} = dry unit weight in densest state
γ_{dmin} = dry unit weight in loosest state

Figure 1-12 shows a chart from which relative density may be determined if the maximum index, minimum index, and in-place or compacted-fill dry unit weights of a material are known. Figure 1-12 also shows the maximum and minimum index dry unit weights for a variety of sands and gravels. Figure 1-13 shows the variation of maximum and minimum index unit weights as various percentages of gravel were added to a sand [22].

f. Compacted-Fill Unit Weight.—The soil's unit weight as it is compacted into a constructed fill is called the compacted-fill unit weight to distinguish it from in-place unit weight of a foundation or any of the various unit weights determined in the laboratory. Usually, it is expressed as dry unit weight, although wet unit weights are used to some extent. The compacted-fill unit weight may be determined by any one of a number of test procedures.

Compacted-fill unit weight is used primarily for construction control to ensure that compacted fill is at least as dense as assumed in preparing designs.

If a cohesive soil contains an appreciable quantity of coarse-grained particles retained on the U.S.A. Standard 4.75-mm (No. 4) sieve, an adjustment must be made in the compacted fill dry unit weight. The theoretical dry unit weight of total material, plus 4.75 mm and minus 4.75 mm, comparable to the laboratory maximum dry unit weight, is computed using the following [23]:

$$\gamma_t = \frac{1}{\dfrac{P}{\gamma_w\, G_m} + \dfrac{1-P}{\gamma_{d_s}}} \qquad \textbf{1-15}$$

where:

G_m = bulk specific gravity (oven dry) of plus 4.75-mm fraction
P = percentage of plus 4.75-mm material to total material (dry mass basis) expressed as a decimal
γ_t = theoretical dry unit weight of total material
γ_{d_s} = dry unit weight of minus 4.75-mm fraction
γ_w = unit weight of water at 4°C

Equation 1-15 gives theoretical dry unit weight of total material for different percentages of plus 4.75-mm (plus No. 4) particles of a given specific gravity assuming that each plus 4.75-mm particle is surrounded by minus 4.75-mm material. The equation is meaningless for percentages of plus 4.75-mm material for which the unit weight of plus 4.75-mm material corresponds to greater than 100-percent relative density of the plus 4.75-mm material when considered alone (fig. 1-14). This occurs at about 70 to 80 percent of plus 4.75-mm material, depending on gradation and on specific gravity. For plus 4.75-mm contents above about 30 percent, studies have shown compacted dry unit weight of soil in the field is less than the theoretical dry unit weight although, for many soils, compaction is not seriously affected until the percentage of plus 4.75-mm material rises to about 40 or 50 percent. For these soils, the usual assumptions of correlation between dry unit weight and other soil properties are not true. For such soils used in fill, special tests must be made. Figure 1-14 (and fig. 6 in USBR 5515)[10] illustrates relationships of various dry unit weights of mixtures of plus 4.75-mm and minus 4.75-mm materials [24].

1-17. Penetration Resistance.—

a. Soil Plasticity.—In 1933, R.R. Proctor developed a simple instrument called a "soil-plasticity needle" that could be used to correlate moisture content-dry density relationships of soils used in construction [18]. He noted that for a given compactive effort upon a given soil, a curve could be developed relating the penetration resistance of a small rod (needle) with the soil's moisture content.

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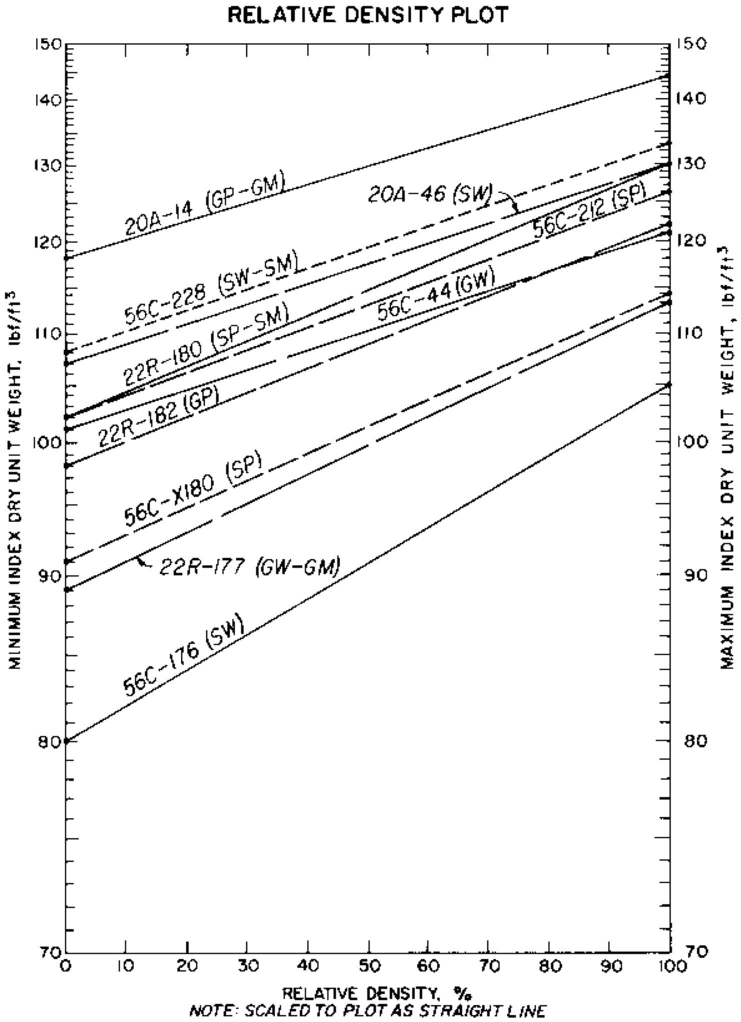
[10] 5515: Performing Laboratory Compaction of Soils Containing Gravel.
7205: Determining Unit Weight of Soils In-Place by the Sand-Cone Method.
7206: Determining Unit Weight of Soils In-Place by the Rubber Ballon Method.
7215: Determining Unit Weight of Soils In-Place by the Sleeve Method.
7220: Determining Unit Weight of Soils In-Place by the Sand replacement Method in a Test Pit.
7221: Determining Unit Weight of Soils In-Place by the Water Replacement Method in a Test Pit.
7230: Determining Unit Weight and Moisture Content of Soils In-Place Nuclear Moisture-Density Gauge.



Laboratory Sample Number	USCS Classification		Index Dry Unit Weight		Gradation			Maximum Size Tested (in)
	Group Symbol	Group Name	Minimum (lbf/ft³)	Maximum (lbf/ft³)	Fines (%)	Sand (%)	Gravel (%)	
56C-44	GW	Well-graded gravel	101	121	5	6	89	3
22R-182	GP	Poorly graded gravel	98	122	2	6	92	3
22R-177	GW-GM	Well-graded gravel with silt	89	113	12	30	58	3
20A-14	GP-GM	Poorly graded gravel with silt	118	144	6	22	72	3
56C-176	SW	Well-graded sand	80	105	7	93	0	3/8
20A-46	SW	Well-graded sand	107	130	2	61	37	1 1/2
56C-212	SP	Poorly graded sand	102	126	4	95	1	3/4
56C-X180	SP	Poorly graded sand	91	114	4	95	1	3/8
56C-228	SW-SM	Well-graded sand with silt	108	133	10	90	0	#4
22R-180	SP-SM	Poorly graded sand with silt	102	130	12	75	13	3/8

Figure 1-12.—Typical minimum and maximum dry unit weights for various coarse-grained soils.

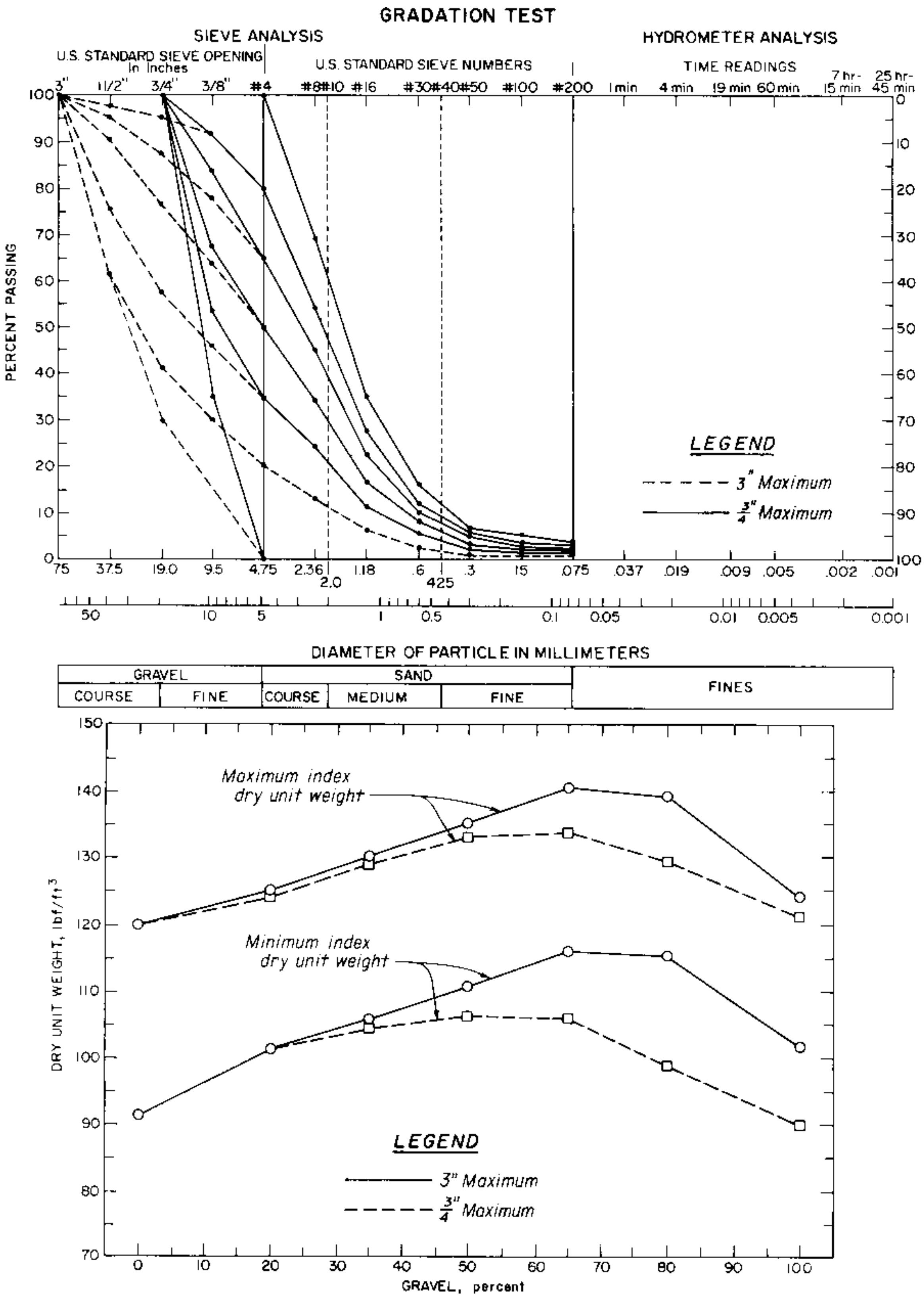


Figure 1-13.—Maximum and minimum index dry unit weights of typical sand and gravel soils [15].

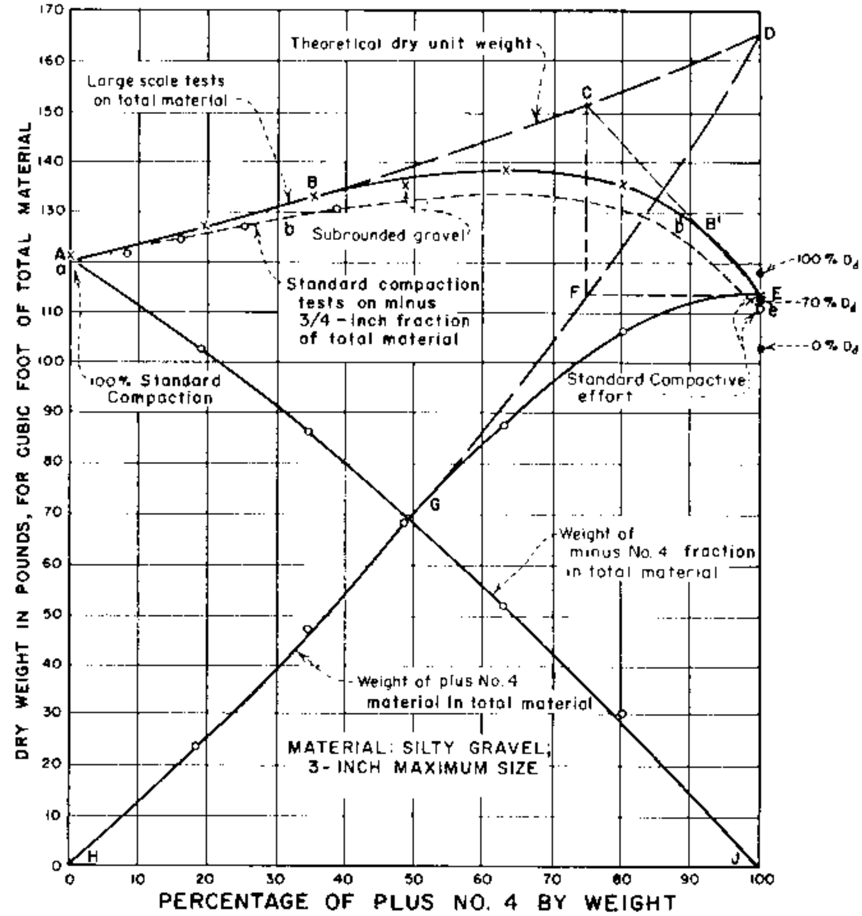


Figure 1-14.—Maximum dry unit weight versus gravel content for compacted soils.

Figures 1-10b and 1-11 show penetration resistance plotted as ordinate versus moisture content as abscissa. The curve begins with a rather high penetration resistance at a point on the compaction curve—dry of optimum moisture content. Then, penetration resistance decreases rapidly and almost uniformly to a rather low value at a point wet of optimum moisture content. From there, the decrease in penetration resistance is gradual to zero near the liquid limit. The curve's straight line part, where penetration resistance decreases rapidly, is normally the useful part of the curve.

The penetration resistance test is described in USBR 5505.[11] The test was formerly used to compare the resistance (of a specimen compacted according to the standard laboratory procedure) to the compaction achieved in the fill at the same moisture content. When fill material contained an appreciable amount of plus 4.75-mm particles, the test was not dependable. This approximate method of controlling compaction, called the "needle-density test," has now been replaced by the rapid method of construction control test.[11]

b. Penetration Test.—The standard penetration test[12] is used in foundation exploration to indicate relative density of *in situ* cohesionless soils and may be used to estimate unconfined compressive strength of cohesive soils *in situ*. The test is performed using a standard sampler and a standard sampling procedure. The sampler is at least 840 mm long, has a constant 50-mm outside diameter, and a constant 35-mm inside diameter (33 by 2 by 1⅜ in). Penetration resistance is measured on the basis of the number of blows of a 64-kg rammer dropped 762 mm required for a sampler penetration of 305 mm (140 lbm at 30 in for 1 ft). The range in which these data are best interpreted extends from a lower limit of 5 to 10 blows per

[11] 5505: Determining Moisture Content-Penetration Relationships of Fine-Grained Soils.

[12] 7215: Performing Penetration Resistance Testing and Sampling of Soils.

305 mm to an upper limit of 30 to 50 blows per 305 mm. It is applicable only to nonlithified soils including fairly clean sands and gravels smaller than about 10 mm (⅜ in) at a variety of moisture contents and in saturated or nearly saturated fine-grained soils. Soil deposits containing appreciable gravel content cannot be tested reliably because of the sampler bearing directly on gravel particles, damage to equipment, and possible plugging of the sampler barrel—all of which result in high blow count values. Deposits containing coarse gravel, cobbles, or boulders typically result in penetration refusal and damage to equipment. Data from this test require interpretation by experienced practitioners and should take into account:

- soil type,
- moisture content,
- methods of drill hole advance,
- ground-water conditions, and
- purpose for performing the test.

Figure 1-15 shows the relationship of blow count to relative density. Figure 1-15a is a relationship determined from chamber testing of clean sands in the laboratory under various overburden pressures. The tests were performed on fine to coarse sands containing less than 5-percent fines; this relationship is not applicable to sands containing more than 10-percent fines or those containing gravel particles. The relationship is converted to a more convenient form, on figure 1-15b, for use in analyzing field data. The graph's vertical pressure scale can be converted to a depth scale by estimating the in-place wet density of soil. For tests below the water table, *in situ* submerged density must be used, taking into account the buoyant effect of water, that is, *in situ* wet density of the soil minus density of water. Using the graph in this manner, penetration resistance is plotted against depth; relative density is determined according to the relative density zone in which the point is plotted. More recently, additional laboratory chamber tests were performed by the Corps of Engineers, and their work confirms the relationships shown on figure 1-15 [25, 26].

The relative consistency of saturated fine-grained soils can be estimated by the standard penetration test. As the values obtained for these soils are only qualitative, table 1-1 and figure 1-16 can be used to provide an approximate evaluation of a soil.

1-18. Unconfined Compressive Strength.—The internal friction component of undrained shear strength for fine-grained soils in a saturated or nearly saturated state is usually small. A compression test made without benefit of confining pressure is assumed to provide an index of the available undrained shear strength of a soil that will be somewhat conservative. A variety of equipment has been developed for making the tests. The devices range from simplified equipment and procedures—from which test results are of value only as an index test—to rather complex equipment and test procedures that provide reliable data on engineering properties. The unconfined compressive strength of a soil is the strength of a cylindrical specimen (maximum axial load divided by cross-sectional area) tested in compression with the lateral pressure equal to atmospheric or zero gauge pressure.

1-19. Soluble Solids.—The quantity of soluble solids present in a soil may be an important factor when considering the suitability of a soil for the foundation of a hydraulic structure or for constructing embankments. The

Table 1-1.—Relationship between consistency, unconfined compressive strength, and penetration resistance, *N*

Consistency	Field identification	Unconfined compressive strength, q_u, ton / ft^2	Penetration resistance, *N*
Very soft	Easily penetrated several inches by fist	Less than 0.25	Below 2
Soft	Easily penetrated several inches by thumb	0.25 to 0.5	2 to 4
Medium	Can be penetrated several inches by thumb with moderate effort	0.5 to 1.0	4 to 8
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 to 2.0	8 to 15
Very stiff	Readily indented by thumbnail	2.0 to 4.0	15 to 30
Hard	Indented by thumbnail with difficulty	Over 4.0	Over 30

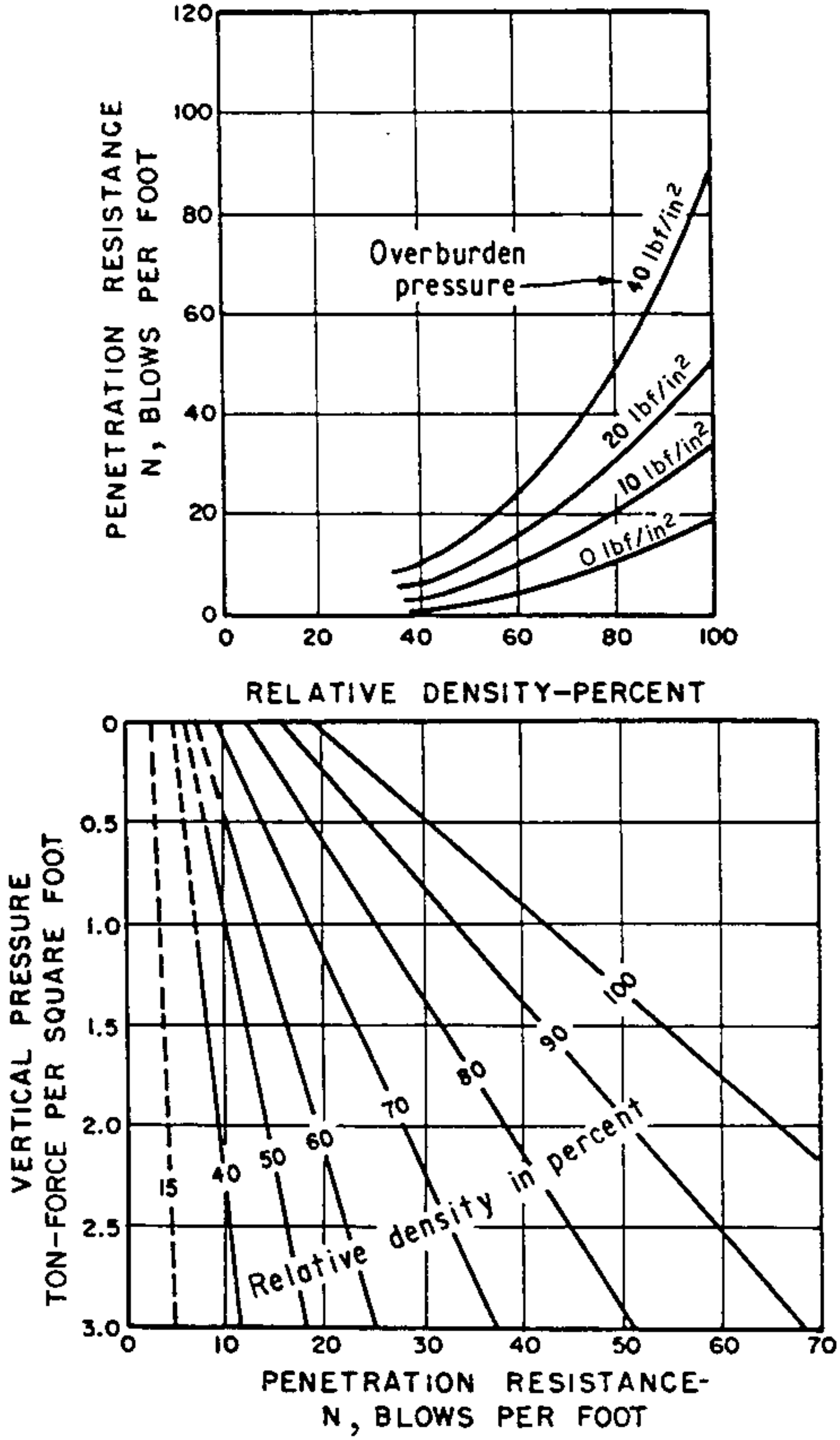


Figure 1-15.—Criterion for predicting relative density of sand from the penetration resistance test.

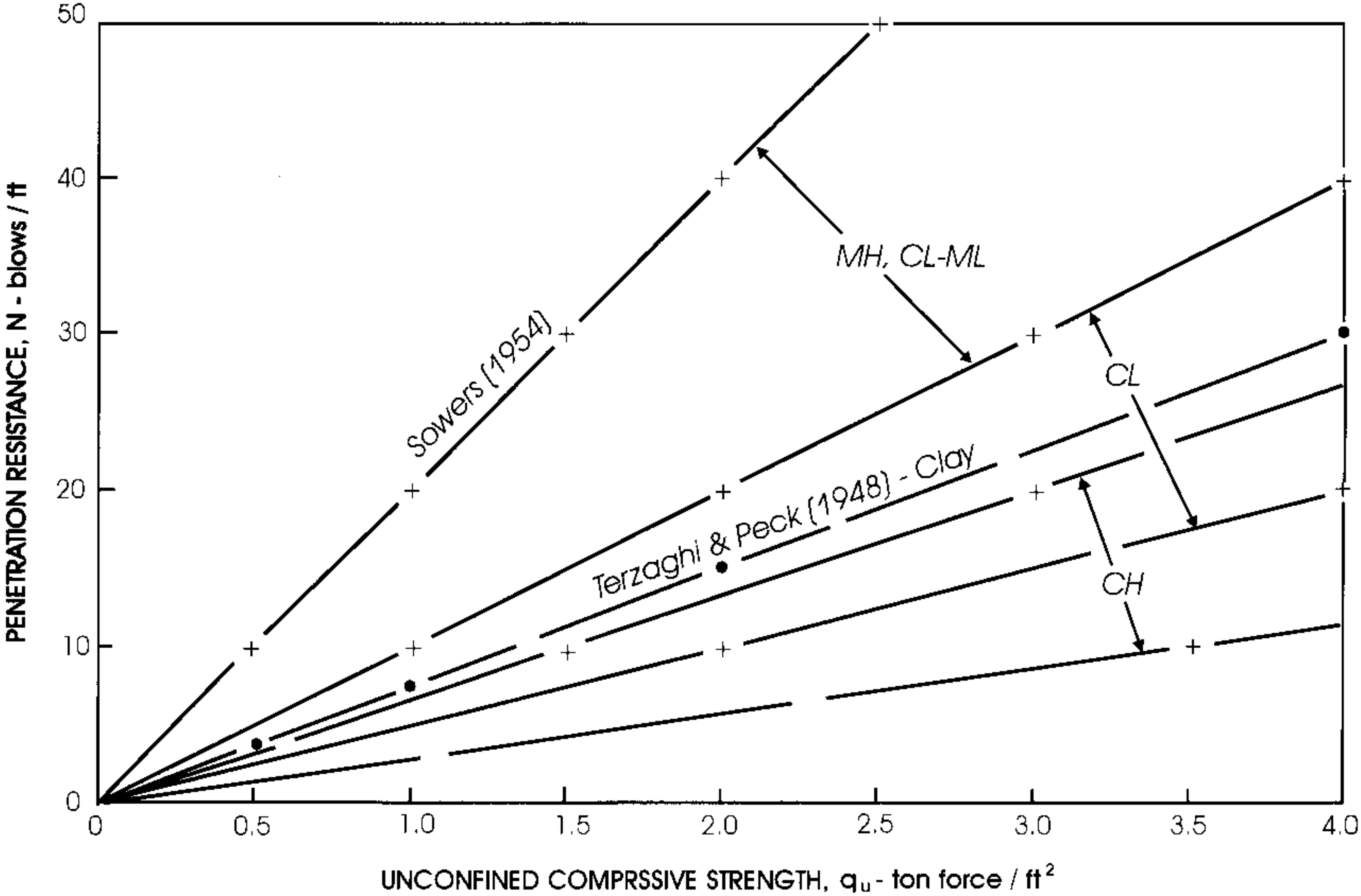


Figure 1-16.—Unconfined compressive strength versus penetration resistance, *N*, for various soils.

quantity of soluble solids is determined using the procedure described in USBR 5450.[13] The effect of soluble solids on an earth structure depends on:

- Temperature,
- Minerals present in the soil and their solubility characteristics in water,
- Coefficient of permeability and, thus, the amount of water passing through the soil,
- Chemical characteristics of the water, and
- Other factors.

Therefore, the percentage of soluble solids is only an indication of possible effects. Soluble solids are more objectionable in materials having moderate to high permeability than in soils with low permeability.

The kinds of soluble minerals present in a soil or carried by groundwater are important when considering the type of cement to be used in constructing concrete structures in contact with soil. The aggressive substances which affect concrete structures are the sulfates of sodium, magnesium, and calcium. The relative degrees of attack on concrete by sulfates from soil, groundwater, and other substances are discussed in Reclamation's *Concrete Manual* [27]. Soluble minerals present in soil and groundwater must be determined by chemical analyses. The presence of appreciable quantities of soluble minerals indicates that engineering properties may change in the presence of percolating water.

C. Engineering Properties

1-20. General.—In evaluating a soil, the transition is gradual between those properties that serve only as broad guides to the character of material and quantitative properties which define specific performance characteristics. For example, moisture content and dry density are at times used as index properties and at other times used as engineering properties. The importance of the two properties in investigation and construction control, as indicators of the nature of material and of the quality of compaction, has frequently resulted in the belief that low moisture content and high dry density are the only desirable characteristics to attain in soils. This is not necessarily true. These properties are only indexes of the probable engineering behavior of a soil. Yet, both dry density and moisture content are so intimately a part of computations involved in design of structures of soil that they must frequently be evaluated as engineering properties.

The principal distinction between index and engineering properties is that the procedures for determining index properties are relatively simpler than those for determining engineering properties, which require considerable knowledge and skill to develop reliable information. This does not imply index properties tests are simple to perform. Skill and care are required when performing index tests. Considerably more special knowledge and skill are required to interpret and apply index information than are required to use information from tests for engineering properties. The most commonly listed soil engineering properties are strength, volume change, and permeability. In addition, but less clearly defined, are such characteristics as deterioration and workability for which quantitative evaluation is invalid.

Selecting samples and specimens for both index and engineering properties testing requires considerable thought to ensure materials selected for testing are representative of materials at the structure site. Also, careful sample and specimen selection are required to obtain maximum information from the testing program at minimum cost.

1-21. Shear Strength.—

*a. General.—*Soil has little strength compared to other materials used for building a structure. Moreover, compared to maximum soil strength, large variation can exist in strength, both from soil to soil and within a given soil type, depending on how it was deposited or placed.

Engineering computations using soil strength deal primarily with shear strength, the resistance to sliding of one mass of soil against another, and rarely with compressive or tensile strengths. In 1776, C.A. Coulomb [28], a French engineer and scientist, observed that the shear strength of a soil consisted of two parts: (1) one part dependent on the stress acting normal to the shear plane, (2) the other part independent of that stress. The parts were called internal friction and cohesion, respectively. The factor relating the

[13] 5450: Determining Water Soluble Solids in Soils.

normal component of stress to shear strength was designated by tan ϕ and the unit of cohesion by *c*. The shear strength, *s*, may then be expressed as:

$$s = c + \sigma \tan \phi \qquad \textbf{1-16}$$

where:

- c = unit of cohesion
- s = shear strength
- σ = normal stress on the sliding surface
- ϕ = angle of internal friction

Many soils are either predominantly cohesive or noncohesive. For either type of soil, engineering computations can be simplified by dropping the smaller term in the shear strength equation, and the resulting solution is conservative. Much of early soil mechanics practice was based on this simplified assumption. The terms cohesive ($\phi = 0$) and noncohesive or cohesionless ($c = 0$) soils are still in common use in referring to these soils.

Occasionally, a structure is so massive or the soil foundation is so weak that this simplifying assumption cannot be used. For a large earth embankment dam, this is almost always the case. In such a situation, determining the values of *c* and ϕ with as much precision as possible is most important for both the proposed structure and foundation under a variety of probable loading conditions. It is extremely important that "undisturbed" samples from the foundation, secured during investigations, be truly representative of materials and conditions. Also, material placed in these earth structures (during construction) should comply with established design limitations, based on laboratory tests.

b. In Situ Shear Strength.—*In situ* shear strength estimates can be derived from:[14]

- standard penetration test—USBR 7015
- vane shear test—USBR 7115
- cone penetration tests—USBR 7020 and 7021
- borehole shear device test

Each field test has been used successfully. *In situ* shear strength has been estimated for a variety of soils using appropriate computations and empirical correlations. The Menard Pressuremeter and the flat blade dilatometer, as well as various other equipment and methods, have been used to determine modulus values of soils.

c. Direct Shear.—In the laboratory, shear strength measurement is accomplished either by using the direct shear test or the triaxial shear test. The direct shear test, USBR 5725,[15] was developed first and is still used widely. It has the advantage of simplicity, but disadvantages include stress and strain concentrations within the specimen, and pore-water pressures cannot be monitored during the test.

Usually, the specimen is tested under consolidated-drained conditions. The specimen is allowed to fully consolidate under each applied normal pressure and is sheared at a rate slow enough to allow pore pressures to remain at equilibrium during shear. Therefore, test results are reported in terms of effective stress.

d. Triaxial Shear.—The triaxial shear test apparatus was developed to permit control of other factors that influence shear strength, which cannot be evaluated with the direct shear test apparatus. Originally used as a research tool, the triaxial device was instrumental in better understanding the mechanics of shear in soils. Because of its capability to simulate a wide range of test conditions, the triaxial shear apparatus is now used for routine testing. The triaxial shear apparatus is used to perform four different test procedures.[16] The procedure used to test a soil must be based on field loading and drainage conditions to which the soil will be subjected. Figure 1-17 indicates the relation between relative density and the angle of internal friction, ϕ (also expressed as tan ϕ), for compacted coarse-grained soils [24, 29].

e. Pore-Water Pressure.—Shear strength is primarily dependent on effective normal stress. Under an externally applied stress, soil grains are forced into more intimate contact, and the soil mass volume decreases. Because soil grain volume cannot be changed appreciably, this volume change must take place primarily in the soil voids or pores. If these pores are completely filled with water, their volume

[14] 7015: Performing Penetration Resistance Testing and Sampling of Soils.
7020: Performing Cone Penetration Testing of Soils - Mechanical Method.
7021: Performing Cone Penetration Testing of Soils - Electrical Method.
7115: Performing Field Vane Shear Testing.

[15] 5725: Performing Direct Shear Testing of Soils.

[16] 5740: Determining Lateral Earth Pressure (k_o) by the Triaxial Test Method.
5745: Performing Unconsolidated-Undrained Triaxial Shear Testing of Soils.
5750: Performing Consolidated-Undrained Triaxial Shear Testing of Soils.
5755: Performing Consolidated-Drained Triaxial Shear Testing of Soils.

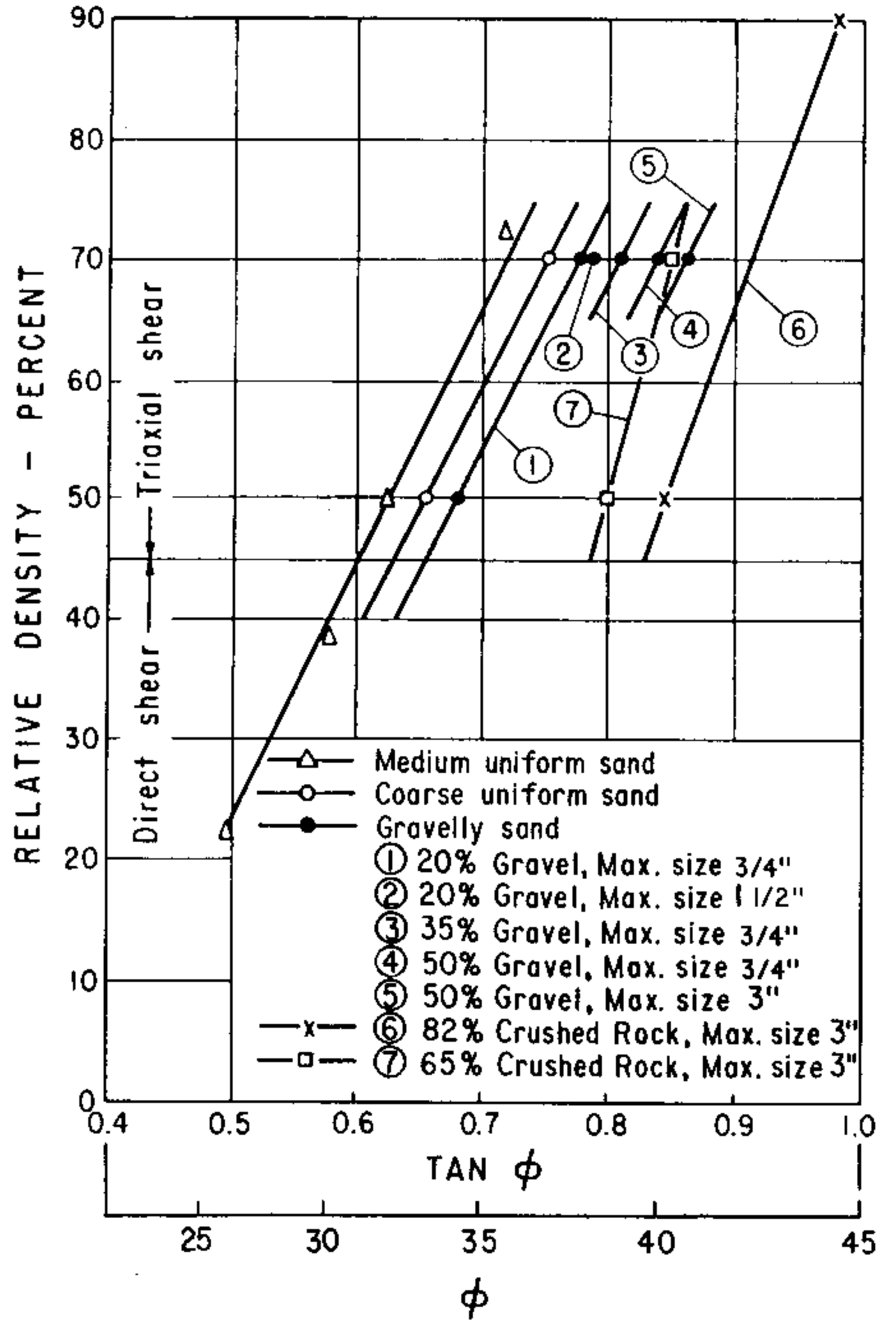


Figure 1-17.—Effect of relative density on the coefficient of friction, tan ϕ, for coarse-grained soils [24, 28].

cannot be changed unless some water is drained from the soil mass, because water is considered incompressible. If drainage is prevented or impeded, stress will develop in the pore water opposing the externally applied stress. The developed stress is called excess pore-water pressure (i.e., pore-water pressure in excess of hydrostatic conditions). Even if the pores are filled only partially with water, pressures in the fluid combination of air and water will develop to a lesser degree, because volume change is possible (air is compressible), and additional stresses can be carried by the soil grains. However, a difference will exist between pore-air pressure and pore-water pressure resulting from capillary tension or capillary stress of the water films. Therefore, when analyzing shear strength of unsaturated soils, consideration must be given to whether pore-air or pore-water pressure is used. Because pore pressures are opposed to the normal stress, total normal stress will be reduced whenever positive pore-water pressure is present. Based on this observation, the general case Coulomb equation must be rewritten:

$$s = c' + (\sigma - u) \tan \phi' \qquad \textbf{1-17}$$

where:

- s = shear strength
- u = pore fluid pressure
- c' = effective cohesion
- σ = normal stress on the sliding surface
- ϕ' = effective angle of internal friction

The analysis can be made considering u as either pore-air pressure or pore-water pressure, depending on the application. Figure 1-18 shows the effect of pore fluid pressure on the shear characteristics of a lean clay. True strength characteristics of a soil are determined only when pore fluid pressures are accounted for during laboratory testing.

f. Capillary Stress.—When the water in a soil does not completely fill the voids, a surface develops between the water and air. This surface is not flat, but is curved due to surface tension. The degree of curvature depends on:

- size of the void,
- nature of the material forming the sides of the void, and
- kind of liquid in the voids.

Because the surface is curved, it is stressed in tension; this tensile stress is imparted to the liquid in the pores. The effect of tensile stress is to pull the soil particles together. The action is contrary to pore-water pressure as described in the previous paragraphs, and it can influence the value of cohesion determined by laboratory tests.

Capillary stress is present when soil is not completely saturated and may have a significant influence on results of laboratory tests. The presence of capillary stress results in a higher shear strength for a soil than will exist when capillary stress is lessened or eliminated by wetting or saturation.

Laboratory test methods for measuring capillary stress are described in USBR 5735.[17] With information from the capillary test, a triaxial shear test can be analyzed either on the basis of pore-air pressure, which will indicate the shear strength when it is influenced by capillary stress, or on the basis of pore-water pressure, which does not include the influence of capillary stress and which can represent the condition of saturation.

g. Sliding Resistance.—A special type of shear strength investigation involves the shear strength between dissimilar materials, commonly between soil and concrete or between soil and geosynthetic materials. Usually, this type of shear is identified as sliding resistance or interface friction. The nature of the problem makes it necessary to investigate this case by direct shear test methods.

h. Residual Shear Strength.—Some soils exhibit brittle stress-strain behavior during shear. Materials such as shale, indurated clay, overconsolidated clay, or stiff fissured clay reach maximum shear stress after extremely small shear displacements. Studies indicate measured shear stress in these materials decreases rapidly with increasing shear strain beyond the point of maximum shear stress. Continued displacement beyond maximum shear stress will reduce the measured shear stress to a low constant value termed the "residual shear stress" [30] (fig. 1-19). Residual shear strength can be evaluated using either USBR 5726 or USBR 5730.[18]

1-22. Volume Change.—

a. General.—Volume change in a soil mass caused by both natural and artificial conditions introduces problems peculiar to soils and not encountered with other construction materials. Volume decrease is caused by pressure increase and is a function of time and permeability; it is associated with changes in moisture and air content, and it can occur as a result of compaction. Volume increase is a function of:

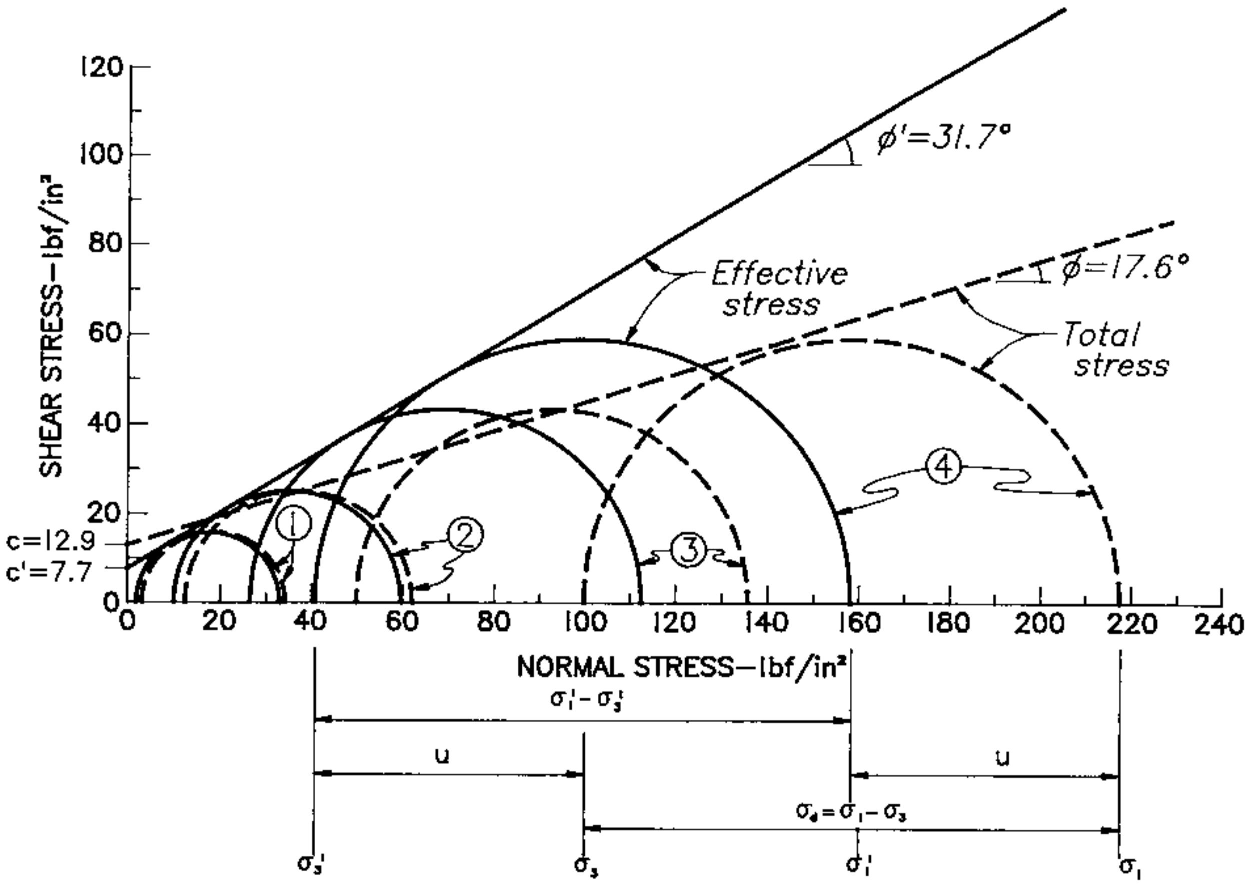
- pressure,
- density,
- moisture content, and
- soil type.

Terms that describe volume change phenomena are:

- *Compression*—Volume change as a result of elastic deformation or expulsion of air produced by application of a static external pressure.

[17] 5735: Determining Initial Capillary Pressure of Soils.

[18] 5726: Performing Repeated Direct Shear Testing of Soils.
5730: Performing Rotational Shear Testing of Soils.



SPECIMEN NUMBER	1	2	3	4
APPLIED LATERAL STRESS, σ_3 (lbf/in²)	3.1	12.5	50.0	100.0
DEVIATOR STRESS, $\sigma_d = \sigma_1 - \sigma_3$ (lbf/in²)	31.3	49.7	86.0	117.5
PORE FLUID PRESSURE, u (lbf/in²)	1.1	3.3	22.8	59.7
MAJOR PRINCIPAL STRESS, $\sigma_1 = \sigma_3 - \sigma_d$ (lbf/in²)	34.4	62.2	136.0	217.5
EFFECTIVE LATERAL STRESS, $\sigma_3' = \sigma_3 - u$ (lbf/in²)	2.0	9.2	27.2	40.3
EFFECTIVE MAJOR PRINCIPAL STRESS, $\sigma_1' = \sigma_3' + \sigma_d$ (lbf/in²)	33.3	58.9	113.2	157.8

EFFECTIVE STRESS SHEAR STRENGTH, $\phi' = 31.7°$ c' = 7.7 lbf/in²
TOTAL STRESS SHEAR STRENGTH, $\phi = 17.6°$ c = 12.9 lbf/in²
SPECIMEN PLACEMENT DATA (AVERAGE OF FOUR SPECIMENS)

DRY UNIT WEIGHT, δ_d (lbf/ft³)		114
MOISTURE CONTENT, w (%)		14
VOID RATIO, e		0.50
INITIAL DEGREE OF SATURATION, S (%)		74
USCS CLASSIFICATION	GROUP SYMBOL	CL
	GROUP NAME	LEAN CLAY

Figure 1-18.—Results of a consolidated-undrained (CIU) triaxial shear test on lean clay with and without considering pore pressure.

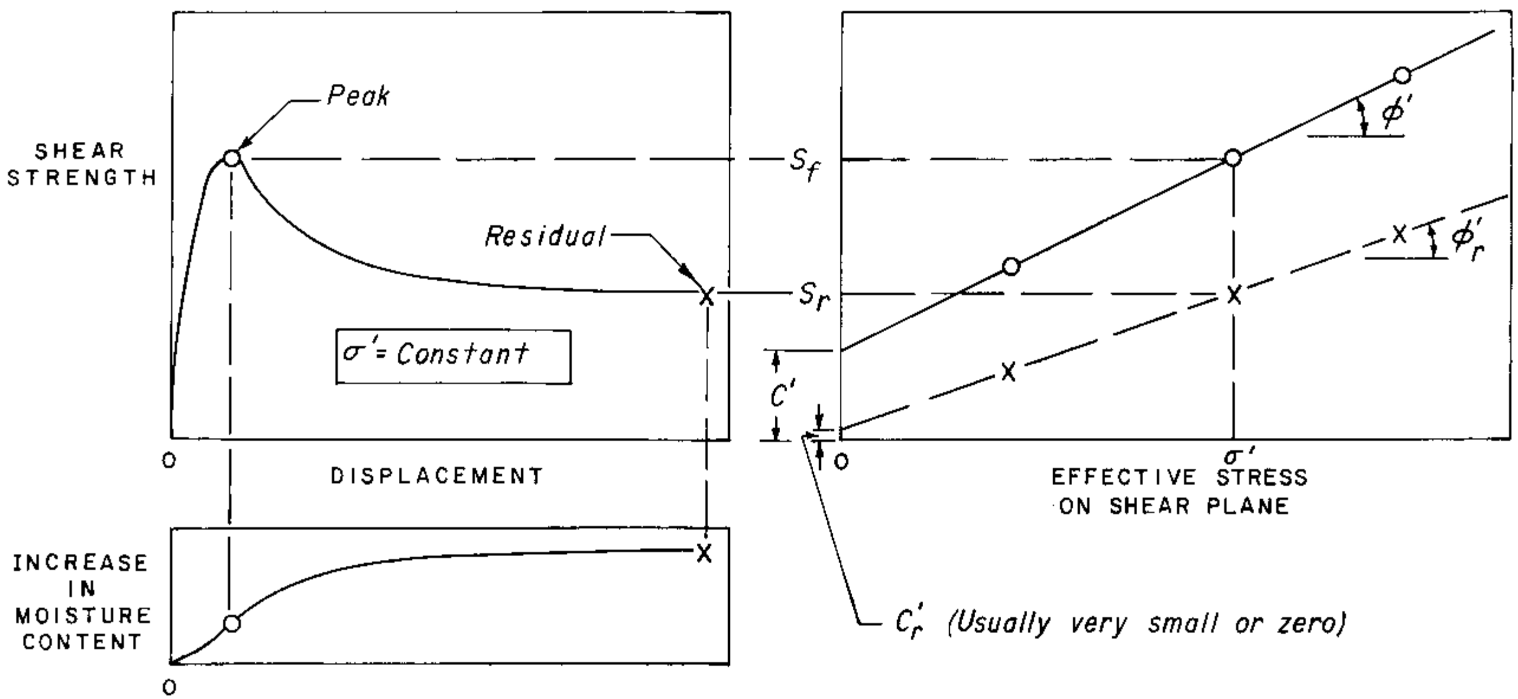


Figure 1-19.—Peak and residual shear strength and resultant angles of internal friction [29].

- *Consolidation*—Volume change produced by application of static external pressure and achieved through pore-water movement with time.
- *Shrinkage*—Volume change produced by capillary stresses during drying of a soil.
- *Compaction*—Volume change produced by mechanical manipulation such as rolling, tamping, or vibrating.

The above terms apply to volume reduction. Corresponding terms apply to volume increase:

- *Rebound* as opposed to compression.
- *Expansion* as opposed to consolidation.
- *Swell* as opposed to shrinkage.
- *Loosening* or *scarifying* describes the operation opposite of compaction.

Ground surface movement is often the result of volume change in the underlying soil. Terms describing ground surface movement are:

- *Settlement*—Ground surface movement associated with volume decrease.
- *Heave*—Ground surface movement associated with volume increase.

Most often, volume change is associated with changes in volume of the voids and only to a limited extent with changes in volume of solid particles. If voids are, to a large extent, filled with air, an increase in pressure on the soil mass will result in compression without appreciable subsequent consolidation. Conversely, if voids are nearly or completely filled with water, little or no compression will take place immediately upon application of a pressure increment; and only when water drains from the soil mass can consolidation occur. If water can drain readily from the soil mass, consolidation may take place within a short period of time; but, if the soil has a low permeability, if the soil mass is extremely large, or if drainage is otherwise impeded, complete consolidation may require many years.

Volume change also is caused by particle rearrangement, particle breakdown, and physical or chemical absorption of moisture. Usually, particle rearrangement is associated with clay soils deposited underwater that have been stressed only with the weight of soil above them. Particle breakdown is most commonly found where residual soil is derived from rock that has been weathered and altered in place. Most clay soils have affinity for moisture which can be removed only with considerable effort. Fortunately, many of the clay minerals attain a state of saturation without great volume change; but a few (e.g., montmorillonite) will absorb or release large volumes of water and experience large shrinkage and swell.

b. Control of Compressibility.—By far, the most frequently occurring problem the geotechnical engineer must deal with involves compressibility. Some spectacular failures have occurred because of compressibility, but the most commonly observed effect is the cracking of structures. Shear strength is affected indirectly because the greater the soil's compressibility, the greater the potential for high pore pressures. Figure 1-20 shows the variation of compressibility with soil type for compacted earth embankments [31].

Although the potential for high compressibility is usually associated with fine-grained, highly plastic soils, a number of procedures have been developed to treat soil to minimize compressibility. In constructed fills, for most soil types, sufficient compaction can be applied to limit compression to a few percent. Where it can be applied, moisture control tends to reduce future consolidation by facilitating compaction. For some soils compacted dry of optimum moisture content, the soil grain structure will not assume its densest state as discernible from compaction curves shown on figure 1-10c. For such a condition, subsequent wetting may result in particle rearrangement and an accompanying volume strain called *saturation collapse*. This action can occur rapidly in some silty type soils.

Soils subject to shrinkage and swell can be used in earth structures if compacted under good moisture control and loaded sufficiently with other materials to prevent swelling. When swelling-type soils are used for the majority of an embankment, flatter slopes and greater soil volumes are required than for nonswelling soils. This may require more stringent zoning of an embankment, and consideration must be given to acquiring better materials from more distant sources as opposed to using swelling-type soils found locally.

Compressibility in a soil foundation is difficult to control. Removing compressible soil, using piles or piers, and use of spread footings or mats has been applied in specific cases with success. Variation in compressibility within the construction area can produce greater difficulty than the amount of compressibility encountered. Under earth dams, removal of questionable materials, drainage of wet soils, wetting of dry soils, and spread embankments have all been used to minimize compressibility. Compressible foundations under proposed buildings and similar structures

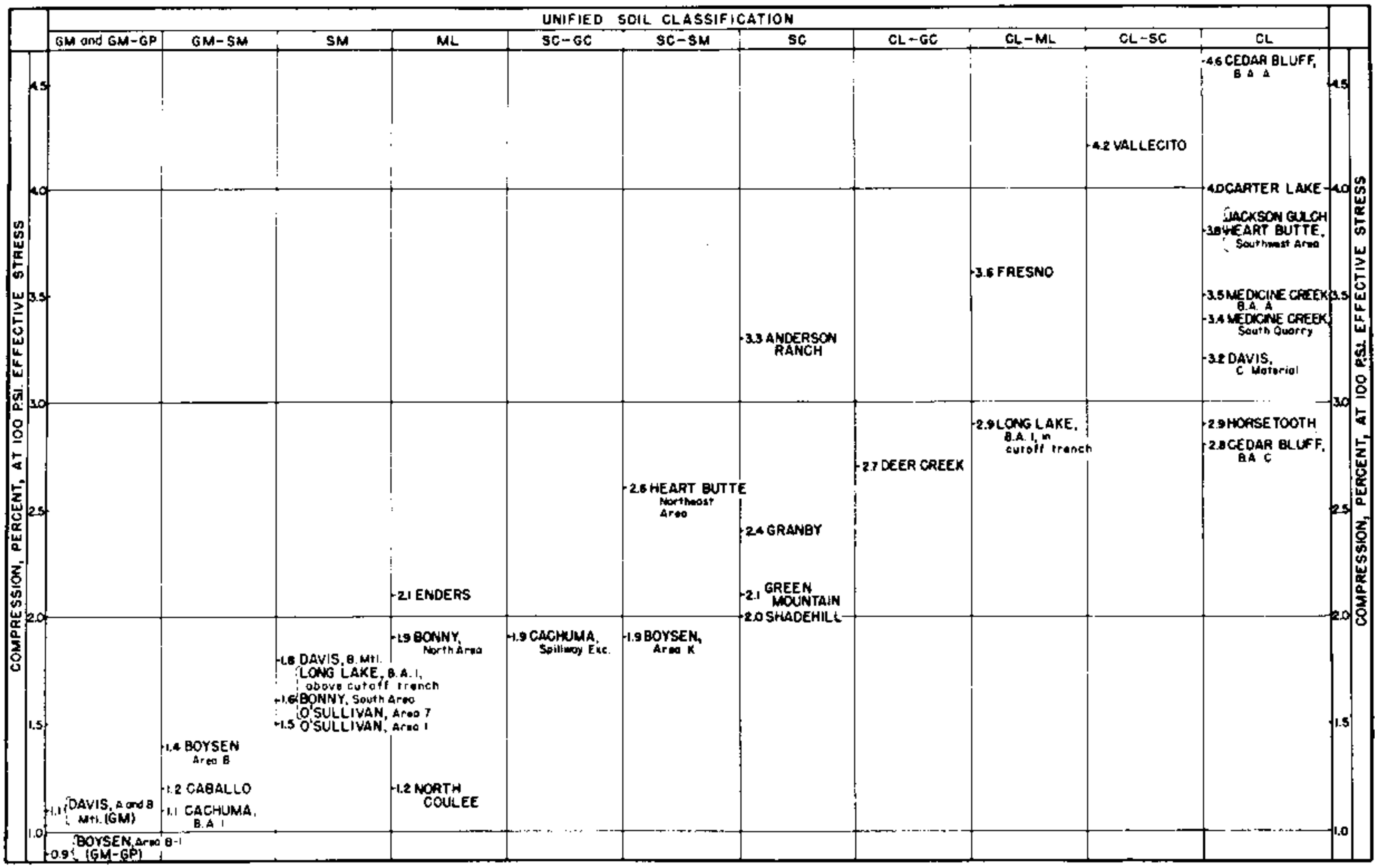


Figure 1-20.—Compression characteristics of various compacted embankment soils based on field measurements [31].

are frequently improved by removing an amount of soil equal to the structure's weight so there is no net increase in stress in the natural soil after the structure is built.

c. Pressure-Compression Characteristics.—When soils are gradually deposited layer upon layer, each layer is compressed by the layers above it; and, as time passes, the soil attains a state of consolidation in equilibrium with the superimposed pressure. Such a soil is described as *normally consolidated*. In such a soil, the density increases with depth or overburden pressure, and the void ratio decreases correspondingly. This relationship is nearly a straight line (virgin compression line) when plotted on a semilogarithmic scale (fig. 1-21 [32]). If all or a portion of the superimposed pressure is removed, rebound will occur, but the change in void ratio usually will be small compared to the void ratio change produced by initial compression and consolidation. Such a soil is *overconsolidated*. If an overconsolidated soil is again loaded, the change in void ratio will be small compared to that produced with similar pressure increments during initial consolidation. The final void ratio, however, will be somewhat smaller than that originally obtained. Where such a condition exists, either as a result of a glacier having overridden the deposit—where extensive erosion has occurred—or where desiccation has occurred, these materials often can function as a foundation without difficulty.

In the arid and semiarid parts of the Western United States, unsaturated deposits of loose silt (ML), silty clay (CL-ML), or lean clay (CL) have caused special problems [33]. These deposits include wind-deposited loess and loess-like soils and colluvial and alluvial soils deposited by flash flood runoffs, often in the form of mudslides. In these deposits, soils have never been completely wetted to allow breakdown of the loose depositional structure. Generally, these soils have relatively high dry strength created by well dispersed clay binder [34]. It is known that loessial soils form high, near-vertical faces that are stable as long as the moisture content is low (fig. 1-22). However, upon wetting, strength is essentially lost and slope failures occur. Similarly, loessial soils support heavy structural loads on footings or piles when dry but lose their bearing capacity and resistance to compression when their loose structure collapses upon wetting (fig. 1-23 [35]). Certain interfan soil deposits adjacent to the southwestern foothills of the Central Valley of California as well as other areas in the Western United States with similarly formed deposits have similar characteristics. When dealing with hydraulic engineering works, where subsoils eventually will become saturated, recognizing such soil deposits is important so that measures can be taken to improve them before building structures upon them.

Methods have been designed to identify these metastable soils [36]. Certain relationships exist between in-place dry density, moisture content, and the volume change that may be anticipated for fine-grained soils. The relationship in which dry density is expressed as the ratio of in-place dry density to laboratory maximum dry density for various moisture contents, presented in relation to optimum moisture content, is shown on figure 1-24a ([37]). If the dry density ratio and moisture content difference of a soil plot above and to the left of the limit line (shown on fig. 1-24a), very small additional volume change will occur upon wetting, and treatment of in-place material is not normally required for small dams, canal embankments, or lightweight structures. If the dry density ratio and moisture content difference plot below and to the right of the limit line, significant volume change may occur on wetting of the in-place material even under low pressures; and treatment of in-place materials may be required. Rigid structures should not be placed on either wet loose soils or dry loose soils, subject to later wetting, as structure cracking may occur. Foundation treatment may be required for rigid structures whose foundation soil properties plot above the limit line.

Another criterion for predicting loose, fine-grained soils needing treatment involves in-place dry density and liquid limit as shown on figures 1-24b and 1-25 (from [37]). The theory is simple: if in-place dry density is so low that the volume of voids is larger than that required to hold the volume of water needed to reach the liquid limit (as shown for case I), the soil can become saturated. Consistency is low, and the void space is sufficient to allow collapse. Conversely, if in-place dry density is high enough so that the volume of voids is less than that required to hold the volume of water needed to reach the liquid limit (as shown for case III), the soil will not collapse upon saturation but will reach a plastic state in which particle-to-particle contact always will take place. Therefore, soils with liquid limits and in-place dry densities that plot above the line show a critical dry density condition, while soils that plot below the line do not. In the latter condition, soils would only settle in a normal manner due to a pressure increase. The graph has additional usefulness in presenting data so the quality of denseness of soils in the case III category can be evaluated. For example, dry densities and liquid limits plotted close to the case II line may not be susceptible to collapse, but may be critical with respect to pressure increases; whereas soils with conditions plotted lower in the case III area would result in more firm material. In the case of expansive soils, with high liquid limits and very high in-place dry densities, plotting very low on the graph indicates susceptibility to future expansion. Therefore, in case III, a moderate range of dry densities exists that is most desirable.

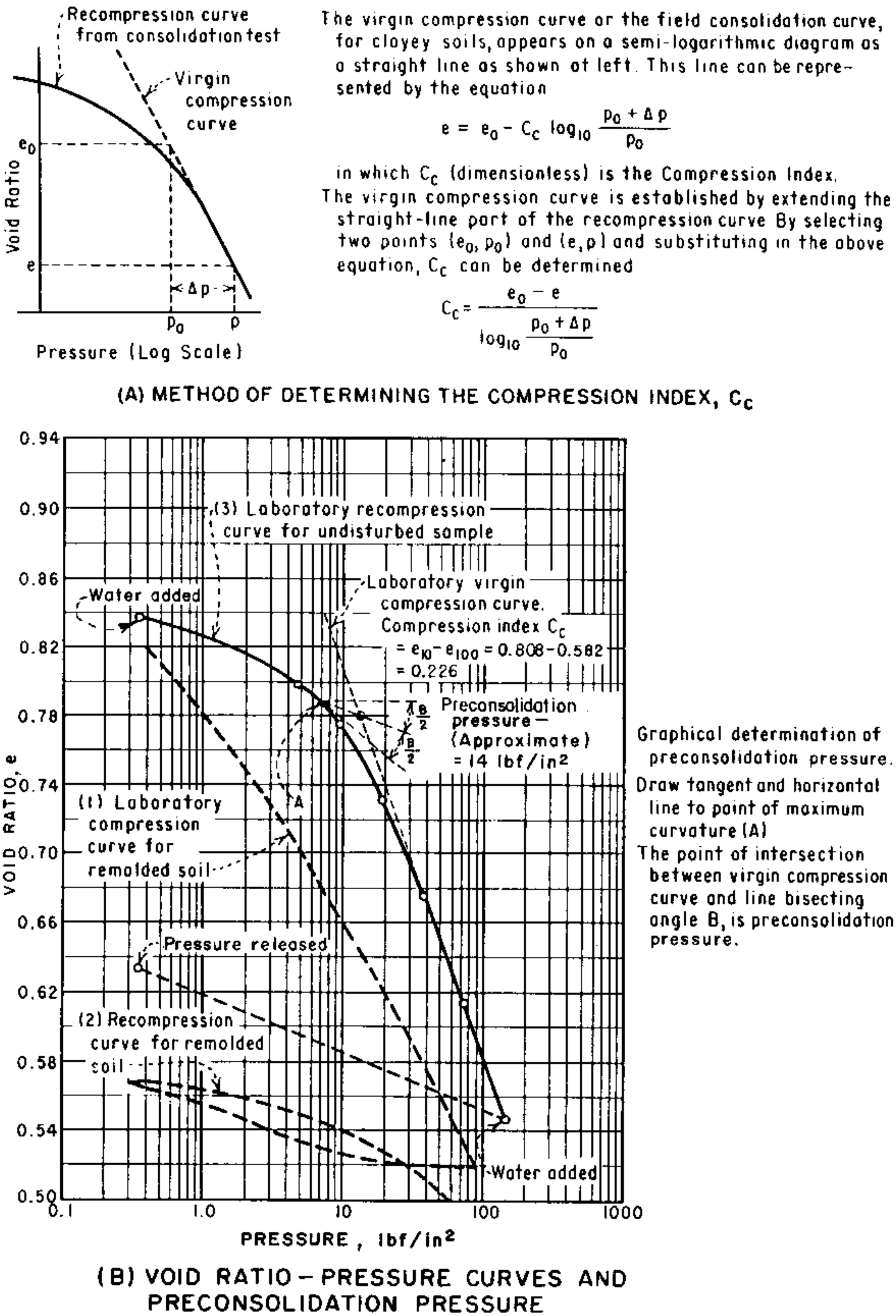


Figure 1-21.—Void ratio-pressure curve, compression index, and preconsolidation pressure [31].

Figure 1-22.—A 34-foot cut on 1/2:1 slope in loess formation—
Franklin Canal, Nebraska.

Loessial soils in Kansas and Nebraska are quite uniform and have liquid limits between 30 and 40 percent. Therefore, in-place dry density and moisture content versus settlement upon wetting can be expressed in terms of in-place dry density and moisture content for these localities. Soils from other areas must be tested.

For loessial soils in Kansas and Nebraska:

- Loess with dry density less than 1,281 kg/m^3 (80 lb/ft^3) is considered loose and highly susceptible to settlement on wetting with little or no surface loading.
- Loess with dry density between 1,281 and 1,442 kg/m^3 (80 and 90 lb/ft^3) is medium dense and is moderately susceptible to settlement on wetting when loaded.
- Loess with dry density above 1,442 kg/m^3 (90 lb/ft^3) is quite dense and capable of supporting ordinary structures without serious settlement, even upon wetting.
- For earth dams and high canal embankments, a dry density of 1,362 kg/m^3 (85 lb/ft^3) has been used as the division between high, dry density loess requiring no foundation treatment and low, dry density loess requiring treatment. However, considerable compression may still occur, and defensive design techniques such as wide filters and drains may be required in the embankment.
- Generally, moisture contents above 20 percent will permit full settlement under load.

Where volume change is a potential problem, a foundation investigation must provide information not only as to soil types found, but also information on their present undisturbed state. Samples must be recovered in an "undisturbed" state using methods described in either USBR 7100 or 7105.[19]

[19] 7100: Obtaining Undisturbed Block Samples by the Hand and Chain Saw Methods.

7105: Performing Undisturbed Soil Sampling by Mechanical Drilling Methods.

5600: Determining Permeability and Settlement of Soils 18-in (203-mm) Diameter Cylinder.

5605: Determining Permeability and Settlement of Soils Containing Gravel.

5610: Determing Permeability of Soil by the Back-Pressure Test Method.

5700: Determining the One-Dimensional Consolidation Properties of Soils (Incremental Stress).

5705: Determining the One-Dimensional Expansion Properties of Soils.

5715: Determining the One-Dimensional Uplift Properties of Soils.

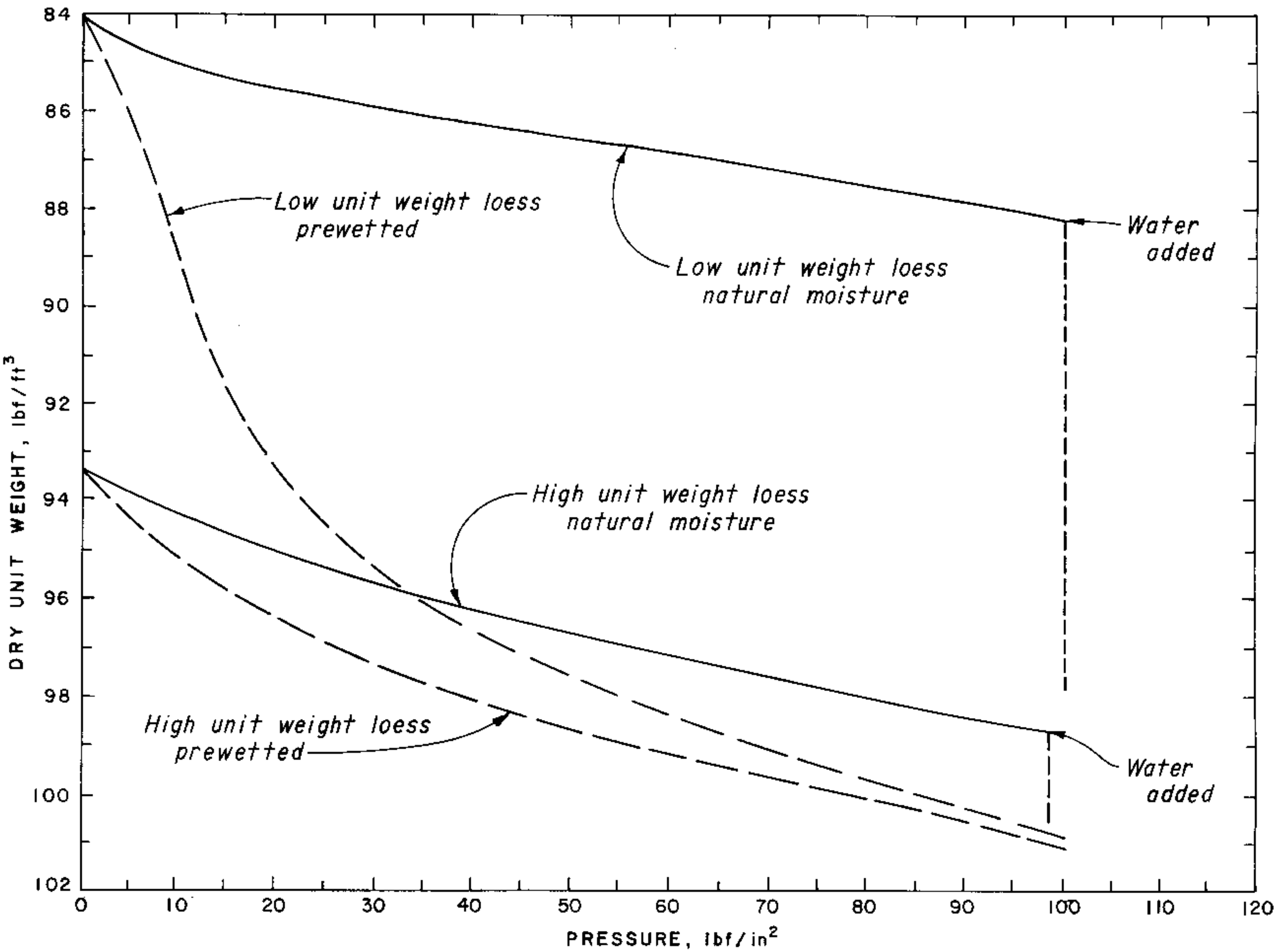


Figure 1-23.—Effect of loading and wetting high and low unit weight loess soil foundations [34].

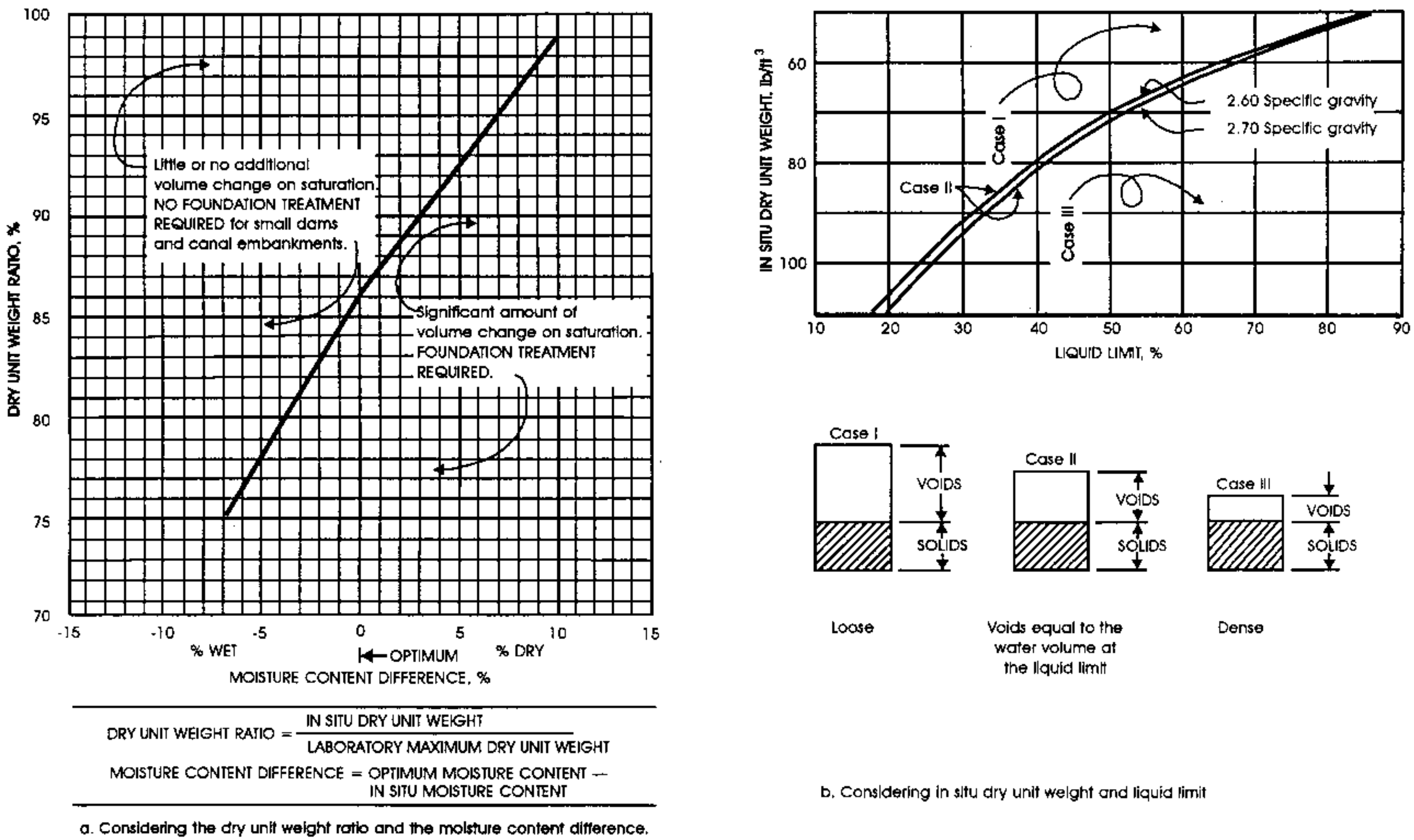


Figure 1-24.—Criterion for treatment of relatively dry fine-grained foundations.

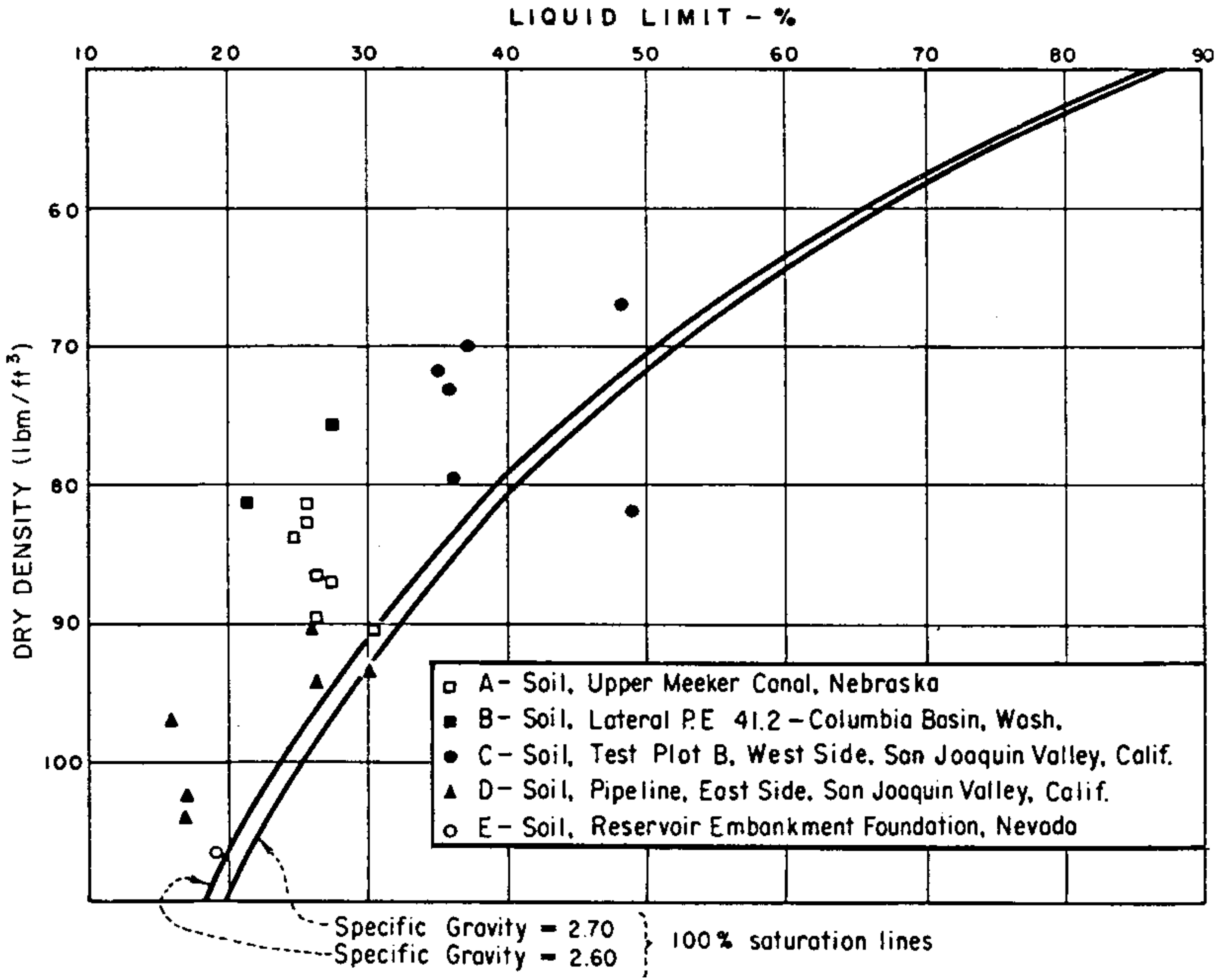


Figure 1-25.—Assessing collapse potential of various soils using dry density and liquid limit [36].

Laboratory consolidation testing of undisturbed or compacted soil specimens is performed using USBR 5700. Permeability and settlement of compacted soil specimens may be determined using USBR 5600, 5605, or 5610. Other volume change characteristics of soils (e.g., expansion, swell, or heave) may be determined using USBR 5705 or 5715. Three-dimensional volume change characteristics of soils may be obtained using consolidation procedures given for triaxial shear tests (see USBR 5750 or 5755). Figure 1-21 shows the type of compression results obtained from testing cohesive soil. Figure 1-26 shows the relation of relative density and void ratio to compressibility for a compacted, fine sand [38].

d. Pressure-Expansion Characteristics.—In addition to the normal rebound phenomenon, which occurs upon release of a compressive pressure as indicated on figure 1-21, certain types of clayey soils and clay shales exhibit expansive characteristics in the presence of water.

The amount of expansion depends on the type of clay mineral, confining pressure, and the availability of water; expansion is a function of:

- time,
- confining pressure,
- initial density, and
- initial moisture content.

Clays containing montmorillonite are the chief sources of difficulty. Since hydraulic structures always provide a water source for soil expansion, clays must be identified and treated to avoid future failures. Figure 1-27 shows a comparison between pressure and expansion for two typical expansive clays [39, 40]. Figure 1-28 shows effects of placement moisture content and dry density on expansion characteristics for a typical compacted expansive clay. Graph (a) shows volume change under a pressure of 7 kPa (1 lbf/in^2), and graph (b) shows total uplift pressure developed when this clay was restrained from expanding [41]. Figures 1-29 and 1-30 show canal lining failures caused by heaving of expansive soils. Table 1-2 gives criteria for identifying expansive clay [42].

An expansive clay can be modified by adding a small percentage of hydrated lime to the soil. Adding lime to soil has two major effects: (1) improves the workability and (2) increases the shear strength.

The first effect is immediate and results from the following reactions of lime with soil:

- Soil plasticity is immediately reduced (fig. 1-31). The liquid limit (LL) of the soil changes very little while the plastic limit (PL) increases, thus reducing the plasticity index (PI) of the soil.
- The finer clay size particles (colloids) agglomerate to form larger particles.
- The large particles (clay clods) disintegrate to form smaller particles.
- A drying effect takes place caused by absorption of moisture for hydration of the lime, which reduces the moisture content of the soil.

The result of these reactions is to make the material more workable and more friable.

The second effect of adding lime to soil is a definite cementing action with strength of the compacted soil-lime mixture increasing with time. The lime reacts chemically with available silica and some alumina in the soil to form calcium silicates and aluminates.

The percentage of lime added to a soil depends on whether the purpose is for modification (small percent to increase workability) or for stabilization (sufficient lime to provide strength). For stabilization, lime percentage can be based on:

- soil pH,
- plasticity index reduction,
- strength gain, and
- prevention of harmful volumetric change.

When the soil-lime mixture reaches a pH of 12.4, sufficient lime has been added to react with all the soil. An optimum lime percentage exists beyond which addition of more lime slightly reduces the plasticity index of the mixture but cannot be economically justified. If a minimum strength material is needed, enough lime can be added to obtain that strength, or enough lime can be added to increase the shrinkage limit to placement moisture or higher to prevent excessive volume change through wetting and drying cycles [43].

1-23. Permeability.—

a. Definition.—Voids in a soil mass provide not only the mechanism for volume change, but also passages for water to move through a soil mass. Such passages are variable in size, and the flow paths are tortuous and interconnected. If a sufficiently large number of such paths act together, an average rate of flow through a soil mass can

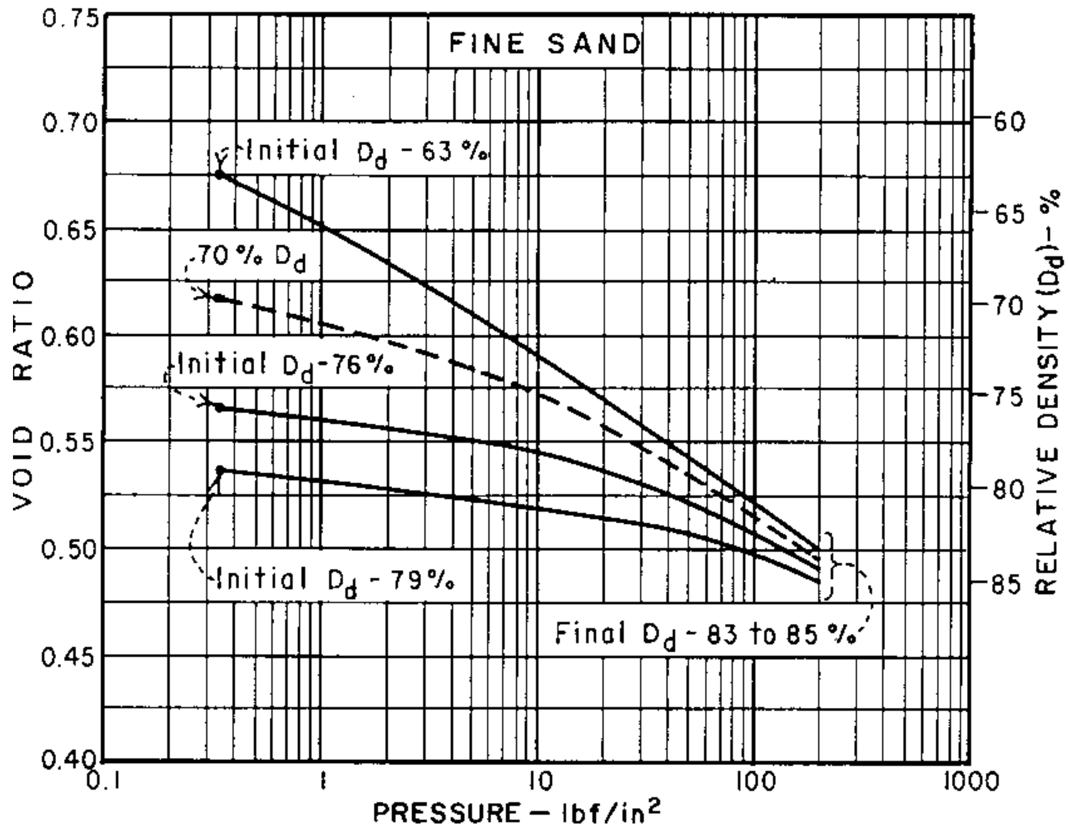


Figure 1-26.—Compressibility characteristics of a fine sand in relation to placement relative density [37].

be determined under controlled conditions that will be representative of larger masses of the same soil under similar conditions.

Under gravitational forces, water movement through soil is called *percolation* or *seepage*. The measure of this movement is called *permeability*; the factor relating permeability to unit conditions is the *coefficient of permeability*, *k*, defined as:

$$k = \frac{Q}{A}\left(\frac{L}{\Delta h}\right) = \frac{v}{i} \qquad \textbf{1-18}$$

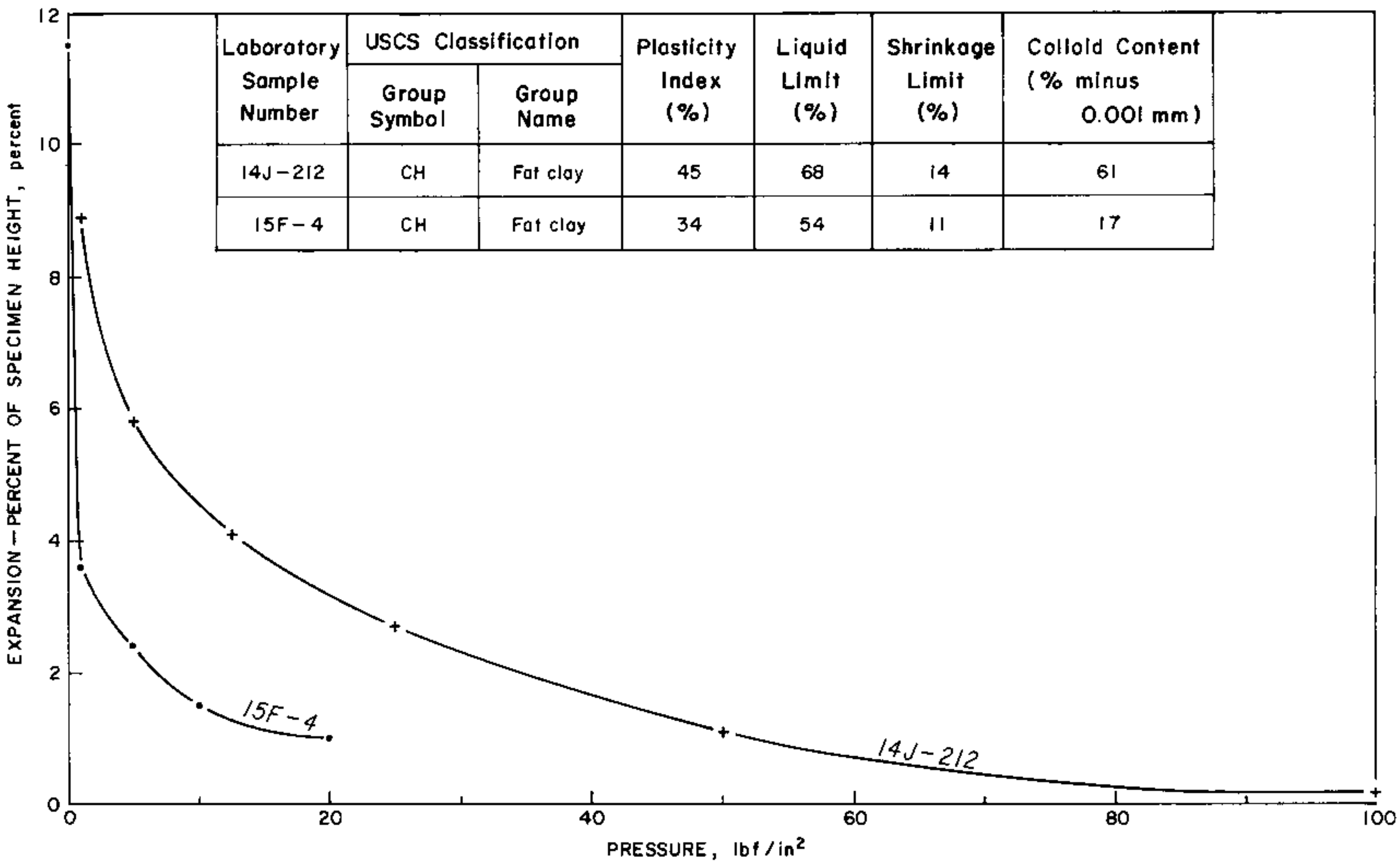
where:

- A = gross cross-sectional area through which Q flows
- Δh = head loss
- i = hydraulic gradient, equal to $\Delta h / L$, or the ratio of head lost to distance over which the head is lost
- k = coefficient of permeability
- L = distance over which head is lost
- Q = volume of water per unit time
- V = discharge velocity

This equation is commonly known as Darcy's law. Temperature and viscosity of water affect the coefficient of permeability; these factors are usually accounted for in permeability determinations by correcting test results to a standard temperature of 20 °C.

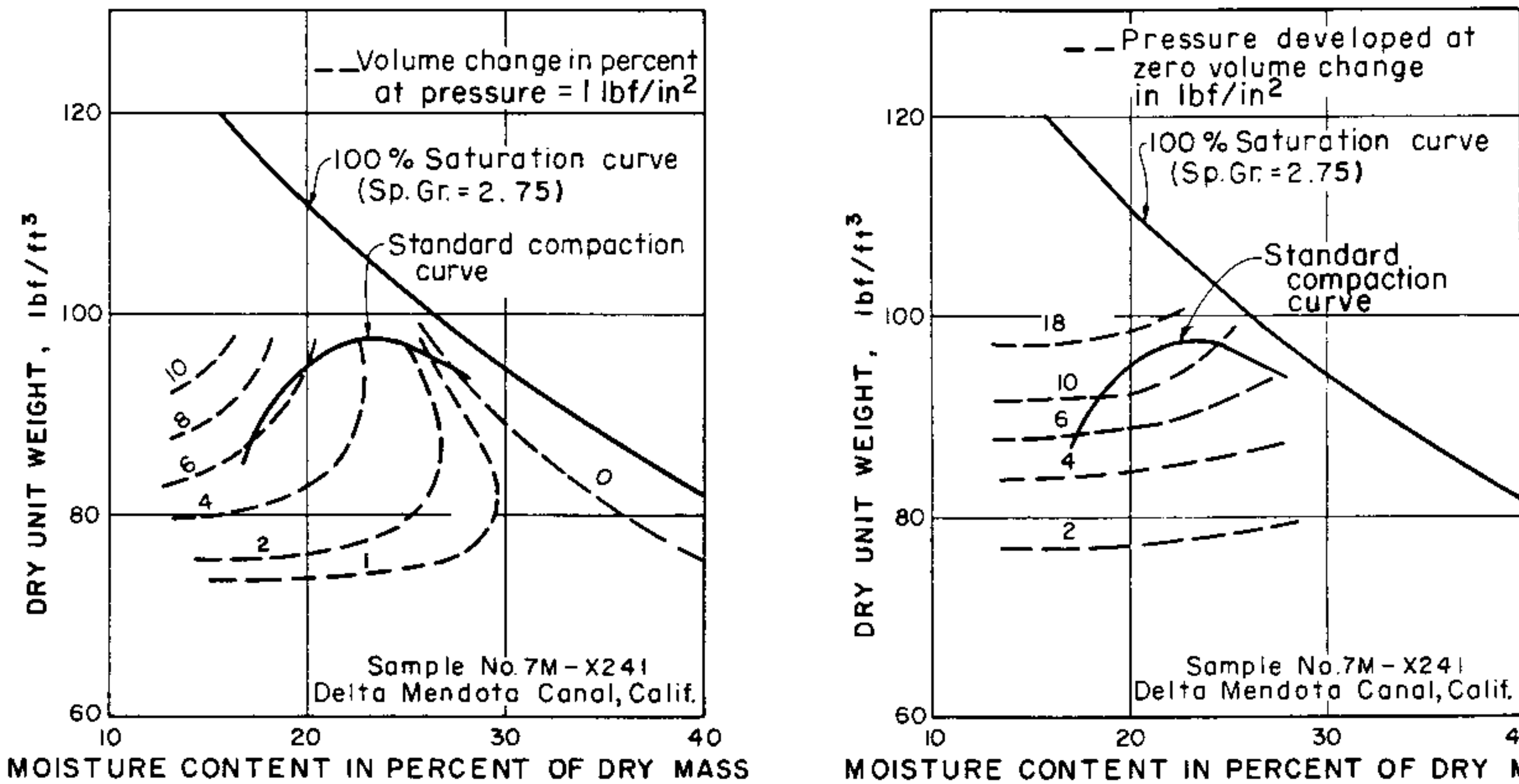
Many units of measure are used to express the coefficient of permeability. In the past, Reclamation preferred to use feet per year (ft/yr), which is the same as cubic feet per square foot per year ([ft^3/ft^2]/yr) at unit gradient. Feet per day (ft/d) is a term used to some extent in canal design, while water-supply engineers favor gallons per square foot per day ([gal/ft^2]/day). Most technical literature uses centimeters per second (cm/s) (fig. 1-32), and Reclamation will be converting to these units.

b. Ranges of Permeability.—The coefficient of permeability in natural soil deposits is highly variable. In many soil deposits, permeability parallel to bedding planes may be 100 or even 1,000 times larger than permeability



Laboratory Sample Number	USCS Classification		Plasticity Index (%)	Liquid Limit (%)	Shrinkage Limit (%)	Colloid Content (% minus 0.001 mm)
	Group Symbol	Group Name				
14J–212	CH	Fat clay	45	68	14	61
15F–4	CH	Fat clay	34	54	11	17

Figure 1-27.—Pressure-expansion curves for two typical expansive clays [38, 39].



a. PERCENTAGE OF EXPANSION FOR VARIOUS PLACEMENT CONDITIONS WHEN UNDER 1- P.S.I. LOAD

b. TOTAL UPLIFT PRESSURE CAUSED BY WETTING — FOR VARIOUS PLACEMENT CONDITIONS

Figure 1-28.—Typical expansive clay. Effects of placement moisture content and dry unit weight on expansion characteristics [40].

Figure 1-29.—Slope failure and bottom heaving of
Friant-Kern Canal, California.

Figure 1-30.—Heaving of canal bottom (about 300 mm at center),
Mohawk Canal, Arizona.

Table 1-2.—Relation of soil index properties to expansion potential of high-plasticity clay soils. Data for making estimate of probable volume change for expansive materials

Data from index tests[1]				
Colloid content, percent minus 0.001 mm	Plasticity index, PI, %	Shrinkage limit, SL, %	Probable expansion[2], percent total volume change, dry to saturated condition	Degree of expansion
> 28	> 35	< 11	> 30	Very high
20 to 31	25 to 41	7 to 12	20 to 30	High
13 to 23	15 to 28	10 to 16	10 to 20	Medium
< 15	< 18	> 15	< 10	Low

[1] All three index tests should be considered in estimating expansive properties.
[2] Based on a vertical loading of 7 kPa (1.0 lbf/in^2)

perpendicular to bedding planes. An exception to this is loess in which vertical permeability is several times greater than horizontal permeability [44]. Permeability of some soils is sensitive to small changes in density, moisture content, or gradation. In certain permeability ranges, a few percent variation in any one of these factors may result in a 1,000-percent variation in permeability. Because of possible wide variation in permeability, measurement with great accuracy is not possible for most design work; rather, the order of magnitude of permeability is important.

Reclamation describes soils with permeability less than 1×10^{-6}cm/s (1 ft/yr) as impervious, those with permeability between $1\text{x}10^{-6}$ cm/s and $1\text{x}10^{-4}$ cm/s (1 and 100 ft/yr) as semipervious, and soils with permeability greater than 1×10^{-4} cm/s (100 ft/yr) as pervious. Figure 1-32 shows a compilation of some of the permeability testing performed at the Reclamation soils laboratory (Denver, Colorado) on compacted specimens of various soils. Figure 1-33 shows results of permeability tests on relatively clean, sand-gravel mixtures [45].

c. Control of Permeability.—The determination of coefficient of permeability is important in water retention and water conveyance structures because water lost through the soil is an economic loss charged to the structure. Continuous movement of water through the soil of a structure may result in removal of soluble solids or may result in internal erosion called *piping*. Piping must particularly be guarded against because it occurs gradually and is often not apparent until the structure's failure is imminent. In nonhydraulic structures, permeability is important only when work below water table is required or when changes in water table occur in the area influenced by constructing the structure.

Seepage control is accomplished in a variety of ways that may be divided into three classes: (1) reducing the coefficient of permeability, (2) reducing the hydraulic gradient, and (3) controlling the effluent.

In embankments, permeability is reduced by selection of material through compaction control and on rare occasions by use of additives. Through foundations, seepage is reduced by several types of cutoffs formed by injection of a material into the foundation by grouting methods and densification by dynamic compaction, vibration, and loading often accompanied by changing the moisture content of the materials to hasten or enhance densification.

Hydraulic gradient reduction is accomplished either by reducing the head or increasing the length of the seepage path. Usually, some type of low permeability reservoir blanket is constructed to increase the seepage path.

Control of effluent requires a design that reduces seepage pressures sufficiently at all points so rupture of impervious zones or foundation layers is prevented and that provides filters to prevent piping. This is accomplished by zoning in earth dams and canal banks, using filters and drains, and installing drains and pressure relief wells. A number of combination control methods are in common use including wide embankment sections, impervious diaphragms, and linings. The control method selected depends primarily on cost of treatment compared to benefit received both in preventing water loss and in assurance against piping failure.

Detailed analysis and design techniques are presented in *Design Standards No. 13—Embankment Dams* ([46] chs. 2, 5, 8, 17).

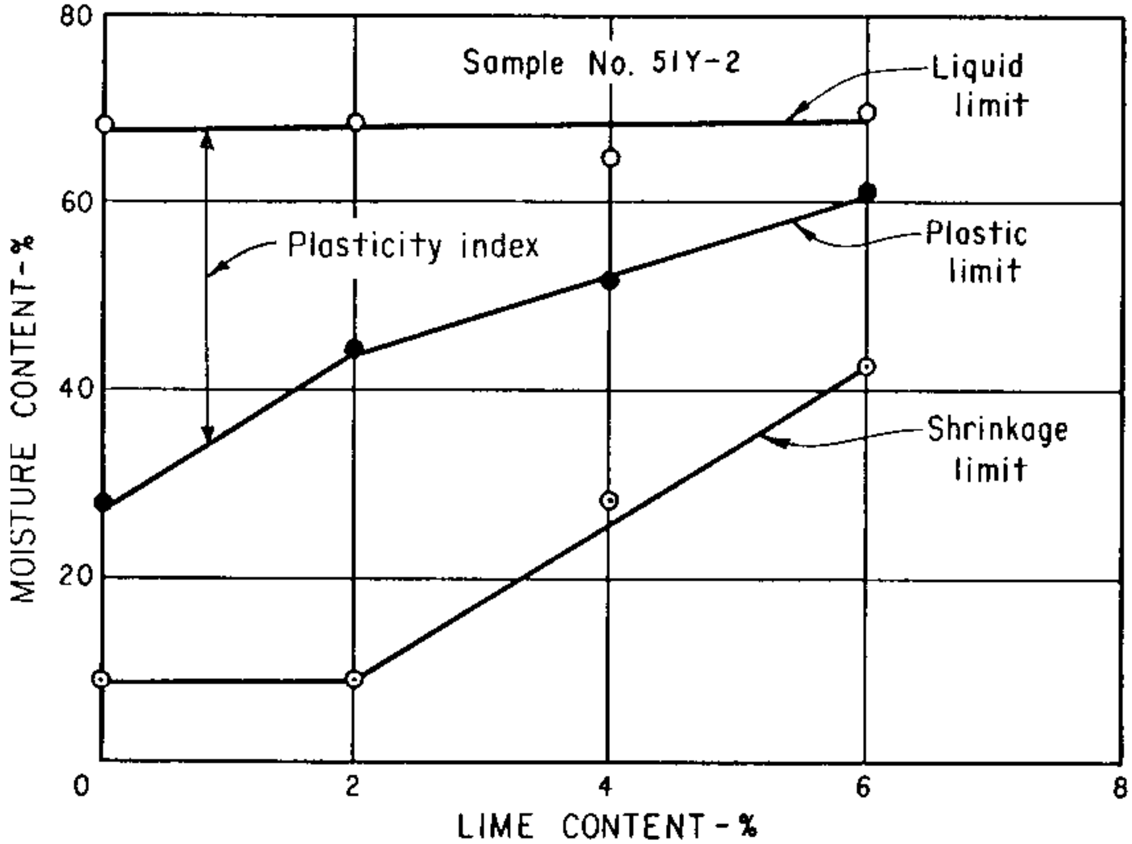


Figure 1-31.—Effect of lime on shrinkage limit, plastic limit, and liquid limit.

d. Determination of Coefficient of Permeability.—Permeability is determined in the field by means of a variety of tests based either on forcing water through the material or removing water under controlled conditions. USBR 7300, 7305, and 7310 describe field methods for determining *in situ* permeability. USBR 5600, 5605, 5610, and 5615 are methods for determining permeability in the laboratory.[20] There are ranges of permeability for which none of these tests may be satisfactory. However, this happens so rarely that procedures have not been established for these special conditions. For groundwater studies, large scale aquifer testing may be necessary (see *Ground Water Manual* [47]).

1-24. Engineering Characteristics of Soil Groups.—Although substitutes for thorough testing have not been devised to determine the important engineering properties of specific soils, approximate average values for compacted specimens of typical soils from each USCS group are available based on statistical analyses of available data (table 1-3). The attempt to determine soil data from average values involves risks: (1) the data may not be representative and (2) the values may be used in design without adequate safety factors. In the early stages of project planning, when different borrow areas and design sections are being studied, these average values of soil properties can be used as qualitative guides. Because the values pertain to the soil groups, proper soil classification is of vital importance. Verification of field identification by laboratory gradation and Atterberg limits tests must be made on representative samples of each soil group encountered.

[20] 7300: Performing Field Permeability Testing by the Well Permeameter Method.
7305: Field Permeability Test (Shallow-Well Permeameter Method).
7310: Constant Head Hydraulic Conductivity Tests in Single Drill Holes.
5600: Determining Permeability and Settlement of Soils [8-in (203-mm) Diameter Cylinder].
5605: Determining Permeability and Settlement of Soils Containing Gravel.
5610: Determining Permeability of Soil by the Back-Pressure Test Method.
5615: Performing Radial Vacuum Permeability Test for Soil-Cement or Porpous Rock.

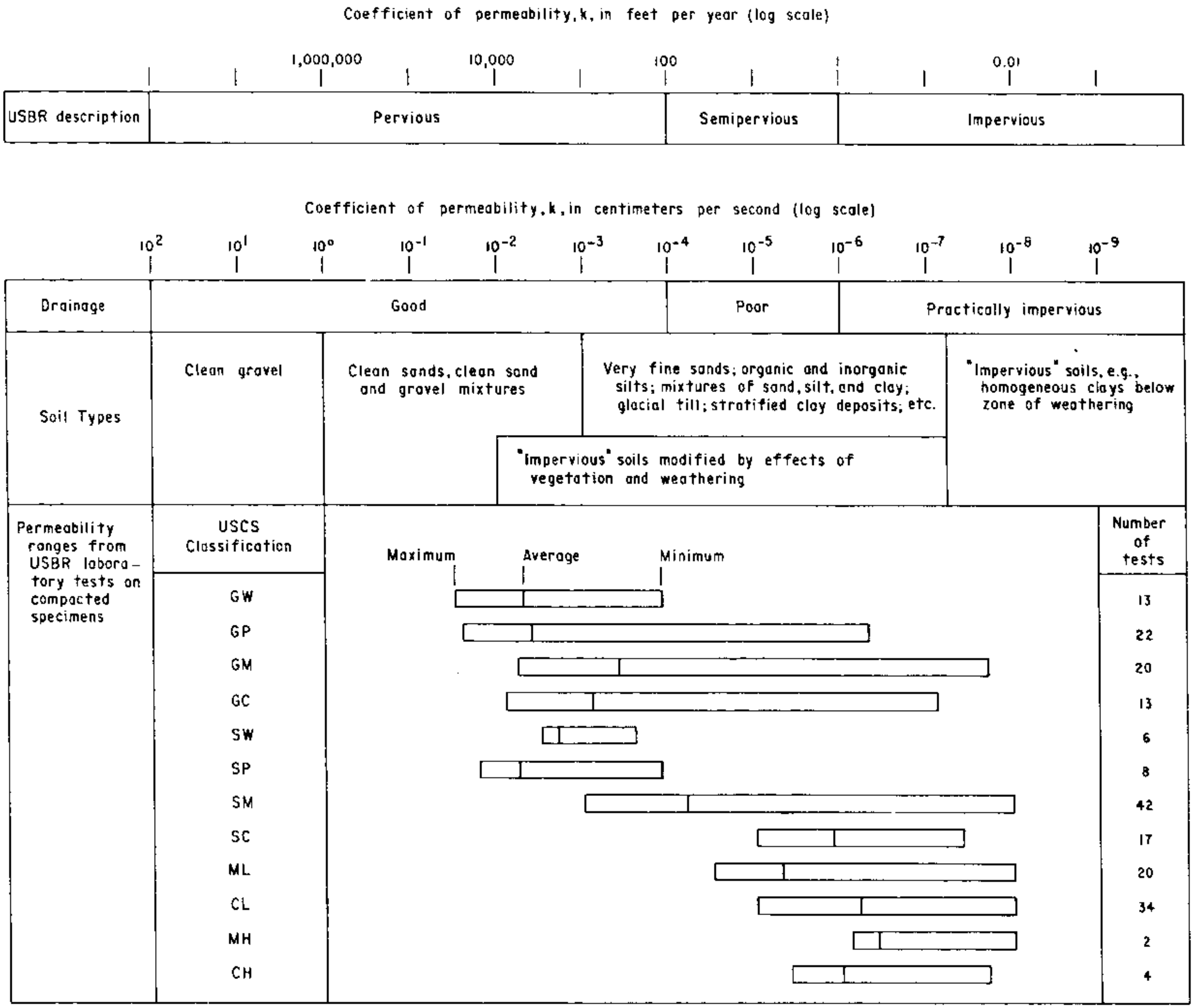


Figure 1-32.—Permeability ranges for soils.

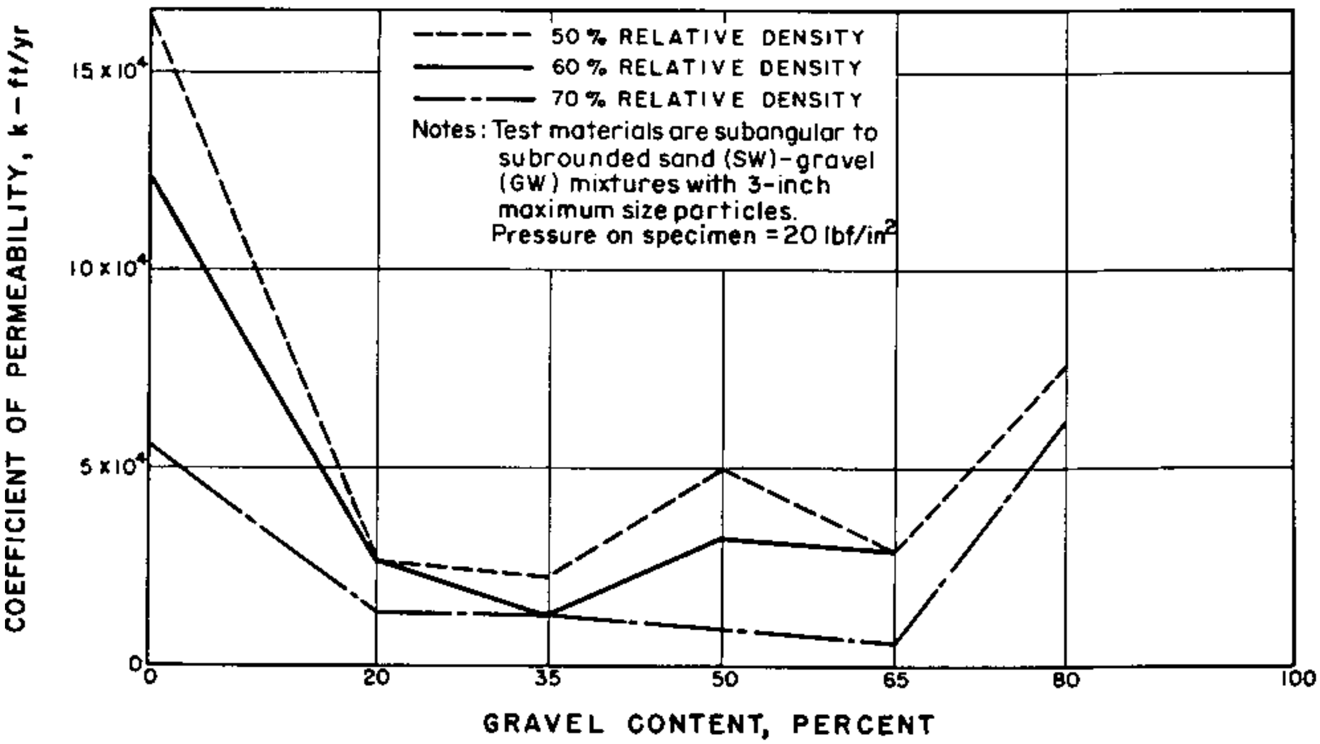


Figure 1-33.—Relationship of permeability to gravel content for specimens of various relative densities.

Table 1-3 is a summary of values obtained from more than 1,110 soil tests performed between 1960 and 1985 in Reclamation's Denver geotechnical laboratory. The data were obtained from reports on soil samples for which laboratory classifications were available and are arranged according to the USCS groups. The soils are from the 17 Western United States.

For each soil property noted in table 1-3, the average, minimum, and maximum test values; standard deviation; and number of tests performed are listed. Because all laboratory tests (except large-size permeability tests) were made on compacted specimens of the minus 4.75-mm (No. 4) soil fraction, data on average values for gravels were not available for most properties. The averages shown are subject to uncertainties that may arise from sampling variation and tend to vary widely.

The values for laboratory maximum dry density, optimum moisture content, specific gravity, and maximum and minimum index densities were obtained using the previously referenced test procedures. The MH and CH soil groups have no upper boundary of liquid limit in the classification system; therefore, the range of those soils is included in the table. The maximum liquid limits for the MH and CH soils tested were 82 and 86 percent, respectively. Soils with higher liquid limits than these have inferior engineering properties.

Two shear strength parameters are given for the soil groups under the heading c' and ϕ'. The values of c' and ϕ' are the vertical intercept and the angle, respectively, of the Mohr strength envelope on an effective stress basis as shown on figure 1-18. The Mohr strength envelope is obtained by testing several specimens of compacted soil in a triaxial shear apparatus in which pore-fluid pressures developed during the test are measured. Effective stresses are obtained by subtracting measured pore-fluid pressures in the specimen from stresses applied by the apparatus. Data used in compiling the values in table 1-3 were taken from unconsolidated-undrained (UU) and consolidated-undrained (CU) triaxial shear tests with pore-fluid pressure measurements and from consolidated- drained (CD) triaxial shear tests (see USBR 5745, 5750, and 5755).

a. Engineering Use Chart.—The Engineering Use Chart, table 1-4, shows the type of soil most appropriate for certain kinds of earthwork construction. The various soil groupings in the USCS generally relate to engineering properties typical for each soil group. These relationships

Table 1-3.—Average engineering properties of compacted soils from the 17 Western United States. Data from reports published between June 1960 and December 1985. Data from 2005 tests on 1110 samples. Table compiled January 1988

				Compaction				Shear Strength								
		Specific gravity		Laboratory		Index density		Consolidated-drained and consolidated-undrained triaxial shear tests				Unconsolidated-undrained triaxial shear tests				
								Av. placement conditions		Effective stress		Av. placement conditions		Effective stress		
USCS soil type	Total No. of samples tested	No. 4 minus	No. 4 plus	Max. dry density kg/m^3	Optimum moisture content %	Max. kg/m^3	Min. kg/m^3	Dry density kg/m^3	Moisture content %	Friction angle degrees	Cohesion kPa	Dry density kg/m^3	Moisture content %	Friction angle degrees	Cohesion kPa	Values listed
GW	22	2.69	2.58	1989	11.4	2167	1746									Average
		0.03	0.08	51	1.2	139	128									Std. dev.
		2.63	2.39	1907	9.9	1810	1417									Minimum
		2.75	2.67	2042	13.3	2332	1896									Maximum
		17	10	5		20										# of tests
GP	62	2.68	2.52	1907	12.2	2212	1808	1933	7.5	42.2	8.1					Average
		0.04	0.21	153	4.3	113	124	238	4.1	2.1	16.3					Std. dev.
		2.54	1.76	1436	9.1	1826	1375	1489	3.3	38.0	0.0					Minimum
		2.77	2.65	2045	26.5	2383	1986	2144	15.1	43.8	40.7					Maximum
		37	15	16		50		5								# of tests
GM	37	2.73	2.43	1819	15.7											Average
		0.07	0.18	189	5.9											Std. dev.
		2.65	2.19	1393	5.8											Minimum
		2.92	2.92	2130	29.5											Maximum
		35	17	35												# of tests
GC	32	2.73	2.50	1854	14.2											Average
		0.09	0.15	126	3.9											Std. dev.
		2.67	2.38	1537	6.0											Minimum
		3.11	2.78	2066	23.6											Maximum
		30	5	32												# of tests
SW	20	2.67	2.57	2019	9.1	1987	1576									Average
		0.03	0.03	96	1.7	128	142									Std. dev.
		2.64	2.54	1896	7.4	1683	1278									Minimum
		2.72	2.59	2162	11.2	2207	1758									Maximum
		13	2	4		13										# of tests
SP	81	2.66	2.62	1827	10.5	1890	1542									Average
		0.04	0.08	160	2.1	120	144									Std. dev.
		2.60	2.52	1649	7.8	1621	1252									Minimum
		2.86	2.75	2159	13.4	2199	1960									Maximum
		50	5	8		43										# of tests
SM	174	2.68	2.50	1877	12.3	1803	1379	1760	13.2	34.0	20.7	1821	12.6	33.5	59.3	Average
		0.06	0.12	140	3.3	147	136	145	5.2	4.9	25.5	201	5.5	6.1	42.1	Std. dev.
		2.51	2.24	1488	6.8	1417	1034	1459	4.6	23.7	0.0	1488	7.6	23.3	0.0	Minimum
		3.11	2.69	2114	25.5	1968	1555	2019	23.0	40.7	90.3	2122	25.0	45.0	146.2	Maximum
		162	10	133		20		10				8				# of tests
SC	112	2.69	2.47	1906	12.4			1773	15.4	32.7	19.3	1967	11.1	35.1	53.8	Average
		0.04	0.18	99	2.4			225	5.2	3.8	14.5	88	2.1	0.7	4.1	Std. dev.
		2.56	2.17	1547	6.7			1459	7.5	25.5	0.0	1843	9.7	34.2	49.0	Minimum
		2.84	2.59	2109	22.1			2111	22.7	38.3	42.1	2035	14.0	35.8	58.6	Maximum
		110	4	90				11				3				# of tests
ML	63	2.70		1645	20.1			1528	25.2	35.2	4.8	1678	17.4	31.8	61.4	Average
		0.09		168	5.7			179	9.5	2.5	3.4	161	5.7	4.3	24.1	Std. dev.
		2.52		1355	10.6			1292	13.5	31.4	0.0	1512	11.1	25.2	21.4	Minimum
		3.10		2018	34.6			1778	40.3	38.3	10.3	1909	25.8	37.2	82.0	Maximum
		60		36				11				4				# of tests
MH	11	2.79		1372	33.1											Average
		0.27		35	1.5											Std. dev.
		2.47		1327	31.5											Minimum
		3.50		1425	35.5											Maximum
		9		4												# of tests
CL	395	2.70	2.48	1768	16.4			1665	18.3	28.1	15.2	1760	15.3	24.4	91.0	Average
		0.05	0.13	97	3.1			174	5.7	5.0	18.6	86	2.4	7.0	49.0	Std. dev.
		2.56	2.34	1398	10.7			1297	10.2	10.8	0.0	1622	11.6	8.0	0.0	Minimum
		2.87	2.75	2002	30.9			1922	35.0	36.8	104.1	1986	20.2	33.8	164.1	Maximum
		361	8	286				31				24				# of tests
CH	101	2.73		1531	24.8			1406	30.6	20.5	32.4	1574	22.7	15.1	124.1	Average
		0.06		102	5.2			107	5.7	6.3	31.0	92	4.6	6.7	25.5	Std. dev.
		2.51		1318	16.6			1249	22.4	10.8	0.0	1438	17.9	5.1	85.5	Minimum
		2.89		1720	41.8			1555	42.0	30.9	108.2	1680	29.1	26.1	148.2	Maximum
		93		36				11				5				# of tests

Conversion factors: 1 kg/m^3 = 0.06243 lb/ft^3; 1 kPa = 0.145 lb/in^2

Table 1-4.—Engineering use chart for compacted soils

Engineering Properties of Compacted Soil[1]						Relative Desirability for Various Uses (No. 1 is considered the best)							
						Rolled earth dams			Canal sections		Foundations and fills		
Soil group name	Group symbol	Perme-ability[2]	Shear strength (saturated)	Compressibility (saturated)	Workability as a construction material	Homo-geneous embank-ment	Core	Shell	Erosion-resistant blanket or belt	Compacted earth lining	Imper-vious	Per-vious	Resistance to frost heave
Well-graded gravel	GW	Pervious	Excellent	Negligible	Excellent	—	—	1	1	—	—	1	1
Poorly graded gravel	GP	Pervious	Good	Negligible	Good	—	—	2	2	—	—	3	2
Silty gravel	GM	Semipervious to impervious	Good	Negligible	Good	2	4	—	4	4	1	4	6
Clayey gravel	GC	Impervious	Good to fair	Very low	Good	1	1	—	3	1	2	6	5
Well-graded sands	SW	Pervious	Excellent	Negligible	Excellent	—	—	3 if gravelly	6	—	—	2	3
Poorly graded sands	SP	Pervious	Good	Very low	Fair	—	—	4 if gravelly	7 if gravelly	—	—	5	4
Silty sands	SM	Semipervious to impervious	Good	Low	Fair	4	5	—	8 if gravelly	5 erosion critical	3	7	12
Clayey sands	SC	Impervious	Good to fair	Low to medium	Good	3	2	—	5	2	4	8	7
Silt	ML	Semipervious to impervious	Fair	Medium	Fair	6	6	—	—	6 erosion critical	6	9	11
Lean clay	CL	Impervious	Fair	Medium	Good to fair	5	3	—	9	3	5	10	9
Organic silt and organic clay	OL	Semipervious to impervious	Poor	Medium to high	Fair	8	8	—	—	—	7	11	—
Elastic silt	MH	Semipervious to impervious	Fair to poor	High	Poor	9	9	—	—	—	8	12	10
Fat clay	CH	Impervious	Poor	High	Poor	7	7	—	10	—	9	13	8
Organic silt and organic clay	OH	Impervious	Poor	High	Poor	10	10	—	—	—	10	14	—
Peat and other highly organic soils	PT	—	—	—	—	—	—	—	—	—	—	—	—

[1] Compacted to at least 95 percent of laboratory maximum dry density or to at least 70 percent relative density.
[2] Impervious: < 1 ft/yr (1×10^{-6} cm/s)
Semipervious: 1 to 100 ft/yr (1×10^{-6} cm/s to 1×10^{-4} cm/s).
Pervious: > 100 ft/yr (1×10^{-4} cm/s).

can help direct an investigation toward specific soil types best suited for the type of earthwork to be constructed.

Three important engineering properties of soils typical of each classification group are listed on the chart adjacent to the group symbol. They are: (1) permeability when compacted, (2) shear strength when compacted and saturated, and (3) compressibility when compacted and saturated. In addition, workability as a construction material is shown. Based on these properties, workability, and experience, the use chart lists the soil groups' relative desirability for use in rolled earthfill dams, compacted earth-lined canal sections, and compacted backfill. The numerical ratings given in the chart are relative and are intended only as a guide to aid the investigator in comparing soils for various purposes.

Gravelly soils are normally preferred construction and foundation materials because of their low compressibility and high shear strength. The GW and GP soils are pervious because they contain little or no fines to fill soil voids. Ordinarily, good drainage is ensured. Their properties are not affected appreciably by saturation and, if reasonably dense, these soils have good stability and low compressibility. In these respects, GW soils are better than GP soils. The GW and GP soils are virtually unaffected by freezing and thawing.

As the sand, silt, and clay fractions increase, the matrix soils begin to dominate the gravel skeleton structure, and the total material assumes more of the characteristics of the matrix. When properly compacted, GC soils are particularly good material for homogeneous, small earthfill dams or other embankments, or for the impervious sections of high earth dams. Permeability of GC soil is low, shear strength is high, and compressibility is low.

An important factor in the behavior of gravelly soils is the gravel content at which interference between the large particles begins to influence total material properties [48]. Extensive compaction studies demonstrated that particle interference begins to influence compaction at about 30-, 35-, and 45-percent gravel content, respectively, for sandy, silty, and clayey gravel soils tested, when placed using standard compactive effort. Similarly, at about the same gravel content, shear strengths show significant effects of particle interference. Soils having angular gravel particles, as compared to rounded particles, show interference characteristics at lower gravel contents.

A mixture's permeability is reduced as gravel content increases because solid particles replace permeable voids until the gravel content reaches an amount at which the matrix soils (i.e., sand, silt, or clay) can no longer fill the voids between the gravel particles. At that point, permeability increases with increase in gravel content.

Structural characteristics of gravelly soils are largely controlled by density, amount and shape of gravel particles, and the amount and nature of the matrix soils. Usually, cohesionless gravel soils have high shear strength and low compressibility when placed at a relative density of 70 percent or above.

Structural characteristics of coarse sands approach those of gravelly soils, but the structural characteristics of fine sands are more like those of silty soils. As in gravelly soils, density and amount and nature of the matrix (silt and clay) control the structural properties of sand. The SW and SP soils are pervious; whereas, SM and SC soils are semipervious to impervious depending upon the amount and character of the fines. When adequately compacted, SC soils are good for impervious earthfill dams and other embankment materials because of their low permeability, relatively high shear strength, and relatively low compressibility.

Other engineering problems encountered with sands are normally related to density. Stability of pervious saturated sands remains high as long as adequate drainage is provided. Shear strength of saturated sands containing appreciable amounts of silty and clayey fines will be controlled by water content; thus, as density becomes lower and water content becomes higher, shear strength decreases.

One of the most troublesome problems encountered by geotechnical engineers is restricted drainage in saturated sands because of low permeability or impervious boundaries. If rapid loadings are applied and if soil density is sufficiently low to allow volume decrease, high pore pressure and reduced stability will result. This may cause strains of unacceptable magnitude or even total failure because of liquefaction. Seismic and equipment vibrations and vehicular traffic loads are examples of rapid loadings that must be resisted.

Coarse, cohesionless soils usually are not affected by moderate water velocities. However, fine sands can be loosened or rearranged by low velocities of waterflow. Therefore, when constructing on sand below ground-water table, the control of seepage is important to prevent particle displacement causing erosion, loosening, or quick action (quicksand).

Even small amounts of fines may have important effects on the engineering properties of coarse-grained soils. For instance, as little as 5- to 10-percent fines in sands and gravels may significantly reduce permeability and increase susceptibility to frost action.

Silts are nonplastic, fine-grained soils that are inherently unstable in the presence of water and, like fine sands, may become quick. Silts are semipervious to impervious, often difficult to compact, and are susceptible to damage by frost. Typical bulky grained, inorganic-silt soils having liquid limits of about 30 percent are less compressible than highly micaceous and diatomaceous silts that have flaky grains and have liquid limits above 100 percent.

1-25. Changes in Soil Properties.—Although soil is commonly considered a stable material, it is constantly changing, either gradually from solid rock to increasingly finer particles or, conversely, gradually changing back to rock. In most soils, this change is sufficiently gradual that it is not a concern. However, in some soils, the change is rapid enough to be important in the life of an engineered structure. Soils where change may be important include those with appreciable quantities of: (1) organic matter, (2) soluble solids, or (3) minerals of volcanic origin. Residual soils may be in a state of chemical alteration such that during placement, they will have one set of characteristics; later, during the life of the hydraulic structure, they may have very different characteristics.

Frequently, existing soil deposits in their natural state have been stable for many years and give every indication that they will remain so. Nevertheless, human changes may result in failure of some soils. One of the soils prone to failure is called *sensitive clay*. This type of clay in an undisturbed condition has substantial shear strength, which to a large extent is lost upon being remolded. Very loose, saturated, fine sand and silt—when subjected to dynamic loading such as an earthquake or vibration from machinery—will lose strength and behave like a viscous liquid. This phenomenon is known as *liquefaction*. Another group of soils exists where minor changes in moisture content result in an abrupt change in shear strength. In some cases, these soils, such as loessial soils, have been deposited in a very loose state and exhibit change in shear strength and can collapse and subside when the moisture content is increased. Swelling clays frequently exhibit a change in strength characteristics caused by an increase in moisture.

Among the soils that, through desiccation, consolidation, and chemical action, have changed to forms commonly regarded as rock are varieties of shale, sandstone, and limestone. When these rocks are exposed to air, marked changes in characteristics can occur. Some shales flake off, air slake, or weather rapidly—turning into soil. Some shales may dry out without any apparent effect, but if rewetted, they deteriorate rapidly into very soft clay. Some sandstones and limestones harden on exposure to air and retain their improved qualities, while other limestones and sandstones break down rapidly with fluctuating temperature and moisture content.

Although deterioration is rare, if unrecognized, failure can occur without advance warning. Engineering practices for treating soils that deteriorate are not well known. Where situations as described above are suspected, the situation should be reviewed by specialists in this field, and specialized treatment may be necessary.

1-26. Workability.—Although laboratory testing indicates the maximum extent to which engineering properties such as shear strength, volume change, and permeability of a given soil may vary, achieving these limits in engineering practice is seldom practical. The ease with which satisfactory values of engineering properties can be economically reached is an important attribute of a soil, a soil deposit, or a foundation.

The cost to procure a unit volume of soil and place it (as in a structure) or for treating a unit amount of foundation varies widely, not only according to soil type but according to type and size of structure. Also, cost is influenced by the kind of equipment available and by current available labor. If a project is sufficiently large so that special equipment can be economically used, maximum efficiency in construction is most likely to be achieved. However, the soils selected must be workable by such methods.

Where separation of oversize is not required and where mixing requirements are minimized, borrow pits which can be preprocessed to optimum moisture content are preferable and usually more economical, even though longer haul distances may be required for their effective use. In practice, situations arise where separation of oversize is economical; also, cases exist where mixing two varieties of soil is worthwhile. Instances occur where extensive efforts to obtain maximum moisture control are justified; however, such operations should be avoided if possible.

Test procedures do not exist for measuring workability. Rather, all pertinent information concerning a soil, a borrow pit, or a foundation is tabulated so the various design possibilities can be evaluated. The engineering use chart (table 1-4) provides qualitative information on the workability of soils as a construction material and the relative desirability of various soil types according to structure. Borrow pits may be evaluated according to amount of work required. Because equipment mobilization is charged against a soil deposit, unit cost decreases appreciably as the volume of work increases. The change in unit cost for excavation up to about 100,000 m^3 (100,000 yd^3) is noticeable; then to 1 million m^3 (1 million yd^3) it is gradual; and beyond that range, unit cost is nearly constant. Transportation costs are nearly constant above about 100,000 m^3 (100,000 yd^3), depending only on distance. Moisture control costs depend primarily on the uniformity and slope of the borrow area and the availability of water. Excavation costs are influenced somewhat by topography of the borrow area. Borrow pits slightly higher in elevation than the work structure are preferable to those below the work.

1-27. Frost Action.—Heaving of subgrades caused by formation of ice lenses and subsequent loss of shear strength upon thawing is known as *frost action*. Water rises by capillarity and by thermal gradient toward the freezing zone and forms lenses of ice, which heave the soil. Soils most susceptible to frost action are those in which capillarity can develop but are sufficiently pervious to allow adequate water movement upward from below the freezing zone. Freezing of the pore water in saturated fine-grained soils, called closed-system freezing, decreases the density of soil by expansion but does not result in appreciable frost heave unless water movement can take place from below.

The severity of frost heave depends on three factors: (1) type of soil, (2) availability of free water, and (3) time rate of fluctuation of temperature about the freezing point. Soils having a high percentage of silt-size particles are the most frost susceptible. Such soils have a network of small pores that promote migration of water to the freezing zone. Silt (ML, MH), silty sand (SM), and clays of low plasticity (CL, CL-ML) are in this category. Tests by the Corps of Engineers from 1950 to 1970 [50] form the basis for figure 1-34; it relates frost susceptibility in terms of average rate of heave in percent by mass finer than the U.S.A. Standard 75-µm (No. 200) sieve. The figure shows most soil types have a wide range of frost susceptibility without a sharp dividing line between frost-susceptible and nonfrost-susceptible soils. Nevertheless, silts, clayey silts, and silty sands have the highest potential for frost heave followed by gravelly and sandy clay, clayey sand, and clayey gravel. Soils with the lowest potential to heave are sandy gravels, clean sands, and silty sands with less than 3 percent finer than the 75-µm (No. 200) sieve [51]. In the absence of a source of free water, frost heave is limited to the increase in volume due to freezing of pore water. This upper limit amounts to about 9 percent of pore water volume [52].

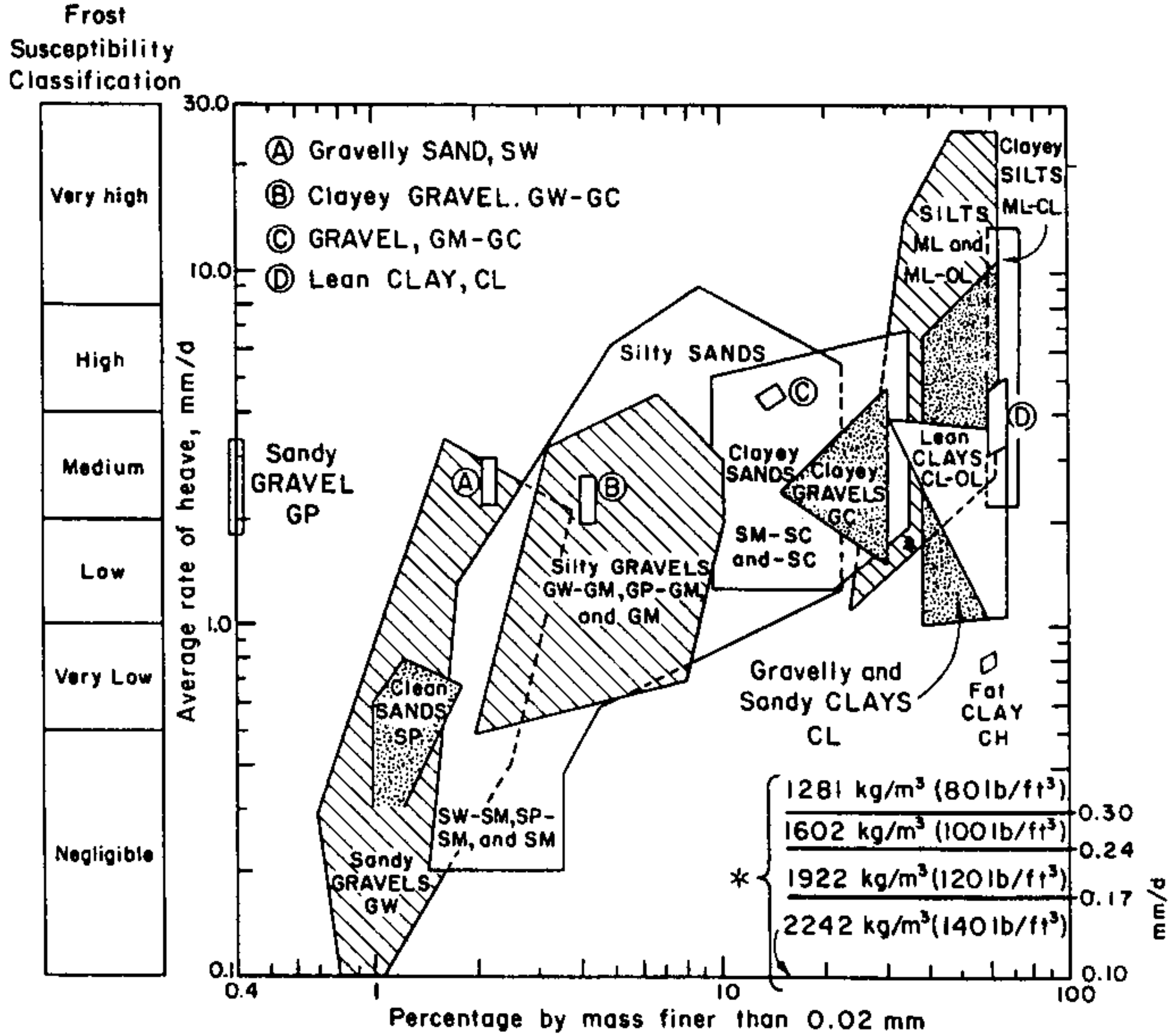
1-28. Erodibility.—*Erosion* has been defined as ". . . a process of detachment and transport of soil particles or particle groups by the forces of water, wind, ice, and gravity [53]." *Erodibility* is the susceptibility of a soil to erode.

The processes that influence erosion of *cohesionless* soil particles have been understood for many years. A cohesionless soil particle resting on the side or bottom of a stream or canal is acted on by gravity and by a tractive or erosive force caused by movement of water past the particle. Thus, erosion resistance of cohesionless soils depends on the applied tractive force and the mass of the particle expressed in terms of mean particle diameter (D_{50}). Figure 1-35 presents data collected by a number of investigators showing the relationship between critical tractive force, the force required to start erosion, and the mean particle diameter [54].

The processes that influence erosion of *cohesive* soils have been studied for a number of years but still are not completely understood. Over the years, investigators have attempted to correlate tractive force to various parameters and properties of cohesive soil. Strong, consistent correlation has not been found. Field performance data have been collected on operating canals and streams as well as laboratory data collected from various erosion devices, including flumes, erosion tanks, submerged hydraulic jets, and rotating cylinders. These studies have helped identify the properties that influence erosion of cohesive soils but have not provided quantifiable correlations between laboratory tests and erodibility in the field. Some soil and fluid properties that may influence the erosion process in cohesive soil include:

- dry density and moisture content,
- grain-size distribution,
- Atterberg limits,
- undrained shear strength,
- clay mineralogy,
- pore-water chemistry,
- stress history,
- soil structure and fabric,
- chemistry of the eroding fluid,
- temperature,
- viscosity of the eroding fluid, and
- applied tractive stress [55].

1-29. Dispersive Clay.—Some unique cohesive soils have been found to be highly erodible. These soils are designated *dispersive clay* because they erode when the individual clay particles disperse (go into suspension)—even in the presence of still water. Dispersive clays cannot be



* Indicated heave rate due to expansion in volume if all original water in 100 percent saturated specimen was frozen, with rate of frost penetration 6.35 mm (0.25-in) per day.

Figure 1-34.—Frost susceptibility classification by percentage of mass finer than 0.02 mm [35].

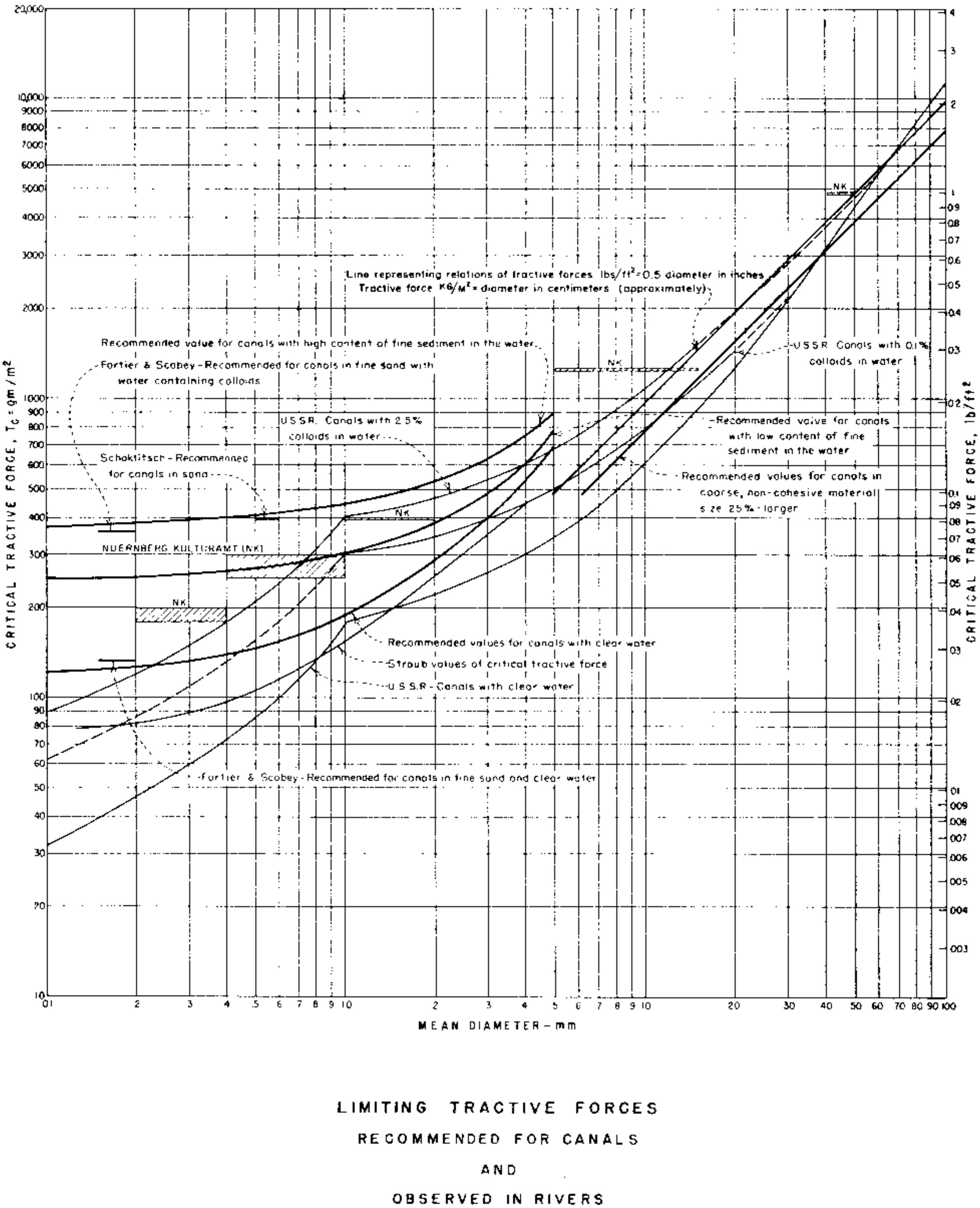


Figure 1-35.—Limiting tractive forces recommended for canals and observed in rivers [54].

distinguished from nondispersive clays by conventional index tests such as gradation, Atterberg limits, or compaction characteristics [56].

Dispersive characteristics are determined by performing three standardized tests on the questionable clay sample material. The three test results are combined to rate the clay as dispersive, intermediate, or nondispersive [57]. The three tests are USBR 5400, 5405, and 5410.[21]

Chemical tests to determine quantity and type of dissolved salts in the pore water are also useful in determining dispersive potential of clay soils. Dispersive clays can be made nondispersive by adding a small percentage of hydrated lime (about 2 to 4 percent by dry mass of soil) to the soil. Detrimental effects of dispersive clay (in hydraulic structures) can also be minimized by proper zoning and by using designed granular filters to prevent piping failures.

1-30. Dynamic Properties.—The response of soil to cyclic or dynamic stress application must be considered in the design of structures subjected to:

- earthquake loading,
- foundations subjected to machine vibrations, and
- subgrades and base courses for pavement.

A variety of laboratory equipment has been used to determine the dynamic properties of soil including:

- cyclic triaxial compression,
- cyclic simple shear,
- cyclic torsional shear,
- resonant column,
- ultrasonic devices.

The dynamic properties of most interest include:

- shear modulus,
- damping ratio,
- dynamic shear strength,
- pore-pressure response.

Detailed dynamic properties test procedures are not included in this manual; information on this subject may be found in references [58, 59, 60, 61].

[21] 5400: Determining Dispersibility of Clayey Soils by the Crumb Test Method.
5405: Determining Dispersibility of Clayey Soils by the Double Hydrometer Test Method.
5410: Determining Dispersibilit of Clayey Soils by the Pinhole Test Method.

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Chapter 2

INVESTIGATION

A. Principles of Investigations

2-1. General.—

a. Objectives.—The purpose of an investigation is to obtain information relating to foundation conditions and to natural construction materials commensurate with the type of structure involved and with the stage of the project. The investigation is conducted in the office, in the field, and in the laboratory. Characteristics of subsurface conditions are developed in progressively greater detail as exploratory work proceeds. Investigations should be performed in phases so the program can be reevaluated and revised to obtain maximum information at least cost. Data obtained must be organized to clearly show significant features of the occurrence and properties of the materials.

Specific objectives of an investigation are to determine (as required):

- The regional geology influence on materials, site, and structure characteristics, particularly seismo-tectonics.
- The location, sequence, thickness, and areal extent of each stratum, including a description and classification of the materials and their structure and stratification in the undisturbed state. Significant geologic features such as concretions, fabric, and mineral and chemical constituents should be noted.
- The depth to and type of bedrock:

  – location,
  – depth of weathering,
  – sequence,
  – seams,
  – thickness,
  – joints,
  – areal extent,
  – fissures,
  – attitude,
  – faults,
  – soundness, and
  – other structural features.

- The characteristics of the ground water:

  – depth to water table,
  – whether the water table is perched or normal,
  – depth of and pressure in artesian zones,
  – quantity and types of soluble salts or other minerals present, and
  – water chemistry particularly for any contaminants.

- Properties of the materials by methods appropriate for the investigation stage, the type of structure, and detailed engineering data:

  – by describing and identifying materials in place visually, and determining in-place density.
  – by obtaining disturbed samples, describing and identifying them visually, and determining their in-place water content and index properties. Engineering properties may be estimated on the basis of the classification along with results of laboratory index tests.
  – by indirect methods performed in the field such as geologic interpretations, in-place tests, or surface geophysical methods, using results of direct explorations and other tests to provide necessary correlations.
  – by observing performance of previously constructed structures built of or placed on similar materials.
  – by observing natural slopes of similar materials.
  – by obtaining undisturbed samples, identifying them visually, describing their undisturbed state, determining in-place density and water content, and obtaining index and engineering properties by laboratory tests.
  – by performing tests in the field such as standard penetration tests, cone penetration tests, bearing capacity tests, pile loading tests, permeability tests, pressuremeter tests, dilatometer tests, and vane shear tests.

b. Classification of Structure Foundations.—For structures used by the Bureau of Reclamation (Reclamation), investigation requirements for foundations vary over a wide range and may include consideration of the foundation material use both for the foundation and for the structure. Foundations for structures can be conveniently grouped into four classes to assist in determining type and degree of foundation investigation required:

(1) The engineering properties of the material are unacceptable, and the soil or rock must be partially or entirely removed to provide a satisfactory foundation for the structure under consideration.

(2) The soil or rock in place will provide the structure foundation, either with or without ground improvement.

(3) The soil or rock provides both the foundation and a major part of the structure—with material from the required foundation excavation provided for use in the structure.

(4) Same as (3), except that substantial amounts of material are needed for the structure in addition to that available from the required excavation.

For structures constructed on rock, in addition to investigations of the rock foundation, a soil investigation is made in which primary concerns are depth to bedrock, stability of slopes, and difficulty of excavation. Stability of reservoir rims should also be considered and investigated if problem areas are identified. Materials from excavations for structures in class 3 should be used for other construction purposes when feasible. For example, a site considered suitable for a concrete dam will usually require temporary cofferdams, and materials from required excavations could be used for that purpose. Aggregate sources for concrete and for filters for earth and rockfill dams also need to be located and investigated.

For structures founded on soil, the primary objective of a soil investigation is to determine soil volume change characteristics that may result in foundation settlement or heave. Dispersivity and soluble salts could also be important, depending on the structure type. If heavy loading and wet soil conditions are anticipated, the shear strength should be investigated.

For structures which use materials from required excavations, materials must be considered from both stability and utilization standpoints. Stability of slopes, both in cuts and fills, is a primary consideration. Compressibility varies in importance, approximately commensurate with the importance of the structure itself—having little significance where roads and laterals are concerned—but major importance where paved highways and large lined canals with large structures are required. In expansive soils and in-place low density soils, the probability and magnitude of uplift and collapse must be evaluated. Permeability is important on canals and laterals. Where a choice in location is possible, workability of materials is of major economic importance. For this reason, cuts into bedrock are normally avoided.

2-2. Sources of Map and Photo Information.—

a. U.S. Department of Agriculture (Aerial Photography).—The U.S. Department of Agriculture (USDA) is an excellent source of aerial photography. The Aerial Photography Field Office (APFO) is the depository and reproduction facility of the U.S. Department of Agriculture's aerial photography, housing aerial film acquired by the Agricultural Stabilization and Conservation Service (ASCS), Natural Resources Conservation Service (NRCS), and the U.S. Forest Service. Combined aerial photography covers about 95 percent of the conterminous United States (see sec. 2-3d for more information). APFO's film holdings include black and white panchromatic, natural color, and color infrared films (CIR) with negative scales ranging from 1:6000 to 1:120,000. National High Altitude Photography (NHAP) is a primary source of new aerial photography. APFO libraries keep the original CIR positives which are flown at a scale of 1:60,000 with each exposure covering 68 square miles.

Ordering information can be obtained from:

U.S. Department of Agriculture – ASCS
Aerial Photography Field Office
Customer Services
2222 West 2300 South
PO Box 30010
Salt Lake City UT 84130-0010

b. The U.S. Geological Survey (USGS).—The USGS produces information in many forms that can be useful in local engineering studies: maps, scientific reports, geodetic data, aerial photographs, bibliographic data, and other forms. Most of these products are available from one or more of the following sources:

- USDA—aerial
- USGS Distribution—maps and published reports
- USGS Earth Science Information Center Open-File Report unit—open-file reports
- Earth Science Information Centers (ESICs)—maps, reports, aerial photographs, and general information on many earth science topics
- Water Resources Division (WRD)—state water data reports
- National Technical Information Service (NTIS)

Most maps and reports are available for reference in USGS libraries, in selected U.S. Government Printing Office (GPO) Depository Libraries, and in large university libraries and are available commercially.

c. USGS Products.—Maps, cross sections, and related indexes.

- *Topographic maps* are the most common USGS maps. For most parts of the United States, topographic maps are available in several scales:

  - 1 : 24,000
  - 1 : 100,000
  - 1 : 250,000
  - 1 : 500,000
  - 1 : 1,000,000
  - 1 : 62,500 and 1 : 63,360 scale maps are available for some areas of the country.
  - 1 : 50,000 and 1 : 100,000 scale county maps are being prepared in selected States.
  - Topographic maps are available commercially and from USGS Distribution and Earth Science Information Centers.
  - Topographic data (in digital form) called digital elevation models (DEMs) and digital line graphs (DLGs), are available from ESICs, the EROS Data Center (EDC), and commercial vendors.
  - Specific information about topographic maps is presented State by State in the USGS: *Indexes and Catalogs to Topographic and Other Map Coverage* [1].

- *Geologic maps* are of many kinds: bedrock geology and surficial geology are the most common, but maps showing depth to bedrock, bedrock structure contours, coal or other mineral resources, basement geology, and similar maps also are considered geologic maps. Cross sections are presented on many geologic and hydrologic maps but are not generally available separately. Bedrock and surficial geologic maps are available in many scales; such maps, at detailed scales (1 : 24,000 or larger), have been published for only about one-third of the United States. To identify maps available for a specific place, use the USGS *Geologic Map Index* for that State or the USGS *GEOINDEX* data base, which contains the same information but is more up to date. The State geoscience agencies can be helpful in identifying such geologic maps.
- *Hydrologic maps*, like geologic maps, can be of many kinds: water table contours, aquifers and aquicludes, water quality, hydrologic units (watersheds), flood hazards, and so on. These kinds of maps are published by the USGS and other Federal, State, and local government agencies. The USGS hydrologic maps are published either as thematic maps available from USGS Distribution or in open-file reports available from the ESIC Open-File Report unit. Hydrologic maps are listed in the annual catalogs.
- *Geophysical maps* show information about geomagnetism, gravity, radioactivity, and many other geophysical characteristics. *Geochemical maps* show information related to stream sediment samples processed in search of mineral resources. *Seismicity maps* show information about earthquake history, severity, and risk. *Hazard maps* can show areas of swelling clays, landslides and related hazards, avalanche danger, volcanic and earthquake hazards, and other geologic hazards. These maps are available either as published USGS thematic maps from USGS Distribution or as open-file reports from the ESIC Open-File Report unit.
- *River surveys* and *wetlands inventory maps* may be of particular interest to investigators studying river sites. River surveys were mostly conducted between 30 and 60 years ago in support of large-scale engineering projects. Some maps are still available showing river courses and profiles. Wetlands inventory maps, which show kinds of wetland ecosystems, are being made by the U.S. Fish and Wildlife Service for 7.5-minute quadrangles.
- *Out-of-print (o.p.) USGS maps* generally are available for reference in libraries and from other sources. Blueprint copies of o.p. topographic maps are available from ESICs. Generally, reproductions of o.p. thematic maps are available from the U.S. National Archives, Cartographic Division. Another source of o.p. maps is map dealers, of whom some specialize in old, rare maps.
- *Other miscellaneous maps* are available from the USGS, including folios (sets of environmental maps of many selected areas showing geologic, hydrologic, land use, historical, and other features) and wilderness area maps.

d. USGS Technical Reports.—The USGS publishes several kinds of technical reports. These reports are useful in many earth science disciplines, including engineering geology and hydrology; most are listed in annual catalogs and bibliographies.

e. USGS Publications.—Publications catalogs have been published annually. This USGS information has been available since 1982; earlier publications are compiled into larger catalogs:

- Publications of the U.S. Geological Survey, 1971-81
- Publications of the U.S. Geological Survey, 1962-70
- Publications of the U.S. Geological Survey, 1879-1961

These are available from USGS Distribution and from ESICs. A monthly list, "New Publications," of new USGS publications is available free from USGS. Bibliographies of USGS publications on selected topics are available as open-file reports, bulletins, and other publications.

Additional unpublished bibliographies on selected geologic topics are also available from the Geologic Inquiries Group (GIG).

A complete bibliography of USGS publications, "Publications of the U.S Geological Survey" [2], includes information on all but the topographic maps described above and is available on CD ROM from the American Geological Institute GeoRef Information System. Other selected references are listed in the bibliography [3, 4, 5, 6].

Several USGS data bases of bibliographic and similar information are available to help identify USGS products:

- Earth Science Data Directory lists earth science data files and data bases from many sources, including many outside the USGS.
- Map and Chart Information System is a data base listing USGS and other maps and charts.
- Cartographic Catalog is a data base listing kinds of maps and such map-related information as map dealers, geographic software vendors, and producers of globes.
- Aerial Photography Summary Record System is a data base for information about aerial photographic coverage of the United States.
- GEOINDEX is a data base of bibliographic and related information about geologic maps of the United States.
- Library data base provides bibliographic information about recent acquisitions of the USGS library; this data base includes many non-USGS publications.
- Selected Water Resources Abstracts presents abstracts of water resources reports and articles published by many organizations.

f. USGS Software, Data, and Related Products.—The USGS has published (and released in other ways) computer software.

The USGS Open-File Report 89-681 (available from the ESIC Open-File Report unit) lists USGS software published as of June 1989 [7]. The individual publications describing or containing computer programs are also listed in USGS publications catalogs and in other references listed above.

For many parts of the country, the USGS also has digital data useful in many earth science investigations:

- DLGs, i.e., digital map data such as roads, streams, and boundaries
- DEMs, i.e., digital map data showing topography
- digital versions of land use and land cover data
- geographic names information
- stream gauge data
- water well data
- geophysical logs
- other kinds of data

One can check on data bases available in the Earth Science Data Directory and contact an ESIC for more information on access to USGS data.

g. USGS Remote Sensing Products.—The USGS has acquired many aerial photographs, orthophotos, satellite photographs, and many kinds of satellite imagery such as:

- Landsat multispectral scanner (MSS)
- Landsat thematic mapper (TM)
- side-looking airborne radar (SLAR)
- advanced very-high resolution radiometer (AVHRR)

Aerial photographs are available for the entire United States, and in many places, an option of scales and acquisition dates is available. Orthophotos are made from aerial photographs that have been geometrically corrected to eliminate displacements present in the aerial photographs; these are available for many parts of the country. For assistance, in ordering any kind of aerial photograph or satellite image, contact an ESIC for help in identifying the best coverage, which varies depending on how it will be used.

The USGS has also collected geophysical data, including gravity data, aeromagnetic and aeroradioactivity. For detailed information about the availability of these kinds of geophysical data for a specific area, contact the USGS

Branch of Geophysics. Information about magnetic declination is available from the National Geomagnetic Information Center. The National Uranium Resource Evaluation (NURE) program collected a large amount of general aeromagnetic data and stream-sediment-sample geochemical data for most of the country; NURE data are available in published form from the ESIC Open-File Report unit and in digital form from the EDC. For more information on remote sensing, see section 2-3.

h. Access to USGS Data Bases.—

Earth Science Data Directory (ESDD)
On several CD-ROM products, on-line, ESDD Project Manager, U.S. Geological Survey, 801 National Center, Reston VA 22092.

GEOINDEX
On several CD-ROM products and available for personal computer style microcomputers as USGS Open-File Report 91-575.

Selected Water Resources Abstracts (SWRA)
On several CD-ROM products, online, on Dialog (file 117), and through the WAIS system on Internet.

i. Sources of USGS Products.—

USGS Distribution
Distribution, U.S. Geological Survey, PO Box 25286, Denver Federal Center, Denver CO 80225.

Open-File Reports
ESIC Open-File Report unit, U.S. Geological Survey, PO Box 25425, Denver Federal Center, Denver CO 80225.

ESICs
Reston ESIC, 507 National Center, Reston VA 22092 (1-800-USA-MAPS).

Washington, DC ESIC, U.S. Department of the Interior, 1849 C St., NW, Rm. 2650, Washington DC 20240.

GIG
Geologic Inquiries Group, U.S. Geological Survey, 907 National Center, Reston VA 22092.

EDC
EROS Data Center, U.S. Geological Survey, Sioux Falls SD 57198.

USGS Libraries
Library, U.S. Geological Survey, 950 National Center, Reston VA 22092.

Cartographic Information Center, 952 National Center, Reston VA 22092.

Library, U.S. Geological Survey, MS 955, 345 Middlefield Rd., Menlo Park CA 94025.

Library, U.S. Geological Survey, MS 914, Bldg. 20, PO Box 25046, Denver Federal Center, Denver CO 80225.

Library, U.S. Geological Survey, 2255 North Gemini Dr., Flagstaff AZ 86001.

NTIS
National Technical Information Service, U.S. Department of Commerce, 5285 Port Royal Rd., Springfield VA 22161.

New Publications and monthly list
USGS New Publications, 582 National Center, Reston VA 22092.

Commercial map dealers
Local map dealers are listed in the telephone yellow pages under "Maps." A map information specialist at ESIC's toll-free telephone number, 1-800-USA-MAPS, also provides names and addresses of map dealers for anywhere in the United States.

Earthquake information
National Earthquake Information Center, U.S. Geological Survey, MS 967, PO Box 25046, Denver Federal Center, Denver CO 80225.

USGS publications CD-ROM
American Geological Institute, GeoRef Information System, 4220 King St., Alexandria VA 22302-1507.

Guide to USGS maps
Documents Index, Inc., PO Box 195, McLean VA 22101.

Soil surveys
U.S. Department of Agriculture, Natural Resources Conservation Service offices or extension offices located in most counties nationwide; State Soil Conservationist office located in most State capitals.

Out-of-print maps

U.S. National Archives, Cartographic Branch (National Science Foundation [NSF] Network Service Center [NNSC]), Washington DC 20408,

j. USGS Topographic Maps.—A topographic map is useful in the design and construction of most civil engineering structures. Before undertaking the painstaking job of mapmaking, a thorough search should be made for existing maps which cover the area of the structure and potential sources of construction materials. The USGS has made a series of standard topographic maps covering the United States and Puerto Rico.

The unit of survey for USGS maps is a quadrangle bounded by parallels of latitude and meridians of longitude. Quadrangles are available generally covering 7.5 minutes, 15 minutes, and 30 minutes of latitude and longitude, and possibly at several scales such as 1 : 24,000 (1 inch equals 2000 feet for 7.5 minutes of latitude and longitude). Each quadrangle is designated by the name of a city, town, or prominent natural or historical feature within it; the margins of each map are printed with the names of adjoining quadrangle maps. In addition to published topographic maps, the USGS has other information for mapped areas; for example, location and true geodetic position of triangulation stations and elevation of permanent benchmarks established by the USGS.

River survey maps are important to dam planning. These are topographic strip maps that show the course and fall of a stream; configuration of the valley floor and the adjacent slopes; and locations of towns, scattered buildings or houses, irrigation ditches, roads, and other cultural features. River survey maps were prepared in connection with the classification of public lands; hence, most of them are of areas in the Western United States. If a valley is less than 1.6 km (1 mi) wide, the topography is usually shown to 30 m (100 ft) or more above the water surface; if the valley is flat and wide, topography is shown for a strip of 1.6 to 3.0 km (1 to 2 mi) parallel to the river or stream. The usual scale is 1 : 31,680 or 1 : 24,000, and the normal contour interval is 6 m (20 ft) on land and 1.5 m (5 ft) on the water surface. Many of these maps include proposed damsites on larger scale topography and show a profile of the stream.

k. USGS Geologic Maps.—Important information is obtainable from geologic maps. These maps identify the rock units directly underlying the project area. Characteristics of rocks are of major importance in selection of a damsite and in design of water-retaining and conveyance structures. Many surface soils are closely related to the type of rock from which they are derived, but if the soil has been transported, it may overlie an entirely different rock type. When the influence of climate, relief, and geology of the area is considered, reasonable predictions can be made of the type of soil which will be encountered or of the association with a particular parent material. Subsurface conditions can often be deduced from the three-dimensional information given on geologic maps. These maps are especially valuable in areas where only limited information on soils or agricultural classifications are available; for example, in arid or semiarid regions where soils are thin.

Commonly available general purpose geologic maps (e.g., USGS-type maps) are not detailed enough for site-specific needs. Geology for engineering exploration, design, and construction must be generated for the specific application. Horizontal scales of 1 in to 50 or 100 ft and contour intervals of 1 to 5 ft are common. Site-specific maps are usually generated using aerial photographs flown for the application, although small maps may be prepared using plane table or ground survey data.

Site geologic maps are used for the design, construction, and maintenance of engineering features. These maps concentrate on geologic and hydrologic data pertinent to the engineering needs of a project and do not address the academic aspects of the geology.

Rocks are identified on geologic maps by name and geologic age. The smallest rock unit mapped is generally a formation, but smaller subdivisions such as members or beds may be delineated. A formation is an individual bed or several beds of rock that extend over a fairly large area and can be clearly differentiated from overlying or underlying beds because of a distinct difference in lithology, structure, or age. The areal extent of these formations is indicated on geologic maps by means of letter symbols, color, and symbolic patterns.

Geologic maps often show one or more geologic sections. A section is a graphic representation of the disposition of the various strata in depth, along an arbitrary line usually marked on the map. Geologic sections are interpretive and should be used with that in mind. The vertical scale may be exaggerated. Sections prepared solely from surface data are less accurate; sections prepared from boring records or mining evidence are more reliable. A section compiled to show the sequence and stratigraphic relations of the rock units in one locality is called a columnar section; it shows only the succession of strata and not the structure of the beds as does the geologic section.

Several types of geologic maps are available. A bedrock or areal geologic map shows a plan view of the bedrock and surficial materials in the area. This type of map shows the boundaries of formations, inferred where the units are covered by soil or plant growth, and may include one or more geologic sections. Areal maps do not show soil except for indicating thick deposits of alluvial, glacial, or windblown materials. In areas of complex geology where exposures of bedrock are scarce, location of the contacts between formations is often indicated as approximate. Surficial geologic maps differentiate surface materials of the area according to their geologic categories such as stream alluvium, glacial gravel, and windblown sand. These maps indicate the areal extent, characteristics, and geologic age of surface materials. Areal (bedrock) geologic maps of moderately deformed areas often show enough structural detail to provide an understanding of the structural geology of that region; in many instances, generalized subsurface structure can be deduced from distribution of the formations on the map. In highly complex areas, where a great amount of structural data is necessary for an interpretation of the geology, special structural geologic maps are prepared.

In addition to giving the geologic age of mapped rocks, some maps briefly describe the rocks. Many maps, however, lack a lithologic description. An experienced geologist may make certain assumptions or generalizations based only on the age of rock by making analogies with other areas. Geologic literature on the entire area must be consulted for more detail and for certain identification of the lithology. Engineering information can be obtained from geologic maps if the user has the appropriate background and experience. It is possible to prepare a preliminary construction materials map by study of a basic geologic map, together with all collateral geologic data that pertain to the area shown. Similarly, preliminary foundation and excavation conditions, as well as surface and ground-water data, can be deduced from geologic maps. Such information is valuable for preliminary planning activities but is not a substitute for detailed field investigations during the feasibility and specifications stages.

l. Agricultural Soil Maps.—A large portion of the United States has been surveyed by the USDA. These investigations are of surficial materials. The lower limit of soil normally coincides with the lower limit of biological activity and root depth. Soils are examined down to rock or down to 2 m (80 in)—possibly deeper in some cases. Soils are classified according to soil properties including color, structure, texture, physical constitution, chemical composition, biological characteristics, and morphology. The USDA publishes reports of these surveys in which the different soils are described in detail; interpretations are given for agricultural and certain nonagricultural uses. Each report includes a map of the area surveyed (usually a county) showing by pedological classification the various kinds of soils present. In addition to published soil maps, many areas are shown in which individual farms and ranches are mapped using the same system of soil classification and interpretation.

Agricultural soil maps can be obtain from the State or local office of the NRCS, from a county agent, or from a congressional representative. Many libraries keep published soil surveys on file for reference. Also, resources conservation district offices and county agricultural extension offices have copies of local soil surveys that can be used for reference. Out-of-print maps and other unpublished surveys may be available for examination from the USDA, county extension agents, colleges, universities, and libraries. Using the agricultural soil classification system, soil surveys have been made for many river basins in the 17 Western States to classify land for irrigation based on physical and chemical criteria. Inquiry should be made at the local Reclamation area offices concerning availability of soil data for these areas.

When applying agricultural soil maps to exploration for foundations and construction materials, some knowledge of the pedological system of classification is necessary. This system is based on the premise that water leaches inorganic colloids and soluble material from upper layers to create distinct layers of soil. The depth of leaching action depends on the amount of water, permeability of the soil, and length of time involved. The surface layer lacks in fines which are accumulated in a subsurface layer containing these fines in addition to its original fines. Deep soil beneath the subsurface layer has been little affected by water and remains essentially unchanged. Exceptions are in the Southeastern United States (or other humid areas) where some soils are weathered to greater depths.

Three layers are typically developed from the surface downward: the **A** horizon, the **B** horizon, and the **C** horizon. In some soils, an **E** horizon (a more leached horizon) is between the **A** and **B** horizons. In detailed descriptions, these horizons may be subdivided into $\mathbf{A}_1$, $\mathbf{A}_2$, $\mathbf{B}_1$, $\mathbf{B}_2$, etc.

Soils of the United States are divided into main divisions depending on the cause of profile development and on the magnitude of the cause. The main soil divisions are further divided into "suborders," then into "great groups" based on the combined effects of climate, vegetation, and topography. Within each "great group," soils are divided into soil series, of which each has the same degree of development, climate, vegetation, relief, and parent material. In the pedological classification system, all soil profiles of a certain soil series are similar in all respects, except for variation in texture or grain size of the topsoil or **A** horizon. Typically, the soil series are named after a town, county, stream, or similar geographical source where the soil series was first identified.

The final soil mapping unit, which is called the soil phase, consists of the soil series name plus the textural classification of the topsoil or **A** horizon plus other features such as slope, flooding, etc. The USDA's textural classification system is different from the Unified Soil Classification System used for engineering purposes. Figure 2-1 shows textural classification of soils used by USDA [8]. The chart shows terminology used for different percentages of:

- clay . . . . . particles smaller than 0.002 mm,
- silt . . . . . . 0.002 to 0.05 mm, and
- sand . . . . . 0.05 to 2.0 mm.

Note the term "loam" is a mixture of sand, silt, and clay within certain percentage limits. Other terms (used as adjectives to the names) obtained in the USDA system are:

- gravelly . . . rounded and subrounded particles from 2 to 75 mm,
- cobbly . . . 75 to 250 mm (3 to 10 in), and
- stony . . . . sizes greater than 250 mm (10 in).

The textural classification—given as part of the soil name on the agricultural soil map—refers to material in the **A** horizon only; hence, this is not of much value to an engineer interested in the entire soil profile. The combination of soil series name and textural classification to form a soil type, however, provides a considerable amount of significant data. For each soil series, the following can be obtained:

- texture,
- degree of compaction,
- presence or absence of hardpan or rock,
- lithology of the parent material, and
- chemical composition.

A table of estimated engineering sieve sizes, the Unified Soil Classification System and American Association of State Highway and Transportation Officials (AASHTO) classification, and Atterberg limits are included in each published soil survey. In addition, actual test data from the survey area is published in some cases.

The following example is taken from the Soil Survey Data Base available from offices of USDA's Natural Resources Conservation Service. The "Cecil Series" is described by a general geographical distribution of the series, the rocks from which it was derived, and information on climate. A comparison is made to series with associated or related soil series. Additional discussion concerns the range in characteristics of the Cecil Series as well as: relief, drainage, vegetation, land use, and remarks; and distribution of the series by States and location. A soil pedon description for the Cecil Series is given as follows.

The Cecil Series consists of very deep, well-drained, moderately permeable soils on ridges and side slopes of the Piedmont uplands. They are deep to saprolite and very deep to bedrock. They formed in residuum weathering from felsic crystalline rocks of the Piedmont uplands. Slopes range from 0 to 25 percent. Mean annual precipitation is 1220 mm (48 in) and mean annual temperature is 15 °C (59 °F) near the *type location*.

Taxonomic Class: Clayey, kaolinitic, thermic Typic Kanhapludults

Typical Pedon: Cecil sandy loam—forested. (Colors are for moist soil unless otherwise stated.)

Oi – 50 to 0 mm (2 to 0 in); very dark grayish brown (2.5Y 3/2) partially decayed leaves and twigs. 0 to 75 mm thick (0 to 3 in).

A – 0 to 50 mm (0 to 2 in); dark grayish brown (10YR 4/2) sandy loam; weak medium granular structure; very friable; many fine roots; strongly acid; clear wavy boundary. 50 to 200 mm thick (2 to 8 in).

E – 50 to 175 mm (2 to 7 in); brown (7.5YR 5/4) sandy loam; weak medium granular structure; very friable; many fine and medium roots; few pebbles of quartz; strongly acid; clear smooth boundary. 0 to 250 mm thick (0 to 10 in).

BE – 175 to 275 mm (7 to 11 in); yellowish red (5YR 4/8) sandy clay loam; weak fine subangular blocky structure; friable; few fine and medium roots; strongly acid; clear smooth boundary. 0 to 200 mm thick (0 to 8 in).

Bt1 – 275 to 77 mm (11 to 28 in); red (2.5YR 4/8) clay; moderate medium subangular blocky structure; firm; sticky, plastic; common thick distinct clay films on faces of peds; few fine flakes of mica; few small pebbles of quartz; strongly acid; gradual smooth boundary.

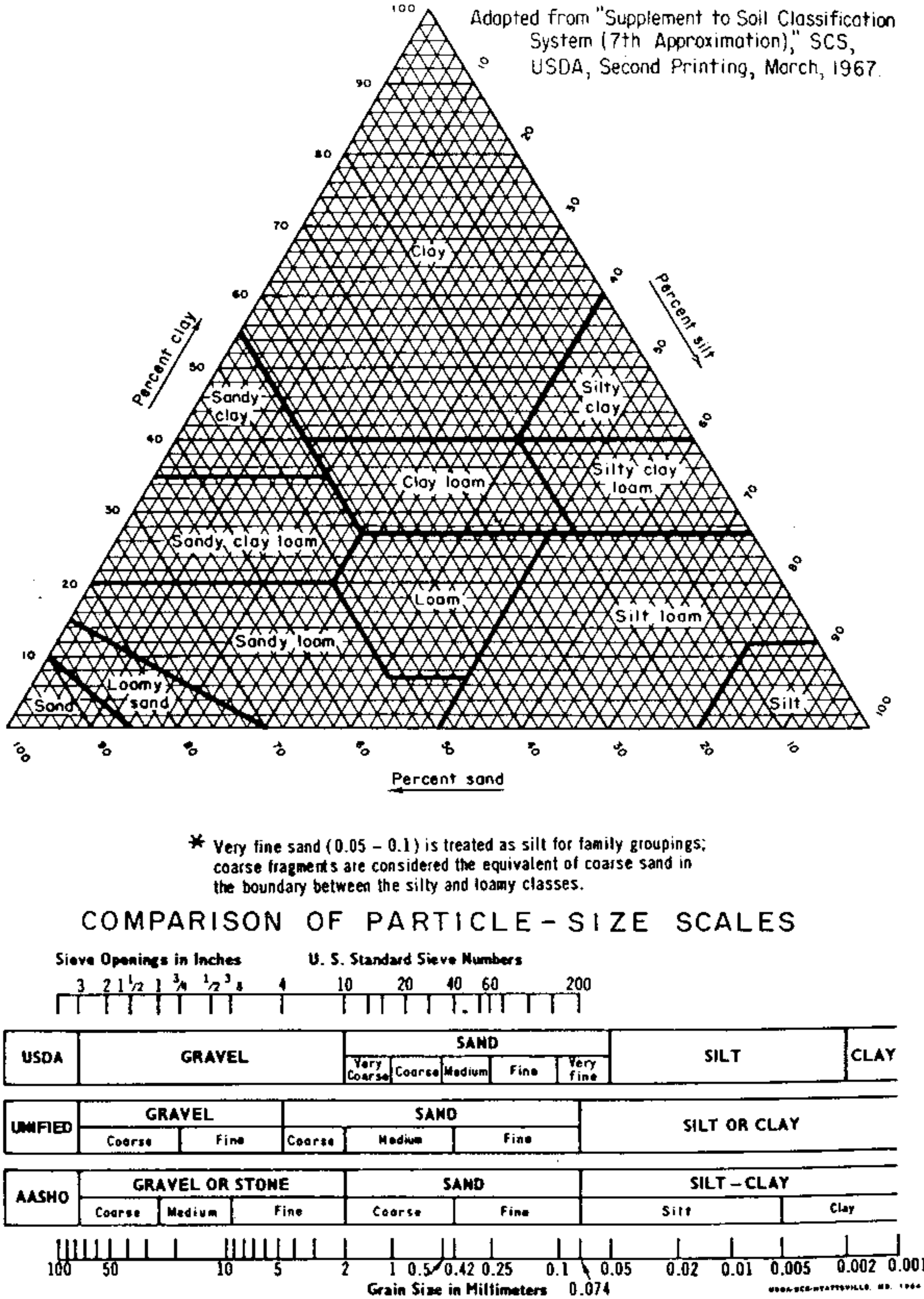


Figure 2-1.—Soil triangle of the basic soil texture classes. (Natural Resources Conservation Service)

Bt2 – 700 to 1000 mm (28 to 40 in); red (2.5YR 4/8) clay; moderate and weak medium subangular blocky structure; firm; sticky, plastic; common thin clay films on faces of peds; few to common fine flakes of mica; strongly acid; gradual smooth boundary. [Combined thickness of the Bt horizon is 600 to 1200 mm (24 to 48 in).]

Bt3 – 1000 to 1275 mm (40 to 50 in); red (2.5YR 5/8) clay loam; common medium distinct strong brown (7.5YR 5/6) mottles; weak medium subangular blocky structure; friable; few thin distinct clay films on vertical faces of peds; common fine flakes of mica; strongly acid; gradual smooth boundary. 175 to 500 mm thick (7 to 20 in).

C – 1275 to 1900 mm (50 to 75 in); mottled red (2.5YR 5/8), strong brown (7.5YR 5/8), and pale brown (10YR 6/3) loamy saprolite of gneiss; common pockets of clay loam; massive; friable; common fine flakes of mica; strongly acid.

Type Location: Note. – The county, state, and detailed directions are given to locate the particular parcel surveyed.

Agricultural soil classifications used for engineering purposes are of limited value. Information of this type is qualitative rather than quantitative; but, if carefully evaluated, agricultural soil classifications can often be used to advantage in the reconnaissance stage and in planning subsurface exploration. Additional information of how soil surveys are made is covered in the *Soil Survey Manual* [9], an in-house publication of the Natural Resources Conservation Service.

The agricultural soil survey report is designed to provide information useful to the farmer and to the agricultural community. However, in addition to the soil maps and soil profile descriptions contained in these reports, other valuable information is included. The reports discuss:

- topography,
- ground surface conditions,
- obstructions to movement on the ground,
- natural vegetation,
- size of property parcels,
- land utilization,
- farm practice and cropping systems,
- meteorological data,
- drainage,
- flood danger,
- irrigation,
- water supply and quality,
- nearness to towns, roads, and railroads,
- electric power, and
- similar data.

2-3. Remote Sensing Techniques.—

a. General—Remote sensing is defined as the act of gathering data concerning the earth's surface without coming into contact with it. Remote sensing is performed using devices such as cameras, thermal radiometers, multispectral scanners, and microwave (radar) detectors. Engineering and geologic interpretation of remotely sensed data may be simple or complicated, depending on the nature of the data and the objective of the study. Remote sensing is a tool which makes some tasks easier, which enables some tasks to be performed that could not otherwise be accomplished, but which may be inappropriate for other tasks. Depending on the situation, remote sensing may be extremely valuable or totally inappropriate. Some remote sensing interpretations can be used directly and with confidence; but for most applications, field correlations are essential to establish reliability. Appropriate specialists should be consulted when evaluating situations to determine if remote sensing methods can provide useful data.

b. Nonphotographic.—

*1. Scanners.—*Electronic sensors are used that have less spatial resolution than photographs, but which can obtain a variety of spectral data and thus allow a wide variety of image processing and enhancement techniques. Scanners can be either airborne or satellite.

*2. Video.—*A television type system is used. Video systems still have lower spatial resolution than photographic cameras, but provide products more quickly and are excellent for reconnaissance type work. They are highly useful for mapping and monitoring linear features such as rivers and canals.

c. Resolution.—

*1. 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.

*2. Spectral.—*The width of a part of the electromagnetic spectrum is the spectral resolution. Certain portions of the electromagnetic spectrum, including the visible, reflective infrared, thermal infrared, and microwave bands, are useful for remote sensing applications. Materials have spectral signatures and distinctive absorptive and reflective spectral characteristics which allow them to be recognized. Given sufficient multispectral data, digital image processing can produce unique spectral signatures to be identified. Instrument limitations, cost limits on computer processing, or lack of spectral contrast may preclude unambiguous identification of some materials.

d. Photography.—Photography provides the best spatial resolution, but less flexibility in spectral data collection and image enhancement.

1. General.—Several types of aerial photographs are available. An aerial photograph is a picture of the earth's surface taken from the air. It may be a vertical photograph or an oblique photograph more or less inclined. High oblique photographs include the horizon; low obliques do not. The vertical photograph is commonly used for topographic mapping, agricultural soil mapping, vegetation mapping, and geological interpretations.

2. Panchromatic.—Panchromatic photography (black and white) records images essentially across the entire visible spectrum. In aerial photography, blue is generally filtered out to reduce the effects of atmospheric haze.

3. Natural Color.—Images are recorded in natural visible colors. In addition to black and white aerial photographs available as contact prints, color photography can be obtained either as positive transparencies or opaque prints, black and white infrared, or color infrared. By using appropriate film and filters, photography may be obtained ranging from ultraviolet to near infrared.

4. Multispectral.—Photographs acquired by multiple cameras simultaneously recording different portions of the spectrum can enhance interpretation. Multiband cameras employing four to nine lenses and various lens, filter, and film combinations make possible photography within narrow wavelength bands to emphasize various soil, moisture, temperature, and vegetation effects to aid photo interpretation.

Except where dense forest cover obscures large areas, aerial photographs reveal every natural and human endeavor on earth within the resolution of the photographic system. Relationships are exposed which could not be detected on the ground under normal or routine surface investigation. Identification of features shown on a photo is facilitated by stereoscopic examination. Knowledge of geology and of soil science will assist in interpreting aerial photographs for engineering uses. Aerial photographs are often used for locating areas to be examined and sampled in the field.

Virtually the entire United States has been covered by aerial photography. An index map of the United States is available from the USGS and USDA. This map shows which of seven Government agencies can provide photographic prints for particular areas. When ordering photographs, specify:

- Contact prints or enlargements, glossy, matte finish, or Cronapaque
- Location should be given by range, township, and section, latitude and longitude
- State and county and the preceding location(s) can be shown on an enclosed index map of the area.
- Where possible, use the airphoto index to determine project symbol, film roll number, and exposure number to expedite the request.
- Stereoscopic coverage should be requested for most uses.

Aerial mosaics covering some of the United States are also available. A mosaic is an assemblage of individual aerial photographs fitted together to form a composite view of an entire area of the earth. An index map showing the availability of aerial mosaics of the United States, including the coverage and the agencies holding mosaic negatives, is available from the USGS Map Information Office.

Equipment is commercially available to produce orthophotographs that have a uniform scale and from which radial and relief displacement due to topography have been removed. Orthographs overprinted with contour lines can be acquired, if desired.

Aerial photograph interpretation of earth materials and geologic features is relatively simple and straightforward, but experience is required. Diagnostic features include terrain position, topography, drainage and erosional features, color tones, and vegetation cover. Interpretation is limited mainly to surface and near-surface conditions. Special cases arise, however, where features on a photograph permit reliable predictions to be made of deep, underground conditions. Although interpretation can be rendered from any sharp photograph, resolution is a limiting factor because small-scale photographs limit the amount of detailed information that can be obtained. The scale of 1: 20,000 is satisfactory for some engineering and geologic interpretation of surface materials. Small-scale photographs are used for highly detailed work such as for reservoir clearing estimates and for geologic mapping of damsites and reservoir areas.

Aerial photographs can be used to identify many terrain types and landforms. Stereoscopic photograph inspection of regional topography, local terrain features, and drainage conditions usually will suffice to identify the common terrain type. This permits the possible range in soil and rock materials to be anticipated and their characteristics to be defined within broad limits.

Geologic features that may be highly significant to the location or performance of engineering structures can often be identified from aerial photographs. In many instances, these features can be more readily identified on an aerial photograph than on the ground. However, aerial photograph interpretation is applicable only to those features that develop recognizable geomorphic features such as surface expressions such as drainage patterns, old river channels, and alignment of ridges or valleys. Joint systems, landslides, fault zones, lineations, folds, and other structural features are sometimes identified quickly in an aerial photograph, whereas it may be difficult to find them on the ground. The general attitude, bedding, and jointing of exposed rock strata, as well as the presence of dikes and intrusions, can often be seen in aerial photographs. The possibilities of landslides into open cuts and of seepage losses from reservoirs can be assessed.

Figures 2-2 and 2-3 are examples of identifiable geologic features determined from aerial photographs by an experienced interpreter using stereoscopic procedures.

5. Color Infrared.—Images use part of the visible spectrum and part of the near infrared, but resultant colors are not natural. Infrared film is commonly used and is less affected by haze than other types.

Other remote sensors use thermal, infrared, microwave, and radar wavelengths. Such phenomena as differences in the earth' s gravity and magnetic properties may be measured to afford additional interpretive tools.

e. Thermal Infrared Imagery.—Thermal infrared systems create images by scanning and recording radiant temperatures of surfaces. Some characteristics of thermal

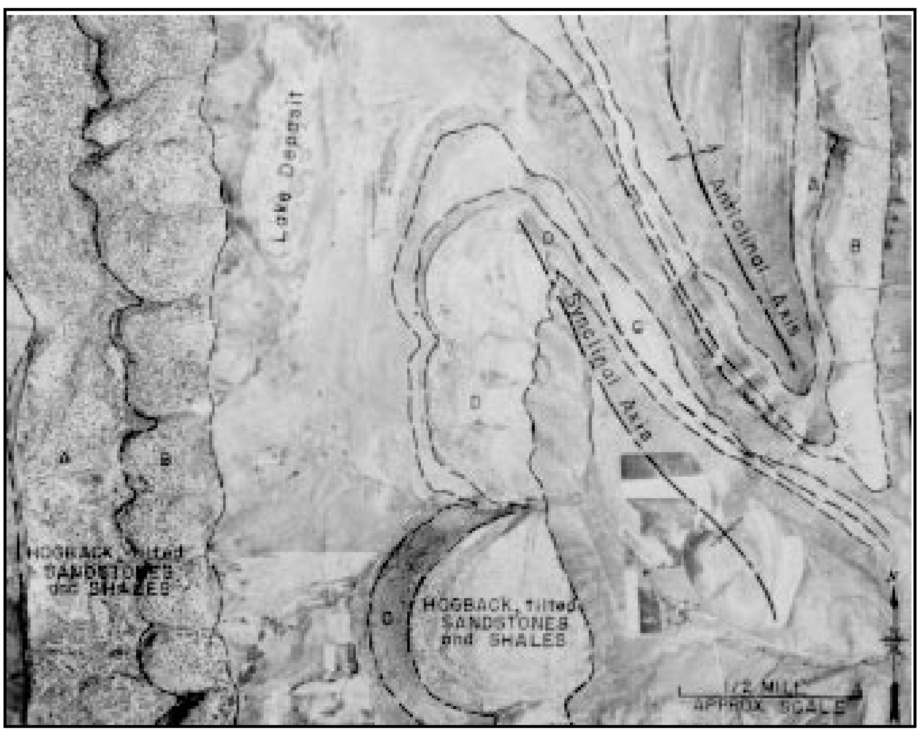


Figure 2-2.—Rock strata illustrating folding in sedimentary rocks. (A) Satanka formation, (B) Lyons formation, (C) Morrison formation, and (D) Lower and Middle Dakota formation. (U.S. Forest Service)

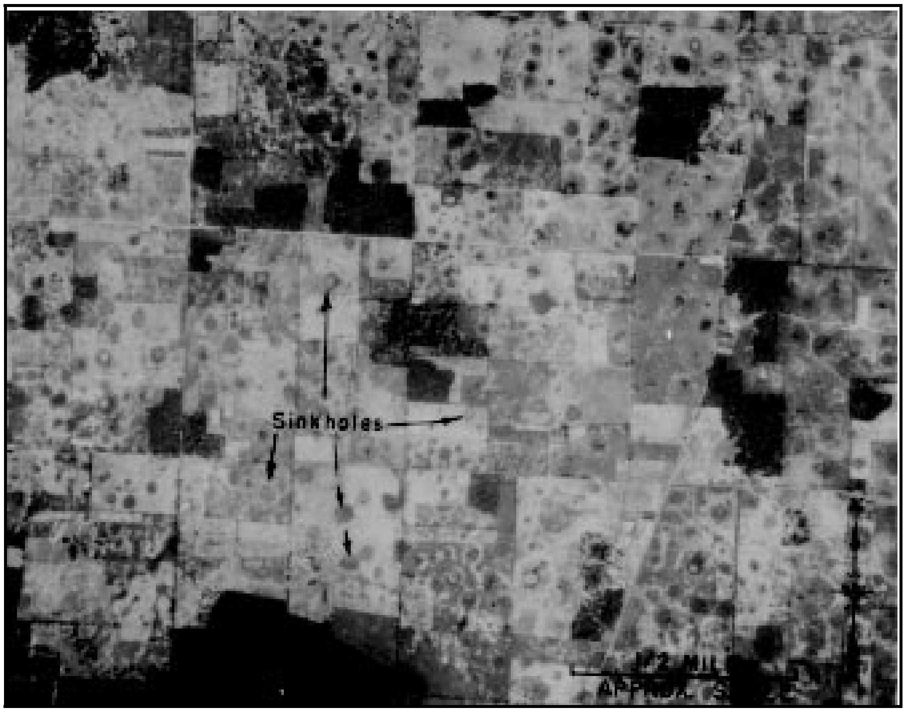


Figure 2-3.—Sinkhole plain indicating deep plastic soils over cavernous limestone, developed in humid climate. (Natural Resources Conservation Service)

image data produce digital image data which enable computer image processing, and others are unique to thermal infrared images which makes them valuable interpretive tools.

Thermal infrared imagery can be analyzed using conventional photo interpretation techniques in conjunction with knowledge of thermal properties of materials and instrument and environmental factors that affect data. Where thermal characteristics of a material are unique, thermal infrared imagery is easily and confidently interpreted. Vegetation patterns can be distinguished and can relate to subsurface conditions such as seepage beneath or through an existing dam. However, thermal characteristics of a material can vary with ambient temperature, moisture content, differential solar heating, and topography, and make interpretation difficult and ambiguous.

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

Because a digital image is actually an array of numerical data, the image can be manipulated by a computer for a variety of purposes. Geometric distortions caused by sensor characteristics can be removed, or distortions can be introduced if desired. Computer processing can be used to precisely register a digital image to a map or another image. Various techniques can be used to improve image quality and interpretability. Various types of data (for example, 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, sometimes can be extracted from an image by digital processing.

1. Airborne MSS Imagery.—Image characteristics.—A number of different airborne MSS systems are available with various spatial and spectral characteristics. Some systems are capable of recording 10 or more spectral bands simultaneously, ranging from ultraviolet to thermal infrared. Typical digital image processing techniques are used. The size of data sets and the number of separate spectral bands on some airborne MSS systems may require data consolidation or careful selection of data subsets for special processing. Typically, interpretation involves normal photointerpretive techniques along with knowledge of spectral characteristics and the data manipulations applied.

Advantages and limitations.—High resolution can be obtained; and with proper band selection and processing, even geochemical information may be possible from imagery. Depending on complexity of the geology or other features and on the size of the area studied, the necessary digital processing can become costly and may require considerable experimentation to obtain satisfactory results.

2. Satellite MSS Imagery.—Landsat is the U.S. satellite for civilian remote sensing of Earth's land surface. The Landsat MSS records four broad bands at 80-meter resolution. Though primarily oriented toward agricultural applications, it has proved useful for some geologic applications. The current Landsat is equipped with a thematic mapper (TM), which enhances spatial and spectral resolution compared to the MSS. The TM records seven narrow bands at 30-meter resolution and is more useful for geologic and geotechnical engineering applications.

A French satellite, Satellite Pour l'Observation de la Terre (SPOT), provides panchromatic imagery with up to 10-meter spatial resolution and MSS imagery at 20-meter resolution and is capable of stereo imaging.

Satellite imagery provides a synoptic view of a large area, which is valuable for regional studies, but the limited resolutions currently available restrict its value for engineering geology and geotechnical work.

g. 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 cloud cover. Some satellite radar imagery is available, but like Landsat, it is more useful for regional geologic studies than for engineering geologic or geotechnical application. Side-looking airborne radar (SLAR) produces a radar image of the terrain on one side of the airplane carrying the radar equipment (equivalent to a low-oblique airphoto). This imagery is useful when studying regional geologic faulting patterns and can be used to assist in determining specific sites for detailed fault investigations.

Image characteristics.—Radar imagery has some unusual distortions which require care when data are interpreted. Resolution is affected by several factors, and the reflectivity of target materials must be considered. Analysis and interpretation of radar imagery require knowledge of the imaging system, wave length polarization, look angle, and responses of target materials.

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

h. Applications to Geotechnical Engineering and Geology.—In general, the most useful form of remote sensing for geotechnical applications is aerial photography because of its high resolution, high information content, and low cost. Various scales of aerial photographs are valuable for regional studies and site studies, for both detection and mapping of a wide variety of geologic features of importance to engineering investigations.

Application of other forms of remote sensing to engineering work depends on the nature of the problem to be solved and the characteristics of site geology or other features. Some problems can be best solved using remote sensing; for others, remote sensing is of no value. In some cases, the only way to determine if remote sensing can be useful is to try it.

i. Availability of Imagery.—The entire United States has been imaged by the Landsat MSS. Partial Landsat TM and SPOT coverage of the United States is also available. The SLAR coverage of a portion of the United States is available. Other types of imagery are also available, but generally only for limited areas.

j. Mission Planning.—Numerous factors make planning a nonphotographic mission more complicated than planning for a conventional aerial photographic mission. Remote sensing mission planning should be done by, or in consultation with, an expert in the field.

The cost of remote sensing data acquisition can be relatively accurately estimated. However, cost of interpretation is a function of the time and image processing required; this cost is difficult to estimate accurately. Some forms of data are inexpensive to acquire and require little processing to be useful. Other forms may be costly to acquire and may or may not require considerable processing.

Two excellent reference books by Sabins [10] and Siegal and Gillespie [11] deal with remote sensing. A thorough treatment of remote sensing by Colwell [12] includes a 287-page chapter on geological applications with numerous color images.

2-4 Site Investigations and Land Form Types.—

*a. General.—*Surface investigations are generally limited to geologic mapping of water resource development sites, outlining areas of potential construction materials, and assessing hazardous waste remedial sites. However, useful relationships exist between surface land forms and the subsurface soil and rock conditions. The ability to recognize these landforms on topographic maps, on aerial photographs, and during preliminary site reconnaissance, when combined with a geologic knowledge of the area, is very important when characterizing a site and when locating potential sources for construction materials.

Soils in their geologic context are defined by their mechanism of deposition which, in turn, usually results in distinct landforms (physiographic surface expressions). A good representation of the subsurface soil engineering characteristics can be made by detailed surface mapping. However, a complete understanding of subsurface soils conditions can only be obtained through careful subsurface investigations. Knowledge of surface conditions can be used to economically lay out a subsurface investigation program.

Most water resource development projects are sited in river valleys containing a variety of soil (surficial deposits). Typically, valley sedimentation consists of alluvial channel, flood plain, and terrace deposits. Slope wash and colluvial materials, landslides, and alluvial fans are often found on the valley slopes but can also impinge upon the valley floor. Side ridges, especially where auxiliary structures may be sited, often contain residual soils forming in place. Other types of surficial deposits that form structural foundations at many Reclamation Projects are glacial, windblown (eolian), and lacustrine in origin. The physical description and properties of the types of soils most common to water resource development projects are described below, and some broad generalizations are made about the engineering characteristics and applications of these soil types associated with their particular landforms.

*b. Alluvial Deposits.—*These deposits constitute the largest group of transported soils. Alluvial deposits are stream transported and are usually bedded or lenticular, but massive deposits may also occur. The different types of sedimentary processes, combined with different source materials and the energy required during the transportation process, leads to materials ranging from well to poorly graded.

*1. Stream Channel Deposits.—*Typically, high energy stream channel deposits contain sands, gravels and cobbles. The size and extent of channel deposits can vary greatly. Channel deposits may comprise the entire valley floor and may be quite deep. In a wide valley, a meandering or braided stream may distribute channel deposits widely, resulting in thin, widespread layers or very irregular lenses of sand and gravel. These deposits can be poorly to well graded and can provide sources of filter materials and concrete aggregate. Stream channel deposits represent paths of potential high seepage. Variation in soil properties, seepage, consolidation, and possible low shear strengths may make them undesirable foundations for water retention structures. Figure 2-4 shows an aerial view and topography of river alluvium and terrace deposits.

The presence of a high water table is often a major difficulty in using channel deposits for borrow. When borrowing of material is being considered from a river deposit downstream from a water retention structure, such operations may change the tailwater characteristics of the stream channel, and the spillway and outlet works may have to be designed for the modified channel conditions.

*2. Flood Plain Deposits.—*The low gradient sections of nearly all streams have flood plains. Their surface expression is usually that of a smooth flat strip of land just above the stream channel. Clay, silt, and sand are typical soils of flood plains. Most flood plain deposits were once carried as sediment load while in the swiftly moving flood water of a channel (high energy) but settled out on the flood plain when water overflowed the channel and slowed (lower energy). In broad valleys, flood plains may be miles wide and may be flooded on an annual or more frequent basis. Flood plain deposits are often the source of the impervious zones for water retention structures.

*3. Terrace Deposits.—*Terraces are located above the flood plain (see fig. 2-4). They are usually elongate deposits along the streamcourse and have either a flat or a slight downstream dip on their upper surface. They also have well-delineated steep slopes, downward from the terrace on the stream side and upward on the valley wall side of the terrace. Along degrading streams, the alluvium in terrace deposits generally has the same size range as adjacent channel deposits. Terrace deposits can be sources of sand and gravel, but often are of limited extent and may require washing and processing.

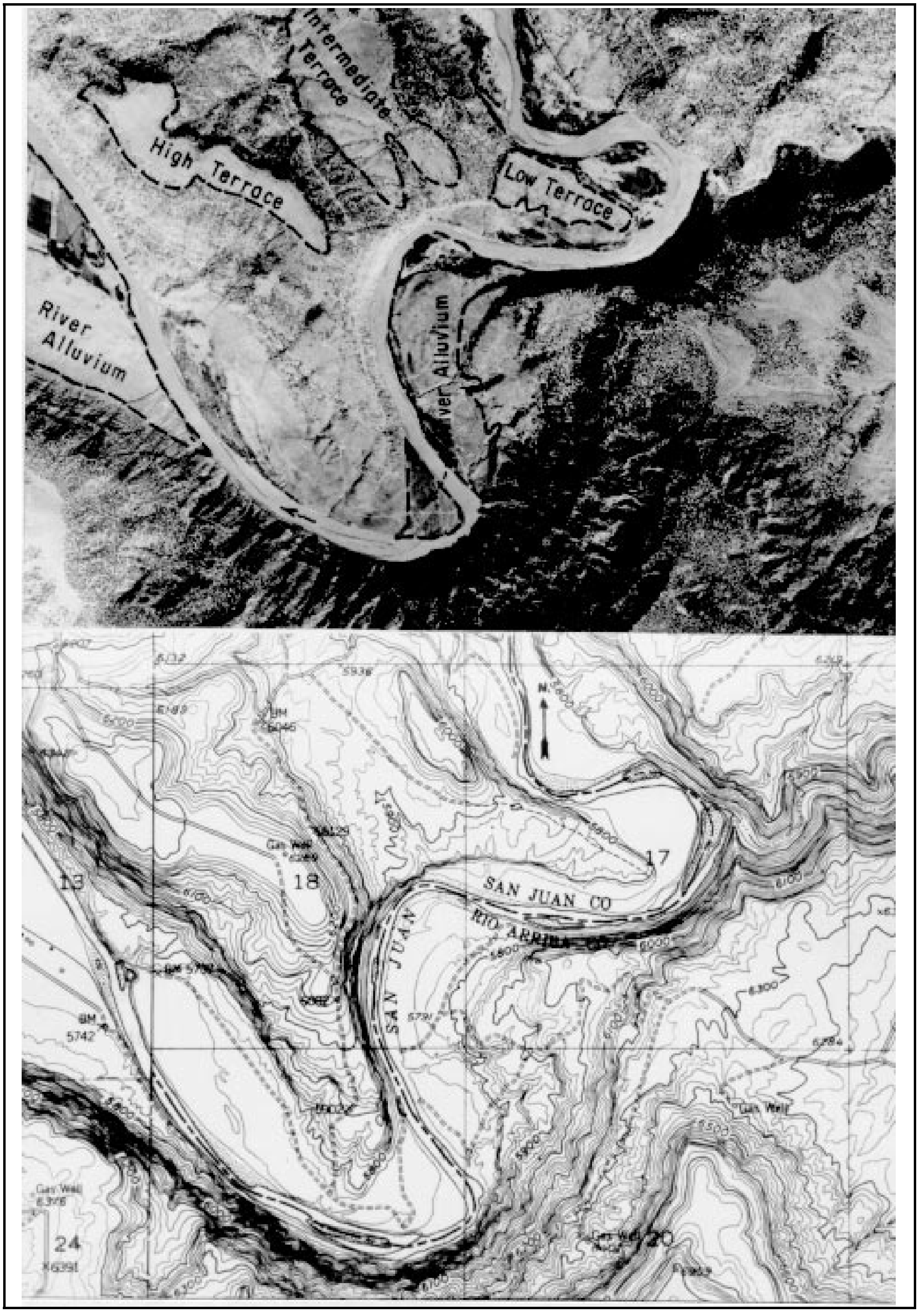


Figure 2-4.—Aerial view and topography of stream deposit showing river alluvium and three levels of gravel terraces. (U.S. Geological Survey)

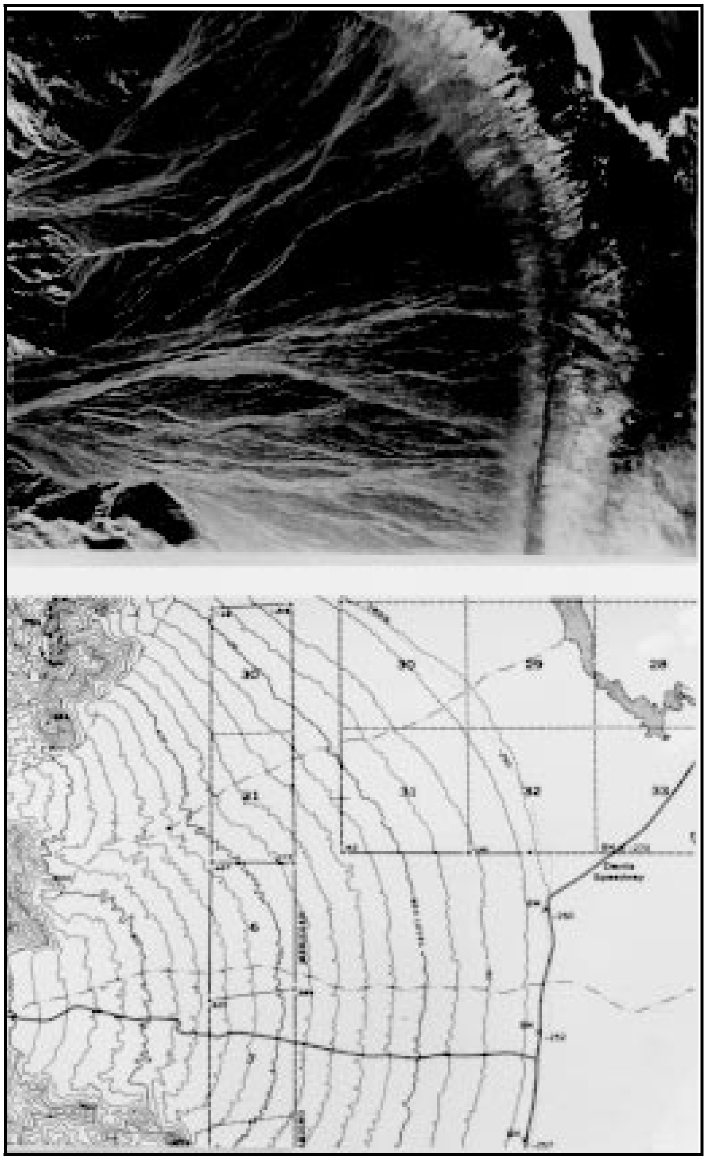


Figure 2-5.—Aerial view and topography of an alluvial fan, a potential source of sand and gravel. (U.S. Geological Survey)

4. Alluvial Fan Deposits.—Alluvial fans are typically gently sloping, fan-shaped masses of soil deposited in locations where an abrupt decrease in stream gradient occurs (see fig. 2-5). Large alluvial fans, many miles in width and length, occur along mountain fronts, where steep streams from mountain drainages spread out on relatively flat valley floors. Aggrading stream channels migrate laterally along the surface of the fan as temporary channels are both eroded and filled. Much mixing of the soils may result, and lenticular, poorly to well-graded materials are typical laterally across the fan. In contrast, there is a longitudinal grading from the head to the toe of the fan. Coarser material is deposited first and found on the steeper slopes at the head of the fan, while the finer material is carried to the outer edges. Grading of a fan depends on a combination of source material, water velocity, and slope. In arid climates, where mechanical weathering generates coarser particles with associated steeper slopes, the fans may be composed largely of rock fragments, gravel, sand, and silt. If the source produces fine materials, large deposits of silty sands or sandy silts may occur. In humid climates, where chemical weathering tends to generate finer particles and landforms have flatter slopes, the alluvial fan can contain much more sand, silt, and clay. In the Western

United States, all types of construction materials are found in alluvial fan deposits, but the percentages of clays are generally small, and when used for impervious zones in water retention structures, may be erodible.

c. Slope Wash or Colluvial Deposits.—Slope wash is a general term referring to loose, heterogeneous soils deposited by sheet wash, or slow, continuous downslope creep, usually collecting at the base of slopes or hillsides. Angular rock fragments are locally very common. The types of slope wash soils generally reflect the bedrock of the slopes on which they occur; that is, clays occur on slopes with shale bedrock, and sandy deposits on slopes with sandstone bedrock. Slope wash may typically vary from thin deposits high on a slope to tens of feet thick near the base of the same slope and are often of low density. Slope wash deposits, where relatively thick, can provide good material for the impervious zones of water retention structures. If rock fragments are too large and numerous, these soils may be undesirable for construction purposes. Unless located within the confines of the reservoir, shallow slope wash deposits often make undesirable borrow sources due to the small volumes of material available.

d. Lacustrine Deposits.—Lacustrine deposits, or lakebed sediments, are the result of sedimentation in still water. Lacustrine deposits are likely to be fine-grained silt and clay except near the deposit margins where currents from tributary streams may have transported coarser materials. Frequently, lacustrine stratification is so fine that the materials appear to be massive in structure. However, a color and grain size difference usually exists between successive beds, and the layered structure often can be observed by drying a slice from an undisturbed sample. Lacustrine soils are likely to be impervious, compressible, and low in shear strength. Their principal use in water development projects has been for impervious cores in earthfill dams, for impervious linings of reservoirs and canals, and for low embankments. These soils often have moisture contents exceeding optimum but can be used with careful moisture control. Treatments may include excavation, transport to drying pads, and discing to accelerate the sun's drying action. In extreme cases, drying can be accomplished using drying kilns if only small quantities are required.

Generally, lacustrine deposits provide poor foundations for structures. The engineering characteristics of these deposits may be so questionable that special laboratory and field testing may be required even during the reconnaissance design stage. Close coordination between the exploration and design teams regarding soil sampling and testing is imperative whenever structures must be founded on lake sediments.

e. Glacial Deposits.—Glacial soils produced and deposited by Pleistocene continental ice sheets are prevalent in a wide area across the Northern United States. Alpine or mountain glaciation has also produced glacial soils in the Rocky Mountains, the Sierra Nevada, the Cascades, and other high western mountain ranges and volcano flanks. Glacial modification of valley shapes and deposition of glacial soils are important to the siting, design, and construction of many water retention structures. Material deposited by glaciers is generally divided into two classes: (1) tills, which are deposited directly from or by the glacier; and (2) outwash or glaciofluvial deposits, which are deposited by water issuing from the glacier. Tills and outwash deposits are often mixed in varying proportions, and generalizations concerning the soil characteristics of any glacial land form are difficult to make. A separate classification is made for deposits laid down in lakes related to glaciation. These lake-related deposits are glaciolacustrine soils and are similar to lacustrine deposits.

1. Tills or Glacial Deposits.—Tills are glacial soils deposited directly from glaciers without subsequent reworking by outwash. Tills often form distinctive landforms and are often described as soils of landforms such as morainal deposits. Tills are predominantly unsorted, unstratified soils consisting of a heterogeneous mixture of clay, silt, sand, gravel, cobbles, and boulders ranging widely in size and shape. The gradation and types of rock fragments and minerals found in till vary considerably and depend on the geology of the terrain over which the ice moved and the degree of postdepositional leaching and chemical weathering. Certain glacial tills may be used to produce impervious materials with satisfactory shear strength, but removal of cobbles and boulders is necessary so that the soil can be compacted satisfactorily. Where till deposits have been overridden by ice, the resulting high, in-place density may make them satisfactory foundations for many hydraulic structures. Typical landforms often composed wholly or partially of till are:

- Till plain or ground moraine, which has a flat to undulating surface.
- Terminal or end moraine (see fig. 2-6), usually in the form of a curved ridge, convex downstream, at right angles to the direction of ice advance and which marks the maximum advance of a glacier.

- Lateral or medial moraines, or till ridges usually formed parallel to the direction of ice movement and consisting of material carried on the surface of the glaciers sides and center (see fig. 2-6).

2. Outwash or Glaciofluvial Deposits.—The meltwater from glaciers produces streams of water which wash out and carry away material of all sizes that have been produced by glacial abrasion. The material is deposited in front of or beyond the margin of an active glacier. In contrast to tills, outwash deposits are usually sorted and poorly stratified by stream action. Outwash deposits consist of the same range of sizes, from fines to boulders, as tills.

Some landforms created by outwash deposits are outwash fans, outwash plains, eskers, and kames. An outwash fan is a fan-shaped accumulation of outwash deposited by streams beyond the front of an active glacier. An outwash plain is formed by numerous coalescing outwash fans. Eskers, remnants of the beds of glacial streams that once flowed under the ice, are winding ridges of sand, gravel, and cobbles that are excellent sources of pervious materials, filter materials, and concrete aggregate. Kames are mounds or short ridges that are composed of materials similar to those of eskers and that were typically deposited by subglacial streams at the margins of glaciers.

f. Eolian Deposits.—Eolian soils have been transported and deposited by the wind. Eolian soils are composed mainly of silt and/or sand-sized particles. They very often are of low density, have low bearing strength, and are generally poor foundations for typical water resource structures. When used as foundations, detailed explorations defining the extent and physical properties of the eolian deposits are necessary. Density data are very important, and this information, combined with Atterberg limits data, can be used to assess collapse potential of the soils. The most common eolian soils are loess and dune deposits.

1. Loess.—Large areas of the Central and Western United States, especially the Mississippi and Missouri river drainage system, the High Plains, the Snake River Plain, the Columbia Plateau, and some of the Basin and Range valleys are covered with loess. Figure 2-7 shows typical loessial topography by map and aerial photograph. These deposits have a remarkable ability for standing in vertical faces, although local sloughing and erosion occur with time. Loess consists mostly of particles of silt or fine sand, with a small amount of clay that binds the soil grains together. Loessial deposits may contain sandy portions which are lacking in binder and are very pervious. Undisturbed loess has a characteristic structure marked by remnants of small vertical root holes that makes it moderately pervious in the vertical direction. Figure 2-8 shows the structure of loess. Although of low density, naturally dry loessial soils have a fairly high strength when dry because of the clay binder. This strength, however, may be readily lost upon wetting, and the soil structure may collapse. When compacted, loessial soils are impervious, moderately compressible, and of low cohesive strength, and they exhibit low plasticity. Usually, loessial soils plot in the ML group or the borderline ML/CL and SM/ML groups of the Unified Soils Classification System [13, 14, 15]. Figure 1-22 shows a steep cut face in loess.

2. Dune Deposits.—Although not as widespread as loess, dune deposits are common in some Western States. Although active dunes are easily recognizable because of their exposed soils and typical elongated ridge shape that is transverse to prevailing winds or a crescent shape that is convex upwind, inactive dunes may be covered with vegetation or detritus so that their extent is not immediately obvious. Dune deposits are usually rich in quartz minerals and uniform in grain size—usually in the range of fine- to medium-grained sand. These sands have no cohesive strength, moderately high permeability, and moderate compressibility, although density and associated compressibility often vary widely. Generally, dune sands fall in the SP group of the Unified Soil Classification System. In their natural state, dune sands are extremely poor materials on which to site hydraulic structures. They erode easily in canal prisms and are generally poor foundations for footings. Cut slopes in dune sands will stand only at the angle of repose or flatter, whether in the wet or dry state. If excavated underwater, slopes will flow (run) until the angle of repose is reached. Dune deposits are usually subject to liquefaction and seepage problems where they are present in the foundation of existing dams. However, dune deposits can be a good source of material for uniform-grain-size filters.

g. Residual Soils.—Residual soils are derived from in-place chemical and mechanical weathering of parent rock, and differ from all of the other soils discussed in this section in that they are not transported and are not associated with unique landforms. The nature of these soils depends on the texture, structure, and mineralogy of the parent rock, climate, rate of surface erosion, ground-water table, and local vegetation. Because of the many factors affecting the development of residual soils, at any one site there may be great variability laterally and vertically.

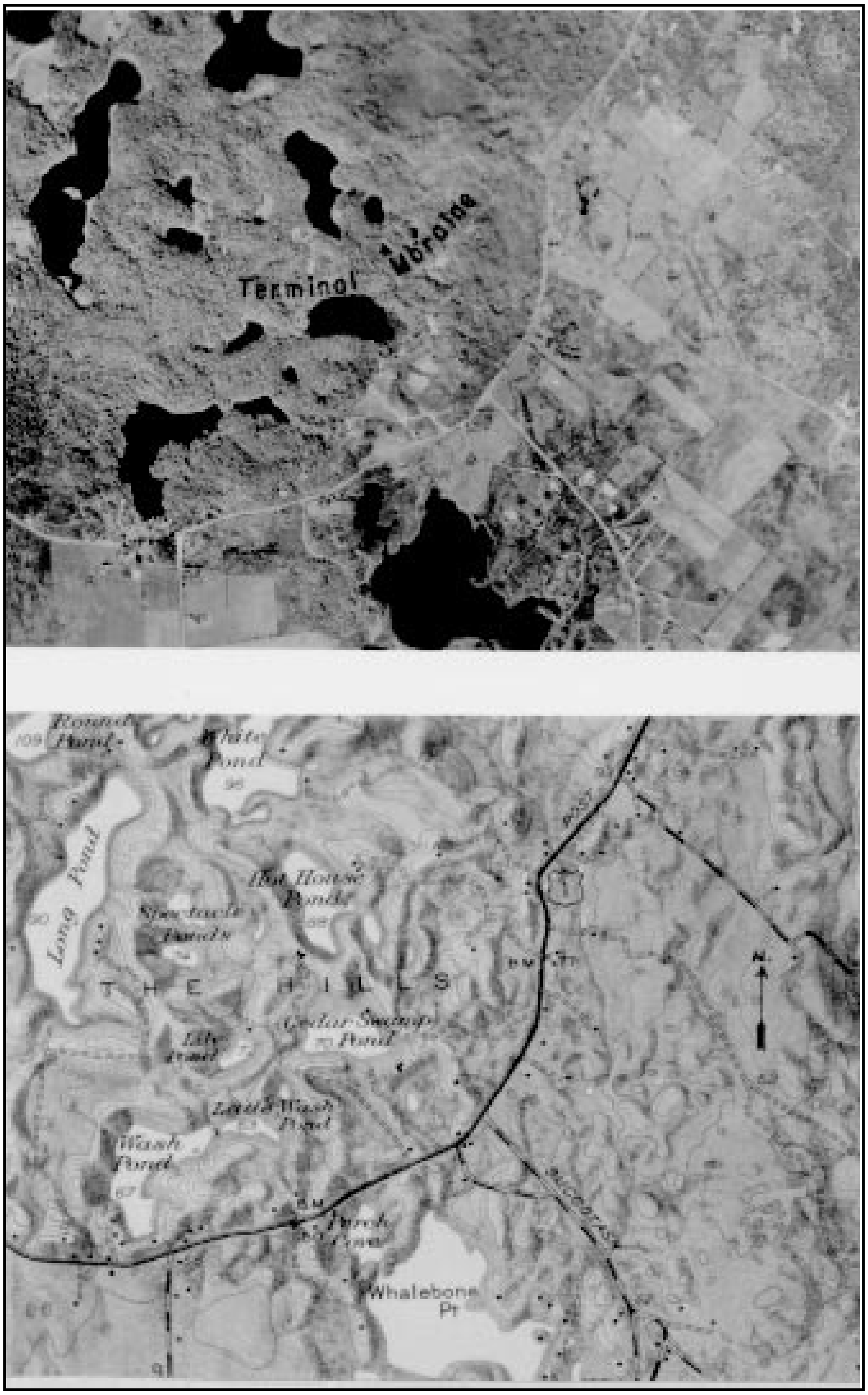


Figure 2-6.—Aerial view and topography of terminal moraine of continental glaciation. (Natural Resources Conservation Service)

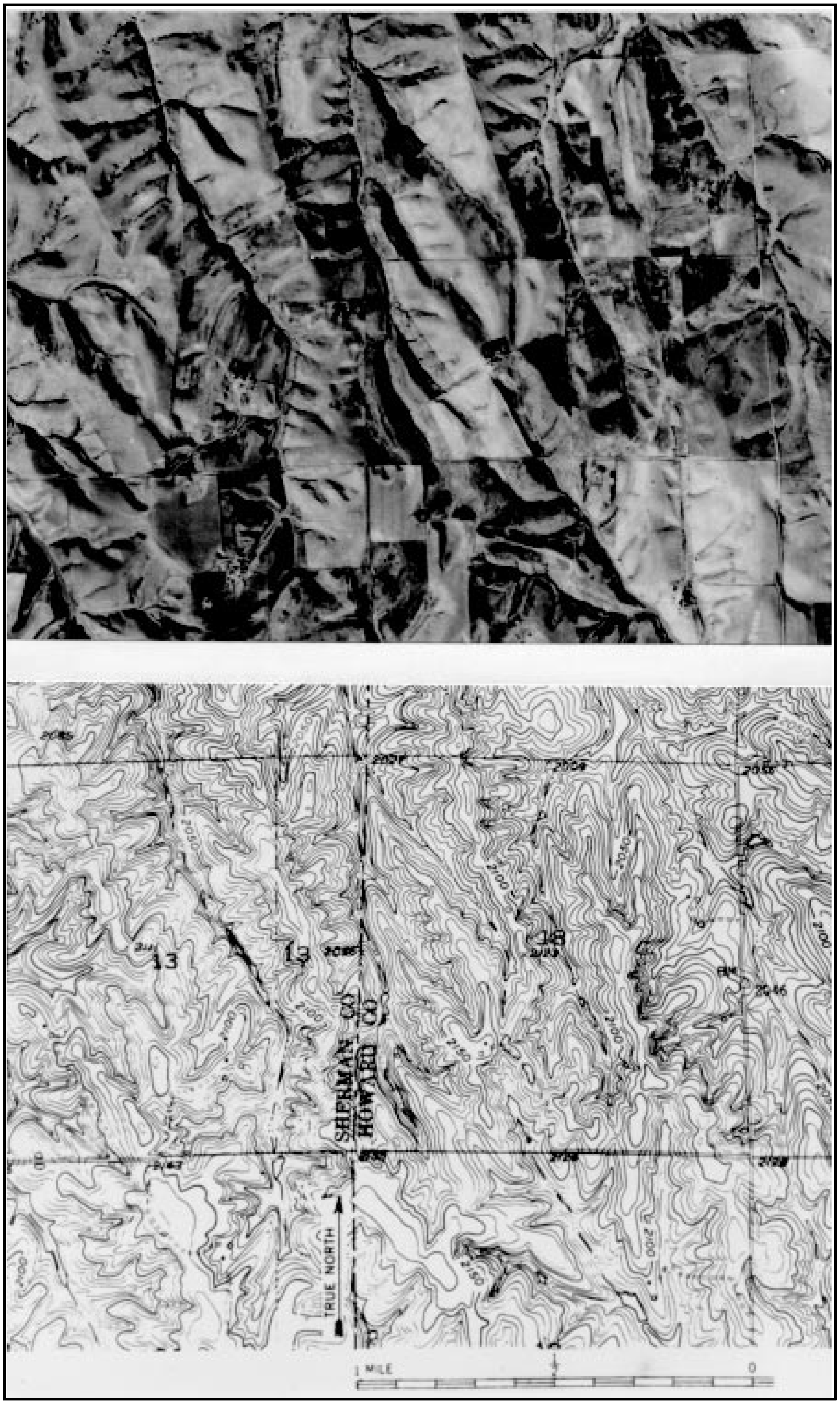


Figure 2-7.—Aerial view and topography of loess identified by smooth silt ridges; usually parallel, right-angle drainage patterns; and steep-sided, flat-bottomed gullies and streams. (Natural Resources Conservation Service)

Figure 2-8.—Photomicrograph showing typical open structure of silty loess.

Large regions of the United States are underlain by residual soils, and many dams are both composed of and founded on them. These soils typically have a natural vertical profile consisting of an A-Horizon at the top, with B- and C-Horizons progressively below. The A-Horizon is usually very thin (a few inches), relatively sandy (clays are moved to the underlying horizon by rainwater percolation), and high in organic material. The B-Horizon is relatively clayey. Its original mineral content, especially the silicate minerals, is highly weathered, and there is no trace of parent rock structures. The C-Horizon contains varying percentages of unweathered mineral grains of the parent rock and shows varying degrees of the parent rock structure.

When residual soils are used for the foundations and zones of embankment dams, the designer should consider that the B-Horizon is usually less permeable than the C-Horizon. Failure to take the cutoff trench through the C-Horizon and the use of some C-Horizon material in impermeable zones have resulted in seepage both through and under dams. Relict joints, foliation, bedding planes, and faults in C-Horizons have resulted in concentrated seepage with subsequent foundation piping. These same relict structures may also cause weak, potential failure planes in dam foundations.

Because the type of parent rock has a pronounced influence on the character of residual soils, the rock type should always be determined. The degree to which alteration has progressed largely governs strength characteristics. Laboratory testing, including testing for dispersion, is required if the material appears questionable or when important or large structures are planned. Petrographic analyses for identification of clay minerals in residual soils is required for an understanding of their engineering properties.

A distinguishing feature of many residual soils is that individual particles in place are angular but soft. During construction, handling of the material may appreciably reduce the grain size so that the soil, as placed in the structure, may have entirely different characteristics than shown by standard laboratory testing of the original soil. Decomposed granites, which at first appear to be free draining granular materials, may break down when excavated, transported, placed, and compacted. As a result, this material may become semipervious to impervious. Special laboratory and field testing programs are often required to determine changes that might occur in the engineering characteristics of residual soils. During this testing, special attention should be paid to soil breakdown by drying, by impact compaction, and by kneading compaction action. Full scale field compaction test sections are sometimes appropriate before proper decisions can be made regarding the use of residual soils.

2-5. Subsurface Investigations.—

a. General.—Subsurface geotechnical exploration is performed primarily for three purposes: (1) to determine what distinct masses of soil and rock exist in a foundation or borrow area within the area of interest; (2) to determine the dimensions of these bodies; and (3) to determine their engineering properties.

In the engineering evaluation of a foundation or borrow area, soil structure should be delineated by means of profiles or plans into a series of masses or zones within which soil properties are relatively uniform. Soils having variable properties can be evaluated, provided the nature of the variation can be defined. Determination of dividing lines between what may be considered uniform soil masses must usually be done on the basis of visual examination and requires considerable judgment. The soil classification system described in chapter 1 is a satisfactory guide for consideration of soils in a disturbed state. For evaluating soils in the undisturbed state, additional qualifying factors required are in-place water content, density, firmness, and stratification. Color and texture are also helpful in delineating soil masses of uniform characteristics. Occasionally, the only uniformity found in a soil deposit is its heterogeneity. However, by exercising careful analysis, a pattern may be found in the soil mass for use in geotechnical analysis.

The dimensions of these masses of soil are determined by methods analogous to those used in surface surveying such as making cross sections or by contouring the upper and lower surfaces (or isopachs). The method used depends somewhat on the type of structure involved. Point structures (buildings) or line structures (canals, pipelines, and roads) are best visualized by cross sections; massive structures (dams) are best visualized by topographic plan maps in addition to cross sections.

Unfortunately, the problem of locating measuring points or the "breakpoints" of buried surfaces is difficult because the subsurface cannot be seen, and the cost of investigating the entire area with a grid of test holes is usually great. The normal procedure used for an investigation is to begin with an estimation of the breakpoints' locations based on a geologic interpretation of the subsurface. An initial series of test borings—supplemented by data from geophysical surveys—can be used for preliminary delineation of buried surfaces. Breakpoint locations can be further defined by locating additional test borings using successive approximations after considering geophysical data. A grid system of test holes is normally used only on large or critical borrow areas and on foundations (for large earth dams) where subsurface irregularities cannot be established by other means. Simple, inexpensive penetration tests such as the electronic cone penetrometer test can provide useful information which can be used for more detailed delineation of subsurface irregularities.

b. Point Structures.—For structures such as small buildings, small pumping plants, transmission towers, and bridge piers, a single test hole is often adequate. Larger structures may require more test holes. When the exact location of a structure is dependent on foundation conditions, the number of test holes required should be increased. Two or three test holes are used for preliminary exploration to establish general foundation conditions; the investigation requirement can usually be reduced for later stages. Figure 2-9 shows suggested depths of preliminary exploratory holes for various point structures. Figure 2-10 is an example of a soil profile at a pumping plant site.

c. Line Structures.—Exploration requirements for the foundations of canals, pipelines, and roads vary considerably according to the size and importance of the structure and according to the character of the ground through which the line structure is to be located. Spacing of holes or other explorations will vary, depending on the need to identify changes in subsurface conditions. Where such structures are to be located on comparatively level ground with uniform soils such as the plains areas, fewer holes along the alignment may suffice for foundation investigation requirements. In certain instances, special investigations may be required such as test pits and in- place densities measurements for pipelines or hand cut block samples to study collapse potential in areas of low density soils.

Test hole requirements are quite variable for design and specifications purposes on major structures; landslides, talus and slopewash slopes, and alluvial fans require thorough exploration. Usually, a test hole at the point of highest fill or at valley bottom is needed. Additional off-line holes may be required for all of these features, depending on topographic, geologic, and subsurface conditions. During investigations, special consideration should be given to line structure excavation such as dewatering requirements and cut slope stability. Water levels should be monitored in test holes for a sufficient length of time to establish seasonal ground-water level variation. Dewatering design parameters must be anticipated throughout investigations. Useful data regarding permeability can be obtained from laboratory gradation tests of critical soils and from borehole permeability tests. Excavations through soft, fine-grained soils or through wet areas should be investigated more thoroughly. Shear strength data may be required in certain critical excavation areas.

When major, costly features are involved, detailed explorations are required even for feasibility estimates. Figure 2-11 shows the suggested minimum depths of exploratory holes for major line structures. Greater depths are sometimes required to determine the character of questionable soils. Figure 2-12 is an example of a geologic profile along the centerline of a proposed pipeline.

d. Damsites.—Sites for dams are initially selected because the valley at that location is narrower than at other places or the abutment and foundation conditions appear superior. The exploration program must be directed toward determining the detailed subsurface conditions. Study of surface geology will aid in deciding locations for initial test holes. In some cases, a surface geologic map, together with four or five test holes and to a depth reaching a competent and impervious formation, will delineate conditions well enough to proceed with a feasibility design of a dam. A line of test holes along the potential dam axis, the outlet works, and spillway structure are also important. Some of the holes may be located to satisfy more than one requirement.

A dam forms only a small part of a reservoir. To assure the adequacy of a reservoir, exploration at considerable distances from the damsite is frequently necessary to locate possible routes of water loss and to identify potential landslide conditions. The entire reservoir rim should always be carefully inspected for landslide potential, rock toppling, areas of high water loss, or other conditions which might jeopardize the safety or economics of the project. Data needed for reservoir studies require detailed investigations and thorough surface geologic mapping as described in the *Design of Small Dams* [16].

As the design progresses, additional boreholes in the valley floor, in dam abutments, and at locations of appurtenant structures will be required. The number, spacing, angle, diameter, and depth of these holes depend not only on the size of the dam and structures, but also on type and complexity of the foundation. Usually, this exploratory program is prepared by the exploration team which includes representatives from both the design and field offices. Figure 2-13, an example of a generalized geologic section for a damsite, shows exploratory holes for an earth dam.

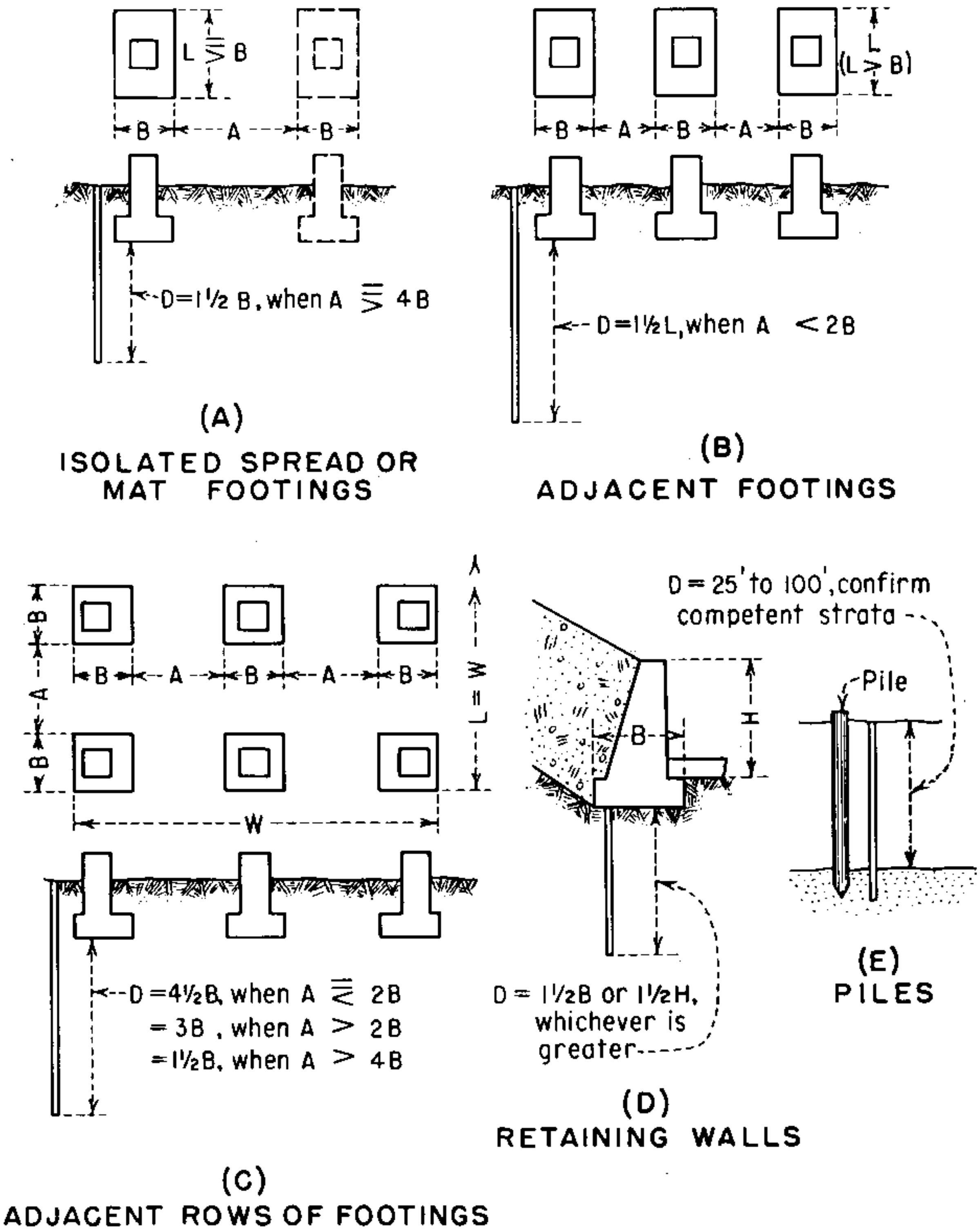


Figure 2-9.—Depth of preliminary exploratory holes for point structures.

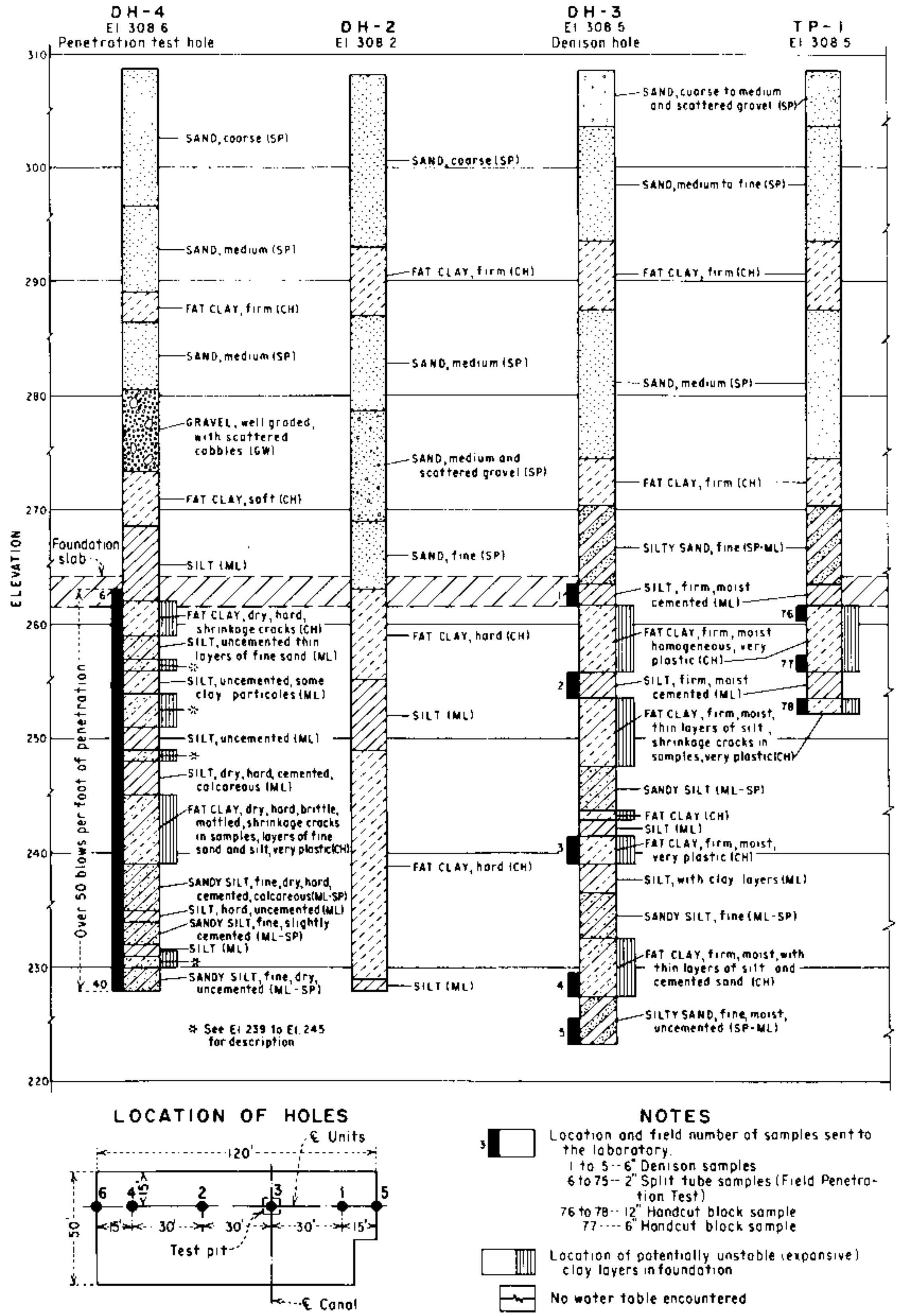


Figure 2-10.—Soil profile at a pumping plant.

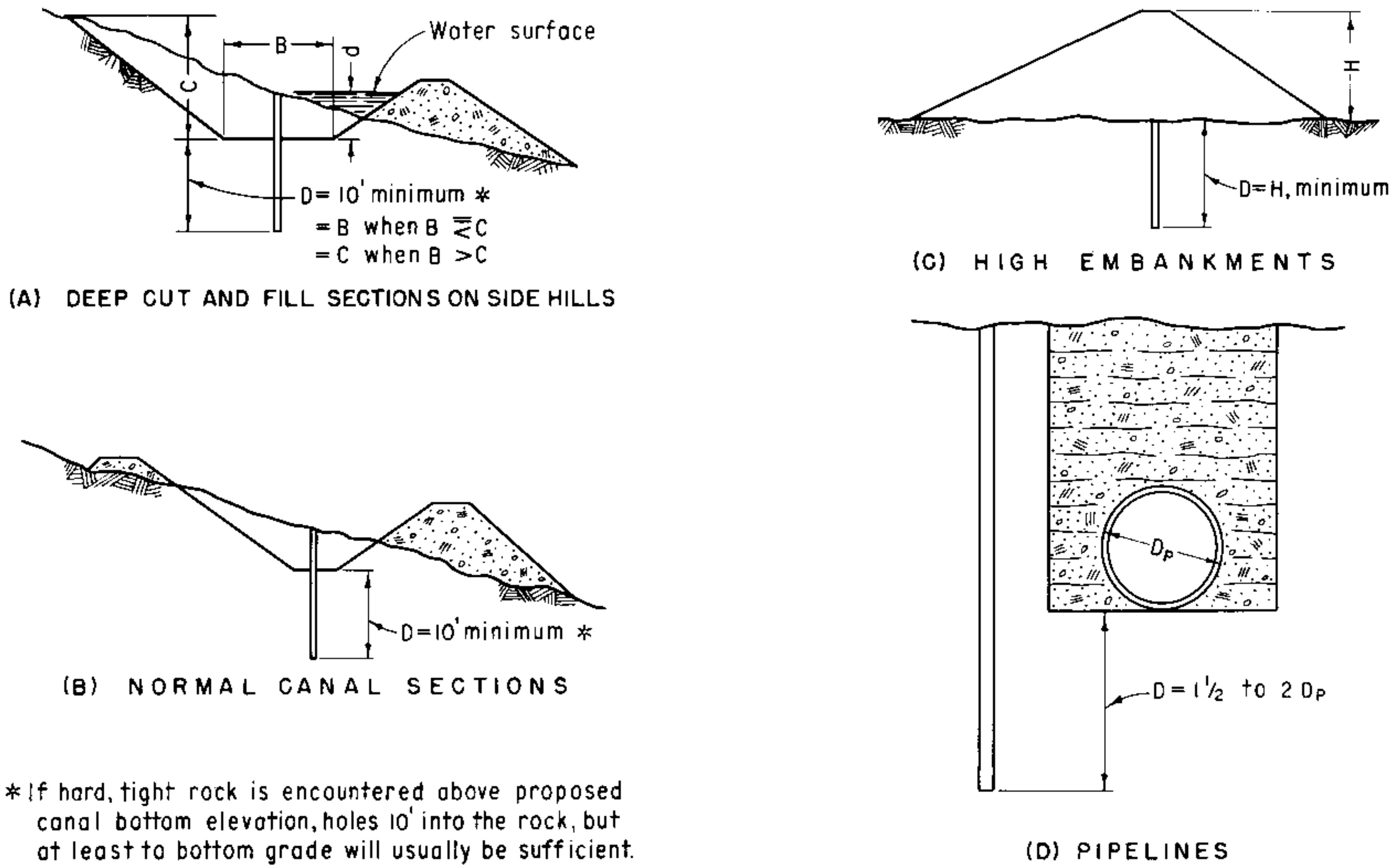


Figure 2-11.—Depth of preliminary exploratory holes for canal, road, and pipeline alignments.

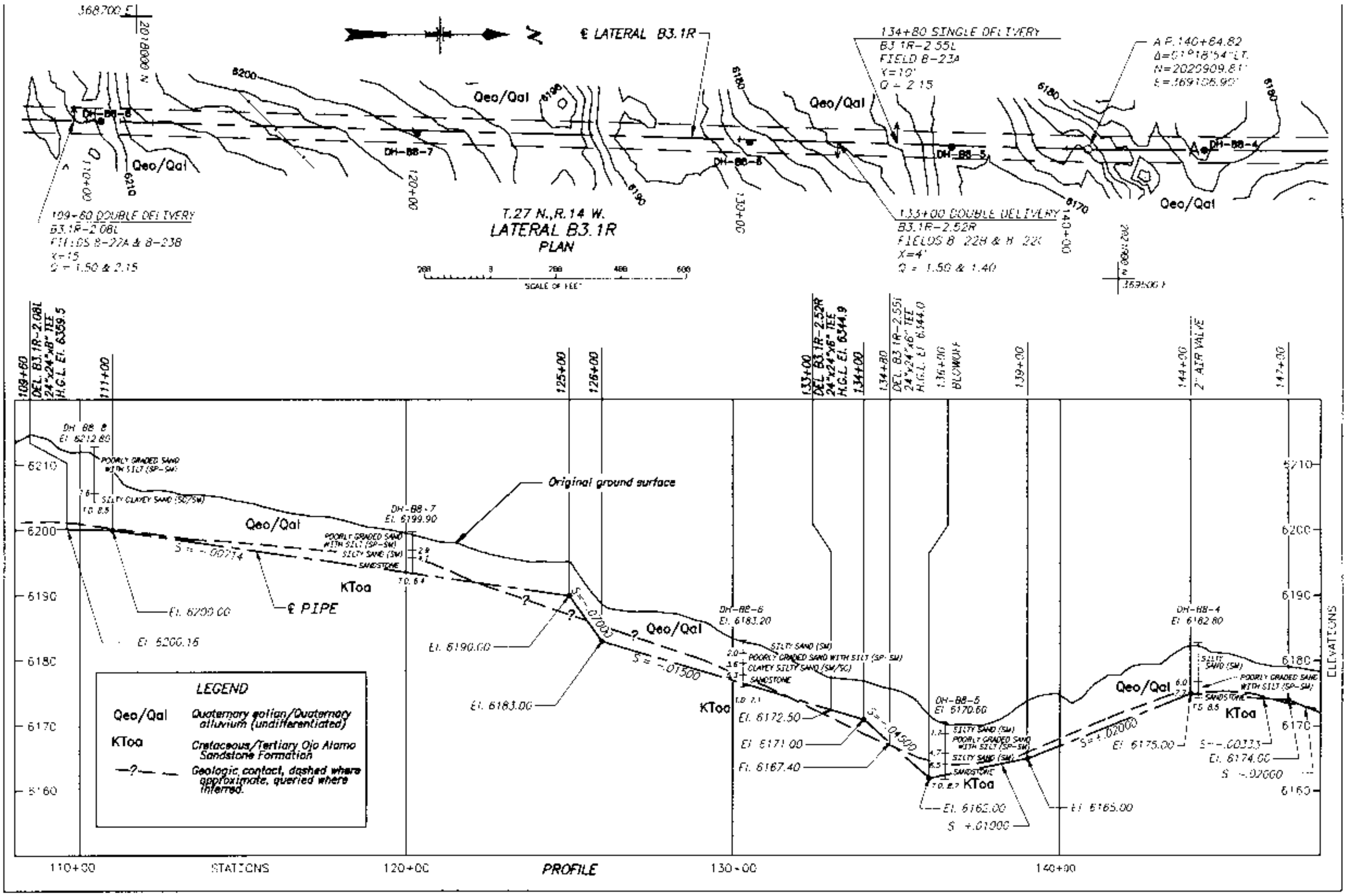


Figure 2-12.—Example of a geologic profile along the centerline of a pipeline.

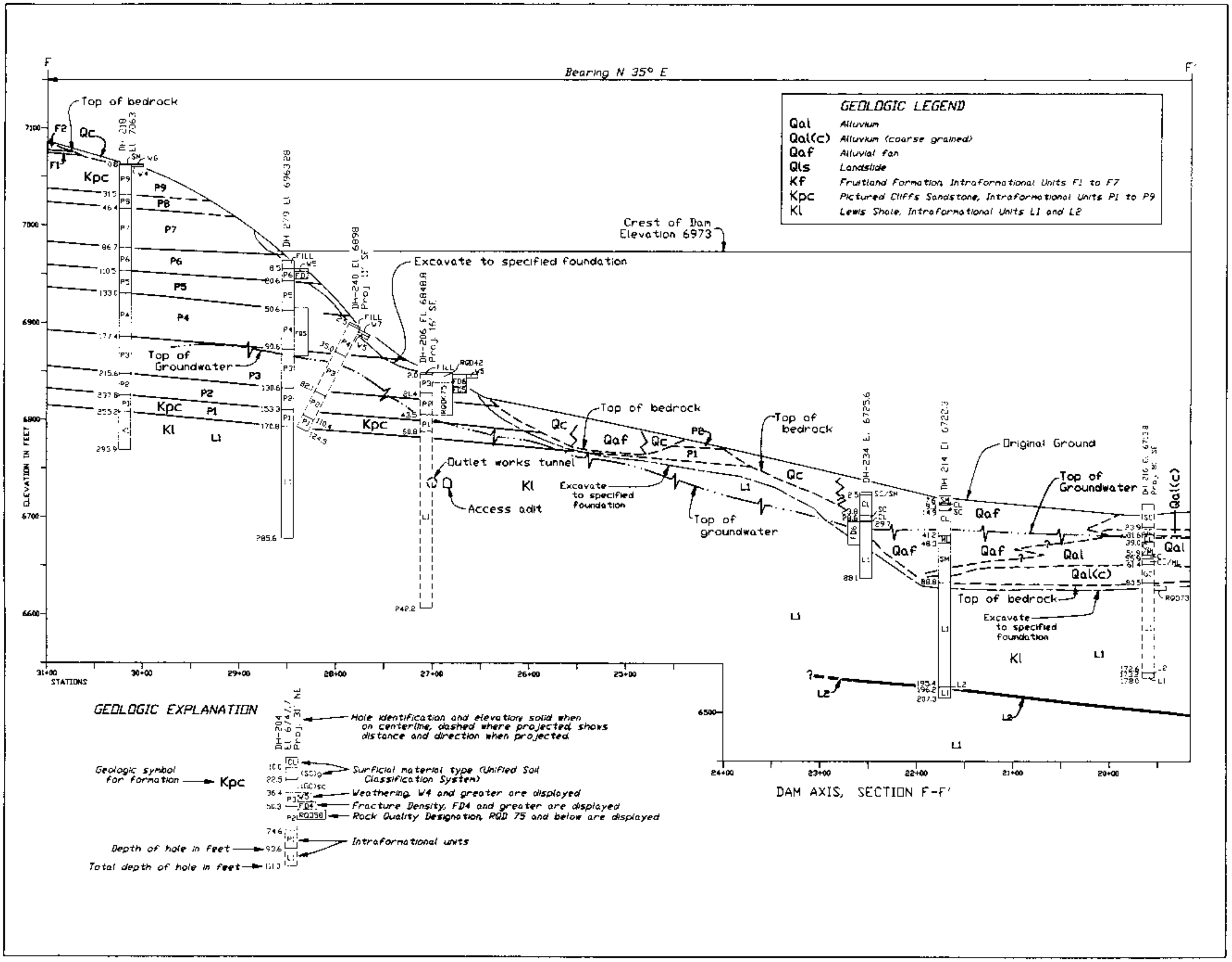


Figure 2-13.—Geologic section of a damsite.

e. Tunnels.—The investigation program must be in balance with anticipated geologic complexities and stage of study. Design studies may require development of detailed geologic investigations. Drill holes are required at each portal area to define the extent and character of overburden materials and along the alignment where overburden materials may be pertinent to tunnel design. This is in addition to geologic and rock mechanics data for tunnel design. Horizontal boreholes and angle boreholes are often used to further define soil and rock properties, and geophysical studies can provide useful supplemental data. Studies of rock hardness, jointing, and stratification should be performed to determine the best method of excavation and for proper tunnel lining and support design.

f. Borrow Areas.—Explorations for borrow areas can be divided into *two types*. The first type includes those to locate a specific kind of material such as:

- aggregate for concrete,
- ballast for railroad beds,
- surfacing and base courses for highways,
- filter materials for drains,
- blanketing and lining materials for canals or reservoirs,
- riprap materials for dams, and
- materials for stabilized or modified soils.

The second type is general to define the major kinds of material available in an area.

The first type of exploration requires locating comparatively small quantities of material with specific characteristics. The site should be geologically mapped to determine the nature and extent of the deposit. Then, either holes are drilled, test pits dug, or other investigations are performed in highly probable locations to establish if materials exist with the required characteristics. When a potential source is located, supplementary exploration is done to evaluate the required quantity. This should include at least one test pit to confirm drill hole data. The limits of the entire deposit need not be determined; but when exploring for specific materials for a major specification, an excess of the required quantity is needed to provide for waste, shrink, swell, or other unforeseen conditions.

The second type of exploration is performed to locate comparatively large quantities of material in which accessibility, uniformity, and workability are as important as engineering properties. A potential source satisfying these requirements is first located on the basis of surface indications. Next, a few holes are drilled or other investigations are performed to establish that appreciable depth of material exists; then the area is explored using a grid of holes or other investigations to establish the volume available. This should include at least one test pit to confirm drill hole data. The grid system layout should provide maximum information with the least number of holes. Normally for a long, narrow deposit, the section lines across the deposit can be located quite far apart, but the holes on these sections should be spaced quite closely. In a valley, more continuity is likely in the longitudinal direction than across the valley; exploring this variability is important. The variability of the deposit will also influence the number of holes.

Test pits and test trenches should always be a part of an investigation as they allow for:

- Visual inspection of in-place conditions,
- Verification of data obtained from drill holes or other investigations, and
- Determination of in-place density to evaluate shrink and swell factors.

Additional test holes should be required during construction. Prior to actual excavation, sometimes test hole spacing is reduced to 15 or 30 m (50 or 100 ft) near the edges of a deposit and in deposits where material is variable. Figure 2-14, an example of a plan and sections, shows exploration in a borrow area. On canal work, borrow material is usually taken from areas adjacent to the canal; and test holes or other investigations for borrow are not required if canal alignment test holes are spaced closely enough to ensure availability of satisfactory materials.

g. Selection of Samples.—Laboratory or field tests to establish engineering properties of a soil deposit may be required on: (1) soil samples without regard to their in-place condition in the deposit, (2) soil samples in which the natural in-place conditions are preserved as well as possible, and (3) soils as they exist in foundations. Laboratory and field tests are made on soils to determine both the average engineering properties and the range over which these properties vary. Because of cost, the procedure used is to determine by visual examination those samples likely to have the poorest, median, and best engineering properties considered critical. In the specifications stage, index tests are used to select samples for detailed laboratory testing instead of depending on visual examination.

During construction, samples are collected from the soils actually used; laboratory tests are performed on a portion of each sample, and the remainder is stored for detailed testing should a need arise. Because samples may become damaged during shipment, storage, or testing, and the exact number

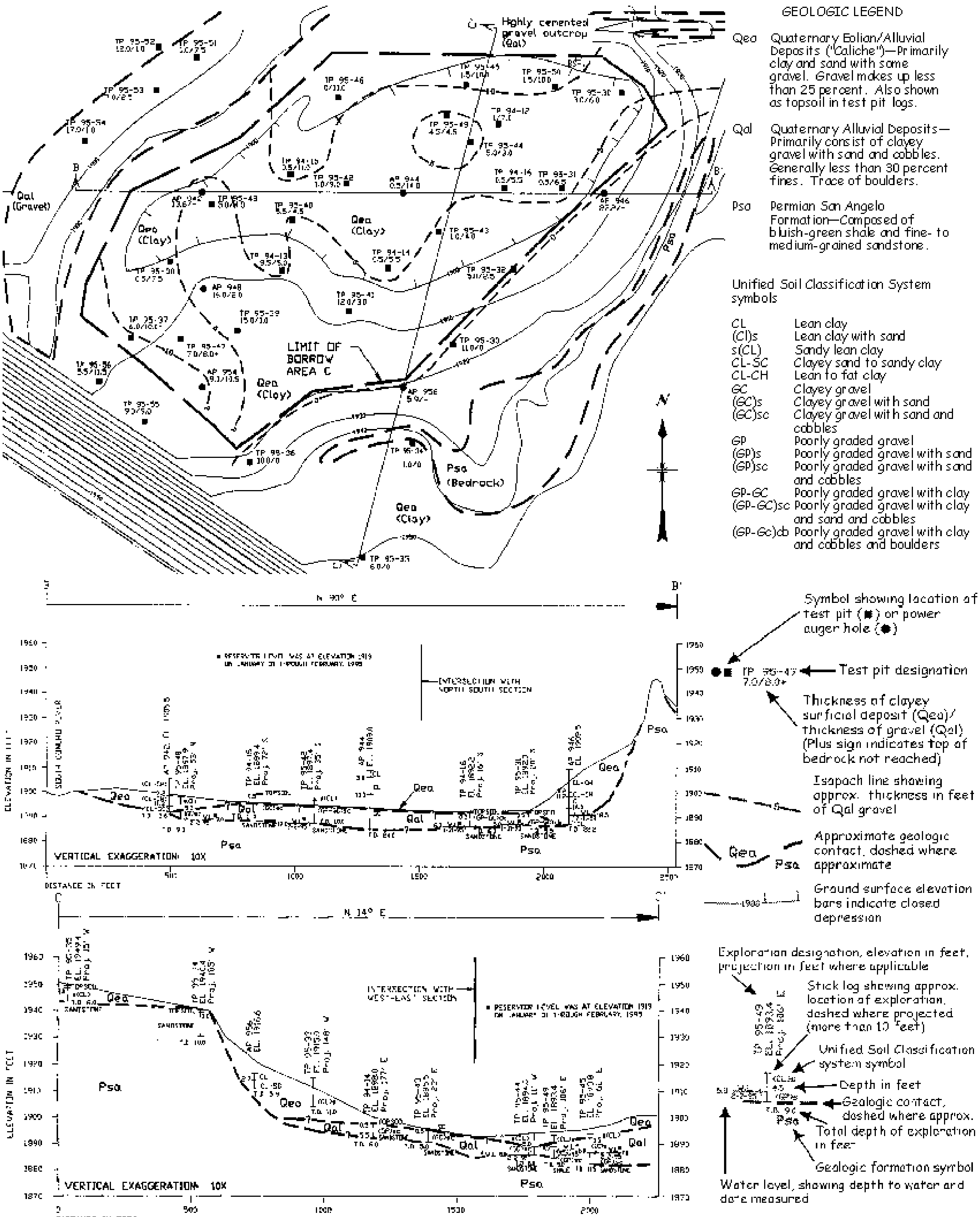


Figure 2-14.—Exploration for embankment materials—borrow area location map and typical cross sections.

to be tested cannot be predetermined, the number of samples collected should be considerably greater than the number actually considered necessary for testing.

Disturbed samples are collected (see USBR 7000 and 7010) when natural in-place conditions of the soil are relatively unimportant, i.e., where the soils will be reworked for use in the structure. The important element in this type of sampling is that:

- Samples are uniform throughout the sampling depth interval.
- Separate samples are collected for each change in material.
- Samples collected are representative of materials being investigated.
- Samples collected show lateral and vertical variations in material.

If test holes are small, total material is collected from the hole. In large test holes, a uniform cross-section cutting is removed from one wall to provide the samples. In some cases, a specific portion of each stratum is set aside as a representative sample. Normally, all of the initial exploratory holes are sampled completely. If the soil deposits have recognizable uniform characteristics, intermediate holes may not require sampling.

Undisturbed samples are collected when foundations are being evaluated for their capability to support structures or where it appears that soil in its natural state may possess special characteristics that will be lost if the soil is disturbed. Undisturbed samples may be collected solely for:

- Visual examination of soil structure
- Determining in-place density
- Load-consolidation testing
- Shear strength testing
- Performing special tests to determine change in engineering properties as the natural condition is changed

Many procedures are available for securing undisturbed samples. The procedures are designed to:

- best sample different types of soil,
- secure samples of an soil structure,
- minimize the amount of disturbance, and
- reduce sampling costs.

Bureau of Reclamation standard practices include securing samples from boreholes with:

- mechanical open end push tube sampling,
- piston sampling, or
- continuous double or triple tube rotary drilling and sampling.

Undisturbed samples are taken also from test pits or open excavations by carefully cutting out large blocks of material or by hand trimming a sample into a metal or plastic cylinder. Hand cut blocks and cylinder samples from accessible excavations provide the highest quality undisturbed samples which can be obtained. Details of undisturbed sampling procedures are given in USBR 7100 and 7105.

h. Field Tests.—Tests made on foundations in the field may include, but are not limited to, the following tests:

- in-place density and water content,
- in-place permeability,
- standard penetration,
- cone penetration,
- pressuremeter,
- flat blade dilatometer,
- vane shear,
- pile driving, and
- pile loading.

Foundations for hydraulic structures (dams and canals) are tested for permeability as a standard procedure.

- Sometimes, standard penetration tests are made on soil foundations and used as an index test—particularly, where bearing strength of the soil is questionable.
- Cone penetration tests are useful for delineating stratigraphy as well as for giving an indication of bearing capacity.
- Pressuremeter and dilatometer tests are used to determine modulus values of soils.
- Vane tests provide data on in-place shear strength.
- Pile driving and pile-load tests are made to assess pile performance.

Because the location of these tests must be closely related to design requirements, they are required mainly in connection with specifications stage investigations. Special tests other than those discussed above are often made in the field. In many cases, field tests can be performed to produce engineering properties more economically than laboratory testing. In some materials and conditions, field tests are preferred over laboratory techniques. An advantage of field testing is that the soil tested is at in-place condition under existing stress. Disadvantages are that the test is limited to a fixed stress path; empirical correlations are required, and sometimes a physical sample is not retrieved.

2-6. Investigations for Construction Materials.—

*a. General.—*A sufficient variety of naturally occurring materials is usually not available in the immediate vicinity of an earthwork structure to allow development of the feature except at an excessive cost.

Often, limited quantities of materials can be economically obtained that have special, desirable characteristics from areas at a considerable distance from the site, or an investigation can be made to determine whether existing materials can be modified at the site. Such materials might include:

- Impervious soils for constructing linings or blankets
- Sand and gravel for concrete aggregate, filters, filter; blankets, drains, road surfacing, and occasionally, protection from erosion; and
- Rock fragments for riprap or rockfill, filters, filter blankets, or concrete aggregate.

If the required materials are found in sufficient quantities in the immediate vicinity of the site, investigating more distant sources is unnecessary. If there is a deficiency of impervious materials, pervious materials, or rock in the immediate area, limited quantities of that deficient material may come from 40 to 80 km (25 to 50 mi) distant. Past cases have shown that securing riprap (rock) 320 km (200 mi) or more distant may be economical. If the haul distance to a suitable borrow source exceeds about 30 km (20 mi), design concept changes or modifications to available materials should be considered. Some modifications might be:

- adding bentonite to sandy soils,
- using cutoff walls, or
- using soil-cement or precast concrete blocks as alternate methods of slope protection.

The *engineering use chart*, discussed in chapter 1, provides information on desirability of earth materials for various uses from a quality standpoint as grouped using the Unified Soil Classification System. Usually, it is infeasible to secure any material having ideal characteristics (including rock); investigators must exercise considerable judgment in selecting material sources. The extent to which desirable characteristics are sought varies according to the purpose for which the material is to be used. In exploiting materials, volume may be substituted for quality to some extent; special processing of nearer sources may be more economical than long hauls. A definite need for and an improvement in the quality of material must exist with increasing distance from the emplacement site to justify long hauls. For distant sources, accessibility and type of available transportation facilities have an important bearing on desirability of the source.

*b. Impervious Materials.—*Situations arise where a special source of impervious materials is required, namely, in the construction of canals; dams on pervious foundations; and terminal, equalizing, or regulating reservoirs. When extensive permeable beds are found in the foundations for such structures, investigators must then locate a source of impervious material. Such material should be impervious—as compared to foundation soils—to justify using the material, but highly plastic clay materials are seldom desirable or necessary. These impervious soils are applied as blankets or linings over pervious foundations. Hydraulic gradients through the blanket or lining will be high; consequently, it is essential that gradations should satisfy appropriate filter criteria so that piping of fines from the blanket or lining into the more pervious foundation materials is prevented.

Piping also may be prevented by use of geotextiles or of soil bedding which meet appropriate filter criteria. The impervious material will be exposed to water in the canal or reservoir and should be capable of resisting erosive forces of flowing water and of waves. The material may be exposed to alternately wet and dry conditions and in some cases to freezing and thawing. Therefore, materials used for exposed linings or blankets should be free of shrinking and swelling characteristics. Special care should be taken to avoid use of dispersive clay soils for any of these purposes. A number of methods can be employed to overcome problems such as by stabilization with lime, cement, or insulation. However, in many cases, costs may increase to a point where buried impermeable geomembranes or geocomposites, or other types of linings such as shotcrete with or without geocomposites, compacted soil-cement, or concrete might become competitive alternatives. Blending two different soils together to form a superior blanketing material has been found to be feasible—such as adding bentonite to sands may be practical.

*1. Canal Construction.—*On canal construction, lining is used to reduce water loss. In turn, it conserves a water supply, prevents waterlogging of adjacent lands, or reduces the size of the conveyance system. Because of these considerations, impermeable lining is very desirable; however, thicker linings of soil material of moderately low permeability have been used. Such linings can be protected from erosion water by reducing the water velocity or by providing a protective cover for the soil lining. These linings are made as thin as possible, both to conserve material and to minimize the necessary additional excavation. Consequently, a mixture of coarse material with impervious soil, either blended together or naturally occurring, will provide a material that is both erosion resistant and impervious. When water velocity in a canal is contemplated to be high or the natural soil is to erode easily, investigations for materials for canals should include a source of coarse material for blending or blanketing unless the impervious material already contains an appreciable percentage of coarse particles.

2. Dam Construction.—On dam construction, it is seldom possible to significantly reduce reservoir water loss with partial earth blankets; but seepage gradients can be reduced to the point that piping is prevented. Materials for blankets should be highly impervious so their permeability is very low compared to that of the foundation. Resistance to flowing water usually is not critical, but care should be taken that dispersive clay soils are not used for this application. Because erosion from flowing water is not critical, blending requirements need not be anticipated. However, the possibility of the blanket piping into the foundation and the possibility of cracking of material having high shrinkage and swell characteristics must be considered.

Investigations may be required to locate a source of blanket material for storage dams; but more often, blankets are needed in connection with diversion dams. The qualifying differences between impervious blanket material for canals and dams as discussed above should be noted.

c. Pervious Materials.—Clean sands and gravels are required for:

- concrete aggregates,
- filter and drainage zones in earth dams,
- filters and drains associated with construction of concrete structures,
- bedding under riprap on earth dams,
- transition zones to prevent piping.

Distance from the construction site to areas where investigations are made for locating a supply of pervious materials having special properties will vary depending on the need for such special material. Using naturally occurring materials rather than those that require processing is usually more economical, especially when found near the site. Designs almost always can be made to accommodate pit run materials such as terrace deposits, etc. If quantities are small, materials acquired from developed commercial sources may be more economical.

On major projects such as large embankment dams, careful investigations must be performed so appropriate information is obtained for developing pervious borrow materials. A borrow area rarely produces concrete aggregates or filter and drain materials without processing. Processing operations are required to wash, sort, and crush granular materials and are critical to the contractor in sequencing construction events. For example, in a dam, filter and drain material will be in early demand since the downstream base of a dam will require a drainage blanket. In many cases, processing and stockpiling pervious materials may be desirable in the early phase of construction.

For all borrow investigations, long, linear test trenches should be excavated—in addition to point explorations such as test pits or auger borings—to gain a better understanding of geologic variability at the site. Trenches in alluvium should be excavated perpendicular to streamflow. Complete trench wall maps should be made, with all pockets and lenses shown, and with geologic interpretations. Special care should be taken to evaluate percentage of fines and their plasticity so formation of clay balls can be avoided during placement of the pervious materials.

On major projects as large dams, such volumetric estimates of oversized particles, e.g., cobbles and boulders, should be supplemented with full-scale gradations of representative materials because processing of oversize is crucial to the contractor's crushing operations. Coarse particles should be examined petrographically, and hardness should be evaluated. Contractors should be supplied with all borrow information that includes summary gradation plots which include comparisons to design gradation requirements. Test pits should be left open for prospective bidders, if possible. Uncertainties in processing borrow materials should be noted in the specifications.

1. Concrete Aggregates.—Procedures for investigation for concrete aggregates are described in the *Concrete Manual* [17]. Investigations for concrete aggregates require more information than other pervious materials. Consequently, data from investigations for concrete aggregate also can be used for other purposes, but data from investigations for other purposes are not usually adequate for concrete aggregate investigations.

2. Filters and Drains.—Although the quantity of pervious materials required for filters and drains is usually small, quality requirements are high. Filters and drains are used to prevent piping and reduce hydrostatic uplift pressures. Therefore, the material must be free draining, but at the same time must be able to dissipate relatively high hydraulic heads without movement of either the filter material or the protected soil. Often, a single layer of material will be inadequate, and a two-stage filter should be designed. Fine sand, silt, or clay in the pervious material is objectionable; processing by washing or screening is often required to produce acceptable material from most natural deposits.

Although grading requirements will be different, filter materials are commonly secured economically from sources acceptable for concrete aggregate. Particle shape of pervious material is not as critical; processed concrete aggregates rejected for shape can usually be used to construct drainage blankets and drains, if suitable gradation and adequate permeability is maintained. However, minerals contained in pervious materials should be evaluated for potential degradation as water percolates through the filter. Likewise, attention should be given to soundness and durability of particles to be sure no significant change occurs in gradation due to particle breakdown as the material is compacted.

Quality evaluation tests, similar to those performed on aggregate for concrete, are performed on these materials to evaluate their durability and suitability as filters or drains. Filter and drain materials must meet filter design criteria as outlined in *Design Standards No. 13—Embankment Dams* ([18] ch. 5).

*3. Bedding Under Riprap.—*Sand and gravel bedding material under riprap should be coarse but still conform to filter criteria in order to perform as a transition between embankment and riprap. Because of this requirement, blanket material is sometimes secured from rock fines resulting from quarrying operations. However, if a coarse gravel deposit can be found within reasonable distance from the damsite, developing the source will usually be economical. Quantity requirements are quite large, and special processing by screening or other means is costly. The principal purpose of this type of blanket is to prevent waves which penetrate the riprap from eroding the underlying embankment. A limited amount of fine material is not objectionable even though some will most likely be lost through erosive wave action. The material should be durable; most fine material found in gravel deposits is adequate. However, gravel deposits have been found containing large quantities of unsuitable material. Such deposits include ancient gravel beds often in terraces that have deteriorated by weathering, and talus or slopewash deposits where water action has been insufficient to remove the soft rock.

*4. Drainage Blankets.—*Materials used for drainage blankets within the downstream zones of earth dams should be pervious with respect to both the embankment and foundation soils. The material should not contain appreciable quantities of silt or clay, and the gradation should meet the required filter criteria [18].

Sometimes, the volume of blanket material can substitute for poor quality. If a sufficient quantity can be found close to the point of use, a search for better materials at greater distances may not be warranted.

*5. Road Surfacing and Base Course.—*Materials for road surfacing or base course are sought primarily for strength and durability. The preferred material for surfacing will consist primarily of medium to fine gravel with enough clay to bind the material together and with relatively small amounts of silt and fine sand. Similar material, but without silt or clay, is preferred for base courses. For these materials, when road construction is for replacement and to be performed by the Bureau of Reclamation, requirements of the local highway agency should be determined before making extensive investigations.

d. Riprap and Rockfill.—

*1. General.—*Rock fragments are required in connection with earthwork structures to protect earth embankments or exposed excavations from the action of water either as waves, turbulent flow, or heavy rainfall. Rock fragments associated with wave action or flowing water protection are called riprap. The term rockfill is commonly applied to the more massive bodies of fill in dam embankments and consists of rock fragments used primarily to provide structural stability. Such rockfills may serve as drainage blankets or blankets under riprap; and if the material to be used is adequate for slope protection, a separate requirement for riprap may not be necessary.

Material from rock sources should satisfy two main requirements: (1) the rock source should produce rock fragments in suitable sizes according to required use, and (2) the rock fragments should be hard and durable enough to withstand the processes involved in procuring and placing them and to withstand normal weathering processes and other destructive forces associated in the place of use.

Specific gravity and density are important attributes although, to some extent, increase in fragment size may be substituted for high density. Suitable substitutes are available for riprap and rockfill blankets such as soil-cement or asphalt for upstream slope protection, and sod cover for downstream slope protection. These alternatives are considered when rock is unavailable or economically prohibitive.

2. Riprap for Earth Dams.—Riprap surfaces on earth dams must withstand severe ice and wave action as well as destructive forces associated with temperature changes, which includes freezing and thawing, heating and cooling, and wetting and drying. Securing highly durable material for riprap is important. Laboratory tests such as the "freeze-thaw" test will disclose weaknesses and lack of durability. Durability can be judged by finding locations where the same rock is subjected to similar conditions in other reservoirs, in stream channels, or in other exposures.

Figure 2-15 shows results of a major blast in a riprap quarry which produced rock fragments of the sizes required for a dam constructed in the Western United States. A more detailed discussion on riprap requirements, use, and placement is in *Design of Small Dams* [16] and in Design Standards No. 13 – Embankment Dams [18], chapter 7, and in [19]. Joint spacing in a rock outcrop will be a determining factor as to whether adequate size fragments can be secured. Old joints that have become cemented but would break apart during excavation should be noted. Because these mechanical weaknesses are often difficult to detect, a blast test should be conducted (fig. 2-16). Riprap blasting techniques are covered in the *Engineering Geology Field Manual* [20] and in *Blasting Review Team Report* [21]. When a bedrock exposure containing satisfactory rock is unavailable in the immediate vicinity of a damsite, it may be possible to secure riprap material from:

- stream deposits,
- glacial till,
- talus slopes (fig. 2-17), and, occasionally,
- surface deposits.

Figure 2-15.—Results of a major blast in a riprap quarry.

(A) Initial blast.

(B) Results of blast.

Figure 2-16.—Blasting a rock ledge at the riprap source for Stampede Dam, California. The rock is basalt having a specific gravity of 2.6.

Figure 2-17.—Talus slope of igneous rock proposed for riprap.

The quality of many rock sources changes laterally and with depth, and overburden on some sources becomes so thick that its removal is uneconomical. It will frequently be necessary to explore rock sources with drill holes, depending on geologic conditions, before establishing the deposit as an approved source.

When investigating riprap at distances greater than a few kilometers, a number of economical sources are usually located. Specifying quality requirements for the riprap rather than the source is desirable, so that a contractor may competitively secure material from the most economical source. If riprap from several sources satisfies quality requirements, all sources are listed in the specifications. Each deposit should be sampled and tested sufficiently to establish essential characteristics. This information will be used in designs to establish minimum acceptable properties for specifying quality requirements. Establishment of these minimum requirements will be influenced by:

- initial cost,
- effective life of the cover,
- repair cost,
- climatic conditions, and
- thickness of cover.

3. Riprap for Stilling Basins.—Riprap blankets are commonly used downstream of spillway and outlet works stilling basins, and other energy dissipating structures where high-velocity turbulent flow must be reduced to noneroding velocities. Quantities involved are usually comparatively small, but quality requirements may exceed those of earth dam reservoir riprap blankets. In some instances, even though marginal material may be used on the dam surface, high-quality riprap must be secured for the blanket material used to protect other structures. Where investigations disclose only moderate quality rock in the vicinity of the site, investigations should be extended to establish the location of a high-quality source.

4. Riprap for Canals.—Riprap is used in canal construction where severe erosion of the channel could occur. Usually, these locations occur in short reaches of canals below concrete structures, adjacent to bridge piers, and at sharp turns in alignment. Protective requirements vary over a wide range; they may be satisfied with a thin gravel cover or may require riprap equal to that required below the controlling works of dams. When these areas must be protected by riprap, blankets are usually 300 to 600 mm (12 to 24 in) thick with corresponding rock sizes.

5. Rockfill Blankets.—When rockfill is used to provide surface protection from rainfall, almost any rock fragments that do not break down excessively on exposure to air or water are acceptable as rockfill. Shales and some siltstones are about the only types of rock considered unacceptable for this purpose. Size is not critical except that fragments should be at least gravel size, and the upper-size limit is controlled by the specified thickness of the blanket. Rounded gravel, if readily available, is frequently used for this purpose. Special investigations for this type of material are almost wholly confined to the plains areas of the Western United States where rock is generally absent within economical depth beneath ground surface. Substitutes may include sod blankets. Securing rockfill blanket material from appreciable distances is justified only where quality rock is absent in the immediate vicinity of the work and when substitutes are inadequate.

6. Investigative Procedures.—Investigation requirements for rock sources for riprap are variable dependent upon:

- the stage of investigation,
- whether a choice of material is locally available,
- the type of specifications to be prepared; i.e., whether contractor or Government furnishes the rock.

Guidance for investigations and testing is no longer included in the *Earth Manual*, Part 2, Third Edition. Reclamation guidance will be included in a new *Rock Manual*, under development. The practice will be issued as USBR 6025, "Sampling and Testing of Rock Fragments or Rock Samples for Riprap Usage." Draft copies of this standard can be obtained from the Earth Sciences Laboratory in Denver.

Another source of information on investigating riprap sources can be found in ASTM standard D-4992-94, "Standard Practice for Evaluation of Rock to Be Used for Erosion Control."[22]

2-7. Materials for Stabilized and Modified Soils.—

a. General.—A stabilized soil is a soil whose properties are partially or completely changed by adding a dissimilar material before compacting the soil or by placing an additive into the soil in place. Depending upon the properties and the amount of additive, all of the properties characteristic of the soil may be completely and permanently changed.

A modified soil is a soil in which specific properties may be either temporarily or permanently changed. Soils are modified by adding a small amount of an additive before compaction or by placing the additive into the soil in place.

Stabilized soils are used as a substitute for:

- riprap to protect the upstream slopes of earth dam embankments,
- linings for reservoirs, and
- temporary protection of construction during river diversion.

They are used also as protective blankets and as pipe bedding. Small quantities of additives are used to:

- modify and improve properties of soils used in fills,
- increase erosion resistance,
- reduce permeability, or
- provide temporary stability during construction.

b. Compacted Soil-Cement.—Compacted soil-cement is a stabilized soil consisting of a mixture of soil, cement, and water compacted to a uniform, dense mass used for linings, protective blankets, and slope protection in lieu of riprap [23, 24]. Placement water content and density are controlled by the laboratory compaction test (see USBR 5500).

The most desirable soil for soil-cement is a silty sand (SM) which has a good distribution of sizes with 15 to 25 percent fines [25]. Other soils may be used; however, more cement may be required to satisfy strength and durability requirements if cleaner, coarse-grained soils are used. Uniformity of soil gradation and moisture when introduced into a continuous feed pugmill type mixing plant is the most important factor in ensuring uniformity of compacted soil-cement.

Soil is usually obtained from a borrow area explored in detail to ensure quantity and uniformity desired. A uniform deposit is most desirable. Stratified deposits may be used, provided selective excavation and processing is practical and economical compared to using other potential sources. Selective excavation and mixing during stockpiling may be necessary to provide a soil as uniform in grading and moisture content as practicable. Screening equipment may be necessary to (1) remove undesirable organic material and oversize particles; (2) remove or reduce the size of sand, silt, and clay aggregations, called "clay balls," which tend to form in the borrow areas containing lenses of clay.

Laboratory testing is required to determine the quantity and type of cement, the moisture limits, and the compaction requirements to be specified for construction. Representative samples of the finest, average, and coarsest material should be submitted for laboratory testing. The water proposed for mixing should be reasonably clean and free from organic matter, acids, alkali, salts, oils, and other impurities. Clear water that does not have a saline or brackish taste and is suitable for drinking may be used; however, doubtful sources should be sampled and tested.

c. Soil-Cement Slurry.—Soil-cement slurry is a cement stabilized soil consisting of a mixture of soil and cement with sufficient water to form a material with the consistency of a thick liquid that will flow easily and can be pumped without segregation. Sands with up to 30-percent nonplastic or slightly plastic fines are best.

Reclamation has used soil-cement slurry for pipe bedding [26, 27]. Even though materials from the trench excavation may be used, locating the borrow areas along the pipeline alignment is generally more economical and usually results in a better controlled and more uniform product. Soil-cement slurry has often been supplied by commercial ready-mix firms when haul distances are economical.

d. Modified Soil.—A modified soil is a mixture of soil, water, and a small amount of an additive. The various components are well mixed before compaction or added to the soil in place to modify certain properties—temporarily or permanently—to within specified limits. Because of the small amount of additive, a modified soil usually retains most of the characteristics of the original soil because it is an aggregation of uncemented or weakly cemented particles rather than a strongly cemented mass. Limited experience has been acquired on even the most commonly used additives including asphalt, portland cement, fly ash, lime, slag, resins, elastomers, and organic chemicals.

As a soil additive, use of lime is the oldest known method of chemical stabilization: it was used by the Romans to construct the Appian Way. Soil-lime is a mixture of soil (usually clay), lime, and water which is compacted to form a dense mass. Experience has shown that mixtures of most clay soils with either quick or hydrated lime and water will form cementitious products in a short period of time. Reclamation applications for water resources works have been limited to use of lime for stabilizing expansive clay soils and dispersive clay soils.

Reclamation's Friant-Kern Canal in California experienced severe damage to both earth and concrete-lined sections from expansive clay soils. Lime, 4 percent by dry mass of soil, was used during rehabilitation of Friant-Kern Canal; and the soil-lime lining has proved durable after more than 20 years of service [28].

Dispersive clay soils will erode in slow-moving or even still water by individual colloidal clay particles going into suspension and then carried away by flowing water. Dispersive clay soils may be made nondispersive by addition of 1 to 4 percent lime by dry mass of soil. Generally, the design lime content is defined as the minimum lime content required to make the soil nondispersive. If lime-treated soil is to be used in surface layers, adding additional lime may be desirable to increase the shrinkage limit to near optimum water content to prevent cracking from drying. In construction specifications, the lime content is often increased 0.5 to 1.0 percent above the design value to account for losses, uneven distribution, incomplete mixing, etc. Dispersive clay soils were found throughout the borrow areas for Reclamation's McGee Creek Dam in Oklahoma. Lime was used to stabilize those soils for certain critical parts of the dam, and specifications required between 1.5 and 3.0 percent lime to be added to the soil [29].

B. Exploratory Methods

2-8. General.—Exploration methods may be grouped in different ways: (1) those that produce usable samples and those that do not, and (2) those that are accessible and those that are not. In investigations of foundations or materials, one purpose of an exploration is to secure samples of soil and rock, either for visual examination or for laboratory testing. Procedures which will not produce samples should be used only where the general characteristics of the materials to be penetrated are already known, where sufficient samples have been secured for testing, or where penetration test or other in-place test data can be used to define or refine an investigation plan or program. Sampling methods will vary according to the type of material to be sampled and according to the acceptable degree of sample disturbance; and test holes or other investigations may be advanced by manual labor or by mechanical power. Exploratory holes may be of various sizes depending upon:

- the need to access,
- the penetration depth,
- the size of sample to acquire, and
- the type of material to penetrate.

Stability of small-size holes entirely above the water table is dependent on the type of material encountered. Holes in soil below the water table usually require support by steel casing, augers, or by drilling fluid. Sometimes, exploratory holes require protection with steel casing because of potential damage to the hole by drilling operations and to prevent contamination of samples with materials from higher elevations. As part of a foundation investigation, many exploratory holes require water testing. When casing is used, specific portions of the foundation may be water tested to simplify evaluation, to concentrate on certain foundation conditions, and to determine required treatment. If water testing or piezometer installation is a part of the requirement, bentonite drilling mud should not be used.

In soft or loose soil foundations, the strength of the materials in the wall of the hole may be insufficient to keep soil from flowing into the bottom of the hole. In many instances, just keeping the hole filled with water will suffice to hold materials in place. In severe cases, a wall stabilizer, a heavy fluid, or both must be used. Information on the various stabilizers may be obtained from drilling fluid manufacturers. If water testing is not required, drilling fluid consisting of a mixture of commercially available bentonite or other materials and water may be used. Drilling fluid may be specially prepared to have a required mass per unit volume due to addition of finely divided solid material or to addition of other additives. (See USBR 7105 and USBR 5890 through 5895.)

Samples may be secured from holes supported by drilling fluid or casing with either double-tube or triple-tube soil samplers, core samplers, drive tube, or with push-tube samplers. To minimize sample disturbance, fixed-piston-type samplers are preferable for very soft soils. The double-tube hollow-stem auger sampler is best for water sensitive, very loose, or unsaturated low density soils. If undisturbed sampling is not successful, a number of in-place tests can be performed that yield valuable information concerning foundation soil conditions. Radiographs (X-ray) may be taken of undisturbed samples after they are received in the laboratory. The radiographs are studied to determine degree of disturbance to each sample. Radiographs are also useful to identify:

- slip planes,
- fault gouge zones,
- slickensides,
- contamination by drilling fluid,
- amount of slough on top of a sample,
- location of gravel particles within a sample,
- various other conditions.

Figure 2-18 shows some of the samplers available for obtaining undisturbed samples.

In drilling exploratory holes through hard materials, where support might normally be unnecessary, crushed zones or faults may be encountered from which rock fragments fall into the hole and either plug the hole, bind the drilling equipment, or both. In such situations, cement grout may be placed in that area; after the grout has set, the hole can be drilled through the grout. Because these crushed zones or faults represent some of the critical conditions being investigated from an engineering standpoint, all pertinent tests (such as water tests) must be performed before grouting the unstable section of hole; a record of the conditions should be fully reported and recorded.

All exploratory holes should be protected with suitable covers and fences to prevent foreign matter from entering the holes and to keep people and animals from disturbing them or falling into them. All holes should be filled or plugged after fulfilling their purpose.

2-9. Accessible Exploratory Methods.—

a. General.—Open test pits, large diameter borings, trenches, and tunnels are accessible and yield the most complete information of the ground penetrated. They also permit examination of the foundation bedrock. When depth of overburden and ground-water conditions permit their economical use, these methods are recommended for foundation exploration in lieu of relying solely on borings. In prospecting for concrete aggregate, embankment, filter, or drain materials containing cobbles and boulders, open pits and trenches may be the only feasible means for obtaining the required information.

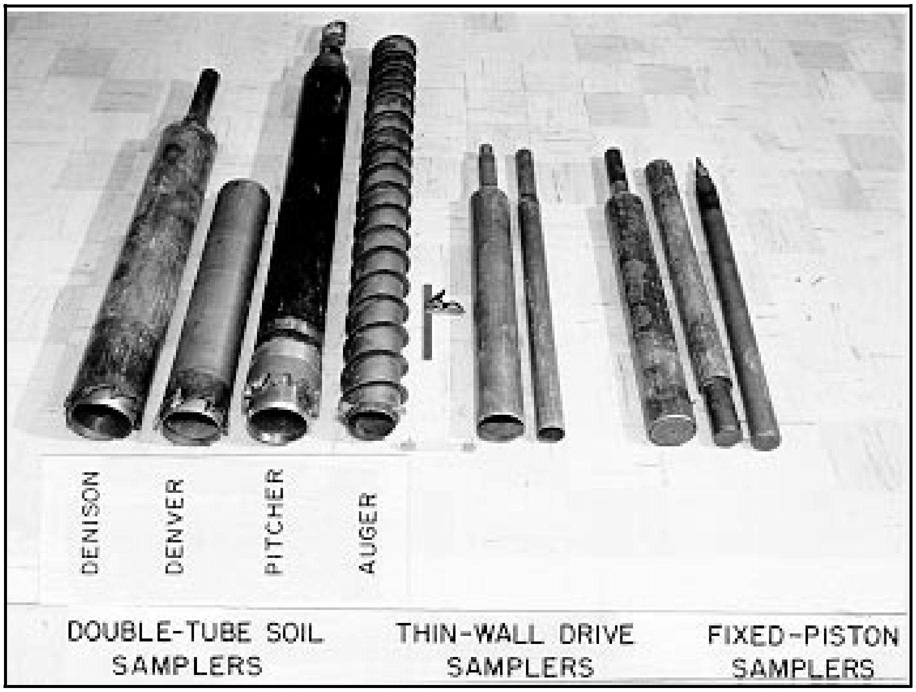


Figure 2-18.—Samplers used for undisturbed soil sampling.

b. Test Pits.—Test pits are an effective means to explore and sample earth foundations and construction materials and to facilitate inspection, sampling, and testing. The depth of a test pit is determined by investigation requirements but is usually limited to the depth of the water table. Dragline, backhoe, clamshell, caisson drilling or auger equipment, and bulldozer pits are usually more economical than digging pits by hand for comparatively shallow materials explorations. Explosives may be required to break up large boulders. At the surface, the excavated material should be placed in an orderly manner around the pit to indicate depth of pit from which the material came to facilitate accurate logging and sampling. The moisture condition should be determined and recorded before drying occurs by exposure to air.

Investigations in open, accessible explorations such as test pits, large diameter borings, trenches, and tunnels are inherently hazardous. Federal, State, and local regulations must be followed when planning and executing accessible investigations. Occupational Safety and Health Act (OSHA) regulations for excavation safety (29 CFR 1926.650-652) should be consulted prior to planning accessible explorations. Regulations require that competent personnel plan, design, and monitor excavations. Excavations greater than 5 ft in depth normally require sloping or shoring systems designed by professional engineers. Large diameter borings and deep trenches may be considered to be confined space and may require special ventilation, monitoring, and rescue safety equipment.

Deep test pits should be ventilated to prevent accumulation of dead air. Ventilating pipe, which begins slightly above the floor extending about 1 m (3 ft) above the mouth of the pit, is usually satisfactory. Canvas and plastic sheeting have been used to deflect wind into the hole. Oxygen meters should be used to determine satisfactory air quality. Test pits left open for inspection must be provided with covers and barricades for safety. All applicable safety and shoring requirements must be met.

When water is encountered in the pit, a pumping system is required for further progress. Small, portable, gasoline-powered, self-priming, centrifugal pumps can be used; however, air or electric powered equipment is preferred whenever possible because of the change of carbon monoxide poisoning. The suction hose should be 15 mm (½ in) larger in diameter than the pump discharge and not more than 5 mm (15 ft) long. This requires resetting the pump in the pit (on a frame attached to the cribbing) at about 4-m (12-ft) intervals. When an air or electric powered pump is not available, and a gasoline engine is used, pipe the exhaust gases well away from the pit when the engine is in or near the pit. When a gasoline engine is operating within a pit, personnel shall not be allowed in the pit for any extended period of time regardless of how well the system is vented. Dewatering test pits is usually expensive and is often unwarranted.

c. Large-Diameter Borings.—Caisson auger rigs using large-diameter discs, gushers, or buckets are often used when accessible explorations are required to be deeper than about 6 m (20 ft). Depths of over 30 m (100 ft) have been achieved using this method. Wall support must be provided for the total depth. Typical wall support for large-diameter borings may consist of welded steel casing installed after the boring is complete or preformed steel liner plate segments bolted together and placed as the boring progresses. Personnel access within a drilled caisson hole may be provided by an elevator platform rigging using power from a crane hoist or by notched safety rail ladder using an approved grab-ring safety belt. Work may be performed at any depth in the drilled caisson boring using steel platform decking attached to the steel wall support, from a steel scaffolding, or from an elevator platform.

Access for material logging or sample collection outside a steel-encased caisson hole may be accomplished by cutting openings in the casing at desired locations or by removing bolted liner plate segments to expose the sides of the boring. Sufficient ventilation must be maintained at all times for personnel working within the excavation. Radio communication to surface personnel should be maintained. Water within a drilled excavation may be removed by an electric or air-powered pump with discharge conduit to the surface. Dewatering may require stage pumping by using several holding reservoirs, at appropriate elevations, and additional pumps as required to lift the water from the borehole to the ground surface.

Large-diameter borings left open for inspection should always be provided with locking protective covers and should be enclosed by a fence or barricade.

d. Trenches.—Test trenches are used to provide a continuous exposure of the ground along a given line or section. In general, they serve the same purpose as open test pits but have the added advantage of disclosing the lateral continuity or character of particular strata. They are best suited for shallow exploration (10-15 ft [3-4.5 m]). Trenching can be used to drain wet borrow areas and at the same time fulfill investigation requirements.

On a slope, field work consists of excavating an open trench from the top to the bottom to reach representative undisturbed material. Either a single slot trench is excavated down the face of a slope, or a series of short trenches can be spaced at appropriate intervals along the slope.

Depending on the extent of investigation required, bulldozers, backhoes, or draglines can be used. Figure 2-19 shows a trench excavated by bulldozer. *All* safety procedures and guidelines *must* be followed when excavating deep trenches to prevent accidents caused by caving ground.

The material exposed by trenches may represent the entire depth of significant strata in an abutment of a dam; however, their shallow depth may limit investigation to only a portion of the foundation, and other types of exploration may be required to explore to greater depths. Test trenches, however, are often extensively used to delineate stratigraphy in borrow areas. As with test pits, trenching permits visual inspection of soil strata which facilitates logging of the profile and selection of samples. Large undisturbed samples or large disturbed individual or composite samples are easily retrieved from test trenches. Trenches in sloping ground have the further advantage of being self-draining.

e. Tunnels.—Tunnels, adits, or drifts have been used to explore and test areas beneath steep slopes in or back of clifflike faces. Any exploratory tunnel or drift is usually roughly rectangular in shape and about 1.5 m wide by 2 m high (5 ft by 7 ft). When lagging is required for side and roof supports, it should be placed to follow excavation as closely as practicable. Excavation of exploratory tunnels can be a slow, expensive process; consequently, this type of investigation should be employed only when other methods will not supply the required information.

Logging and sampling of exploratory tunnels should proceed concurrently with excavation operations if possible (see fig. 2-20).

If explosives were used to excavate the tunnel, selecting locations for undisturbed samples must be carefully made. This includes removal of all material disturbed by the explosives, thereby exposing undisturbed material.

2-10. Nonaccessible Exploratory Methods.—

a. General.—The usual nonaccessible exploratory methods are cone penetrometer, standard penetration, auger drilling, rotary drilling, core drilling, and in-place field testing. Of the methods, auger drilling, rotary drilling, and core drilling are the most commonly used to obtain samples

Figure 2-19.—Shallow test trench excavated by bulldozer.

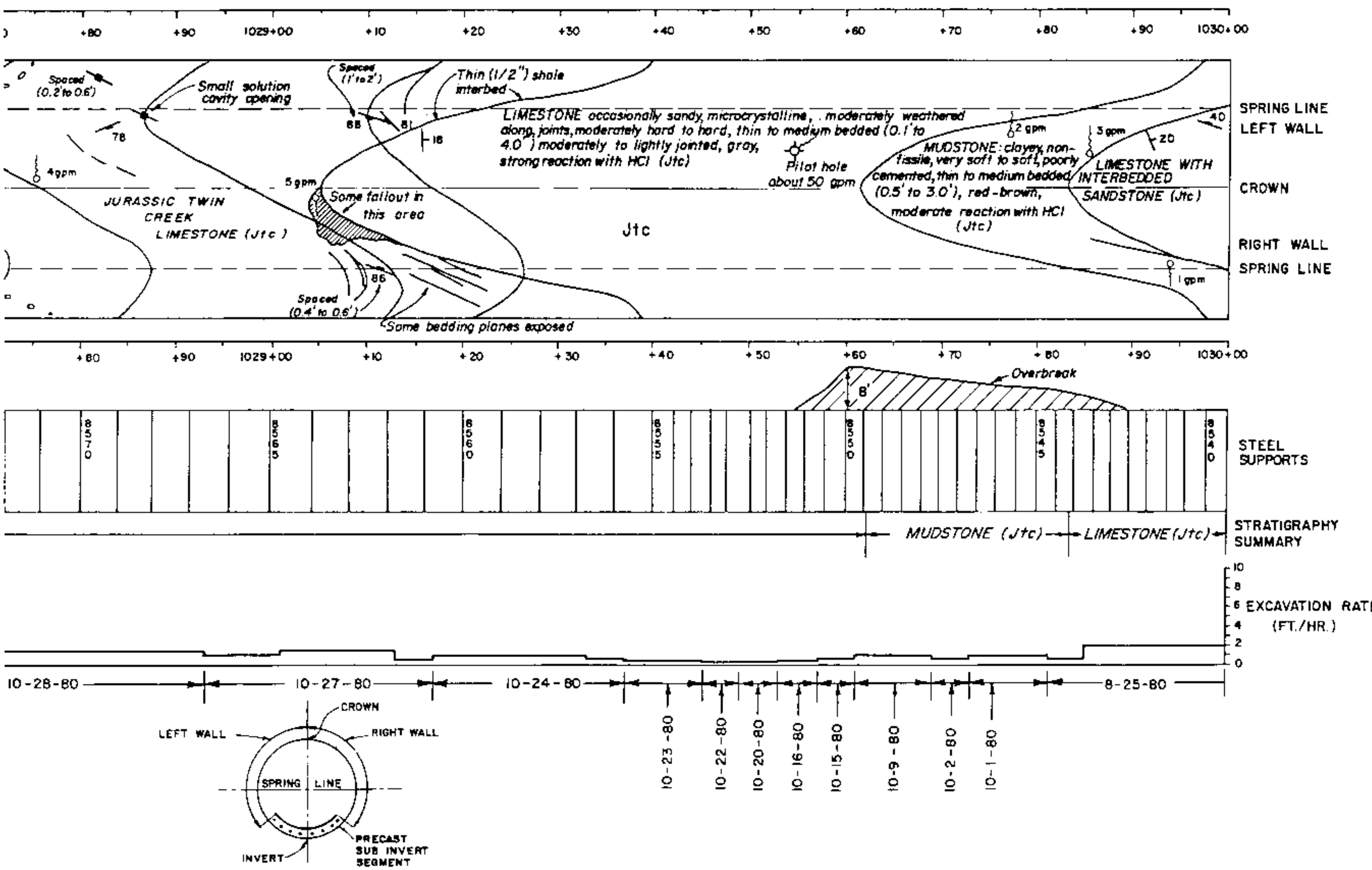


Figure 2-20.—Example of tunnel mapping from machine-excavated tunnel.

for laboratory examination and testing. It should be stressed that geologic complexity should have been determined beforehand, and exploratory drilling operations should never be solely relied upon to provide data for accurate and reliable geologic interpretation of a complex geologic structure.

Economics and depth requirements are the principal reasons for performing extensive drilling programs in lieu of constructing accessible test pits or trenches to establish geologic conditions. If drilling is considered the only feasible method for conducting subsurface explorations, the following considerations should be given priority when planning the remainder of the exploration program:

- All relevant subsurface and geologic information should be assembled and used when selecting strategic drilling locations so the maximum amount of subsurface information can be obtained from a minimum number of drilling locations.
- The type of exploration drilling, in-place testing, sampling, or coring necessary to obtain pertinent and valid subsurface information should be carefully considered (see section 2-12 for embankment drilling).
- The type of drilling rig capable of accomplishing exploration requirements must be determined.

Although drilling may be accomplished to some extent using manual methods (i.e., hand augers, tripod assemblies, and hand-crank hoist systems), many factors such as equipment technology, economics, depth requirements, type of sample needs, and the need for accurate subsurface information have made manual exploration methods obsolete. Various types of mechanical power-driven drilling equipment are available, and most efficient use and capability of each type of drilling unit is discussed.

b. Auger Borings.—General.—Auger borings often provide the simplest method of soil investigation and sampling. They may be used for any purpose where disturbed samples are satisfactory and are valuable in advancing holes to depths at which undisturbed sampling or in-place testing is required. Mechanical hollow-stem auger drilling and sampling are the most preferred methods for drilling in existing dam embankments to avoid hydraulic fracturing. Hollow-stem augers are frequently used for drilling potentially contaminated ground because fluids are not used. Sometimes, depths of auger investigations are limited by the ground-water table, by rock, and by the amount and maximum size of gravel, cobbles, and boulders as compared with the size of equipment used.

Hand-operated post hole augers 100 to 300 mm (4 to 12 in) in diameter can be used for exploration to shallow depths (fig. 2-21).

An auger boring is made by turning the auger the desired distance into the soil, withdrawing it, and removing the soil for examination and sampling. The auger is inserted into the hole again, and the process is repeated.

A soil auger can be used both for boring the hole and for bringing up disturbed samples of soil. It operates best in somewhat loose, moderately cohesive, moist soils. Usually, holes are bored without addition of water; but in hard, dry soils or in cohesionless sands, introduction of a small amount of water into the hole will aid with drilling and sample extraction. It is difficult to avoid some contamination or mixing of soil samples obtained by small augers. Rock fragments larger than about one-tenth the diameter of the hole cannot be successfully removed by normal augering methods. Large-size holes permit examination of soils in place; therefore, they are preferred for foundation investigation. An excellent guide for using augers in water resources investigations is available from USGS: Application of Drilling, Coring, and Sampling Techniques to Test Holes and Wells [30].

Pipe casing may be required in unstable soil in which the borehole fails to stay open—and especially where the boring is extended below ground-water level. The inside diameter of the casing must be slightly larger than the diameter of the auger used. The casing is driven to a depth not greater than the top of the next sample; the boring is cleaned out by means of the auger. Then, the auger can be inserted into the borehole and turned below the bottom of the casing to obtain the sample.

c. Mechanical Auger Drilling.—Auger drills are mechanical, engine-powered drills that are designed to produce high rotational torque at low revolutions per minute as required to drill into and collect subsurface soil samples. Drill cuttings and soil samples are removed by the auger's rotation without using fluid circulation media; thus, the requirement for high torque capability of the drill. Multipurpose drills are available which are capable of auger, rotary, or core operations. Discussion in this section is directed toward describing and explaining individual general uses for each of the four distinct types of auger-drilling operations used in subsurface explorations:

- Continuous-flight auger drilling
- Hollow-stem auger drilling
- Helical, disk, and bucket auger drilling
- Enclosed augers

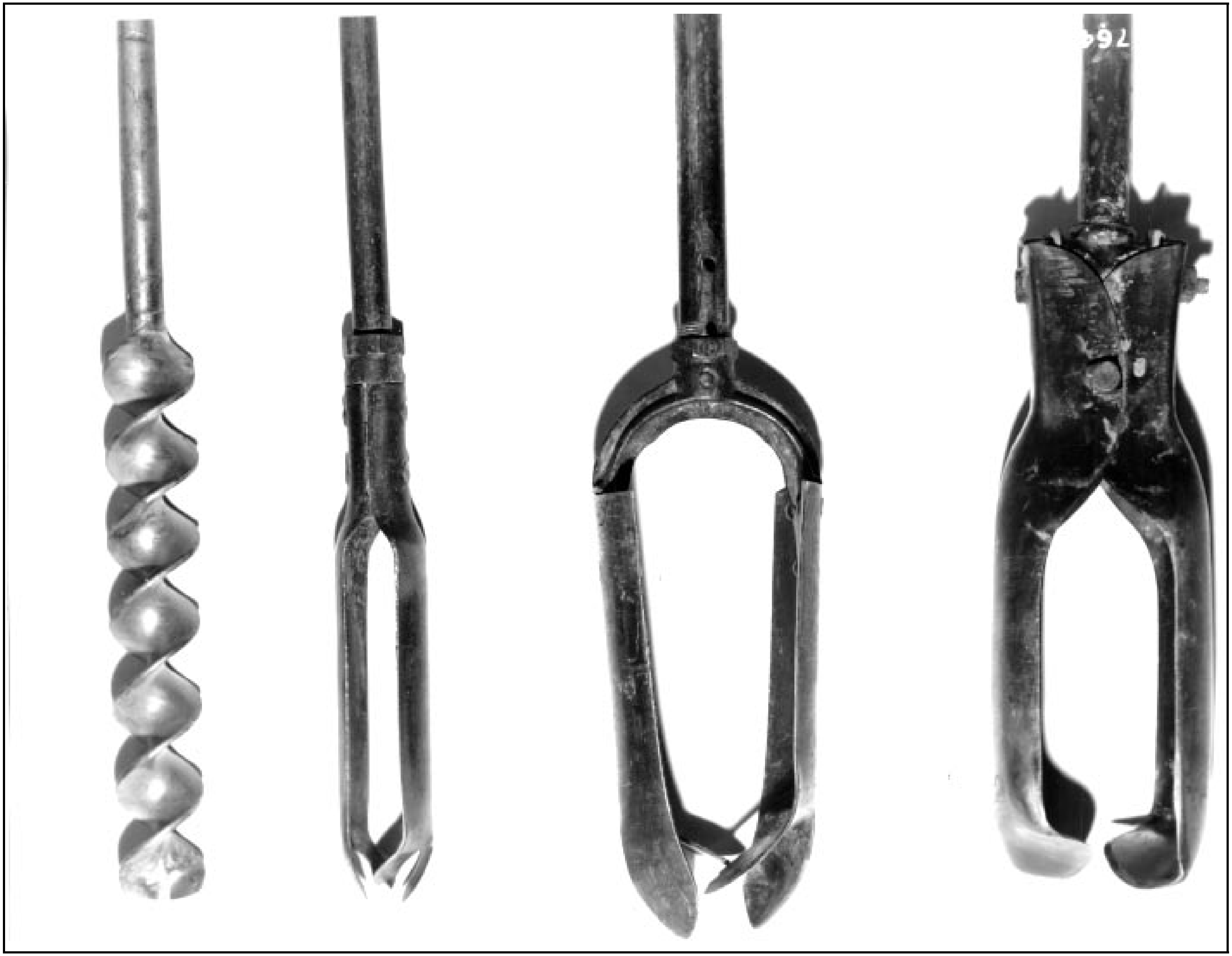


Figure 2-21.—Types of hand augers (2-inch helical, 2- and 6-inch Iwan, and 6-inch Fenn [adjustable]).

1. Continuous-Flight Augers.—Continuous-flight auger drilling often provides the simplest and most economical method of subsurface exploration and disturbed sampling of surficial deposits. Flight augers consist of a center drive shaft with spiral-shaped steel flights welded around the outside circumference of the drive shaft.

As each auger section is drilled into the ground, another section is added with an identical spiral flight that is manufactured to match the in-hole auger. The joining of each matched auger section results in a continuous spiral flight from the bottom of the hole to the surface. The auger's rotation causes drill cuttings to move upward along the spiral flights so that disturbed soil samples can be collected at the hole collar. Figure 2-22 shows a continuous-flight auger drilling operation.

Flight augers are manufactured in a wide range of diameters from 50 to greater than 600 mm (2 to > 24 in). The most common auger used for obtaining disturbed samples of overburden is 150 mm (6 in) in diameter. Normally, flight auger depths are limited by:

- Equipment torque capability,
- ground-water table,
- firmness of materials penetrated,
- cobble or gravel strata,
- caliche zones, and
- bedrock.

Continuous-flight auger drilling is an economical and highly productive exploration method. A common and efficient use of flight augers is to define borrow area boundaries and depths. Borrow area investigations are conducted by augering holes on a grid pattern to define borrow boundaries and to estimate quantities of usable material. Flight augers are especially efficient when used to collect composite samples of mixed strata material to establish a borrow depth for excavation by belt-loader equipment. Composite samples are collected by advancing auger borings to the depth capability of the belt loader. Hole advancement is accomplished by turning the auger at a low, high-torque rate while adding downpressure for penetration to depth. At the end of the penetration interval, the auger is turned at a higher rate without further downward advancement to collect a composite sample of the material augered through. After the hole is thoroughly cleaned by bringing all cuttings to the surface, the material is mixed to form a representative composite sample and sacked according to requirements for laboratory testing. If additional sample intervals are required, the process can be repeated at a second sample depth.

Although the above procedure for collecting composite samples of mixed material strata is efficient, it may not result in an accurate representative sample of the material being drilled—especially at greater depths. This is because the augered material may mix with sidewall material as cuttings are brought to the surface. In addition, auger borings through a noncohesive, low density sand stratum can result in a sample with a greater volume of sand than would be obtained from a borehole having a constant diameter throughout the hole depth. (It is assumed the diameter of the hole enlarges in the sand stratum.) If material contamination is evident or if too large a volume of material is recovered from a given auger penetration interval, a hollow-stem auger with an inner-barrel wire-line system or a continuous-sampler system should be used in lieu of a continuous-flight auger.

Other beneficial and efficient uses of flight-auger drilling include:

- *Delineation of soil properties for lengthy line structures such as canals and pipelines*.—Economical power auger borings can be spaced in between more detailed sampling and testing locations.
- *Determination of shallow bedrock depths*.—This method is especially valuable for estimating overburden excavation volume required to expose potential rockfill or riprap sources. Confirmation of potential rock quality and usable volume must be determined using rotary core drilling exploration methods.
- *Drilling through cohesive soils to install well points to monitor water table fluctuation*.—This method is recommended for use only through cohesive soils that can be completely removed from the auger hole to leave a clean, full-size open hole so the well point can be installed and backfill material placed.
- *Determination of overburden depth to potential sand and gravel deposits for concrete aggregate processing*.—This method would be used to estimate the volume of overburden excavation required to expose the sand/gravel deposit. Assessment of quality of potential concrete aggregate and usable volume could be determined from open pit excavation or by use of a bucket drill.

2. Hollow-Stem Augers.—Hollow-stem auger drilling can provide an efficient and economical method of subsurface exploration, advancing holes for in-place testing, and sampling of overburden material in an undisturbed condition. Hollow-stem augers are manufactured similar to flight augers. The difference between flight augers and

Figure 2-22.—Continuous-flight auger mounted on an all-terrain carrier.

hollow-stem augers is in the design of the center drive shaft. The continuous-flight auger drive shaft consists of a steel tube with end sections for solid pin connections to abutting auger sections. The hollow-stem auger drive shaft, however, consists of a hollow steel tube throughout the total length with threaded or cap screw connections for coupling to abutting auger sections. The advantages of hollow-stem auger drilling over continuous-flight auger drilling are:

- Undisturbed sampling tools and in-place testing equipment can be lowered and operated through the hollow stem without removing the in-hole auger.
- Unstable soils and water zones can be drilled through by the hollow-stem auger without caving.
- Instruments and ground behavior monitoring equipment can be installed and the hole backfilled through the hollow stem.
- Removal of samples through the hollow stem eliminates contamination from upper-strata material.
- The hollow stem may be used as casing so rotary drilling or core drilling operations can be used to advance the hole beyond auger drilling capabilities.

In addition to advantages listed, the hollow-stem auger can function as a continuous-flight auger. This is accomplished by using a plug bit within the center tube of the lead auger, as shown on figure 2-23. The bit can be retracted at any time for undisturbed sampling or in-place testing without removing the auger tools from the drill hole.

In the 1960s, Reclamation developed a double-tube auger to combine soil sampling with auger-hole advancement. This arrangement was quite successful, especially in water sensitive soils such as loess [31]. During the 1980s, auger manufacturers developed (for commercial sale) double-tube auger soil sampling tools so that undisturbed soil samples could be recovered simultaneously with advancement of the hollow-stem auger and without the need for drilling fluids. This development resulted in a method to recover high-quality, undisturbed soil samples of surficial deposits more efficiently and economically than any other method. The method is especially good for sampling low density soils that may be susceptible to collapse upon wetting.

Hollow-stem augers are commonly manufactured in 1.5 m (5 ft) lengths and with sufficient inside clearance to pass sampling or in-place testing tools from 50 to 175 mm (2 to 7 in) in diameter. The spiral flights are generally sized to auger a hole 100 to 125 mm (4 to 5 in) larger than the inside diameter of the center tube. Normally, drill depths are limited by:

- equipment rotational torque capability,
- firmness of materials penetrated,
- cobble or gravel strata,
- caliche zones, or
- bedrock.

3. Helical, Disk, and Bucket Augers.—Helical, disk, and bucket augers are useful for obtaining disturbed samples from large-diameter borings for classifying overburden or borrow soils. Generally, these augers are limited to sampling above the water table without casing and can be further limited by caving ground or oversize particles. Figure 2-24 illustrates the basic differences of these auger systems. Disk-auger drilling can be an economical method of drilling large-diameter holes for disturbed sampling or for installing large-diameter casings for accessible explorations. A helical or disk auger has spiral-shaped flights similar in design to a flight auger; however, it is used as a single-length tool rather than being coupled to abutting sections. Rotational power is provided by a square or hexagonal drive shaft (Kelly bar) on the drill rig. With the disk-auger, drill cuttings are retained by the upper-disk flight and are removed by hoisting the disk auger from the hole after every 1 to 1.5 m (3 to 5 ft) of penetration.

Hole diameters range from 300 to 3,050 mm (12 to 120 in); the larger disk-auger rigs drill to 37 m (120 ft) depth or more using telescoping Kelly drive bars. Unless casing is installed, disk-auger capabilities are generally limited by cobble or boulder strata, saturated flowing sands, or ground-water tables. Weathered or "soft" rock formations can be drilled effectively with a helical auger equipped with wedge-shaped "ripper" teeth. Concrete and "hard" rock can be drilled by helical augers equipped with conical, tungsten-carbide tipped teeth.

In addition, for use in drilling and installation of deep accessible explorations, disk augers are used to recover large-volume samples from specific subsurface strata. They may be used to drill and install perforated casing or well screens for ground-water monitoring systems. The most common use of disk augers is drilling caissons for building foundations.

4. Bucket Drills.—Figure 2-25 shows a bucket drill in operation. Bucket drills or "bucket augers" are used to drill large-diameter borings for disturbed sampling of overburden soil and gravel material. The bucket is designed as a large-diameter hollow steel drum, usually 1 m (3 ft) long, that is rotated by a square or hexagonal drive (Kelly bar) on the drill rig and connected to a steel yoke fixed to

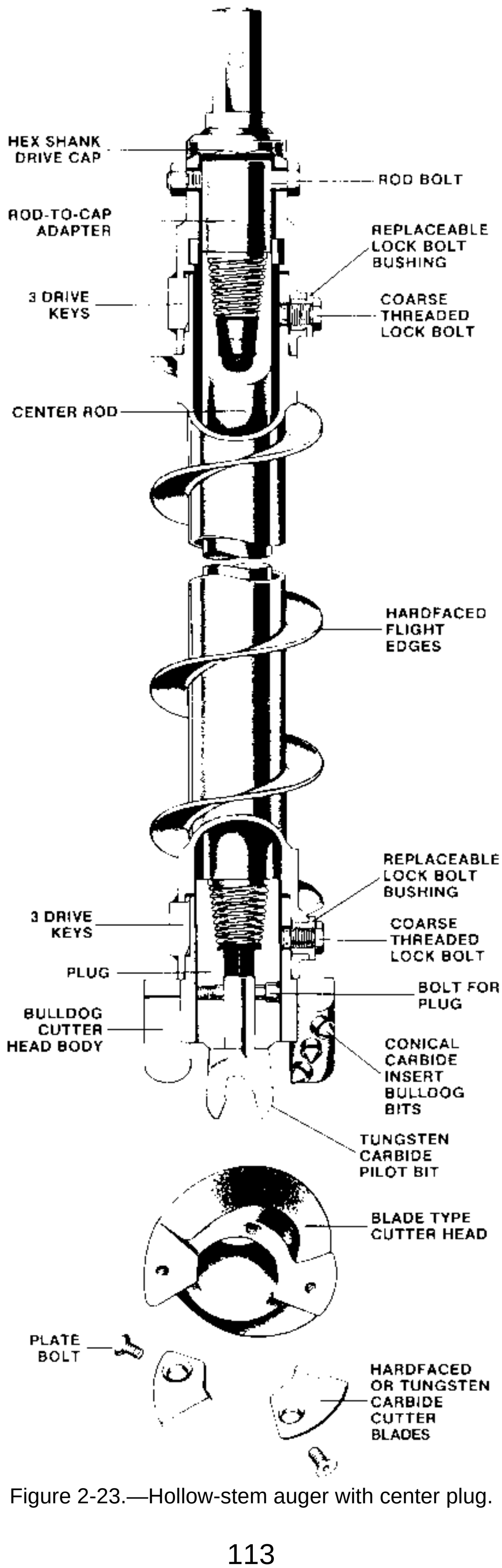


Figure 2-23.—Hollow-stem auger with center plug.

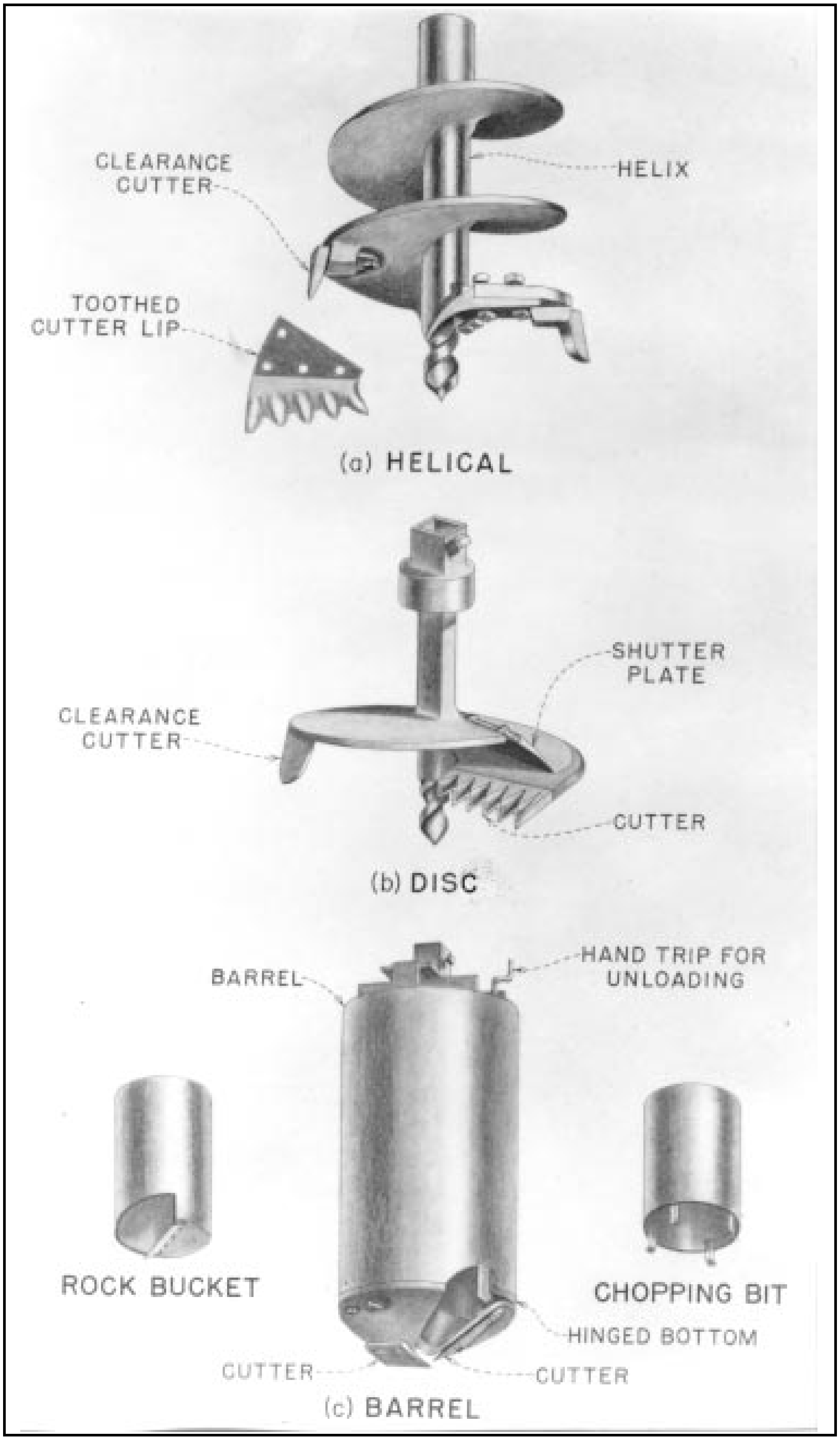


Figure 2-24.—Illustration of the helical, disc, and barrel types of machine-driven augers showing basic differences. (U.S. Department of the Navy)

Figure 2-25.—Bucket drill rig in drilling position with a 24-foot triple Kelly and 36-inch bucket.

the top of the bucket. The bottom of the bucket is designed with a hinged, lockable, steel cutter plate equipped with wedge-shaped ripper teeth. The cutter plate is mechanically locked during the rotational drilling operation and has a 225- to 300-mm (9- to 12-in) bottom opening through which drill cuttings are forced and collected in the bucket. After about 1/2 to 1 m (1.6 to 3.3 ft) of drilling penetration, the bucket is hoisted out of the drill hole, attached to a side-jib boom, and moved off the hole to discharge the cuttings. Cuttings are discharged from the bucket by mechanical release of the hinged cutter plate or by opening one side of the bucket, which also may be hinged. The drilling operation is continued by locking the cutter plate or the hinged side panel of the bucket before lowering the bucket to the hole bottom and continuing drilling.

Holes can range from 300 to 2,100 mm (12 to 84 in) in diameter using a standard bucket. A reamer arm extension, equipped with ripper teeth, can be attached to the bucket drive yoke for overreaming a hole to 3 m (120 in) in diameter using special crane-attached bucket drills. When using overreaming bar extensions, drill cuttings enter the bucket from the bottom cutter plate during rotary penetration. Cuttings also fall into the top of the bucket as a result of the rotational cutting action of the overreamer.

Generally, bucket drill capabilities are limited by saturated sands, boulders, caliche, or the ground-water table unless casing is installed. Weathered or "soft" rock formations can be effectively penetrated with bucket drills. The larger crane-attached bucket drills have achieved depths of 60 m (190 ft) using telescoping Kelly drive bars and crane draw-works hoist systems.

Bucket drills are used for boring caisson holes for foundations. They have proved extremely beneficial for performing subsurface investigations into sand and gravel deposits for concrete aggregate investigations. Also, they may be used to drill and collect intermixed gravel and cobble samples with particle diameters up to about 200 mm (8 in). Bucket drills can be an effective method of drilling deep, accessible, exploration holes. In large caisson applications, the boring can be an "accessible" excavation—as these are often inspected in construction.

5. Enclosed Augers.—Enclosed augers have been used successfully in lieu of casing (fig. 2-26). The outer barrel, which acts as casing, is connected to the sampler on a swivel-type head and remains in a stationary position as the auger rotates.

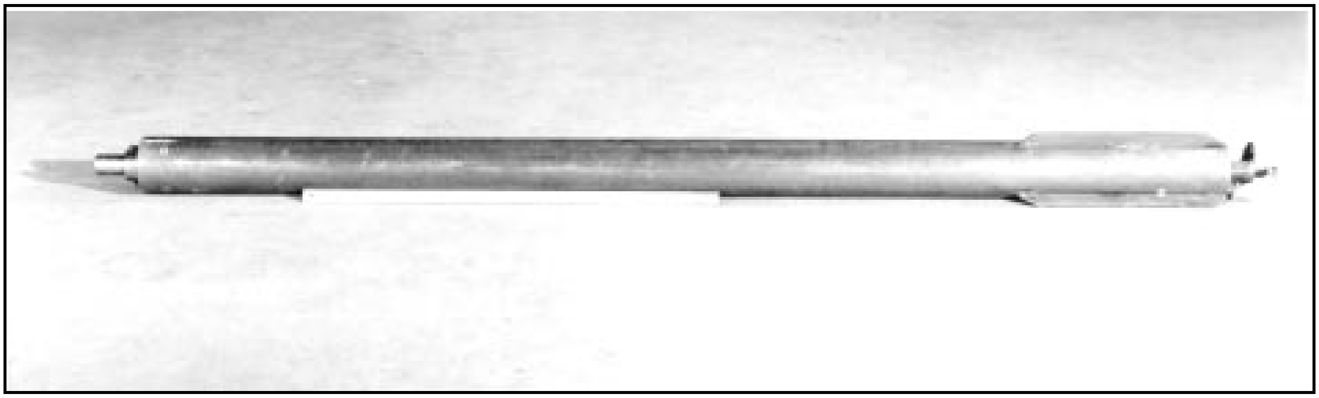
The sampler is lowered to the bottom of the hole and auger rotation started. As the auger penetrates unsampled soil, the outer barrel is pulled down with the unit, thereby holding out any caved or foreign material. The sampled material is retained on the helical auger inside the outer barrel. After completing the sample run, the final penetration depth is carefully noted and the sampler removed from the hole. The auger is rotated in reverse to eject the sampled material. Increased torque capacity of mechanical drills has enhanced capability of continuous or hollow-stem augers to operate below the water table thereby diminishing use of enclosed augers.

d. Nonsampling Borings.—Test holes excavated merely to determine depth to some particular stratum or bedrock, or for advancing a hole to provide access to a buried layer for sampling, can be accomplished by any of the previously described methods. A number of economical procedures are in common use. These procedures include percussion or churn drilling, wash boring, and jetting. All operations are based on moving the tool up and down to chop away the material in the hole, using increasing amounts of water in the order listed—except probing which uses none at all. Often, probing is an economical method for establishing the depth to a firm stratum. Variations in procedure depend primarily on the nature of the soil to be penetrated, with percussion drilling being used on the hardest and most dense soils, and probing on the softest.

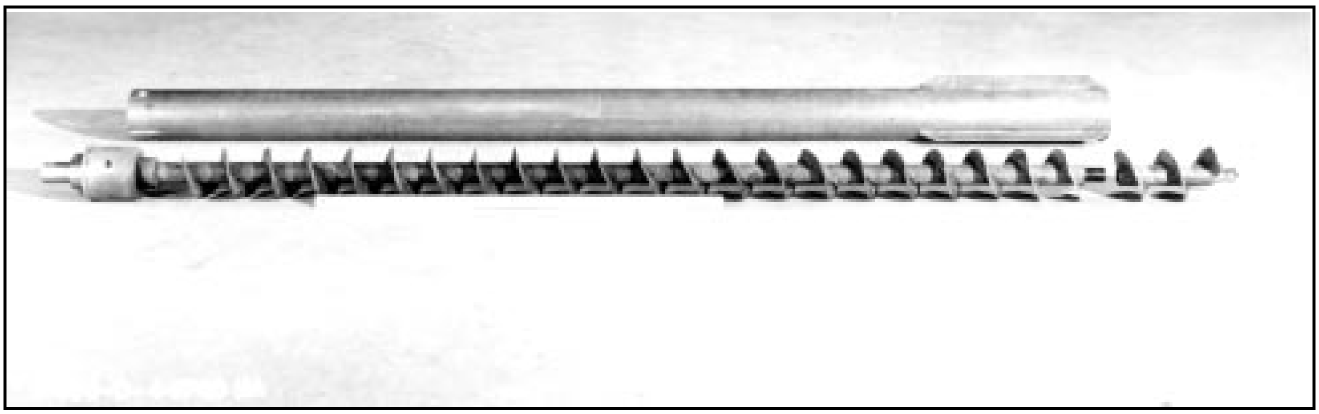
In churn or cable tool drilling, the tool is attached to the end of a cable. Water is added, and the cuttings form a slurry which is removed intermittently by pumping or bailing. In wash boring and jetting, the cuttings are removed with a continuous flow of water from the top of the hole. Wash boring advances the hole by a combination of chopping and washing. Jetting depends primarily on the cutting action of a high-pressure stream of water. Care must be exercised to avoid disturbing and moistening the underlying stratum to be sampled when using these methods. Some indication of the nature of material penetrated is obtained by examining cuttings in the sludge or wash water, but accurate classifications require other sampling methods. Probing consists of driving or pushing a rod or pipe into the soil and measuring the effort required. Probing can be accomplished with or without jetting tips. If the layers to be penetrated are soft enough, crude probing may be replaced by cone penetration or other dynamic penetrometers to gain additional information on surficial materials.

2-11. Rotary Drilling Methods.—Nonaccessible Borings.—

a. General.—One of the most important tools for subsurface exploration is a rotary drill rig complete with core barrels, diamond bits or hardened metal bits, and a



Enclosed auger assembled.



Auger disassembled.

Figure 2-26.—Enclosed auger.

hydraulic or screw feed. The drill may be operated with a variety of samplers and bits, depending on the hardness of the material penetrated. Rotary drill equipment is manufactured in a wide variety of forms that vary from highly flexible to extremely specialized equipment, from lightweight and highly mobile equipment to heavy stationary plants, and range in size of hole and core from less than 25 (1 in) to 900 mm (36 in) or more in diameter. Normally, drill equipment is mounted on trucks but a wide variety of carrier units such as track, all terrain, and skid mounts are available. They are capable of drilling to depths of hundreds or thousands of meters depending upon the type of rig and the material penetrated.

The Diamond Core Drill Manufacturers Association (DCDMA), which is composed of members from the United States and Canada, has established dimensional standards for a series of nesting casings with corresponding sizes for bits and drill rods [32]. These standards allow for interchangeable use of equipment from different manufacturers. Table 2-1 shows the nomenclature for hole size, group, and design. Nominal hole sizes run from **R** to **Z** [25 to 200 mm (1 to 8 in), respectively]. These nominal hole sizes are often specified in exploration requests.

The DCDMA standards for drill rods, casings, and core bit sizes are given on tables 2-2, 2-3, 2-4, and 2-5. Casing, drill rods, core bits, and barrels are designed to be used as a system for a particular hole size and group of tools. For example, **HX** core-barrel bits will pass through flush-coupled **HX** casing (flush-coupled casing is denoted by the group letter **X)** and will drill a hole large enough to admit flush-coupled **NX** casing (the next smaller size) and so on to the **RX** size. Flush-joint casing, denoted by group letter **W**,

Table 2-1.—Diamond Core Drill Manufacturers Association nomenclature for bits and drill rods

<table>
<tr><th colspan="3">Three-Letter Names</th></tr>
<tr><th>First letter</th><th>Second letter</th><th>Third letter</th></tr>
<tr><th>Hole size</th><th>Group</th><th>Design</th></tr>
<tr><td>Casing, core barrel, diamond bit, reaming shell,and drill rods designed to be used together for drilling a nominal hole size.</td><td>Key diameters standardized on an integrated group basis for progressively reducing hole size with nesting casings.</td><td>The standardization of other dimensions, including thread characteristics, to allow interchangeability of parts made by different manufacturers.</td></tr>
<tr><td><table>
<tr><th>Letter</th><th>Inch</th><th>Milli-meter</th></tr>
<tr><td>R</td><td>1</td><td>25</td></tr>
<tr><td>E</td><td>1-1/2</td><td>40</td></tr>
<tr><td>A</td><td>2</td><td>50</td></tr>
<tr><td>B</td><td>2-1/2</td><td>65</td></tr>
<tr><td>N</td><td>3</td><td>75</td></tr>
<tr><td>K</td><td>3-1/2</td><td>90</td></tr>
<tr><td>H</td><td>4</td><td>100</td></tr>
<tr><td>P</td><td>5</td><td>125</td></tr>
<tr><td>S</td><td>6</td><td>150</td></tr>
<tr><td>U</td><td>7</td><td>175</td></tr>
<tr><td>Z</td><td>8</td><td>200</td></tr>
</table></td><td>Letters x and w are synonymous when used as the GROUP (second) letter.

Any DCDMA standard tool with an x or w as the GROUP letter belongs in that DCDMA integrated group of tools designed using nesting casings and tools of sufficient strength to reach greater depths with minimum reductions in core diameter.</td><td>The DESIGN (third) letter designates the specific design of that particular tool.
It does not indicate a type of design.</td></tr>
</table>

Two-Letter Names	
First letter	Second letter
Hole size	Group and design
Approximate hole size, same as in 3-letter names.	GROUP standardization of key diameters for group integration and DESIGN standardization of other dimensions affecting interchangeability.

Table 2-2.—Diamond Core Drill Manufacturers Association drill rod specifications

Rod type	Outside diameter in	Outside diameter mm	Inside diameter in	Inside diameter mm	Coupling identification in	Coupling identification mm	Mass per foot lbm	Threads per inch	Thread type
W Series Drill Rod									
RW	1.094	27.8	0.719	18.3	0.406	10.3	1.4	4	Regular
EW	1.375	34.9	0.938	22.2	0.437	12.7	2.7	3	"
AW	1.750	44.4	1.250	31.0	0.625	15.9	4.2	3	"
BW	2.125	54.0	1.500	44.5	0.750	19.0	6.1	3	"
NW	2.625	66.7	2.000	57.4	1.38	34.9	7.8	3	"
HW	3.500	88.9	3.062	77.8	2.375	60.3	9.5	3	"
WJ Series Drill Rod									
AWJ	1.75	44.5	1.43	36.4	0.63	16.1	3.6	5	Taper
BWJ	2.13	54.0	1.81	46.0	0.75	19.3	5.0	5	"
NWJ	2.63	66.7	2.25	57.0	1.13	28.8	6.0	4	"
KWJ	2.88	73.0	2.44	61.9	1.38	34.9		4	"
HWJ	3.50	88.9	2.88	73.1	1.75	44.5		4	"
Old Standard									
E	1.313	33.3	0.844	21.4	0.438	11.1		3	Regular
A	1.625	41.3	1.266	28.6	0.563	14.3		3	"
B	1.906	48.4	1.406	35.7	0.625	15.9		5	"
N	2.375	60.3	2.000	50.8	1.000	25.4		4	"

Table 2-3.—Wire-line and American Petroleum Institute drill rods

Wire-line Drill Rods

Rod type	Outside diameter		Inside diameter		Gallons per 100 ft	Weight per lbm	Threads per inch	Thread type
	in	mm	in	mm				
AQWL[1]	1.750	44.5	1.375	34.9	7.7	3.3	4	Taper
AXWL[2]	1.813	46.0	1.500	38.1	9.18	2.8	4	Regular
BQWL[1]	2.188	55.6	1.812	46.0	13.4	4.0	3	Taper
BXWL[2]	2.250	57.2	1.906	48.4	14.82	3.8	4	Regular
NQWL[1]	2.750	69.9	2.375	60.3	23.0	5.2	3	Taper
NXWL[2]	2.875	73.0	2.391	60.7	23.30	6.8	3	Regular
HQWL[1]	3.500	88.9	3.062	77.8	38.2	7.7	3	Taper
HXWL[2]	3.500	88.9	3.000	76.2	36.72	8.7	3	Regular
PQWL[1]	4.625	117.5	4.062	103.2				
CPWL[2]	4.625	117.5	4.000	101.6				

[1] Q Series rods are specific manufacturer's design.
[2] X Series rods are specific manufacturer's design.

API Tool Joints — Internal Flush (in-lb system)

Type/size	Joint body o.d.	Pin i.d.	Thread type
API 2-3/8	3.375	1.750	Taper
API 2-7/8	4.125	2.125	"
API 3-1/2	4.750	2.687	"
API 4	5.750	3.250	"
API 4-1/2	6.125	3.750	"

Table 2-4.—Diamond Core Drill Manufacturers Association casing specifications

DCDMA Casing Design

Size	Outside diameter		Inside diameter W series		Inside diameter X series		Gallons per l00 ft	Mass per ft	Threads per inch	
	in	mm	in	mm	in	mm			W series	X series
RW, RX	1.44	36.5	1.20	30.5	1.20	302.0	5.7	1.8	5	8
EW, EX	1.81	46.0	1.50	38.1	1.63	41.3	9.2	2.8	4	8
AW, AX	2.25	57.2	1.91	48.1	2.00	50.8	14.8	3.8	4	8
BW, BX	2.88	73.0	2.38	60.3	2.56	65.1	23.9	7.0	4	8
NW, NX	3.50	88.9	3.00	76.2	3.19	81.0	36.7	8.6	4	8
HW, HX	4.50	114.3	4.00	100.0	4.13	104.8	65.3	11.3	4	5
PW, PX	5.50	139.7	5.00	127.0	5.13	130.2		14.0	3	5
SW, SX	6.63	168.3	6.00	152.4	6.25	158.8		16.0	3	5
UW, UX	7.63	193.7	7.00	177.8	7.19	182.6			2	4
ZW, ZX	8.63	219.1	8.00	203.2	8.19	208.0			2	4

W series casing is known as "flush-coupled casing." W series casing has flush inside diameter throughout, while X series casing has upset diameter with coupling inside diameter equal to flush wall inside diameter.

Table 2-5.—Approximated core and hole diameters for core barrels

*Core barrel type/group	Set bit dimension inside diameter ≈ **Core Diameter**		Set reaming shell ≈ **Hole Diameter**	
	in	mm	in	mm
Conventional Core Barrels[1]				
RWT (d)	0.735	18.7	1.175	29.8
EWD_3	0.835	21.2	1.485	37.7
EWG (s.d.), EWM (d)	0.845	21.5	1.485	37.7
EWT (d)	0.905	23.0	1.485	37.7
AWD_3, AWD_4	1.136	28.9	1.890	48.0
AWG (s.d.), AWM (d)	1.185	30.1	1.890	48.0
AWT (d)	1.281	32.5	1.890	48.0
BWD_3, BWD_4	1.615	41.0	2.360	59.9
BWG (s.d.), BWM (d)	1.655	42.0	2.360	59.9
BWT (s.d)	1.750	44.4	2.360	59.9
NWD_3, NWD_4	2.060	52.3	2.980	75.7
NWG (s.d.), NWM (d)	2.155	54.7	2.980	75.7
NWT (s.d.)	2.313	58.8	2.980	75.7
HWD_3, HWD_4	2.400	61.1	3.650	92.7
HWG (s.d.)	3.000	76.2	3.907	99.2
HWT (s.d.)	3.187	80.9	3.907	99.2
DCDMA Large Diameter—Double-Tube Swivel—Core Barrels				
2-3/4 × 3-7/8	2.690	68.3	3.875	98.4
4 × 5-1/2	3.970	100.8	5.495	139.3
6 × 7-3/4	5.970	151.6	7.750	196.8
Wire-line Core Barrel Systems[2]				
AXWL (joy)	1.016	25.8	1.859	47.2
AQWL	1.065	27.1	1.890	48.0
BXWL	1.437	36.5	2.375	60.3
BQWL	1.432	36.4	2.360	60.0
BQ_3WL	1.313	33.4	2.360	60.0
NXWL	2.000	50.8	2.984	75.8
NQWL	1.875	47.6	2.980	75.7
NQ_3WL	1.75	44.4	2.980	75.7

Table 2-5.—Approximated core and hole diameters for core barrels (continued)

*Core barrel type/group	Set bit dimension inside diameter ≈ **Core Diameter**		Set reaming shell ≈ **Hole Diameter**	
	in	mm	in	mm
Conventional Core Barrels[1]				
HXWL	2.400	61.0	3.650	92.7
HQWL	2.500	63.5	3.790	96.3
HQ_3WL	2.375	60.3	3.790	96.3
CPWL	3.345	85.0	4.827	122.6
PQWL	3.345	85.0	4.827	122.6
PQ_3WL	3.25	82.6	4.827	122.6

[1] Conventional double-tube core barrels are available in either rigid or swivel designs. The swivel design inner barrel is preferred for sampling because it aids in preventing core rotation. In general, smallest core for given hole size results in best recovery in difficult conditions (i.e., triple-tube core barrels). Use of double-tube-swivel type barrels, with split liners, are recommended in geotechnical investigations for best recovery and least sample damage.
[2] Wire-line dimensions and designations may vary according to manufacturer.
*s = single tube d = double tube

is such that 20- by 150-mm (3/4- by 6-in) nominal core-barrel bits will pass through **ZW** casing and will drill a hole large enough to admit flush-jointed **UW** casing (next smaller size) and so on to the **RW** size. An illustration of nested casings and casing terminology is shown on figure 2-27.

The straight **A** through **N** drill rods with no group designation are no longer commonly used. The recent trend in the drilling industry has been toward the **J** group of drill rods with taper threads although some **W** group rods are still frequently used. Wire-line drill rod (casing) sizes are also listed on table 2-3. Wire-line core barrel systems have not been standardized by DCDMA, but most manufacturers adhere to nominal hole and core sizes listed in the tables.

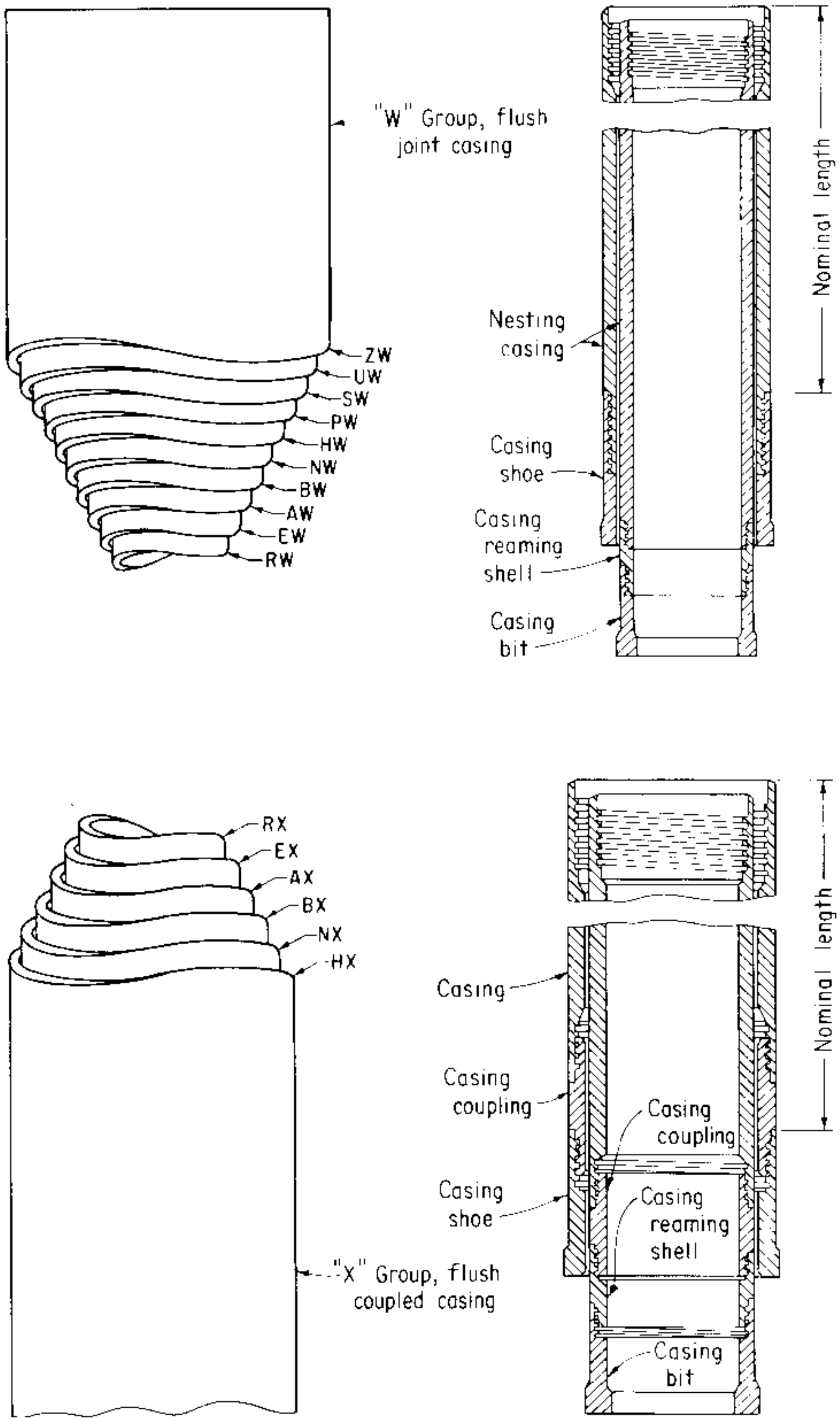
Rotary drills are mechanical and/or hydraulic, engine-powered drills designed for medium rotational torque at variable rotational speeds from low (≈100 r/min) for hole penetration using tricone rock bits or carbide-tipped drag bits to medium-high (≈800 r/min) for undisturbed soil sampling or rock core drilling with core barrels.

All rotary drills are equipped with high-pressure fluid injection pumps or air compressors to circulate drill fluid media. These media, which may consist of water, drill mud, compressed air, or air-foam are used to cool and lubricate drill bits and to hold drill cuttings in suspension for circulation to the top of the hole at ground surface. Accessories essential for a drill rig are:

- A watermeter
- A cathead winch and derrick for driving casing and for hoisting and lowering drill rods
- A pump for circulating drilling fluid to the bit and for flushing and water testing the hole
- The required driving hammer, bits, drill rods, and core barrels

Usually, supported holes are required except when drilling through competent rock or stiff cohesive soils. A short surface casing about 1.5 to 3 mm (5 to 10 ft) long is commonly used at the top of the hole. Use of drilling fluids—including hole wall stabilizer compounds—often nullifies the need for casing in soil, but the foundation cannot be effectively water tested when bentonite drill mud is used.

At least two drive hammers should be available, a 64-kg (140-lbm) safety or automatic hammer for standard field penetration tests (see USBR 7015) and a 113 to 181 kg mass



Note: Use of casing shoe allows nesting; use of casing bit does not.

Figure 2-27.—Core drill casing.

(250 to 400 lbm) for driving and removing casing pipe. The hammer is raised by pulling tight on an attached rope which is threaded through a sheave at the top of the derrick and wound two to four times on the revolving cathead winch. Sudden loosening of the rope permits the hammer to drop on the driving head attached to the casing. Various types of chopping bits are used to facilitate driving casing through soils containing cobbles and boulders. Large boulders must be drilled with a diamond bit or a roller rock bit or blasted. Casing is raised before blasting. Figure 2-28 shows a drill rig with derrick.

Although the rotary drill was designed primarily for penetrating rock rather than soil, many sample barrels and cutting bits have been developed for investigating a wide variety of soil deposits. Double-tube core barrel samplers of the Denison, Denver, Pitcher, and double-tube auger types are capable of obtaining 150-mm (6-in) diameter undisturbed samples of sands, silts, or clays for laboratory testing. These samplers are described in USBR 7105.

The following discussion, excerpted from the *Design of Small Dams* [16] provides a comprehensive review of rotary drilling equipment technology.

b. Rotary Drills.—Seven distinctively different types of rotary drills are used for subsurface explorations:

- Rotary-table drills
- Top-head drive drills
- Hollow-spindle drills
- Fluted Kelly drills
- Reverse-circulation (rotary and percussion) drills
- Top-head drive with percussion casing hammer drills
- Horizontal rotary drills

Each type of rotary drill is described; and a typical application for which each drill was designed follows. In addition, although not classified as a rotary drill, operation and use of a churn/cable-tool drill is described.

1. Rotary-Table Drills.—Initially, rotary drills were developed for the petroleum industry as a stationary-plant, heavy-duty drill machine that used a large rotational table mechanism to provide rotary power to a rigid tubular string of rods with a bit attached. Hole penetration, using rotary-table drills, is accomplished by using heavy, weighted drill collars coupled to the drill rod and high-pressure pumps that discharge drill cutting circulation fluid through small jet ports in the bit. The mass, cutting action of the rotary bit, and high-pressure jetting-action of the circulation fluid all combine to rapidly advance the drill hole through all types of surficial deposits and bedrock. Although this type of drill proved extremely effective for the petroleum industry, the rotary-table drill was not designed to perform relatively shallow subsurface investigations. This type of drilling operation is prohibitively costly for shallow exploration work and results in extremely poor core quality in low-strength rock.

A smaller version of the rotary-table drill was developed by drilling rig manufacturers to perform shallow explorations and for water well drilling. These drills are generally truck-mounted rigs with either a chain-driven or gear-driven rotary table. Use of smaller, lower pressure pumps with bypass systems provides better control over downhole fluid pressure that can easily erode or fracture subsurface soil or rock. Mechanical chain pulldowns were added to eliminate use of heavy drill collars and for more precise control over downward bit pressure. However, a drill with a mechanical chain pulldown is not designed with the precision control features necessary to recover high-quality core samples of soil or laminated hard to soft rock. This type of rotary-table drill is used primarily in the water well industry and can be a useful method for installing ground-water monitoring systems.

Rotary-table drills can drill holes from 150 to 600 mm (6 to 24 in) in diameter. Depth capabilities can range from 750 to greater than 3,000 m (2,500 to > 10,000 ft).

2. Top-Head Drive Drills.—The top-head drive drill was developed to provide greater operator control over the drilling operation. This is accomplished through use of variable-speed hydraulic pumps and motors to control rotational speed and downward bit pressure. Incorporation of hydraulic systems into drilling machinery vastly improved drilling capabilities, performance, and reliability—with less down time for costly repairs. A skilled operator can precisely control even the largest top-head drive drill by:

- Monitoring drill-head hydraulic pressure (indicating bit torque resistance),
- Monitoring drill fluid circulation pressure (indicating open-hole, blocked-hole, open-bit, or plugged-bit condition), and
- Controlling applied hydraulic pulldown pressure, making it compatible with required bit pressure to drill a given formation at a constant and efficient rate of penetration.

In addition to controlled hydraulic down pressure (crowd pressure), the new top-head drive drill rigs are equippedwith "float" controls that provide pulldown pressure equal to the weight of the drill head and in-hole drill tools, and

Figure 2-28.—Photograph of typical rotary drill showing some of the essential equipment for rotary drilling.(Central Mine Equipment Company)

with "hold-back" controls, which apply a back pressure to the down pressure to reduce applied weight at the bit. All of these features make the top-head drive rotary drill one of the most advanced drilling units for high-quality subsurface explorations.

Top-head drive rotary drills are generally long-stroke drills capable of a continuous penetration of 3 to 10 m (10 to 30 ft) without requiring additional rods or "rechucking." Conventional drilling to advance boreholes to specific depths is normally accomplished using 60 to 140 mm (2-3/8 to 5-1/2 in) outside diameter (o.d.) drill rods. For drilling stability, maintenance of hole alignment, and efficient circulation of drill cuttings out of the hole, drill rod diameter should not be less than one-half the diameter of the cutting bit. A drill rod/bit combination of a 115-mm- (4-1/2-in-) o.d. rod and a 200-mm (8-in-) diameter bit results in a nominal annulus of 44 mm (1-3/4 in) between the rod and drill hole wall. This size annulus is adequate for efficient removal of all drill cuttings by high-velocity circulation of drill fluid while maintaining minimum pump pressure. For holes larger than 200 mm (8 in) in diameter, centralizers or stabilizers—about 25 mm (1 in) smaller in diameter than the bit—should be added to the drill rod string on approximately 9-m (30-ft) centers. These devices stabilize the drill string and aid removal of drill cuttings from the hole through a reduced annulus area.

Downhole percussion hammers are commonly used with top-head drive drills for rapid penetration through hard materials and to maintain a better drill-hole alignment than can be achieved with use of tricone rock bits.

Tricone rock bits are generally rotated 3 to 4 times faster than a downhole hammer but tend to drift off alignment when one or more cutting cones contact the edge of a boulder or other obstruction. Downhole hammers are operated with air or an air-foam mix as the drilling fluid and are generally rotated between 12 and 20 rpm. The bit is slightly concave and embedded with rounded tungsten-carbide buttons that chip away at the rock with the rapid in-out percussion impact blows. The slow rotation and direct impact hit of the single-piece button bit can result in a truer hole alignment than using a 3-roller tricone bit. Under reamer percussion, bits also allow rapid drilling or casing through overburden materials.

In subsurface exploration programs, top-head drive drills are commonly used to install ground-water monitoring systems and structural-behavior monitoring instruments. They are used for geothermal investigations, drilling waste injection wells, and to recover large-diameter samples of surficial deposits or rock core. When continuous cores are required, a large-diameter wire-line system should be used to enhance the efficiency of the operation and to eliminate the need for removing all drill rod from the hole for core recovery.

In all coring operations using air-foam as the circulation media, the o.d. of the core bit must be sized to drill a hole no less than 22 mm (7/8-in) larger in diameter than the o.d. of the drill rod. Water or low-viscosity drill mud circulation could be accomplished with a core bit no less than 13 mm (1/2-in) larger diameter than the o.d. of the rod.

Hole diameters using top-head drive drills generally range from 150 to 600 mm (6 to 24 in); depth capabilities may range from 460 to more than 1,525 m (1,500 to 5,000 ft). Figure 2-29 shows a top-head drive drill with head in mast.

3. Hollow-Spindle Drills.—The hollow-spindle drill is a multiple-use drill developed for rapid changeover from auger drilling to rotary or core drilling operations. Basically, the hollow-spindle transmits rotary drive power, pull down, and retract to the specific drill tools being used. Unlike other rotary drills designed to drill only with tubular-shaped drill rods or heavy-duty Kelly bars, the hollow-spindle drillhead was designed for attachment of a flight auger or hollow-stem auger drive head. Manual or hydraulically activated chuck assemblies can be used to clamp tubular drill rods, or automatic chuck assemblies for clamping and drilling with fluted Kelly drive bars.

Another advantage of a hollow-spindle drill is that the spindle opening provides access for passage of sampling tools or of testing tools through larger-diameter drill rod or through the hollow-stem auger without having to disassemble major equipment. This is especially advantageous with hollow-stem auger drilling, wire-line core drilling, or penetration resistance testing operations.

Hollow-spindle drills are manufactured either with variable-speed hydraulic drillheads or mechanically driven drillheads with multiple rotary-speed transmissions. Pulldown feed rate and retraction are hydraulically controlled and can be set to automatically maintain a constant rate of feed and pressure on the drill bit. Hollow-spindle drills are manufactured with capability to drill 1.7 to 1.3 m (6 to 11 ft) in a single feed stroke without having to add drill rods or rechuck to achieve additional depth.

A wide variety of sampling and in-place testing operations can be performed with a hollow-spindle drill. Disturbed samples can be obtained by flight auger drilling. Undisturbed samples can be obtained using 75 to 125 m

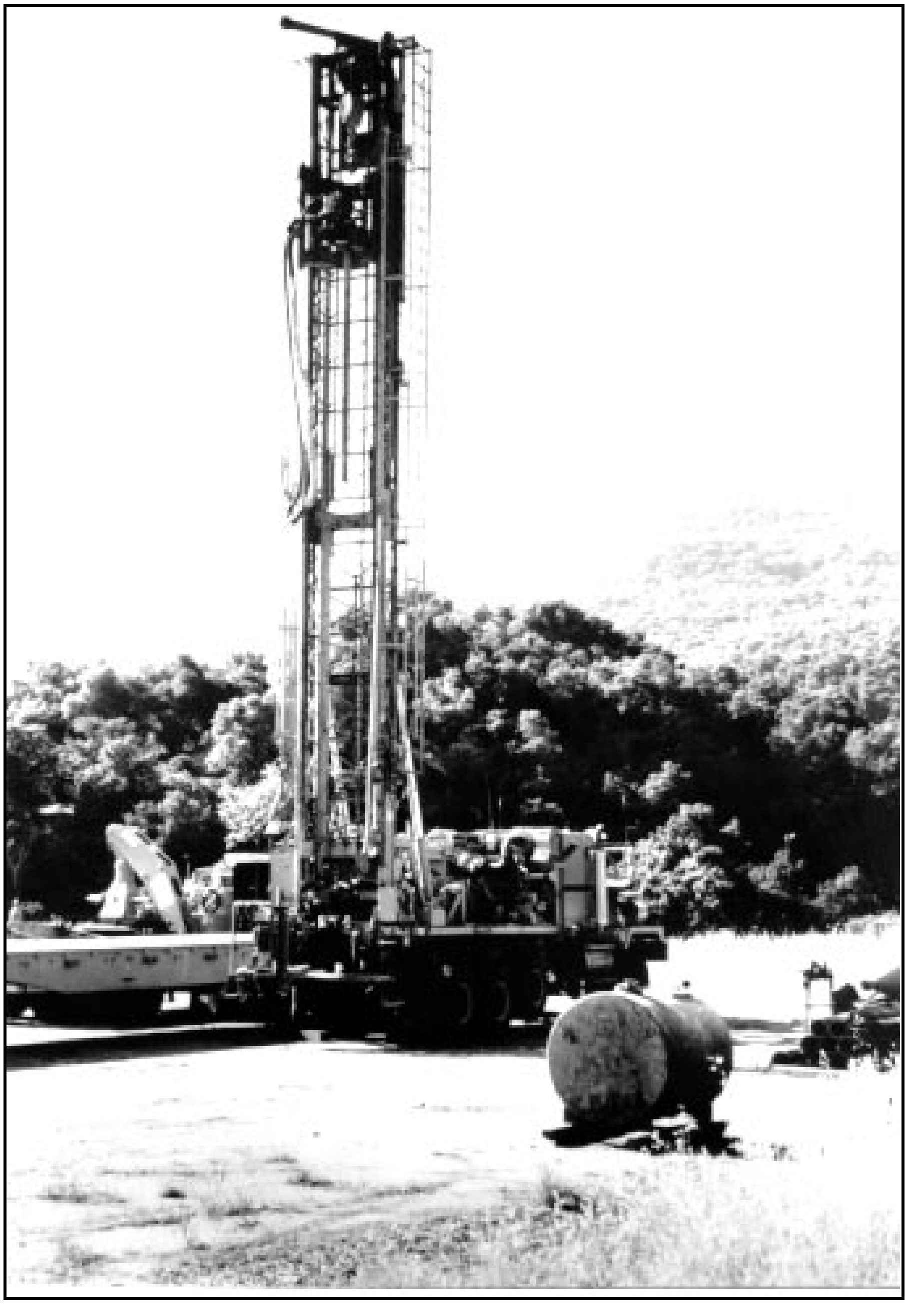


Figure 2-29.—Top-head drive drill with head-in mast for drilling.

(3 to 5 in) thinwall push tubes or with soil samplers designed to lock within the hollow-stem auger and simultaneously recover a soil core sample with advancement of the auger. Large-diameter undisturbed soil samples [100 to 150 mm diameter (4 to 6 in)] can be recovered using drill mud or air-foam circulation media and conventional soil-sampling core barrels. The hollow-spindle design also permits piston sampling of noncohesive sands or sampling of saturated soils with sampling tools that require an inner rod within the drill rod. Rock coring operations can be performed using wire-line systems or conventional core barrels with water, air, or air-foam circulation media.

In-place testing can be conducted from within the hollow-stem auger or casing without major equipment changeover. Specific in-place tests, which can be efficiently performed with a hollow-spindle drill, are vane shear, penetration resistance, flat plate dilatometer, and hydraulic Dutch-cone testing.

Hollow-spindle drill holes generally do not exceed 200 mm (8 in) in diameter. Depth capabilities vary:

- 45 m (150 ft) approximately, through surficial deposits with a hollow-stem auger,
- 60 m (200 ft) through surficial deposits with a flight auger,
- 245 m (800 ft) through surficial deposits and bedrock with a 150-mm- (6-in-) diameter rotary bit, and
- up to 305 m (1,000 ft) through bedrock with a 75-mm- (3-in-) diameter wire-line coring system.

4. Fluted Kelly Drills.—Figure 2-30 shows a fluted Kelly drill setup. A rotary drill equipped with a fluted Kelly rod is designed to continuously drill 3 to 9 m (10 to 30 ft) (depends upon length of Kelly rod) without having to add additional drill rods. The Kelly rod is a thick-walled tubular steel rod with 3 or 4 semicircular grooves milled on equally spaced centers into the outer wall of the rod and parallel to the longitudinal axis of the rod. The milled grooves (flutes) run continuously along the total length of the Kelly rod except through the upper and lower tool joint connections.

Drills equipped with fluted Kelly rods are generally designed so rotational power is supplied to the Kelly rod through combined use of a stationary drillhead and rotary quill. The quill is equipped with automatic pulldown to apply downward pressure and a Kelly drive bushing for rotational drive to the Kelly rod. The Kelly drive bushing contains hardened steel pins sized to fit into the rod flutes which transmit rotational drive power to the Kelly rod. While rotational torque is being applied by the drive bushing pins within the flute grooves, the Kelly rod has unrestricted up or down movement throughout the total length of the flutes. Hole advancement is accomplished by engaging the automatic pulldown to clamp and apply hydraulically controlled down pressure to the Kelly rod. It is common practice to disengage the automatic pulldown, in relatively easy drilling material, and let the weight of the total drill string (Kelly rod, attached drill rods, and bit) advance the hole with holdback control maintained by braking the draw-works hoist cable attached to the top of the Kelly rod. (Draw works: an oil-well drilling apparatus that consists of a countershaft and drum and that is used for supplying driving power and lifting heavy objects.)

Fluted Kelly drills are commonly used for subsurface exploration to bore 150- to 200-mm- (6- to 8-in) diameter holes through surficial deposits and bedrock, to set casing, and to recover large-diameter 100- to 150-mm (4- to 6-in) undisturbed soil or rock cores with conventional core barrels. Usually, drill mud or air-foam is used to remove cuttings. A fluted Kelly drill is not considered efficient for exploration programs where continuous core recovery is required because they are not generally equipped for wire-line core operations. This limitation significantly reduces coring production because all rods and the core barrel must be removed from the hole after each core run.

Fluted Kelly drills are best used for drilling and installing water and/or wells. Hole sizes may be drilled to 300 mm (12 in) in diameter and to depths ranging from 300 to 450 m (1,000 to 1,500 ft).

5. Reverse-Circulation Drills.—A reverse-circulation drill (rotary and percussion) is a specialized rotary or percussion drill that uses a double-walled tubular drill rod. The circulation drilling media, compressed air or air-foam, is forced downhole through the annulus between the inner and outer rod wall. For a reverse-circulation rotary drill, the circulation media is ejected near the tool joint connection between the rotary bit and the center rod. The media circulates around the outside face of the bit to cool the bit and moves drill cuttings upward through a center opening in the bit. The cuttings are forced up the center tube to a discharge point at the hole collar. For a reverse-circulation percussion drill, the circulation media is ejected just above the drive shoe on the outer rod. The circulation media forces drill cuttings in the drive shoe upward through the center tube to a discharge point at the hole collar, as shown in figure 2-31 from reference [34].

The reverse-circulation rotary drill uses a hydraulically powered top-head drive drillhead and hydraulic pulldown/retract system. This drill is especially useful for drilling through loss circulation zones (loose sands, voids, etc.), for recovering uncontaminated disturbed samples, and for testing water aquifer yield. Drill depths to 300 m (1,000 ft) can be achieved using a dual-wall drill rod with an o.d. of 140 mm (5-1/2 in) and a center tube inside diameter of 80 mm (3-1/4 in).

The reverse-circulation percussion drill uses an air- or diesel-powered pile drive hammer to drive dual-wall drive pipe ranging from 140 mm o.d. (5.5 in) outer tube by 83 mm i.d. (3.25 in) inner tube to 610 mm o.d. (24 in) outer tube by 305 mm i.d. (12 in) inner tube. Depth capabilities range from 15 m (50 ft) (with the 610-mm o.d. drive pipe) to 105 m (350 ft) (with the 140-mm o.d. drive pipe). This drill is especially good for drilling gravel to

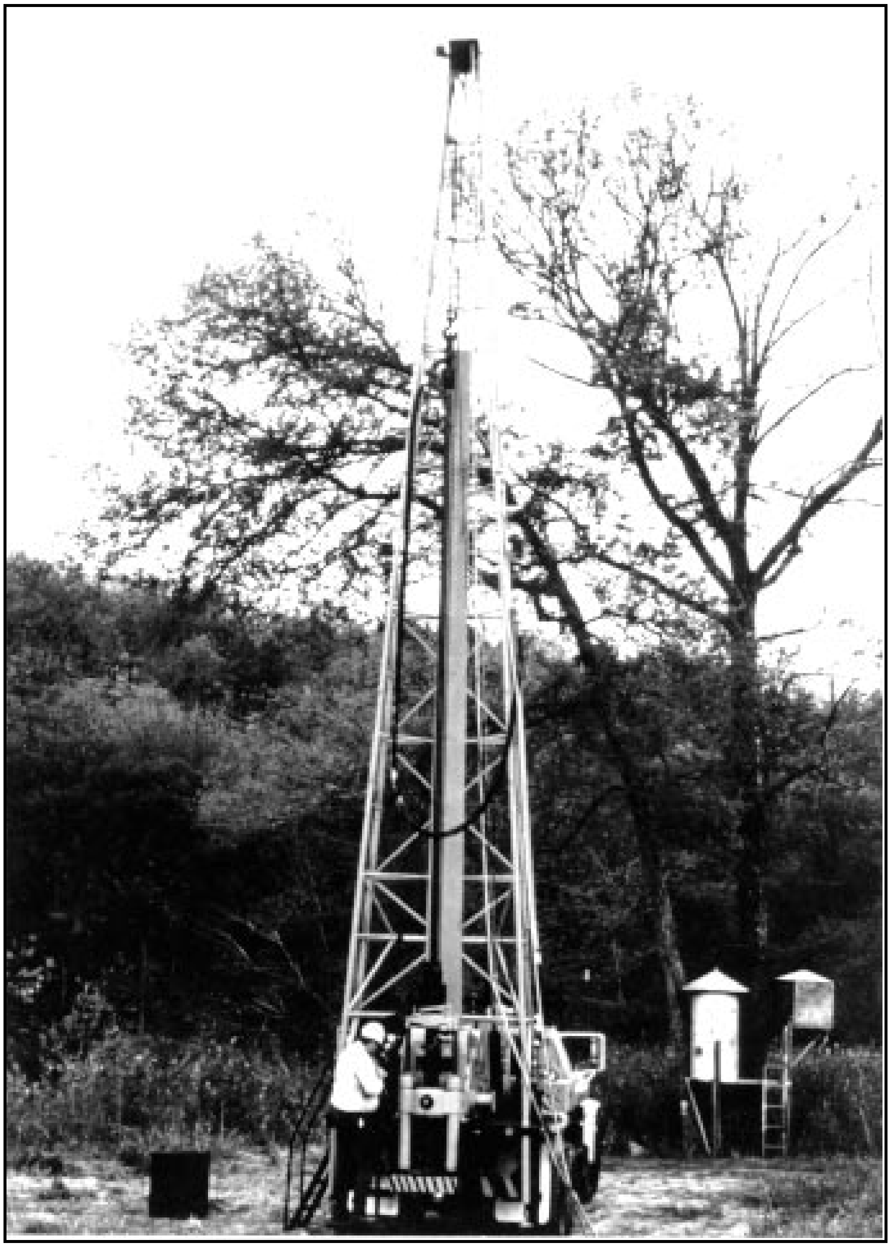


Figure 2-30.—Fluted Kelly drill setup. Automatic pull-down chuck assembly and breakout table.

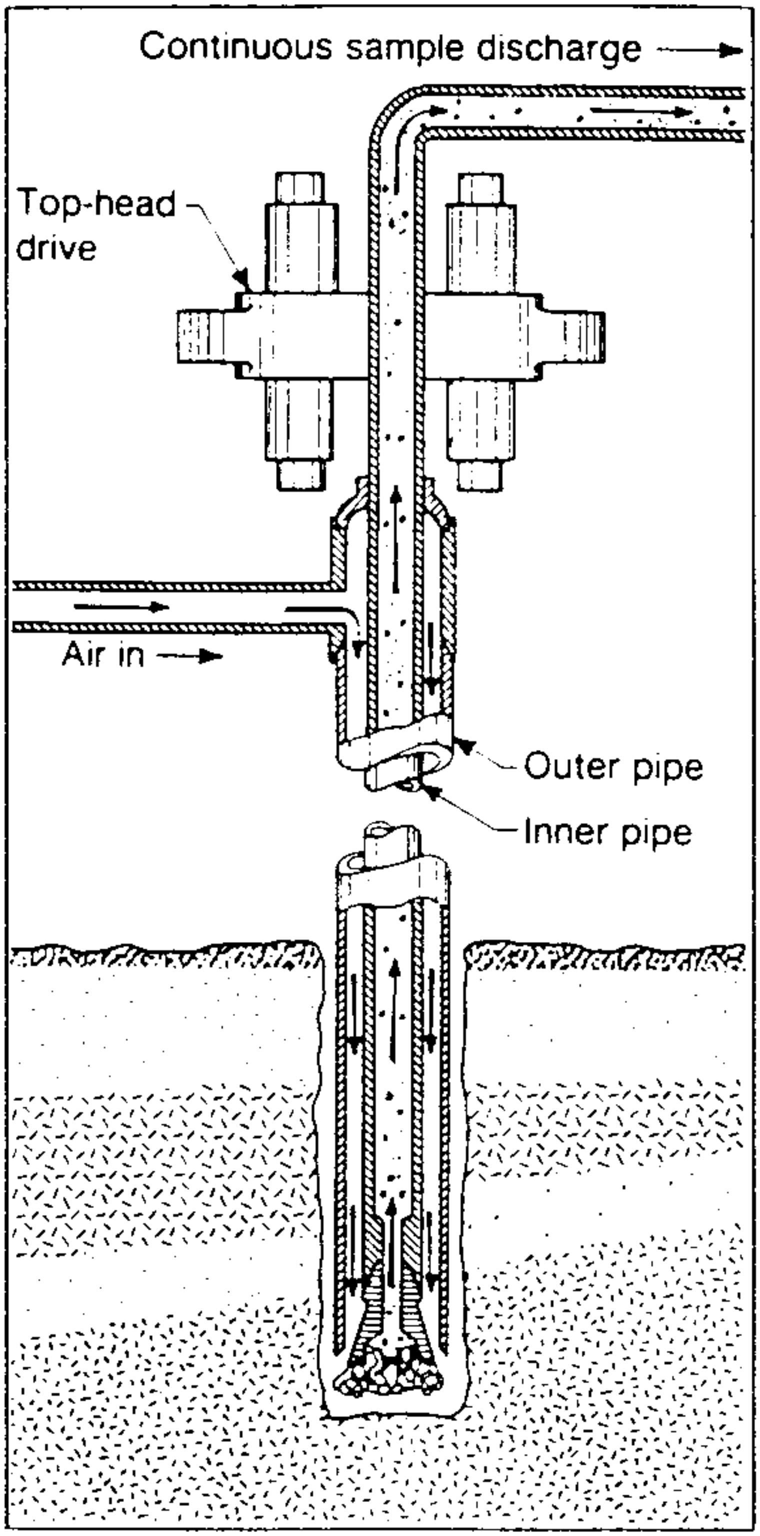


Figure 2-31.—Dual-wall reverse-circulation method.

boulder-size material and for recovering uncontaminated disturbed samples of sand, gravel, and cobble-size material.

Another advantage of a reverse-circulation percussion drill and dual-wall drive pipe system is that the drive pipe can be used as a temporary casing through coarse aggregate deposits. Smaller drills then can be set over the casing to conduct coring operations, perform in-place tests, or install subsurface instrumentation systems.

A special version of a reverse-circulation drill known as the "Becker" drill uses a double-acting diesel hammer. Research is being performed to obtain penetration resistance test data using this drill to evaluate loose or dense conditions in gravels. Reclamation has used these drills on several dam investigations for evaluating the penetration resistance of gravels.

6. Top-Head Drive With Percussion Casing Hammer Drills.—This drill is essentially the same as the conventional top-head drive drill previously described except it is equipped with an automatic casing hammer. Addition of an automatic casing driver allows equipment to be used to simultaneously advance casing during rotary drilling operations. This is especially advantageous when drilling through materials susceptible to caving or squeezing such as sand-cobble-boulder strata, saturated sands, and soft saturated silts and clays.

Automatic casing drivers are designed for use only with top-head drive rotary drills. The casing driver has a circular opening through the center of the driver assembly so drill rods can rotate inside the casing. This permits simultaneous drilling advancement with casing advancement. As the casing driver lowers during percussion driving of the casing, the drillhead also lowers to ream a pilot hole for the casing drive shoe and cuttings are removed from within the casing; see figure 2-32 [34].

The compressed-air-powered driving ram is designed to impact the casing drive anvil with a driving energy ranging from 1750 N-m (1,300 lbf) for the smaller drivers, to 10000 N-m (7400 lbf) for the larger drivers. Circulation media for removing cuttings is compressed air or air-foam. Cuttings travel upward through the casing to a discharge spout that is a component of the casing driver.

Casing can be removed using the casing driver to drive upward for impact against a pulling bar anvil positioned in the top of the driver assembly. The bottom of the pulling bar, opposite the upward-drive anvil, is connected to an adaptor "sub" for attachment to each section of casing.

Casing driver systems are very efficient in terms of production. Production rates of up to 60 m (200 ft) per day can be realized. They are very advantageous for instrument installations where the goal of the program is to rapidly produce a boring, and sampling or testing can be performed along the way through the casing. Some casing driver systems are equipped with underreaming downhole hammers. The hammer has an eccenetric bit that can cut a borehole slightly larger than the casing; and, in some cases, the casing can be dropped under its own weight. These systems are useful in deposits containing cobbles and boulders and are often used for drilling rockfill sections of embankments.

Casing drivers can be operated using several methods.

The casing advancer has an advantage that the casing maintains an open hole and prevents caving and possible blocking of circulation which could result in fracturing problems. One method to reduce the potential for hydraulic fracturing of embankments consists of using a rotary rock bit which is kept inside of the casing so that a small soil plug in the end reduces the possibility of fracturing (see section 2-12 for drilling methods in existing dams). This technique is used when drilling impervious zones of dams. When cobble, boulder materials, or bedrock is encountered, the bit or a downhole hammer must be extended past the casing.

After backfill is placed to a height of about 9 m (30 ft) above the hole bottom, 3 to 9 m (10 to 20 ft) of casing is removed followed by continuation of backfilling. This procedure leaves the upper part of the backfill within the casing at all times to prevent any caved material from damaging the instrument or contaminating the backfill. The percussive blows of the casing driver contribute to consolidation of backfill material by vibrating the casing during removal.

The action of the casing driver can be similar to displacement piles if the bit is withdrawn inside the casing during hole advance. Some soil is displaced laterally as the casing is advanced, and corresponding vibrations may cause densification of surrounding soils. The zone of influence is assumed to be small (300 to 900 mm [1 to 3 ft]) when adequate precautions are taken, but influence may affect testing between closely spaced boreholes (i.e., crosshole shear wave velocity which is normally performed at 3-m [10-ft] spacings). Densification effects may be more pronounced in loose cohesionless soils.

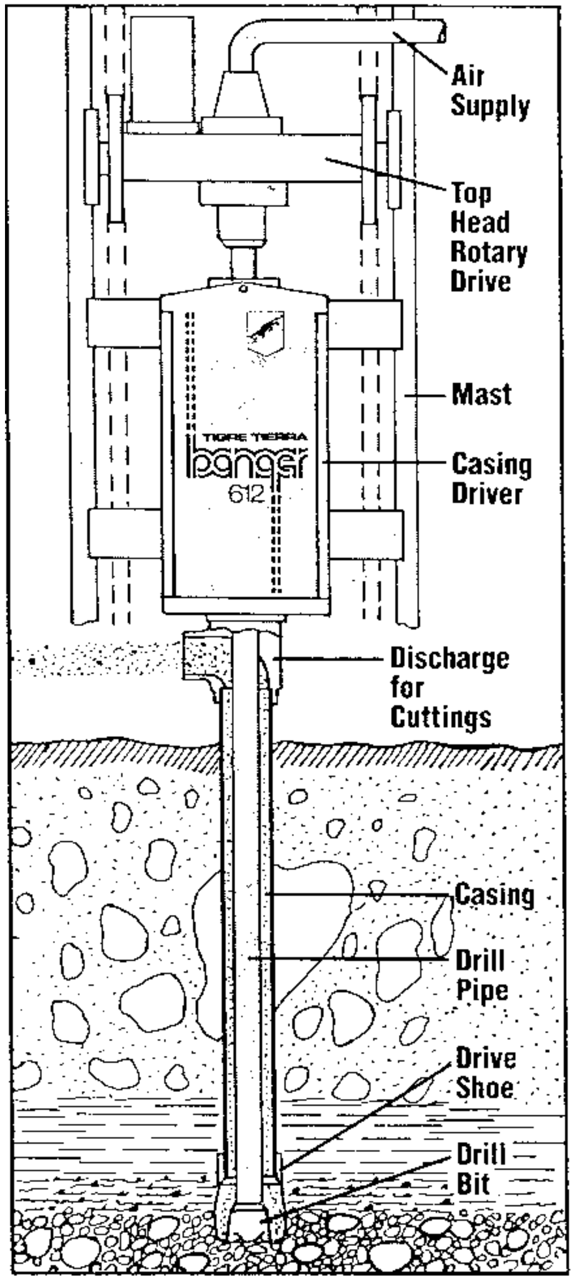


Figure 2-32.—Casing drivers can be fitted to top-head drive rotary rigs to simultaneously drill and drive casing. (Aardvark Corporation)

7. Horizontal Rotary Drills.—Special self-propelled horizontal rotary drills are used to install perforated or slotted pipe drains to stabilize water-saturated landslide areas. The success of this innovative idea resulted in development of specialized drilling equipment and slotted polyvinylchloride drainpipe. Horizontal rotary drills are crawler tractor-mounted for all-terrain mobility and are designed with proper mass distribution for stability to provide the required horizontal thrust. The track carrier power unit provides mechanical tracking power for the tractor and for total hydraulic power for the drill unit. The rotary drillhead is positioned on a box-beam slide attached to the side of the tractor. The slide can be positioned by hydraulic cylinders to drill at any angle from vertical downward to 45° above horizontal. Drilling is continuous over a 3-m travel length of the drillhead on a smooth plane surface of the beam slide. Forward thrust and retract of the drillhead is hydraulically controlled through combined use of a hydraulic ram, equipped with wire rope sheave wheels, and a cable (wire rope) attached to the drillhead. Drilling is accomplished using a custom-size drill rod, 57 i.d. by 76-mm o.d. (2-1/4 by 3 in), or 114 i.d. by 127-m o.d. (4-1/2 by 5 in). The smaller rod is used to install 50-mm (2-in) diameter slotted polyvinylchloride drain pipe; the larger rod is used to install up to 100-mm- (4-in-) diameter drain pipe, piezometers, or inclinometer casing. Special carbide tipped drag bits or tricone bits are locked to a drill sub on the lead rod that is manufactured with two L-shaped slots milled into opposite sidewalls of the drill subbody. The bit shank (threaded tool joint connection of the bit body) is welded to a tubular steel sleeve that is milled with an inside diameter slightly larger than the o.d. of the L-slotted drill sub. A hardened steel pin is welded across the inside diameter of the bit sleeve for locking into the L slots of the sub. The bit is attached for drilling by pushing the bit sleeve over the drill sub to bottom contact of the hardened pin into the L slot and locked by one-quarter turn in the opposite direction of the drilling rotation. Figure 2-33 shows an Aardvark™ model 500 horizontal drill in operation.

Figure 2-33.—Horizontal rotary drill. Aardvark model 500 drill with adjustable box-beam slide, crawler-tractor mounted.

Drilling for drain installations, such as landslides, is performed using water as the circulation media to remove cuttings. Horizontal or angle drilling into slide zones is generally a high-production operation [average drilling penetration rate is 2.5 to 3 m/min (8 to 10 ft/min)], primarily because of the saturated and loose condition of the material. Holes can be drilled to 245-m (800-ft) depths (horizontally or nearly so) using a 114-mm (4-1/2-in) bit for the 76-mm- (3-in-) o.d. drill rod, and to 150-m (500-ft) depths using a 165-mm (6-1/2-in) bit for the 127-mm- (5-in-) o.d. drill rod. Drain installations are commonly drilled in a fan pattern through the slide material or wet area.

After the hole is completed to the designed depth, the drill head is unthreaded from the drill rod, and slotted polyvinylchloride drain pipe is installed within the drill rod to contact with the drill bit at the end of the hole. A one-way check valve assembly, positioned behind the discharge ports of the bit, inhibits entrance of ground water or drill cuttings into the rod during drain pipe installation. Drain pipe installation into the drill rod is measured to equal total hole depth plus 1 m (3 ft) to ensure the water discharge point is outside the hole collar. The drill head is power threaded onto the drill rod containing the slotted drain pipe, and an additional 300 to 460 mm (1 to 1-1/2 ft) of drilling penetration is made without using circulation media. This operation forces dry cuttings to plug and seize the drill bit so that it can be detached from the drill rod. After the dry drilling, water is pumped into the drill rod to about 2070 kPa (300 lbf/in^2) pressure behind the plugged bit. A reverse rotation on the drill rod unlocks the expendable bit from the L-slot drill sub. This is followed by a rapid (high power) pullback on the drill rod while monitoring pump pressure for indication of a sudden pressure drop. The pressure drop confirms bit drop off, which is immediately followed by rapid withdrawal of the drill rods. The bit cost is insignificant when compared to cost of removing all drill rods, saving the bit, and attempting to install drain pipe into a hole that has collapsed. As the rods are withdrawn, the drain pipe is maintained in the hole (against the expendable bit) by continuing to inject water against a floating piston device seated against the outlet end of the drain pipe. This floating piston maintains pressure on the drain pipe to prevent withdrawal of the drain during rod removal. After all rods are removed, the drain discharge is plumbed into a manifold pipe assembly and conduit to direct the water away from the slide zone or wet slope area.

In addition to drilling for drain installations, horizontal rotary drills have proved extremely efficient and effective for use in performing other types of subsurface work discussed below.

c. Core Drilling for Tunnel Alignment Geology.—A river diversion tunnel alignment at Reclamation's Buttes Dam, Arizona, was horizontally core drilled to a depth of 283 m (927 ft) using a horizontal rotary drill and NWD-3 core-barrel assembly. Core recovery was 98.9 percent. Production rate was good at an average of 8 m (26 ft) per workshift; however, the addition of a pump-in wire-line core barrel would have had the potential to triple conventional core-barrel production.

d. Slope Inclinometer Casing Installation.—Using a horizontal rotary drill is a productive and efficient method for drilling, installing, and grouting inclinometer casing in place. The drill can be track-walked under its own power to difficult access sites. The inclinometer hole can be drilled with a 158-mm (6-1/4-in) expendable bit and a 108-mm- (4-1/4-in-) i.d. drill rod. The inclinometer casing can be installed to the hole bottom through the large-diameter drill rod. After releasing the expendable bit, the annulus between the hole wall and inclinometer casing can be grouted by pumping grout through the drill rod. When grout fills to the hole collar, drill rod can be removed from the hole to complete the inclinometer casing installation. After completion, a water-injection pipe should be lowered to the bottom of the hole inside the inclinometer casing, and clean water should be circulated to remove any grout that may have entered through the casing joints.

e. Piezometer Installation.—The drilling and installation procedure is the same as that described for an inclinometer casing. However, the backfilling procedure is modified to be compatible with the type of backfill material used. Generally, a uniformly graded clean sand is placed around the piezometer tip or to a specified height above the slot openings of a well screen. This can be accomplished by placing in the drill rod a measured volume of backfill material 28 to 57 liters (1 to 2 ft^3) greater than the volume required to fill the hole after removal of a single drill rod. Then, the drill head is threaded onto the collar rod, and one rod is removed while rotating slowly while clean water is simultaneously pumped to force the backfill out of the rod. This procedure leaves 300 to 600 mm (1 to 2 ft) of material in the bottom rod and protects the piezometer from an open hole condition and possible caving. The backfill and rod removal procedure is repeated in like increments to completion of the hole.

f. Settlement-Plate Monitoring Systems.—Reclamation's Choke Canyon Dam, Texas, was constructed with 0.84-m^2 (1-yd^2) steel settlement plates embedded at the interface between the embankment and compacted overburden material just below the embankment. After completion of embankment construction, a horizontal rotary drill was set on the 3:1 (horizontal : vertical) downstream slope face to drill and install a steel reinforcement measurement rod to contact on the plate for survey monitoring of embankment settlement. Drilling was conducted using 76-mm- (3-in-) o.d. rod and a 108-mm (4-1/4-in) drag bit with water circulation media. The plates were located at six separate stations along the embankment at an average depth of 43 m (140 ft). After the bit contacted each plate, the rods were pulled and the bit removed.

The second drill phase was conducted with an open drill sub on the lead rod to contact the steel plate. The 50-mm- (2-in-) diameter casing pipe was lowered through the drill pipe to plate contact. A bentonite seal was injected to the bottom of the hole during removal of a 3-m (10-ft) rod section. The bentonite was used to seal the casing to inhibit grout intrusion. The installation was completed by filling the annulus between the casing and hole wall with grout from the top of the bentonite seal to the hole collar. After removal of all drill rods from the hole and initial grout set, a reinforcement steel rod was installed through the casing to plate contact. The top of the steel rod is a survey point to monitor embankment settlement.

g. Churn/Cable-Tool Drills.—Although incapable of performing rotary drilling operations, the churn drill or cable tool is sometimes used in lieu of, or in combination with, rotary or core drills. The churn/cable-tool drilling procedure is one of the oldest known methods of boring holes and continues to be a favored method to drill water wells. Drilling is performed by raising and dropping a heavy string of tools tipped by a blunt-edge chisel bit. The tools are attached to a steel cable that is alternately raised and released for free fall by a powered drum assembly. The cable is suspended from a sheave assembly mounted on an oscillating beam that absorbs the shock load created by quick load release on the taut cable upon impact of the drill tools in the bottom of the hole. The impact of the blunt-edge chisel pulverizes soil and rock material, and the borehole is advanced. The cuttings are suspended in a slurry injected into the borehole. After each 3- to 6-m (10- to 20-ft) penetration, the cable tools are hoisted out of the hole, and a cylindrical bailer equipped with a bottom check valve is lowered into the hole to remove the slurry. This process is repeated to total hole depth.

A sampling barrel also can be attached in place of the blunt-edge chisel bit. In this mode, the churn/cable-tool drill can be used to sample and to advance the hole without using water, resulting in a muddy hole. The sampler mode has been used to advantage in sampling glacial terrains where great thicknesses of heterogeneous surficial deposits overlie bedrock. The sampler mode of churn/cable-tool drilling has also been used to advantage for sampling and instrumenting dam embankments.

The churn/cable-tool drill is sometimes used to drill and then drive casing pipe through cobble-laden or fractured material so core drilling of deeper formation material can be done with diamond-core drills. When vertical hole alignment is critical, the churn/cable-tool drilling method is very effective. The churn/cable-tool drill was used successfully by the petroleum industry to drill 380-mm- (15-in-) diameter holes to depths of 2,000 m (7,000 ft). Simplicity of equipment makes churn/cable-tool drilling operations one of the least expensive methods for boring holes.

h. Rock Core Drilling.—Rock core drilling is accomplished with mechanical, engine-powered rotary drills designed to drill rock and to recover cylindrical cores of rock material. Most core drilling equipment is designed with gear or hydraulically driven variable-speed hollow-spindle rotary drill heads (fig. 2-34). Average core-diameter capability of these drills ranges from 19 to 85 mm (3/4 to 3-3/8 in) and to 300-m (1,000-ft) depths. Larger-diameter coring operations [100 to 150 mm (4 to 6 in)] are usually performed using rotary drills, and cores to 1.8 m (6 ft) in diameter can be drilled and recovered using a shot/calyx drill.

A general misconception is that for coring operations with diamond core bits to be efficient, drilling must be performed at the highest rotary speed—regardless of core size. However, this operational procedure usually results only in shortened bit life, poor penetration rate, and excessive vibration that results in broken cores or core blockage. Diamond drill manufacturer’s literature serves as an excellent guide for selecting bit styles and evaluating bit wear [32, 35, 36].

Diamond-core drilling can be compared to using drill presses or center-bore lathes in a machine shop. A small-diameter drill bit must be rotated at high speed with minimum pressure applied to the bit, while a large diameter drill bit must be rotated at a low rate of speed with significant pressure on the bit. Any variation from this procedure results in bit chatter, dulled drill bits, and poor penetration rate. The same is true for a core drilling operation. Rotational speed and "crowd" pressure must be compatible with type and hardness of rock being drilled to achieve a smooth and steady rate of penetration throughout the core length. Any variation results in loss of extremely expensive core bits, poor production, and poor quality core recovery.

All core drills are equipped with pumps or compressors to circulate drill media through use of water, drilling mud, air, or air-foam to cool and lubricate the coring bits and to transport the drill cuttings to the top of the hole. Most core

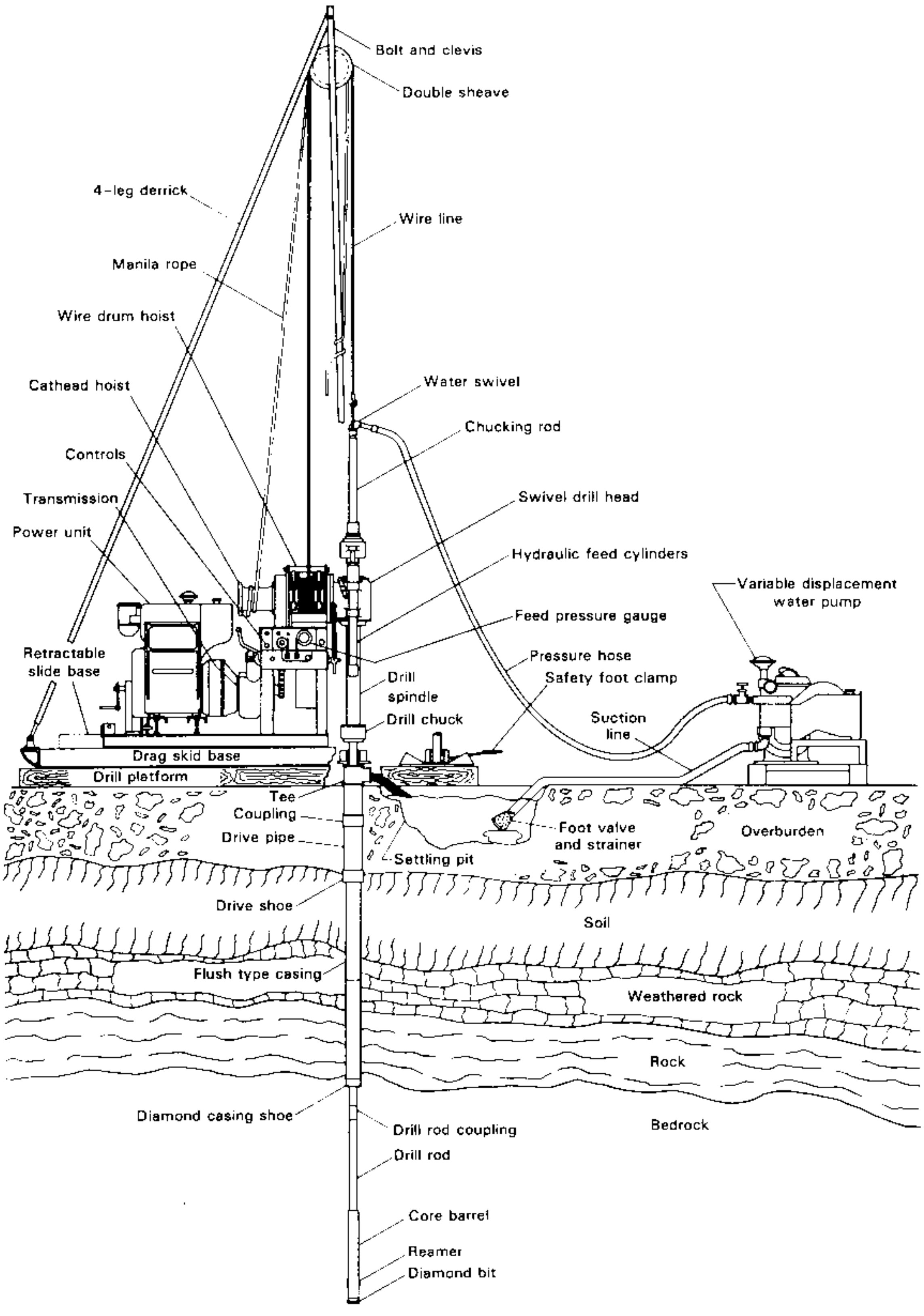


Figure 2-34.—Typical diamond drilling rig for exploration. (Acker Drill Co., Inc.)

drills are equipped with a mast assembly, powered hoist assembly for hoisting heavy loads, and, sometimes, a wire-line hoist assembly for hoisting or lowering a wire-line core barrel through the drill rods.

Although some core rigs have been manufactured with gear or chain pulldown/retract systems, precise control over bit pressure can best be accomplished with a hydraulic pulldown/retract system. The hydraulic system must have a precision regulator control so desired pressure can be set and maintained on the bit. Deep-hole rigs should be equipped with a hydraulic holdback control so the full weight of the drill tools is not exerted on the drill bit.

Many variations are available in design and mountings for drill rigs manufactured specifically for corin; however, there are only two basic types. They are conventional or wire-line core drills, for drilling and recovering cores up to 150 mm (6 in) in diameter, and shot/calyx core drills for drilling, and recovering cores to 1.8 m (6 ft) in diameter.

1. Conventional and Wire-line Core Drills.—Conventional and wire-line core drills are capable of high-speed rotary core drilling (up to 1,800 rpm) for recovery of relatively small-diameter cores ranging from 19 to 150 mm (3/4 to 6 in) in diameter; however, wire-line core recovery is limited to 86 mm (3-3/8 in) in diameter.

Conventional core drilling is performed using standard rotary drill rods and a core barrel. After each core run, all rods and core barrel must be removed from the hole to recover the core. A wire-line core drill uses large inside-diameter drill rods through which an inner-core barrel assembly is lowered by wire-line cable and locked into a latch mechanism in the lead rod. After each core run, an "overshot" tool is lowered by wire-line to unlock and retrieve the inner-barrel assembly for core recovery.

Conventional core drilling is usually limited to relatively shallow coring depths or when intermittent core runs are separated by intervals of hole advancement by rock bitting. However, the nonrecovery advancement of boreholes between coring intervals also can be achieved with a wire-line system by removing the inner core barrel and lowering a rock bit—designed with a wire-line latching mechanism—into the wire-line drill rod.

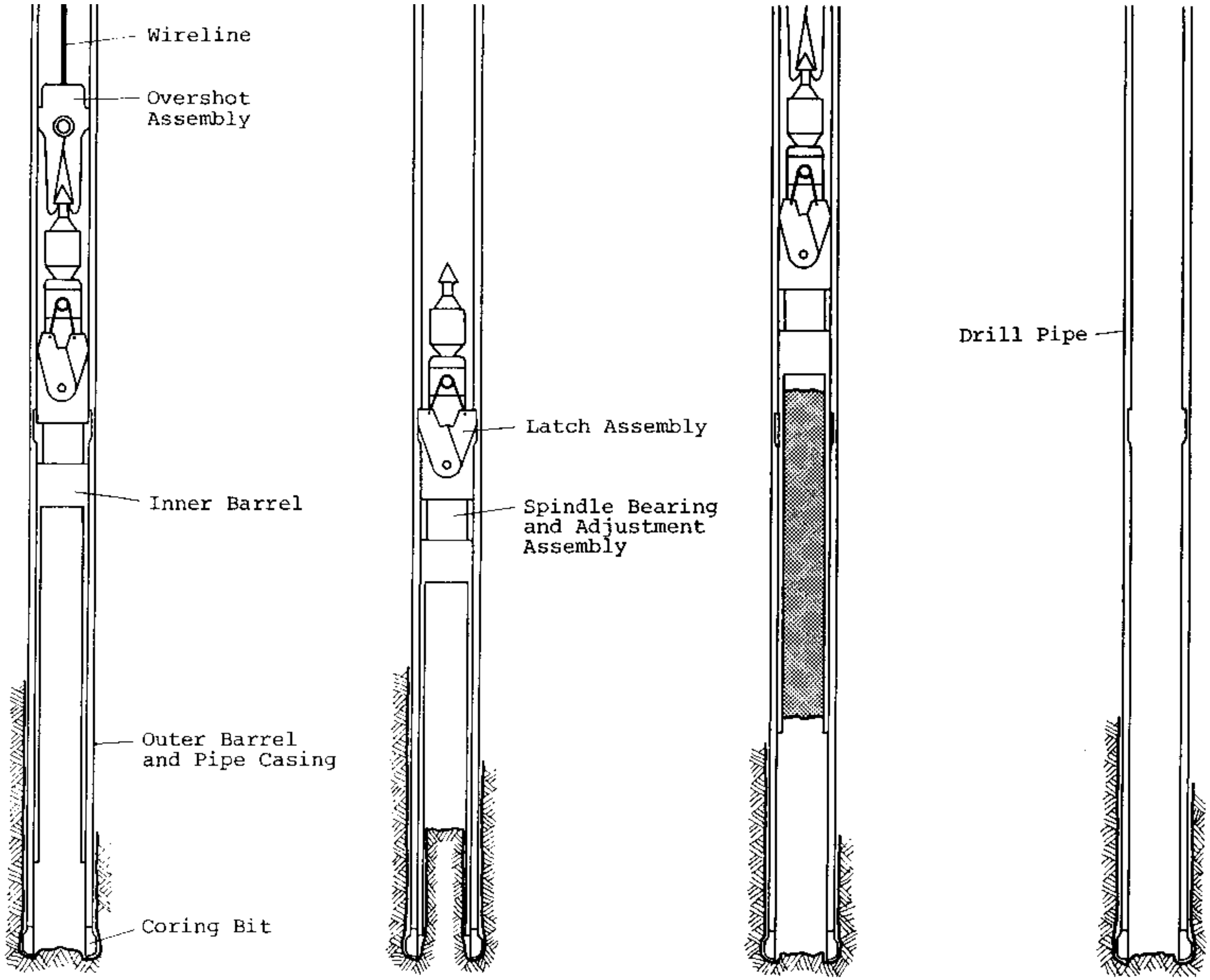
Wire-line equipment is especially valuable in deep hole drilling since the method eliminates trips in and out of the hole with coring equipment. Figure 2-35 illustrates the working components of the wire-line core barrel. With the wire-line technique, the core barrel becomes an integral part of the drill rod string. The drill rod serves as both coring device and casing, and usually is not removed except when making bit changes. Core samples are retrieved by removing the inner-barrel assembly from the core barrel through the drill rod. This is accomplished by lowering an overshot or retriever, by wire-line, through the drill rod to release a locking mechanism built into the inner-barrel head. The inner barrel is brought to the surface; the core is removed, the inner barrel is returned to the bottom of the hole through the drill rod, and coring is continued.

Other advantages of wire-line core drilling over conventional core drilling are:

- Production — Wire-line core drilling is three to four times faster.
- Hole Protection — The larger drill rod functions as a casing to protect the hole from caving material or squeezing zones at all times.
- Drilling Stabilization — The wire-line drill rod helps to eliminate rod vibration and rotational whipping action by minimizing the open hole annulus between the outside of the rod and the hole wall.
- Extended Bit Life — The only time wire-line rods must be removed from a core hole is to replace a worn core bit. Rod trips in and out of a core hole, as with conventional core drilling operations, reduce bit life because the o.d. gauge stones (diamonds) on the bit are in contact with abrasive rock formations during rod "tripping" operations. This is especially true during angle or horizontal hole conventional coring operations. In addition, removal of rods from the hole may cause rock fragments to loosen and fall or wedge in the hole. As a result, reaming through the fallout material is necessary while the rods are lowered to the hole bottom.
- Water Permeability Testing — Water testing through a wire-line rod can be accomplished by hoisting the rod above the test interval, then lowering a wire-line packer unit through the bit for expansion and seal against the hole wall. Conventional core drill operations would require removal of all rods and core barrel before setting the packer at the zone to be tested.

Some core drills are designed with angle-drilling capabilities, including up-hole drilling with underground drills used in the tunneling and mining industry. Angle-hole drills are generally small and can be quickly disassembled for moving by helicopter or other means into areas of rough terrain. Core drills can be mounted on motorized carriers, trailers, skids, or stiff-leg columns for underground operations.

Core drills have limited capability for drilling through gravels, cobbles, or any surficial material that requires



a) INNER BARREL BEING LOWERED INTO POSITION

b) OBTAINING CORE, OUTER BARREL ROTATING, INNER BARREL STATIONARY

c) LIFTING INNER BARREL AND CORE OUT OF HOLE

d) OUTER BARREL AND CORING BIT REMAIN IN HOLE

Figure 2-35.—Wire-line core barrel operation.

significant rotary torque. Generally, casing must be set through surficial materials to preclude hole caving and loss of drill fluid circulation. Core drill depth capabilities are limited mainly by hoisting capacity of the mast and draw works and by the ability to maintain a clean hole free of cuttings.

2. Shot or Calyx Drills.—A shot drill, also called a calyx drill, is a large rotary drill used primarily in large-diameter (0.1 to 2 m [4 to 6 ft]) rock or concrete core drilling operations. After development and use of industrial diamond-core bits, the shot or calyx drill became obsolete in the United States, but still is used in some European and Asian countries. The primary differences between a shot/calyx drill and rotary core drills previously discussed are the tools and methods used to perform core drilling operations. Coring is performed using a coring bit that is a flat-face steel cylinder with one or two diagonal slots cut in the bottom edge. As the bit and core barrel are rotated, small quantities of hardened steel shot (also called adamantine shot, buckshot, chilled shot, or corundum shot) are fed at intervals into the drill-rod water injection system. The water circulation media flows through the core barrel around the bit face for cooling and return circulation of cuttings, leaving the heavier steel shot on the hole bottom. The rotating core barrel creates a vortex at the bit, which results in movement of steel shot under the flat face of the bit. As the core bit rotates, the steel shot aids in coring penetration by abrasive cutting action on the rock.

A steel tube called a calyx barrel is attached to the upper (head) end of the core barrel. The o.d. of the calyx barrel is the same as that of the core barrel; the calyx barrel serves as a stabilizing guide rod for the core barrel. The top end of the calyx barrel is open except for a steel yoke welded across the inside diameter of the barrel to a steel ring encircling the drill rod. In addition to functioning as a stabilizer for the core barrel, the calyx barrel functions as a bucket to catch and contain drill cuttings too heavy for circulation out of the hole by drill water. Cores are recovered by hoisting all rods and the core barrel out of the hole using a cable draw-works system.

Depth limitation for a shot/calyx drill depends on the mast and draw works hoist capacity and on capability to maintain a clean open hole. Although smaller-diameter cores can be drilled with a shot/calyx drill, costs would be much higher than diamond drilling. Only on jobs where large-diameter [1 to 2 m (3 to 6 ft)] cores are required would efficiency and price be comparable for diamond and for shot drilling.

i. Planning a Drilling Program.—Planning is critical to the success of a rotary drilling program. *Engineering Geology Field Manual* [20] provides guidance on planning a drill program. An investigation program with drilling operations program can be a major cost, and proper planning is required to ensure that the program can be performed without cost overruns. In planning a drill program, important areas to consider include:

- preplanning,
- site visit,
- topography and accessibility,
- protection of the environment,
- drilling concerns,
- equipment concerns,
- traffic control and safety plans,
- buried and overhead utilities,
- special considerations, and
- preparation of drilling specifications.

2-12. Procedures for Drilling in Embankment Dams.—Concern exists throughout the geotechnical community regarding drilling in embankment dams and the potential for hydraulic fracturing of the impervious core during drilling. This concern has prompted an evaluation of conditions and drilling methods which pose the greatest potential for hydraulic fracturing. The following procedures should be followed when drilling in the impervious portions of embankment dams.

a. Conditions Conducive to Hydraulic Fracturing.—Reclamation's embankment design and construction practices, historically as well as currently, minimizes development of stress patterns within an embankment caused by drill fluids during drilling. However, certain embankment locations and conditions have a higher potential for hydraulic fracturing than others, and improper drilling procedures or methods increase the potential for fracturing. Site locations and conditions where hydraulic fracturing by the drilling media are most likely to occur include the following:

- Impervious cores with slopes steeper than 0.5H:IV, within cutoff trenches, and upstream inclined.
- Near abutments steeper than 0.5H:IV; where abrupt changes in slopes occur; or above boundaries in the foundation which sharply separate areas of contrasting compressibility.
- Near rigid structures within embankments.
- Impervious zones consisting of silt and mixtures of fine sand and silt.

b. Recommended Procedures.—Recommended procedures for developing exploration and instrumentation programs and for drilling in the impervious portion of embankment dams are as follows:

- A site-specific determination as to whether hydraulic fracturing potential exists should be made by exploration team.
- If a potential for hydrofracturing exists, the type of equipment and the method and technique proposed to be used must have the approval of the exploration team. Once drilling has commenced, drilling personnel are responsible for controlling and monitoring drill media pressure, drill media circulation loss, and penetration rate to assure that the drilling operation minimizes the potential for hydraulic fracturing.
- If a sudden loss of drill fluid occurs during any embankment drilling in the impervious core, drilling should be stopped immediately. The reason for the loss should be determined; and if hydraulic fracturing may have been the reason for the fluid loss, the Principal Designer and Principal Geologist should be notified. Action should be taken to stop the loss of drill fluid.

c. Acceptable Drilling Methods.—Based on the evaluation of the various drilling methods, with the exception of augering, any drilling methods have the potential to hydraulically fracture an embankment if care and attention to detail are not taken.

Augering is the preferred method of drilling in the core of embankment dams. Augering does not pressurize the embankment, and no potential for hydrofracturing exists. Use of a hollow-stem auger permits sampling of the embankment and the foundation through the hollow stem with the auger acting as casing.

With proper planning, the following drilling methods may be approved for drilling in embankment dams if augering is not practical:

- cable tool
- mud rotary (bentonite/biodegradable)
- drilling with water
- air-foam rotary
- reverse-circulation percussion/rotary
- rotary percussion

Selection of any one of the above methods should be based on site-specific conditions, hole utilization, and availability of equipment and trained personnel. Any drilling into the impervious core of an embankment dam should be performed by experienced drill crews that employ methods and procedures that minimize the potential for hydraulic fracturing. Therefore, it is essential that drillers be well trained and aware of the causes of and the problems resulting from hydraulic fracturing.

Because hydraulic fracturing can be induced when in-place horizontal stress and tensile strength in the embankment material are less than fluid or gas pressure, general practice is to limit downhole pressures of circulation media to 1 to 2 kPa/m [½ to 1 (lbf/in^2)/ft] of depth of drill hole drilling operations. Advance rates should be slow to ensure that blocking of the bit or barrel does not occur. Drill rod and hole diameter should be selected to ensure appropriate annulus for efficient cuttings removal. Circulation pressures should be monitored continuously during the drilling process.

Rotary percussion and reverse-circulation drills can induce fracture by air pressure. General precautions to reduce fracture potential include reducing air pressures and maintaining lead distance of casing shoe well in advance of the downhole hammer or inner casing. Rotary percussion and reverse-circulation drilling can normally circulate cuttings efficiently with air pressures of 100 to 200 kPa (15 to 30 lbf/in^2).

In cases where steep abutment contacts are encountered, the static weight of fluid column alone may be sufficient to induce hydraulic fracture without any excess hydrostatic pressure. In these cases, the only successful method for advancing a drill hole without fracturing is to incrementally drive casing and perform cleanout inside the casing, while leaving a sufficient plug of soil in the casing to prevent exposure of drill media with the consequent transmission of excess pressure to the embankment.

C. Sampling and Testing Methods

2-13. General.—Sampling serves many purposes when investigating foundations and evaluating construction materials for water resources structures. Samples are required to accurately identify and classify soil or rock. Samples are essential for obtaining in-place density and moisture content, for performing laboratory tests on earth and rock materials, for testing potential concrete sand and aggregate deposits, for designing concrete mixes, and for testing potential riprap sources. Data obtained from laboratory testing of samples are used to finalize the design of foundations and embankments and to select construction materials for use in earth and concrete dams and in other structures.

The importance of obtaining representative samples cannot be overemphasized. Samples that are not truly representative of in-place subsurface conditions can result in erroneous conclusions that affect the design of the structure. Sample recovery requires considerable care to avoid altering in-place conditions of natural deposits. Obtaining representative samples from accessible trenches, test pits, or tunnels is relatively easy because in-place material can be visually inspected to determine the best method of sampling by hand. However, in boreholes, visually inspecting in-place material is not possible; consequently, the recovery of representative samples is more difficult.

Samples are broadly classified as either disturbed or undisturbed. Disturbed samples do not reflect the in-place condition of the soil or rock. Obtaining undisturbed samples requires significant experience and meticulous care to maintain in-place material conditions. Even using the most careful procedures, undisturbed soil or rock samples are changed from their in-place condition because removing them from parent material changes stresses which confine the sample.

Both hand and mechanical sampling methods commonly used to recover disturbed and undisturbed subsurface samples are described in the following paragraphs.

2-14. Hand Sampling Methods for Obtaining Disturbed Samples.—*Disturbed Samples* (hand-sampling methods) are normally used to obtain samples from accessible excavations, from existing stockpiles and windrows, or from shallow hand-auger borings.

a. Accessible Test Pits, Trenches, and Large-Diameter Borings.—Using methods described in USBR 7000, obtaining disturbed samples from accessible test pits or trenches (including road cut and river bank deposits) can be accomplished in the following manner:

- An area of sidewall of the test pit, trench, large-diameter boring, or open cut should be trimmed to remove all weathered or mixed material.
- The exposed strata should be examined for changes in gradation, natural water content, plasticity, uniformity, etc., and a representative area should be selected for sampling.
- Sketches and photographs showing changes in strata, geologic descriptions, and sampling locations should be recorded.
- Either individual or composite samples can be obtained by cutting a sampling trenching down the vertical face of a test pit, trench, or cut bank with a sampling cut of uniform cross section and depth.
- The soil can be collected on a quartering cloth spread below the sampling trench.
- The minimum cross section of the sampling trench should be at least four times the dimension of the largest gravel size included in the soil.

In obtaining individual samples, it is important that an adequate sample of representative material be obtained only from the stratum of interest and that extraneous material is not included. For composite samples, a vertical sampling trench is cut through all strata of interest.

If the material sampled is a gravelly soil that contains large percentages (about 25 percent or more of total material) of particles 75 mm (3 in) in diameter or larger, it is usually appropriate to take representative parts of the excavated material (such as every 5th or 10th bucketful) rather than to trim the sample from the in-place sidewall of the excavation. In critical investigations, such as for processed aggregates or cohesionless soils, screening all oversize may be necessary to determine volume of oversize cobbles and boulders—while maintaining a constant width of cut.

Size requirements for disturbed samples for various testing purposes are given in USBR 5205 for soils and in USBR 4075 [37] for concrete. The quantity of the field sample depends on the testing that is to be performed and the maximum particle size present in the material. When samples are larger than required for testing, they may be reduced by quartering as described in USBR 5205 or USBR 4075. This is done by piling the total sample in the shape of a cone on a canvas or plastic tarpaulin. Each shovelful should be placed on the center of the cone and allowed to distribute equally in all directions. Then, the material in the cone is spread out in a circular pattern by walking around the pile and gradually widening the circle with a shovel until a uniform thickness of material has been spread across the tarpaulin. The spread sample is then quartered. Two opposite quarters are discarded, and the material in the remaining two quarters is mixed again by shoveling the material into another conical pile, taking alternate shovelfuls from each of the two quarters. The process of piling, spreading, and discarding two quarters is continued until the sample is reduced to the desired size.

b. Stockpiles and Windrows.—When sampling stockpiles or windrows, care must be taken to ensure that samples are not selected from segregated areas. Procedures for sampling stockpiles are given in USBR 4075. The amount of segregation in materials depends on gradation of the material and on methods and equipment used for stockpiling. Even with good control, the outer surface and fringes of a stockpile are likely to be somewhat segregated, particularly if side slopes are steep and the material contains a significant amount of gravel or coarse sand. Representative samples can be obtained from stockpiles by combining and mixing small samples taken from several small test pits or auger holes distributed over the entire pile. A windrow of soil is best sampled by taking all the material from a narrow cut transverse to the longitudinal axis of the windrow. Samples from either stockpiles or windrows should be fairly large originally, and they should be thoroughly mixed before quartering down to the size desired for testing.

c. Hand Auger Borings.—Small auger holes cannot be logged and sampled as accurately as an open trench or a test pit because they are inaccessible for visual inspection of the total profile and for selecting representative strata. Procedures for augering and sampling are discussed in USBR 7010. Small hand augers [100-mm (4-in) diameter or smaller] can be used to collect samples adequate for soil classification and, possibly, for physical properties testing (fig. 2-36). As the auger hole is advanced, soil from the hole should be deposited in individual stockpiles to form an orderly depth sequence of removed material. When preparing an individual sample from an auger hole, consecutive piles of the same type of soil should be combined to form a representative sample. All or equal parts from each of the appropriate stockpiles should be mixed to form the sample of desired size for each stratum (fig. 2-36).

2-15. Mechanical Sampling Methods for Obtaining Disturbed Samples.—*Disturbed Samples* (mechanical sampling methods) using mechanical methods are often obtained from drilled holes; however, samples also are obtained using construction excavation equipment (backhoes, draglines, trenchers, dozers) when they are required primarily for identification or for making volume computations of usable material. Samples obtained with construction equipment are generally unsuitable for use in laboratory testing because of severe mixing of material that occurs during the excavation process. Heavy excavation equipment is best used to excavate an accessible test pit or trench so individual material stratum can be sampled by hand methods to avoid contamination from adjacent materials.

a. Power Auger Drills.—One of the most common methods of obtaining disturbed subsurface samples is by using power auger drills. Continuous-flight auger drilling can be used to obtain disturbed samples of borrow area materials. As the drill hole is advanced, soil cuttings travel up the spiral flight of the auger to the collar of the hole, and soil from selected intervals, or material change, is collected. Soil cuttings are most efficiently transported up flights when explorations are performed in partially saturated strata. When an interval has been reached, auger rotation can be continued without depth advancement until most of the soil is brought to the surface. However, soil cuttings moving upward along the flight can loosen and mix with previously drilled material. If contamination or mixing with other soil material is undesirable, a hollow-stem auger with an internal sampling system should be used.

Disk augers are commonly used to recover disturbed samples of soil and moderately coarse-grained material. After each penetration, the disk is removed from the hole with the disturbed sample cuttings retained on the top of the disk. Then, the sample collection is made at the hole collar followed by repeated drilling intervals.

Bucket augers are suitable for recovering disturbed samples of coarse-grained soils, sands, and gravel deposits. During each drilling interval, sample cuttings enter the cylindrically shaped bucket through the bottom cutter block. Removal and collection of samples are accomplished by hoisting the bucket from the hole and releasing the hinged bottom plate or side of the bucket. Samples of gravels and sands obtained below the water table are normally unreliable because of loss of soil fines through the bucket openings.

b. Reverse-Circulation Drills.—Reverse-circulation drills work well for recovering sand, gravel, and cobble-size disturbed samples. However, this sampling method is relatively expensive and is not used for borrow area investigations. These drills use a double-walled drill stem and compressed air to circulate drill cuttings for collection at the hole collar. Compressed air is pumped down the annulus between the inner and outer walls of the double-walled drill rod, and cuttings are forced upward through the center rod as drilling progresses. Drill cuttings are collected at the discharge spout of a special funnel-shaped cyclone assembly designed to dissipate the energy of the compressed air and deposit cuttings in the order drilled.

This method of disturbed sampling is considered most reliable to produce a noncontaminated sample because the drill stem seals previously drilled material zones. Normally, coarse gravels and cobbles are broken by impact of the double wall casing shoe, but this is easily identified by fresh fracture surfaces. Gradation tests are unreliable if coarse particle fracturing occurs.

c. Protecting and Preparing Disturbed Samples for Shipping.—Disturbed samples of 35 kg (75 lbm) or more should be placed in bags or other suitable containers that

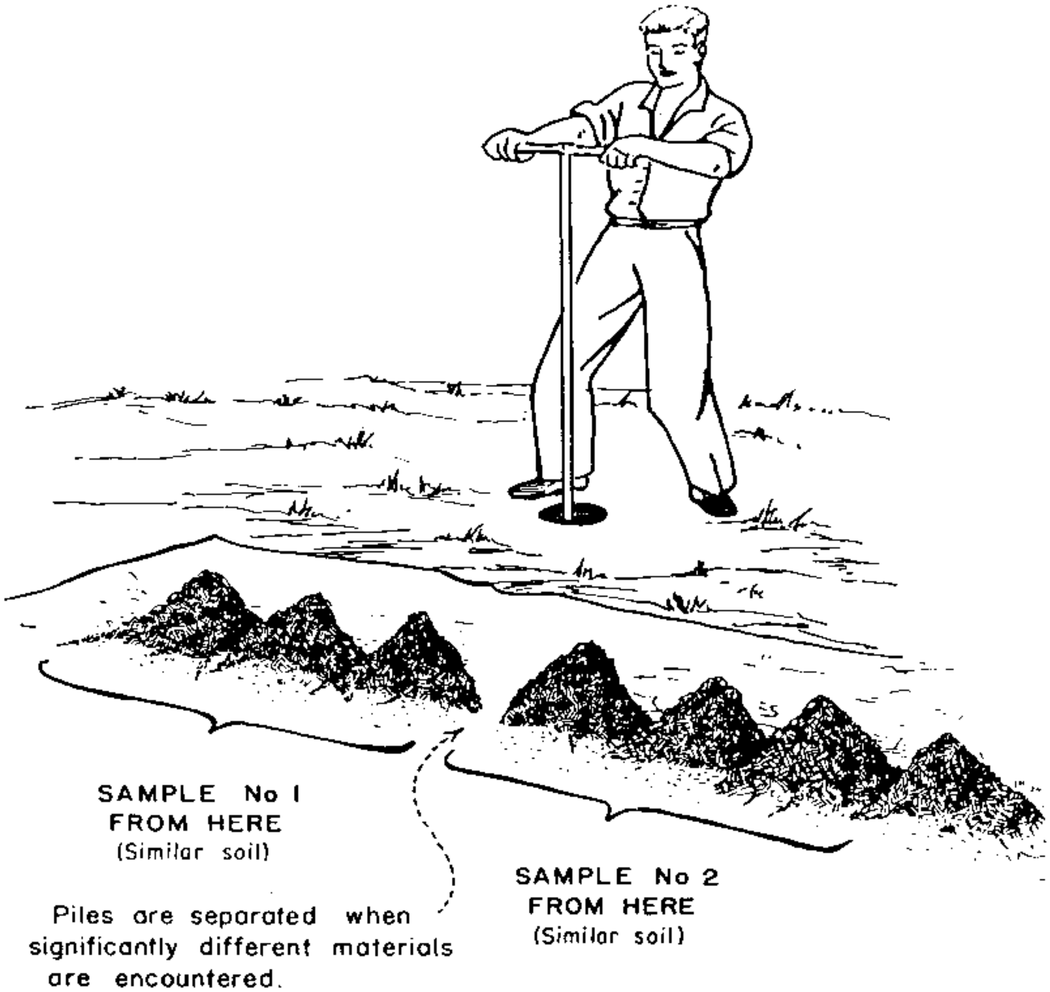


Figure 2-36.—Auger sampling.

prevent loss of moisture or fine fraction from the soil. Asphalt coated burlap sacks with an inner plastic liner are commonly used for protecting and shipping disturbed soil samples. Samples of silty or clayey borrow soils can be allowed some moisture loss as long as studies show no irreversible processes occur from air drying. These soils will be dried in the laboratory for development of compaction curves. The coated burlap sack with plastic liner has proved satisfactory for most borrow studies of silty and clayey soils.

When proposed for use as borrow material, samples of silt and clay for laboratory testing should be protected against drying by placement in waterproof bags or other suitable containers. Sand and gravel samples should be shipped in closely woven bags and should be air dried before they are placed in the bags. When sack samples are shipped by public carrier, they should be double sacked.

2-16. Hand Sampling Methods for Obtaining Undisturbed Samples.—

a. Undisturbed Hand-Sampling Methods.— Undisturbed samples in the form of cubes, cylinders, or irregularly shaped masses can be obtained from strata exposed in the sides or bottoms of open excavations, test pits, trenches, and large-diameter auger holes. Such samples are useful for determining in-place density and moisture content and for other laboratory tests.

Hand cut cylinder samples and block samples provide the highest quality undisturbed samples for laboratory testing and are often preferred in critical studies of weak zones when access is available. Generally, sampling is only possible in unsaturated zones because dewatering can cause increases in effective stress and possible consolidation of sensitive material. Sampling by hand-carved cylinder sample is depicted on figure 2-37. An elevated bench is constructed in a test pit or accessible shaft. A cylinder, sharpened on one end, is placed on a level surface and is pressed into the soil as the sample is trimmed with a knife to a diameter slightly larger than the cutting edge. The incremental process of trimming and pressing is continued until the cylinder is overfilled with soil. The sample bottom can be severed from parent material by spade or shovel and ends trimmed flush the end of the cylinder. This method is effective for soils generally wet of laboratory optimum water content and which contain maximum particle sizes less than coarse sand.

If soils are very fine-grained and wet, cylinders can possibly be pushed the complete depth without trimming in advance of the cutting edge. Several devices such as the U.S. Army Corps of Engineers drive cylinder or the "Elly Volumeter" have been developed. Samples can be extruded from cylinders into laboratory test chambers. Reclamation has obtained large-diameter cylinders [200 to 300 mm (8 to 12 in)] for testing by pushing into compacted fills. Cylinders were pushed with drawbars of bulldozers or buckets of front end loaders. This sampling method may be particularly effective for compacted clay liners placed wet of optimum water content.

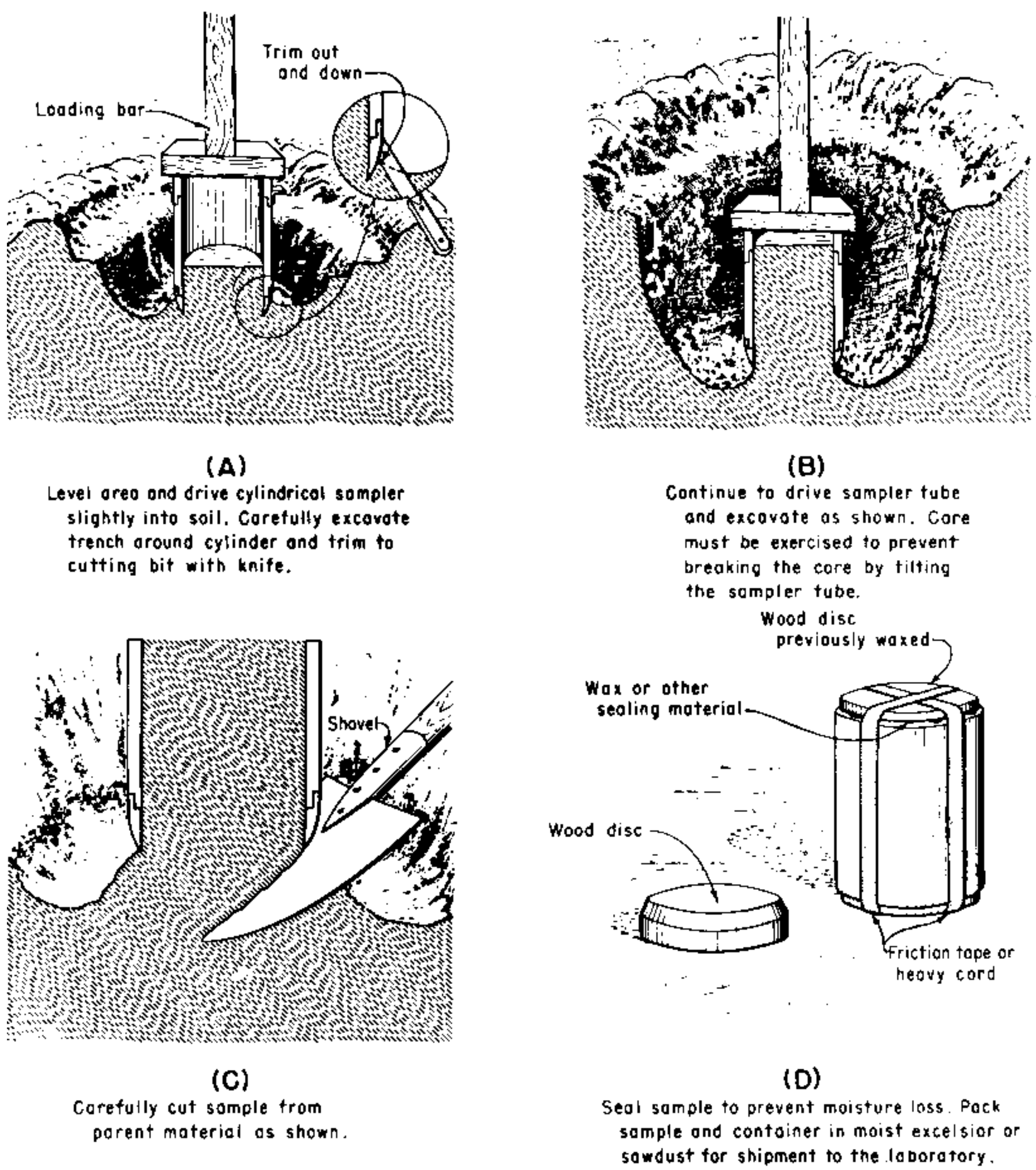


Figure 2-37.—Method for obtaining a hand-cut, undisturbed cylindrical sample.

Figures 2-38 and 2-39 show procedures commonly used to hand-cut block samples. Cutting and trimming samples to the desired size and shape requires extreme care particularly when working with easily disturbed soft brittle materials. Appropriate cutting tools should be used to prevent disturbance or cracking of the sample. Soft, plastic soils require thin, sharp knives. Sometimes a thin piano wire works well.

A faster and more economical method for obtaining undisturbed block samples is by use of a chain saw equipped with a specially fabricated carbide-tipped chain to cut block samples of fine-grained material and soft rock (fig. 2-40) (see USBR 7100). Usually, this method results in least disturbance to a sample because of the saw's rapid, continuous cutting action. Diamond concrete saws can cut gravel particles in a soil matrix effectively, although considerable precautions must be taken to prevent disturbance. When using saws, the operators should take appropriate safety measures, including wearing chaps and eye and ear protection. Block samples for laboratory testing are usually limited in size to 300- to 460-mm (12- to 18-in) cubes to facilitate handling. This size should allow for sufficient shear, consolidation, and permeability specimens. Block sampling is routinely performed as record testing of compacted fill to determine density. Small block samples of undisturbed soil can be cut from the larger block and coated with wax and tested in accordance with USBR 5375.

In dry climates, moist cloths should be used to inhibit drying of the sample. After the sample is cut and trimmed to the desired size and shape, it should be covered with thin plastic sheeting (such as Saran Wrap), wrapped with a layer of cheesecloth, and painted with melted, microcrystalline sealing wax. Rubbing the partially cooled wax surface with the bare hands helps seal the pores in the wax. At least two additional layers of cloth and wax should be applied.

b. Protecting and Preparing Hand-Cut Undisturbed Samples for Shipping.—As illustrated in figure 2-39, a firmly constructed wood box with top and bottom panels removed should be placed over the sample before the base of the sample is severed from the parent material and lifted for removal. The annular space between the sample and the walls of the box should be packed with moist sawdust or similar packing material. The top cover of the box then should be placed over the packing material. After the sample is cut from the parent material, the bottom side of the sample should be covered with plastic sheeting and the same number of layers of cloth and wax as the other surfaces; and the bottom of the box should be placed over the packing material. Tags and markers should be attached to the block to denote top, bottom, and orientation. Samples may vary in size, but most often are 150- to 300-mm (6- to 12-in) cubes. The same trimming and sealing procedures as described for block samples apply to cylindrical samples.

2-17. Mechanical Sampling Methods for Obtaining Undisturbed Samples.—Soil samplers used by Reclamation are designed to obtain relatively undisturbed samples of soils ranging from saturated, noncohesive soils to shale or siltstone. Each soil type dictates use of different types of sampling equipment to effectively recover high-quality samples. The following paragraphs describe the type of sampler best suited for good sample recovery from various soils.

a. Saturated Cohesionless Soils.—Cohesionless soils such as poorly graded sands (SP, SP-SM) and silty sands (SM) are difficult to sample below the water table. This is due to lack of friction in the inner tube or barrel. Research has shown that undesirable volume change can occur in clean sands during insertion of thinwall tubes or piston samples [38]. This is because the high permeability of clean sands allows movement of porewater and volume change results (i.e., drainage can occur). Sand may either dilate or contract during the sampling process. For sands containing more than 15 percent fines, undesirable volume change may not occur if the structure is not sensitive because there is no drainage due to decreased permeability. For clean sands, the only successful method to preserve structure is to freeze the soil before sampling. Freezing can be accomplished without disturbance in clean sand, but costs are so excessive that it is infrequently used. Efforts to characterize engineering properties of cohesionless soils currently depend on penetration resistance testing such as Standard Penetration Test or Cone Penetrometer Test.

Efforts have been made to track volume change in sands during fixed piston sampling. The technique requires use of piston sampling with inner rods with accurate measurements of stroke, recovery, and deflection on a reaction frame set up on a drilling rig. Measurements are made to 3 mm (0.01 ft) on the drill rig frame. Using this procedure, Reclamation found that predicted volume changes were also occurring in fine-grained soils. Accuracy and precision of such an approach has not been established, and Reclamation is not currently using this procedure.

In situations where cohesionless soils must be recovered, piston sampling or sampling barrels with baskets or retainers can be used. A fixed-piston sampler is designed to obtain a sample within a thin-wall cylindrical tube by pushing the tube into the soil with an even and uninterrupted hydraulic

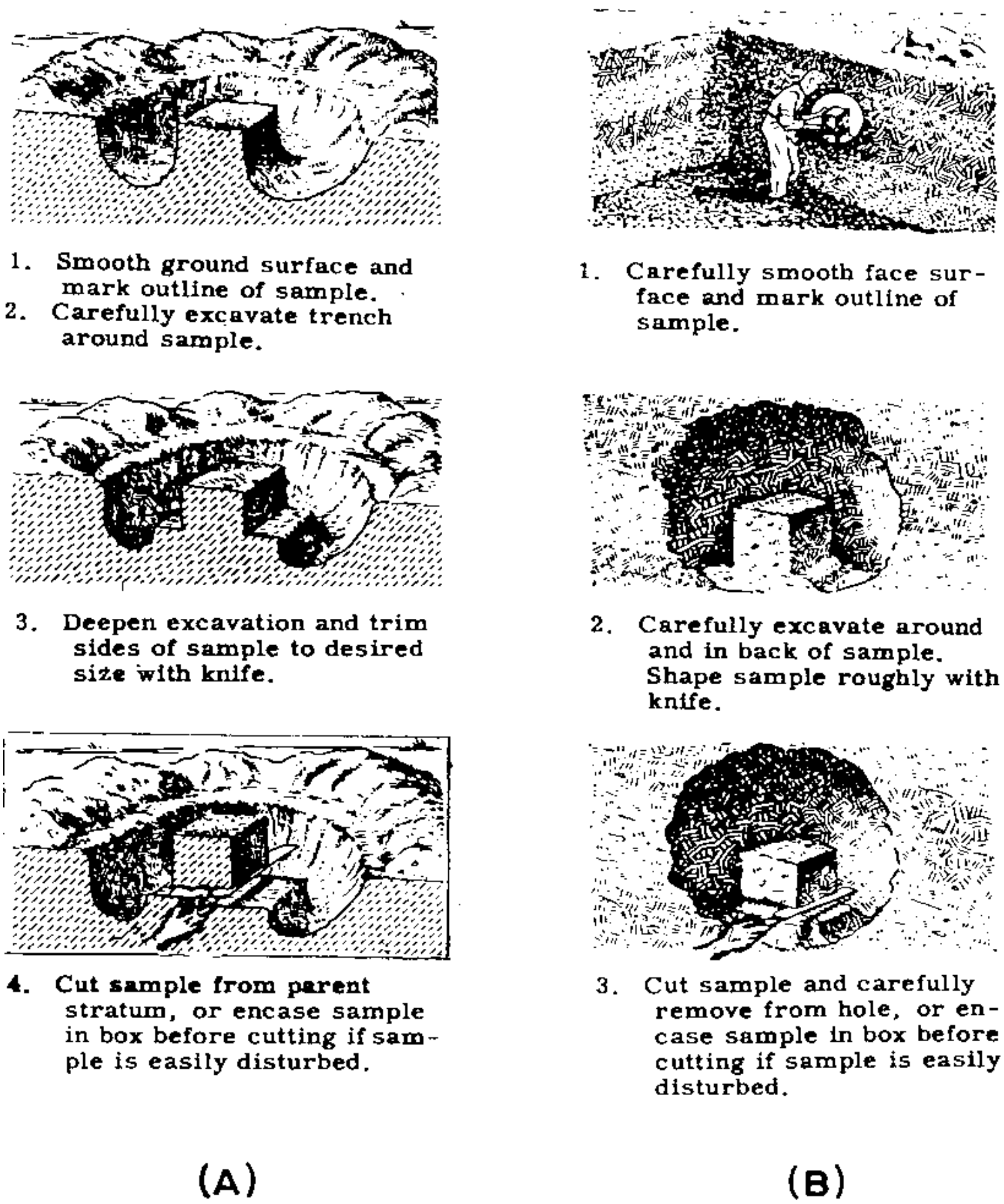


Figure 2-38.—Initial steps to obtain a hand-cut, undisturbed block sample from (A) bottom of test pit or level surface, and from (B) cutbank or side of test pit.

thrust. The sample is held within the tube during removal from the drill hole by a vacuum created by a locked piston, which is an integral part of the sampler (USBR 7105).

Reclamation uses several types of fixed-piston samplers to recover samples of soft, saturated soils. With the Hvorslev, Butters, and other inner rod piston samplers, the piston is held stationary by a piston-rod extension connected to the upper part of the drill rig mast while the sample tube is pushed into the soil. These samplers require a drill rig with a hollow-spindle. The Osterberg sampler has a piston that is attached to the head of the sampler. Sample recovery is accomplished by use of drill fluid pressure through the drill rods to push the thinwall sample tube into the soil. A fluid bypass system fabricated into the sampler stops penetration of the sampler tube at 750 mm (30 in). Figure 2-41 illustrates the operating principle of the Osterberg sampler. The Osterberg sampler is preferred for sampling in soils

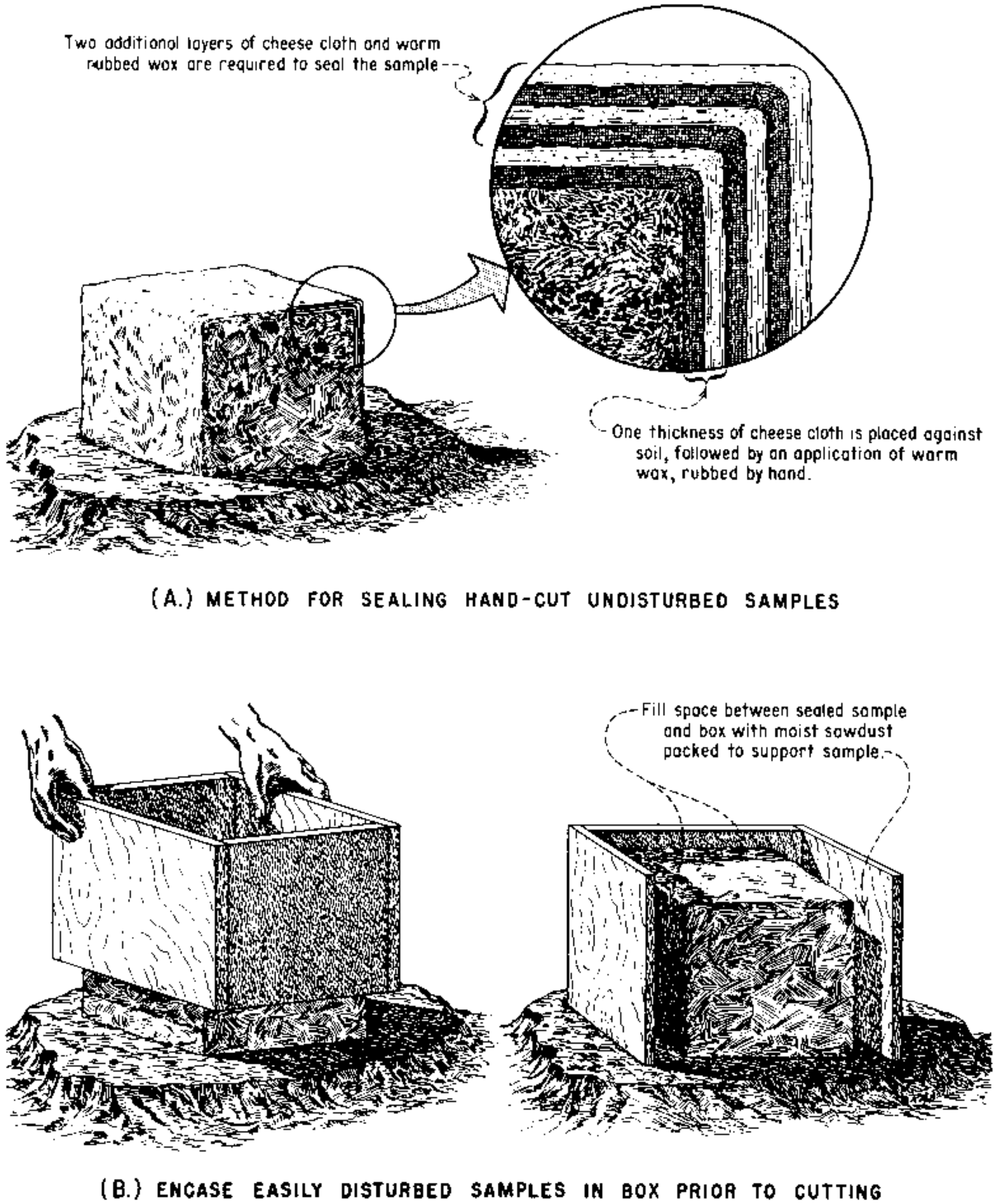


Figure 2-39.—Final steps in obtaining a hand-cut, undisturbed block sample.

where inner rod deflection measurements are not required since it is much faster to operate. The sample is recovered from the borehole by removing all rods and the sampler from the hole.

Thinwall push tube sampling is normally successful in nonsensitive cohesive soils. The optimum clearance ratios and drill mud consistencies described in USBR 7105 should be used. Since cohesive soils are undrained during the penetration process, high-quality samples can be obtained with smooth penetration rate. The cohesive nature of fines normally causes sufficient friction between soil and tube to retain the sample. Initial attempts to sample soft cohesive soils should be made with the thinwall push tube.

Handheld penetrometer tests should be performed on soft clays at the bottom of the tube to assist with selection of laboratory test specimens. A guide for sampling methods for materials difficult to recover can be found in reference [39].

Figure 2-40.—Chain saw equipped with carbide-tipped blade being used to cut block sample.

Samples of soft cohesive soils should be shipped to the laboratory and tested promptly. If stored for excessive periods of time, changes in material structure can occur because of oxidation and microbial processes. In the past, many materials have been used to coat steel tubes, such as lacquers and zinc magnesium oxides. If appreciable sand content exists in the soil, these coatings can be scraped away, and rust (iron oxides) forms on the thinwall tube. Stainless steel tubes can be used, but they are more difficult to cut open. The ends of thinwall soil samples are inevitably exposed to air during trimming, and the oxidation processes begins. Aging along with oxidation processes can cause detrimental changes in peak strength and sensitivity of soft clays.

b. Soft to Moderately Firm Cohesive Soils.—Soft to moderately firm cohesive soils in surficial deposits can be sampled in a relatively undisturbed condition using fairly simple sampling methods. Sampling equipment for this type of soil includes the thinwall push sampler and the hollow-stem auger sampler. The following paragraphs discuss each sampler and the necessary operational procedures to ensure recovery of a high-quality representative soil sample.

1. Thinwall Push Samplers.—Thinwall push samplers were developed primarily for obtaining undisturbed soil core samples of soft to moderately firm cohesive soils and are described in detail in USBR 7105. The sampler consists of a thinwall metal tube attached to a sampler head containing a ball check valve. The principle of operation is to push the sampler, without rotation, into the soil at a controlled penetration rate and pressure. The sample is held in the tube primarily by soil cohesion to the inner tube walls and assisted by a partial vacuum created by the ball check valve in the sampler head.

Reclamation commonly uses thinwall sampling equipment designed to recover either 75- or 125-mm- (3- or 5-in-) diameter soil cores. Size requirements depend primarily upon intended use of the sample. For moisture density determinations, a 75-mm- (3-in-) diameter sample will suffice, but great care must be exercised, and proper clearance ratios must be used to ensure that the sample does not densify during sampling. Appropriate tube clearance ratios and drill mud consistencies for thinwall sampling are given in USBR 7105. Samples 125 mm in diameter are preferred for laboratory testing because multiple specimens can be trimmed from a single sample. Triaxial shear equipment is available with end platens to accommodate extrusion of 75-mm thinwall specimens directly from the tube, especially for soils which are difficult to trim.

The ends of the soil sample in thinwall tubes should be trimmed to fresh soil. A moisture specimen should be taken from the bottom end of the tube. Average tube density can be measured according to USBR 7105. Expandable O-ring

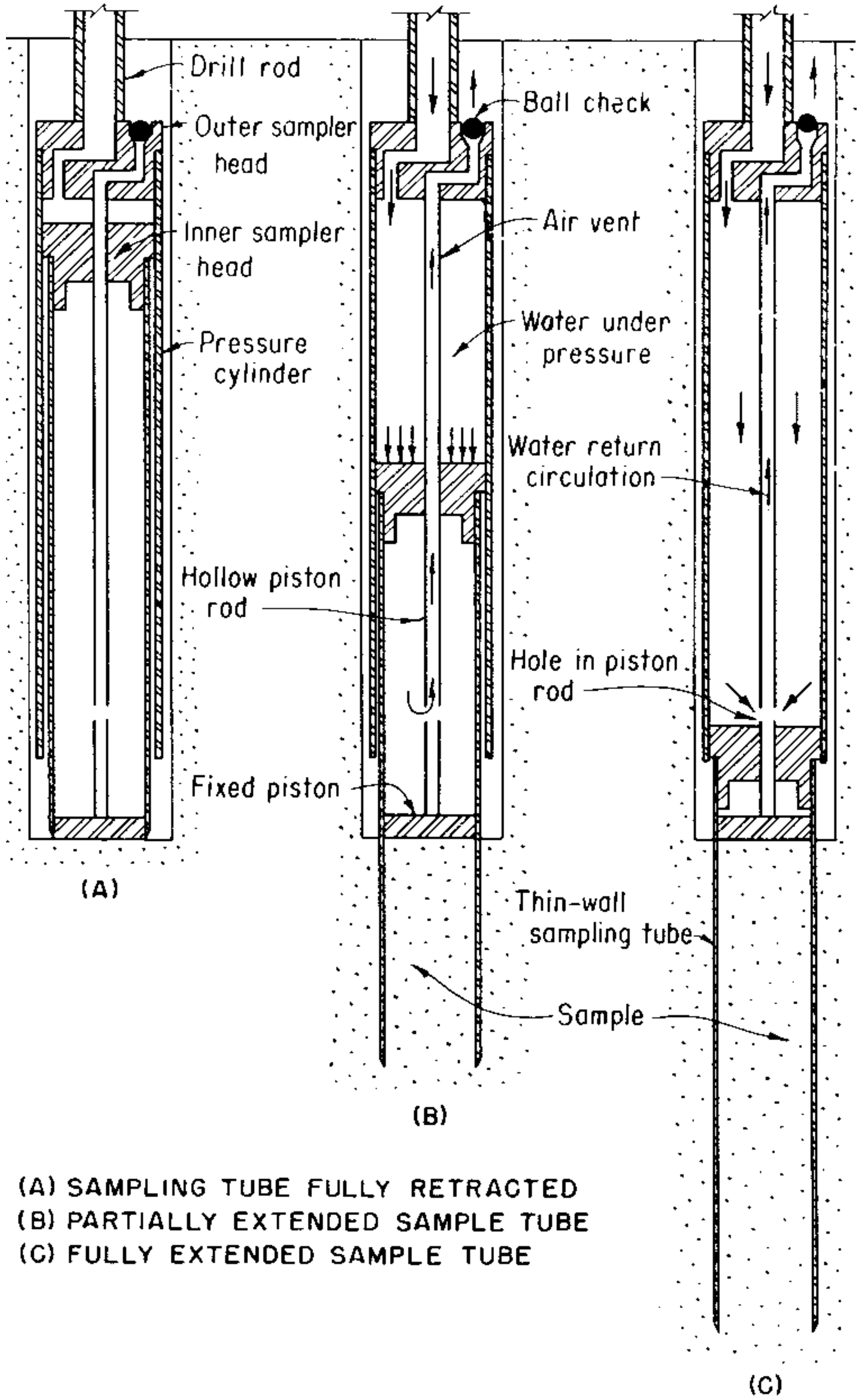


Figure 2-41.—Thinwall fixed-piston sampler (Osterberg type).

packers should be used to confine the soil. Special care should be taken to remove slough and cuttings by trimming the top of the sample.

2. Hollow-Stem Auger Samplers.—As described in USBR 7105, three types of sampling operations are available for recovering soft to moderately firm cohesive soils with hollow-stem augers. In the first method, a conventional sampler is lowered inside the hollow stem. In the second and third methods, known as "continuous sampler systems," a sampler barrel is specifically designed to lock into the lead auger allowing sampling to progress with advancement of augers. The continuous samplers are widely used in the drilling industry and are capable of sampling a wide variety of materials. Schematic drawing of wire-line and rod type continuous sampling systems are shown on figure 2-42.

The first type of sampling operation is accomplished by drilling to the sampling depth with a hollow-stem auger equipped with a center pilot bit. The pilot bit is attached to drill rods positioned within the hollow-stem auger. At the sampling depth, the drill rods and pilot bit are removed, and a thinwall push sampler is lowered to the bottom of the hole. After the sample is recovered, the pilot bit is replaced, and augering is continued to the next sampling depth.

A second type of hollow-stem auger sampling operation involves a wire-line latch system that locks the pilot bit and soil sampler within the lead hollow-stem auger. After the auger has been advanced to the sampling depth, an overshot assembly is lowered by wire-line to unlock and latch onto the pilot bit for removal from the hole. Then, a thinwall sampler with a head bearing assembly is lowered by wire-line and locked within the lead auger section. Sampling is accomplished by continuing auger rotation and penetration, which allows the center core material to enter the thinwall sampler. The head bearing assembly on the sampler allows the sample tube to remain stationary while the auger is rotating. At the end of the sample run, the overshot is lowered by wire-line to release the sampler lock mechanism, latch onto the sampler, and remove it with the soil sample from the hole.

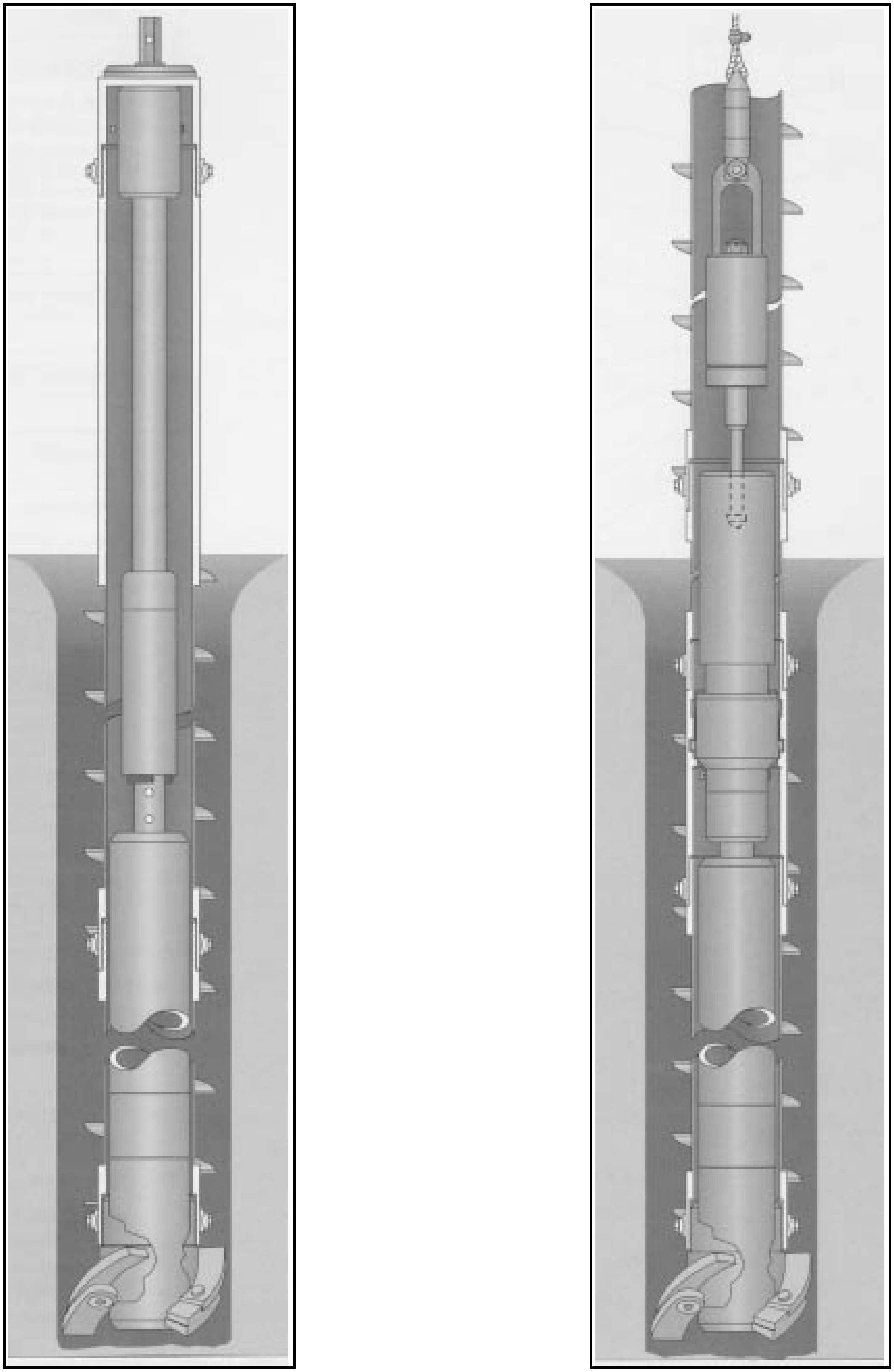
The wire-line hollow-stem auger sampling system can be used to successfully sample a wide variety of soils. Some difficulties may be experienced with latching systems in soils below the water table which heave into the hollow stem. Wire-line line systems are economical due to rapid continuous sampling capability. They are used extensively for hazardous waste site characterization studies. Drill rod systems discussed below are preferred for large-diameter soil samples where detailed studies of engineering properties are required. Inner barrels can be equipped with split liners or Plexiglas liners. Sample diameters of up to 100 mm (4 in) are currently available in wire-line systems. Typical sample lengths are 1.5 m (5 ft) but may be shortened if sample disturbance is evident.

The third and most recently developed hollow-stem auger sampling system (USBR 7105) involves using rods to lower, hold, and hoist a continuous sampler unit designed to recover samples during auger penetration. This system positively eliminates rotation of the sampler as the auger rotates. It is considered the best mechanical sampling system available for recovery of undisturbed soil samples by hollow-stem auger.

The stability of any sampling tool is critical to recovery of representative undisturbed samples. With hollow-stem augers, the inner barrel or sample tube that receives the soil core must not rotate as soil enters the sampler. A sampler with a head bearing assembly can rotate if cuttings are allowed to accumulate in the annulus between the outer rotating auger and the inner sample barrel. To eliminate any chance of movement of the inner barrel, the continuous sampler system is rigidly connected to rods that extend up through the hollow-stem auger to a yoke located above the rotating auger drillhead. Then, the outer auger is allowed to rotate for drilling penetration, but the sampler within the auger is held to prevent rotation as soil core enters the sample tube. As with the wire-line systems, typical sampling intervals are 1.5 m; but for large-diameter samples for laboratory testing, this interval should be shortened to reduce friction buildup in liners and/or the tendency to use larger clearance ratios and overcut the sample. Samplers up to 150 mm (6 in) in diameter are available with the drill rod hollow-stem auger system. The large diameter capability and positive antirotation of the inner rod system make this the preferred method when detailed engineering properties studies are required.

To recover a sample with the continuous sampler system, all sampler connecting rods and the sampler are removed from the auger to retrieve the soil core. This is followed by lowering the sampling unit to the hole bottom for continuation of sampling operations.

Both continuous hollow-stem auger sampling systems have adjustments for lead distance of the soil cutting shoe and different clearance ratios for the shoe. As with the Denison soil core barrel, the lead distance and clearance ratios must be optimized for best sample recovery. In general, softer soils require more lead distance of the cutting shoe below auger flights. More cohesive soils require larger clearance ratios to avoid buildup of friction inside liners. Both the



a b

Figure 2-42.—Overshot wire-line a) and rod type b) continuous sampling system. (Mobile Drill Co.)

wire-line and inner rod operated hollow-stem auger systems will accept clear acrylic liners. Plastic liners should have walls thick enough to confine the core without deflecting [usually about 6 mm (1/4 in)]. Numerous measurements are taken with micrometers to ensure that liner diameters required for accepting samples are maintained. The plastic liners can be cut flush with soil ends, and plastic caps or expandable O-ring packers can be used to seal specimen ends to preserve in-place moisture content. Use of plastic liners permits sample inspection and adjustments for clearance ratio and shoe lead distance.

Manufacturer supplied shoes are designed for successful recovery of a wide variety of soils but may not provide optimum sampling for advanced engineering properties testing. For critical samples, shoes of varying lead distance and clearance ratios can be readily machined by local commercial machine shops. The most frequent problem is the tendency to use too large a clearance ratio during sampling. This can result in a large air gap inside the sample liner and possible alteration of soil properties. The goal is to provide a sample which fills the sample liner without objectionable friction that results in compression and densification of the soil. If an air gap exists, the clearance ratio should be reduced. If excessive friction develops in the liners, either decrease sample length or increase clearance ratio.

c. Medium to Hard Soils and Shales.—Medium to hard soils and shales located either above or below the water table can usually be sampled in an undisturbed condition using double-tube coring barrels. The three types of core barrels commonly used are the Pitcher sampler, Denison core barrel, and DCDMA series 100- by 140-mm and 150- by 200-mm core barrels (4- × 5-1/2- and 6- × 7-3/4-in). The DCDMA series barrels can also be converted to perform diamond coring for rock sampling. The following paragraphs discuss each sampler and the necessary procedures to ensure recovery of high-quality representative soil samples.

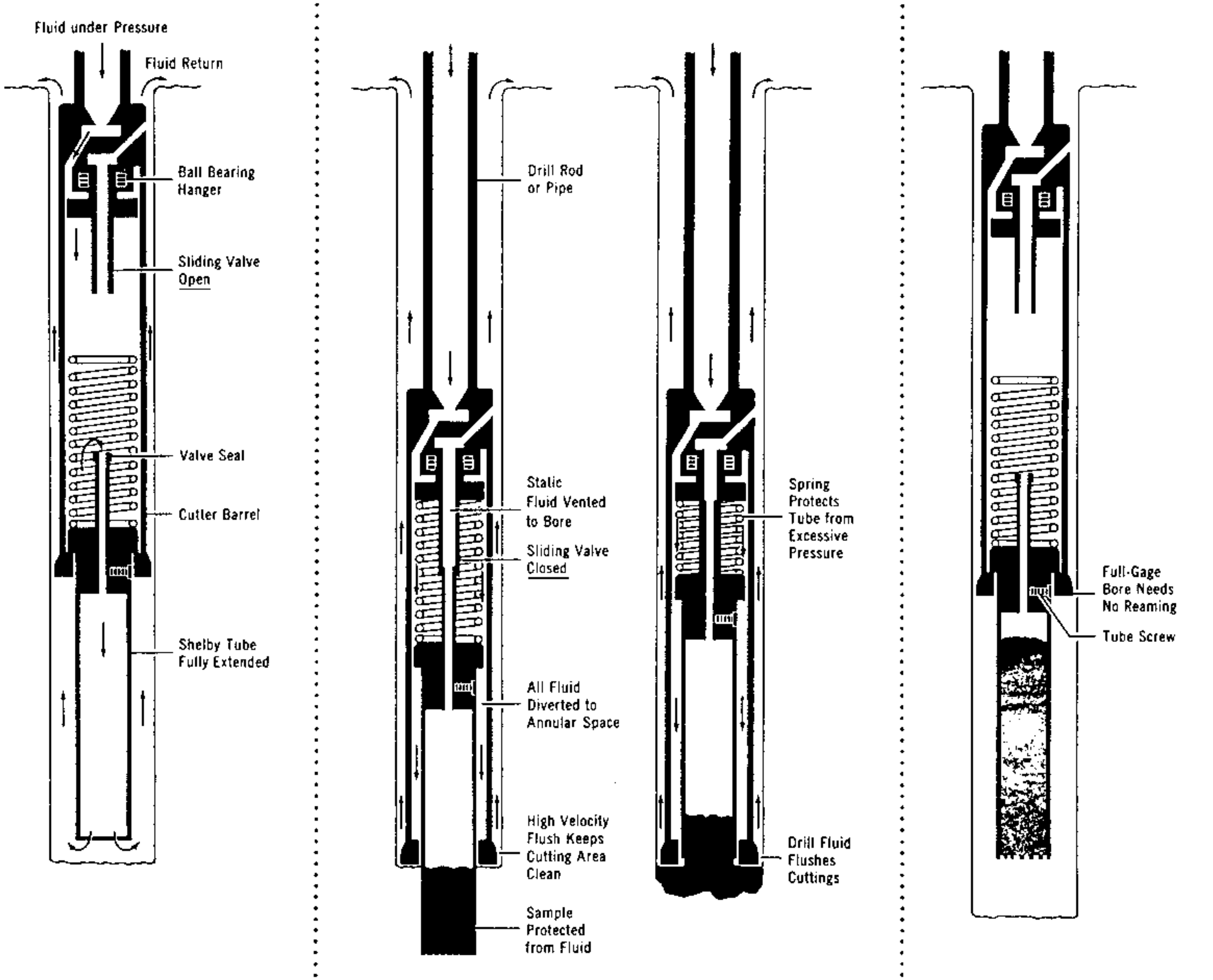
1. Pitcher Sampler.—The Pitcher sampler was developed primarily for obtaining undisturbed soil core samples from medium to hard soils and shales. Figure 2-43 illustrates the action of this sampler. The Pitcher sampler has a unique feature in that it has a spring-loaded inner barrel which lets the sample trimming shoe protrude or retract with changes in soil firmness. In extremely firm soils, the spring compresses until the cutting edge of the inner barrel shoe is flush with the crest of the outer barrel cutting teeth. In soft soils, the spring extends and the inner barrel shoe protrudes below the outer barrel bit and prevents damage to the sample by drilling fluid and drilling action. Because spring action on the cutting shoe automatically adjusts lead distance, the pitcher sampler is always preferred for use in firm to stiff soils where layer stiffness will change or where softer zones will be encountered. If soft zones are encountered without spring loaded inner barrels, the material will be washed out and contaminated with drill fluid.

Although the Pitcher sampler is available in various sizes for obtaining cores from 75 to 150 mm (3 to 6 in) in diameter, Reclamation's laboratory requirements normally dictate 150 mm (6 in), a 150- by 200-mm Pitcher sampler. This sampler was designed to use 150-mm thinwall tubes as the inner barrel. Normally, the soil core is contained within the thinwall tube, and a new tube is placed within the sampler for each sampling run. However, Reclamation changed the inner-barrel configuration to one that accommodates sheet metal liners for the soil core, rather than thinwall tubes. Aluminum irrigation pipe was found to have acceptably small-diameter variation for high-quality sampling. The modified inner barrel is threaded for attachment of a trimming shoe with a milled recess to contain the sheet metal liner. Sheet metal liners are preferred for samples for laboratory testing because they are easier to open for examination, and core is more easily removed without damage.

2. Denison Sampler.—The Denison sampler, shown in figure 2-44, was developed to obtain large-diameter undisturbed cores of cohesive soils and shales of medium to hard consistency. Disadvantages of the Denison sampling barrel are having to manually adjust the position relationship between the outer barrel cutting bit and the inner barrel trimming shoe according to the consistency of the soil to be sampled. The required setting must be determined by the operator before each sampling run. The setting is achieved by interchanging varied lengths of outer barrel cutting bits to conform with the type and consistency of soil being sampled. The proper cutting bit for various soil consistencies is selected as described below and as illustrated in figure 2-45.

- Soft soil samples can be obtained with a short cutting bit attached to the outer barrel so the inner barrel trimming shoe protrudes about 75 mm (3 in) beyond the bit. The shoe acts as a stationary push sampler, trims and slides over the sample, and protects the core from drill-fluid erosion or contamination.

- Firm soil samples can be obtained by attaching a cutting bit having a length that will position the crown of the bit teeth approximately flush with the inner-barrel shoe trimming edge. With this setting,



a) LOWERING DOWN HOLE

b) SAMPLING IN SOFT STRATA

c) SAMPLING IN HARD STRATA

d) WITHDRAWING SAMPLER FROM HOLE

Figure 2-43.—Pitcher sampler. (Pitcher Drilling Co.)

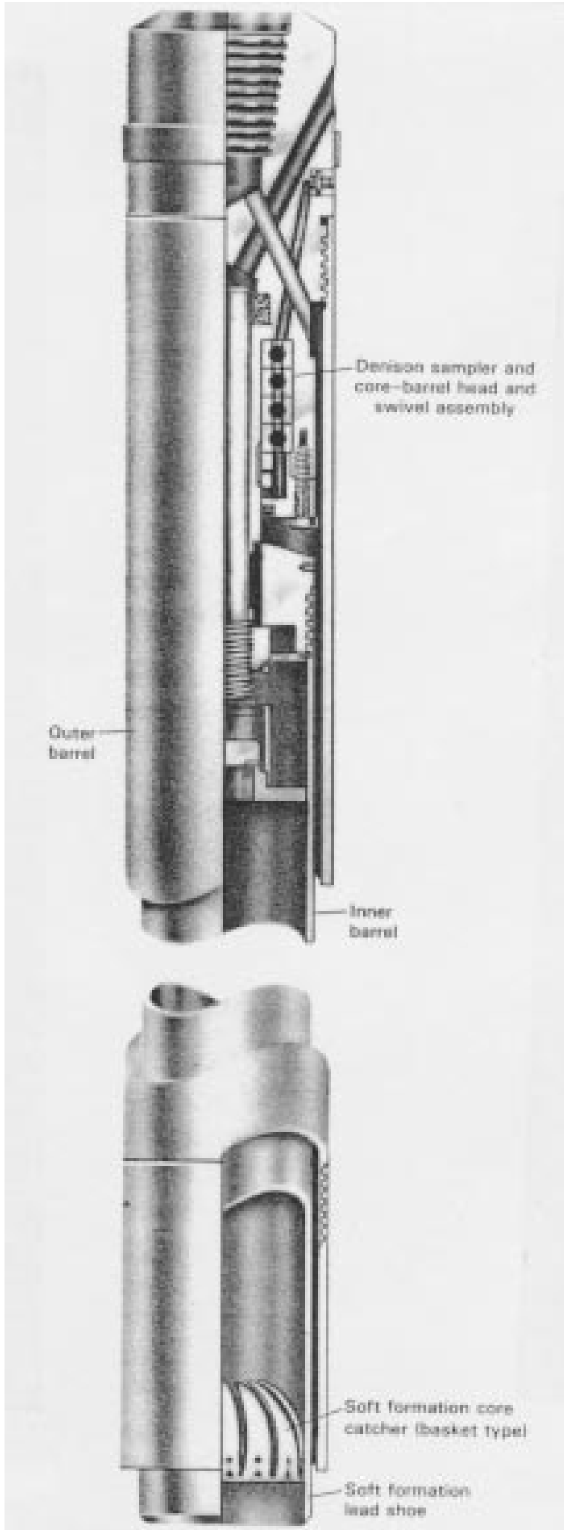


Figure 2-44.—Denison sampler and core barrel.

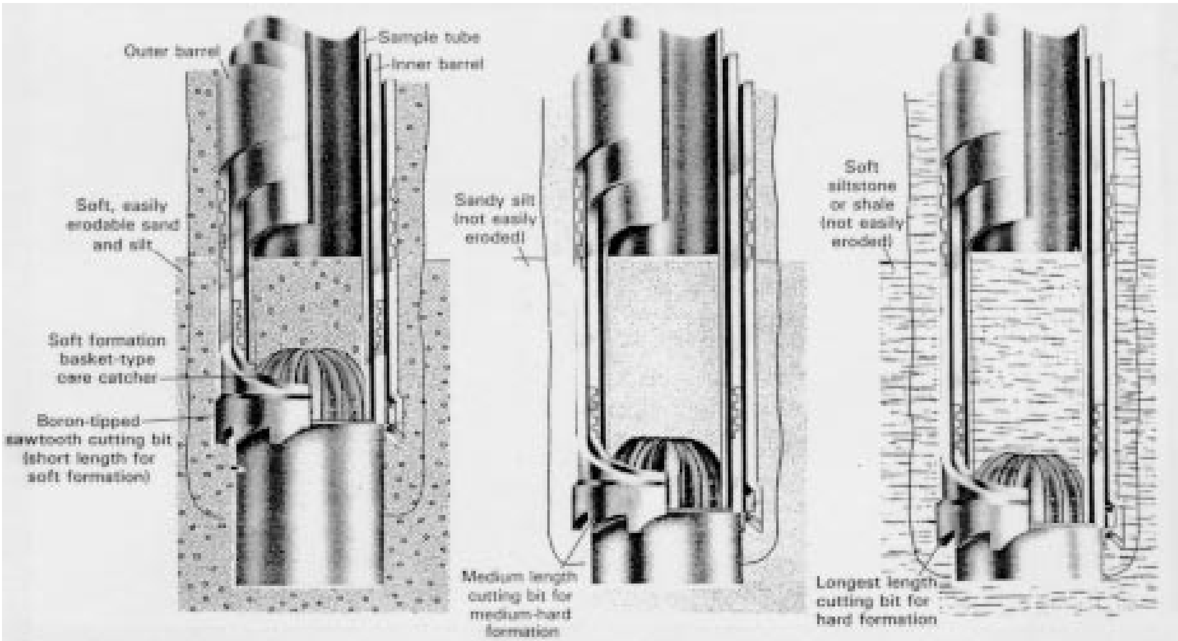


Figure 2-45.—Relationship of inner-barrel protrusion using different-length, Denison-sampler cutting bits for drilling.

the bit teeth cut the core simultaneously with the trimming of the core by the shoe. The shoe provides some protection to the sample from the drill fluid because most of the fluid circulates between teeth openings rather than through the crown area.

- Hard soil samples are obtained by attaching a cutting bit having a length that will position the teeth about 25 to 50 mm (1 to 2 in) below the trimming shoe. This setting is intended only for nonerodible soils because the entire circumference of the sample is subjected to drill fluid circulation before it is contained within the trimming shoe.

3. Large-Diameter Hi-Recovery Core Barrels.—Increased demand for large-diameter soil samples for laboratory testing became obvious to manufacturers of conventional rock coring equipment in the late 1960s. To provide alternatives to soil-sampling core barrels (e.g., Denison and Pitcher core barrels), the DCDMA developed standards for a large-diameter core barrel with versatility to sample both soil and rock. These core barrels use a variety of interchangeable parts to convert the basic rock core barrel to core medium to hard soils and shales, fragmented rock, rock with soil lenses, and homogeneous rock. Some of the interchangeable parts and their functions are:

- A clay bit, with face extension, to trim and advance over softer clay soils and to protect the core from drill-fluid erosion.
- A spring-loaded inner barrel to protrude in front of the core barrel for soft soils and to retract into the core barrel for harder soils.
- A split inner barrel for coring shales, soft rock, fragmented rock, and lensed rock.
- A single-tube inner barrel for coring homogeneous hard rock.

Reclamation has used successfully both the 120- by 140-mm core barrel and 150- by 200-mm barrel core (4- by 5-1/2- and 6- by 7-3/4-in), depending upon core size requirements. A metal liner should be inserted inside the inner barrel to contain and seal the core sample for shipment to the laboratory.

d. Unsaturated Water Sensitive Soils.—Dry drilling techniques are preferred for sampling water sensitive unsaturated soils. Water sensitive soils include windblown loess deposits and slopewash that can collapse when exposed to water. These soils are characterized by low density in-place conditions. Continuous hollow-stem auger samplers are preferred for sampling these deposits because exposure to drilling fluids is not permitted. If surface exposures are accessible, block samples provide the highest quality sample. Sampling studies in loess have shown that thinwall tubes and Pitcher sampling result in unacceptable sampling disturbance [40]. Thinwall samplers compacted loess—when dry methods were used to advance the borehole. Pitcher sampling allowed for some exposure to fluid if not properly operated.

2-18. Casing Advancer.—Manufacturers have adapted wire-line drilling principles to accommodate a wide variety of sampling methods. The wire-line casing advancer can be used to advance standard **BW**, **HW**, and **HW**, casing through difficult deposits such as coarse alluvium or highly fractured materials while still protecting the drillhole. Casing advancers have a center pilot bit which is normally configured as a tricone rockbit. In one case, where it was necessary to drill through several steel settlement plates, a pilot bit made with diamonds was used. Casing bits are normally diamonds, but carbide insert drag bits have been used in soils. After the casing is advanced to depth for testing or sampling, the center pilot bit can be removed by wireline, and a flush casing is left. The casing advancer has proved effective for the following operations:

- Conventional or wire-line coring through casing
- Casing through landslide or alluvial materials
- Conventional or wire-line thinwall tube sampling through casing
- Penetration resistance testing and sampling ahead of casing
- Tie back anchor installation through casing
- Installation of well screen, perforated drain, or piezometers through casing

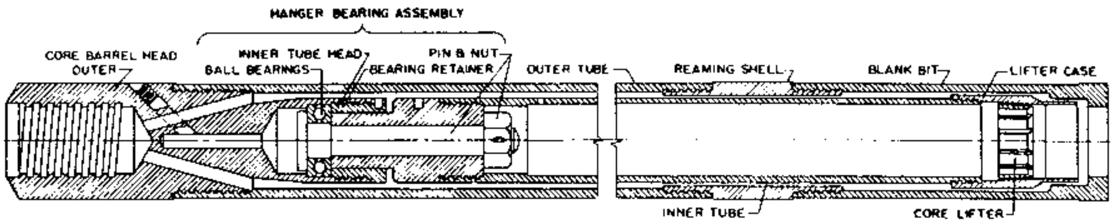
The casing advancer has been used to successfully perform penetration resistance tests in loose sands below the water table. Drilling was performed with carbide casing bit and no pilot bit. The primary reason for success was that a fluid column was maintained in the casing that prevented heaving sands from entering, and careful attention was paid to circulation to avoid hydraulic fracturing.

2-19. Rock Sampling Methods.—Core barrels are available to obtain cores from 20 to 150 mm (3/4 to 6 in) in diameter. There are two principal types of core barrels: (1) single tube and (2) double tube. The DCDMA has standardized dimensions for four series of conventional rock core barrels. The series are denoted as **WG**, **WT**, **WM**, and **Large Diameter** [32]. The **W** series barrels can be obtained for nominal hole sizes **R** through **H**. The differences between these designs are primarily related to methods of fluid circulation and optimizing core diameters. For example, the **G** series of tubes is the most basic in design and allow for more core exposure to drill fluid. The **T** series barrels have thinner tube walls, which result in larger core. The **G** and **T** series are available in single tube or double-tube configurations. In the **M** series barrel, the inner tube is threaded to receive the lifter with resulting less exposure of core to drill fluid. The large-diameter core barrels have better control of drill fluid circulation and core protection. The **M** and **Large Diameter** series barrels have double-tube design. As one progresses through **G**, **T**, **M**, and **Large Diameter**, the ability to retrieve difficult cores increases. Double-tube core barrels can be either rigid or swivel design. Reclamation uses either **M** or **Large Diameter** double-tube core barrels for the majority of rock coring operations with conventional barrels. Figure 2-46 shows a schematic view of typical **M** and **Large Diameter** double-tube swivel core barrels with a split liner recommended for most investigations. When DCDMA, standard drawings are compared to drill manufacturers literature, it is difficult to distinguish which DCDMA core barrel series is available. Most manufacturers offer conventional barrels equivalent to the **M** series.

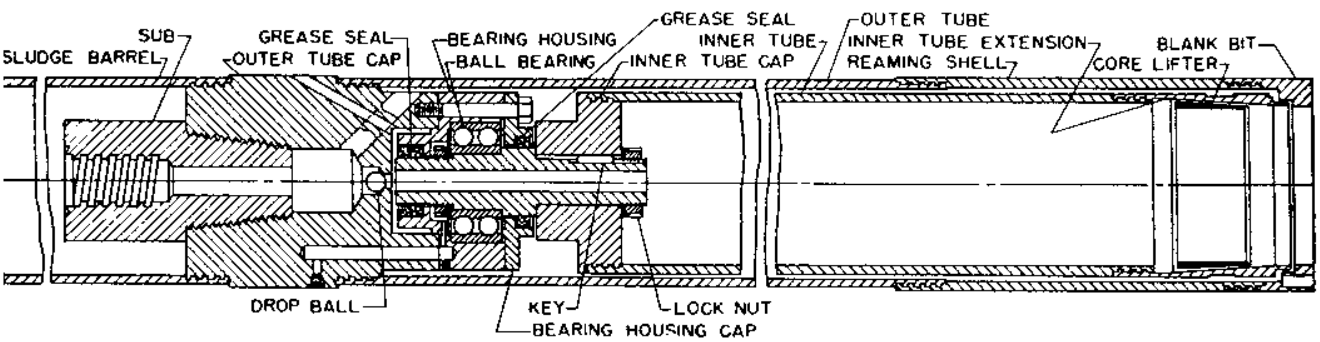
Normally, **R**, **E**, **A**, and **B** hole size cores normally apply only to instrumentation or stabilization applications such as overcoring studies or rock bolt installations. For investigation purposes, Reclamation specifies a minimum hole size of **N** for conventional core barrels.

The single-tube core barrel is a basic design and consists of a core barrel head, a core barrel, and an attached coring bit that cuts an annular groove to permit passage of drilling fluid pumped through the drill rod. This design exposes the core to drilling fluid over its entire length and can result in serious core erosion of unconsolidated or weakly cemented materials. The single-tube core barrel is no longer used except in unique situations, such as in concrete sampling or when using "packsack-type" drills. Single-tube core barrels or masonry core barrels are frequently used for coring soil cement dam facings for construction control. Single-tube core barrels are not recommended for coring operations in concrete dams [41].

The rigid-type, double-tube core barrel provides an inner barrel which rotates with the outer barrel but protects the core from drilling fluid. The rigid-type, double-tube core barrel has been used successfully in soft to medium-hard formations and in hard, broken formations. A problem arises in core abrasion caused by the rotating inner tube, and soft, easily disturbed material cannot be cored well with the barrel. The rigid design is not recommended for coring when laboratory tests are required or where good recovery is required.



M-DESIGN, CORE LIFTER IN INNER BARREL



LARGE DIAMETER M-DESIGN

Figure 2-46.—Double-tube swivel-type core barrels. (Bureau of Mines [39])

The swivel-type, double-tube core barrel sampler consists of an outer rotating barrel and an inner stationary barrel that protects the core from drilling fluid and reduces torsional forces transmitted to the core. The swivel-type, double-tube barrel is used to sample most rock; it may be used to obtain cores in hard, brittle, or poorly cemented materials such as shale and siltstone or cores of soft partially consolidated or weakly cemented soils.

Most double-tube core barrel inner tubes may be replaced with a split inner tube. Use of a split inner tube is required in Reclamation investigations unless special conditions exist which warrant deviation from this requirement. Advantages to split inner tubes are:

- The undisturbed core allows detailed visual analysis.
- The core is easily transferred into the core box without sample disturbance.
- The core can be easily wrapped in plastic to preserve moisture content before it is placed in the box.
- Expansive or sticky formations can be easily removed.

A few of the double-tube core barrels have been modified to allow a split liner to be inserted inside a solid inner tube to accept the core sample. Barrels modified in this fashion are sometimes referred to as "triple-tube" core barrels. A triple-tube configuration is desirable in formations that may be detrimentally affected by drill fluid which could penetrate a single-split inner tube.

The **Large Diameter** core barrels can be adapted for a wide variety of sampling purposes. Three sizes commonly available are described by core and barrel outer diameter (in inches) as follows: 2.75 by 3.87, 4 by 5.5, and 6 by 7.75. The large-diameter series is available also with split inner tube or triple-tube configuration. The larger barrels can even be adapted for sampling soils by converting bits and liners into configurations similar to Denison or Pitcher samplers. These core samplers also have been designed with a spring-loaded retractable inner barrel, which enables the same type of core barrel to be used for coring either soil or rock. The retractable inner barrel and soil-coring bits are replaced with a standard inner barrel and diamond bits for rock coring. They can be equipped with split liners. These barrels are highly recommended in deposits which are difficult to sample. Samples of soils have been successfully obtained with this type of barrel. In cases where soil samples are retrieved, soil core should be waxed to preserve in-place moisture content.

A wide variety of wire-line core barrel systems are available for rock coring. By design, the wire-line core barrel is essentially a swivel type double-tube or triple-tube system. Most coring operations now use a split inner liner inside of a solid inner tube. The triple-tube system is highly recommended over a single-split inner tube system to prevent core loss if the single-split tube springs open. Some of the triple-tube systems with split inner liners are equipped with hydraulic core extraction pistons to remove the split liner from the solid inner tube. The **AQ**, **BQ**, **NQ**, **HQ**, and **PQ** wire-line core barrels are available in core diameters of about 27, 33, 44, 60, and 83 mm (1.1, 1.3, 1.75, 2.4, and 3.25 in). The actual core sizes vary slight among manufacturers. **NQ** size is considered a minimum—primarily, because rapid water testing is routinely performed on many investigations using wire-line downhole packers. The wire-line packer system is shown on figure 2-47. Water tests are performed using this system according to USBR 7310. If the objective of the program is good core for testing, Reclamation normally uses **HQ** size wire-line equipment, and water testing can be performed on these holes also. **PQ** size equipment is useful for obtaining better recovery in softer matrices such as soil-cement-bentonite cutoff wall backfill materials. Materials with compressive strengths as low as 1400 Kpa (200 lb/in^2) have be successfully recovered with **PQ** size equipment. Compressive strengths lower than this present significant problems with good core recovery, and other testing methods such as in-place tests must be considered to characterize these materials.

Many guides are available for selecting conventional and wire-line core barrel bits including from the manufacturers themselves. Key parameters in bit selection include the abrasiveness of the cuttings and the matrix of the material. For softer materials, diamond bits can be replaced with hardened metal, polycrystalline, or carbide drill bits, and shorter core runs [1.5 m (5 ft) or one-half that in length] are used to minimize disturbance. Metal bits are an attractive alternative for sampling softer materials, but diamonds are a must for coring harder rock. Another important parameter is the location of fluid discharge. Core barrel bits can either be internal or face discharge with face discharge recommended for easily erodible matrix. Excellent references for bit selection are available [32, 39, 42].

Accuracy and dependability of data from rock core drilling depend largely on the size of core in relation to the kind of material drilled, the percentage of core recovered, behavior during drilling, and experience of the drill crew. Since rock that cores well in an **NX** size hole may break up badly in an **EX** size hole, it is important to use the largest practical diameter hole and core barrel. Recovery of core is more important than making rapid progress when drilling a hole. Portions of core that are lost probably represent shattered or soft, incompetent rock, whereas recovered portions represent the best rock from which an overly optimistic evaluation of the foundation likely will be made. Nevertheless, a reasonably high percentage of core recovery provides a more continuous section of the materials passed through. Cores provide information on the character and composition of different materials, with data on spacing and tightness of joints, seams, fissures, and other structural details. When drilling in soft materials, drill fluid circulation must be reduced or stopped entirely, and the core recovered "dry," even though significant delay in operations may occur.

Many of the principles of conventional coring can apply to wire-line coring. The core barrel is fitted with a coring bit and is lowered into the hole with the hollow drill rod. Circulation of drilling fluid should begin before the core barrel reaches the bottom of the hole to lift cuttings or sludge and prevent them from entering the core barrel at the start of coring. The optimal rotational speed of drilling varies with type of bit used, diameter of core barrel, and kind of material to be cored. Excessive rotational speed results in chattering and rapid bit wear and will break the core. Low rotational speed results in less wear and tear on the bit and better cores, but lower rates of progress. It is critical to minimize vibration from uneven rods or poor drive head assembly. Vibration of the drill stem will cause vibrating and chattering, resulting in bit blockage, broken core, and poor core recovery. If rotation rates must be decreased because of vibration, the mass or pressure on the bit must also be decreased, or polishing and dulling of the diamonds will occur.

Rotational speed must also be adjusted for core barrel diameter. Drillers must evaluate all factors when considering coring rates and pressures. Bit wear can provide many clues when selecting proper rotational rates once the appropriate bit is specified for the material. If wire-line coring is performed, bit inspection will cause trip time delay.

The rate at which the coring bit advances depends on firmness of material, amount of pressure applied on the bit, and rotational speed. Pressure must be carefully adjusted by the driller; excessive pressure causes the bit to plug and may shear the core from its base. Bit pressure is controlled by

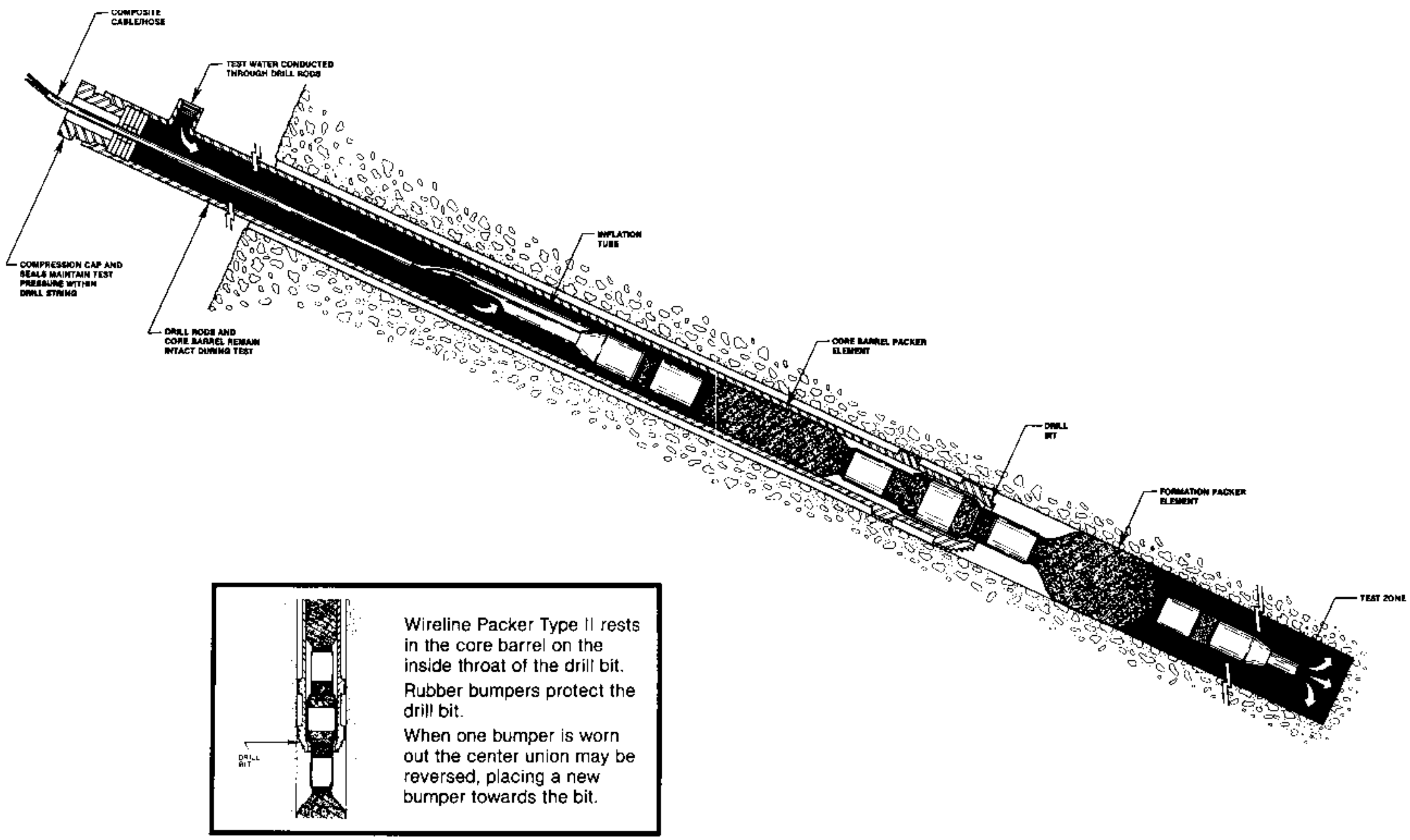


Figure 2-47.—Wire-line packer system. (Longyear Co.)

a hydraulic or screw feed on the drilling machine. The weight of the column of drill rod is seldom in excess of the optimum bit pressure for coring medium and hard rock; frequently, additional downward pressure is applied. If coring is performed in abrasive, friable, or fractured rock, the rotational speed and downfeed pressure must be reduced. If the coring bit penetrates these materials too fast, pieces of uncut rock result and cause core blockage and poor recovery. After the core run is completed, the drill rods are pulled up without rotation which causes the core lifter to slide down a beveled shoe and increasingly grip the rock until breakage occurs. Generally, breakage can be heard as a snapping sound, and it will always occur below the lifter.

If the drill hole in rock is clean, and seams and fissures are not sealed off by drill action, percolation tests can be performed to test the permeability of various strata. Gravity or pressure tests are made as described in USBR 7310. Large drilling fluid losses or water inflows into drill holes during drilling indicate either the presence of large openings in the rock or the existence of underground flow. Completed holes should be protected with lockable caps to preserve them for use in ground-water level observations, as grout holes, or for reentry, if later it is necessary to deepen the hole. Usually, casing is required for those sections of hole in loose material or unconsolidated subsurface soils.

As rock cores are removed from core barrels, they are placed in core boxes and logged. Guidelines for logging, core box construction, and core box arrangements are given in the *Engineering Geology Field Manual* [20]. Core can be placed by hand into core boxes, and use of cardboard or plastic half-rounds is encouraged. Long pieces of core may be broken to fit into the core box but should be marked as a mechanical break. If the core contains weak materials or materials for laboratory testing, the core should be completely wrapped in layers of plastic wrap, aluminum foil, and wax. Wrapping can be performed on core in the half-round as shown on figure 2-48, except that the complete core is wrapped.

Figure 2-49 shows a standard core box and illustrates the method of placing cores in the box to ensure proper identification of each core sample.

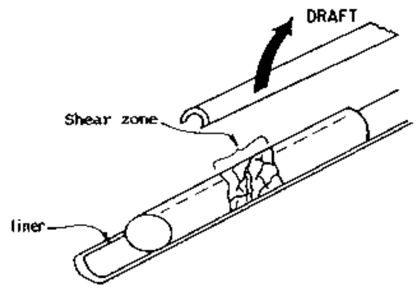
2-20. Geophysical Exploration Methods.—Geophysical methods of subsurface exploration are an indirect means of gathering data pertaining to underground conditions. Using geophysical techniques involves taking measurements at the earth's surface or in boreholes to determine subsurface conditions. Geophysical methods may be employed during any stage of an investigation and include:

- seismic,
- electrical,
- magnetic, and
- gravity.

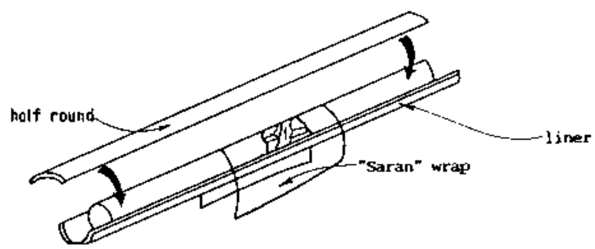
Geophysical methods can be a useful and economic addition when used in conjunction with a test-boring investigation program. In contrast to borings, geophysical surveys are used to explore large areas rapidly and economically. They indicate average conditions along an alignment or in an area, rather than along the restricted vertical line at a single location as in a boring. Geophysical data can be used to help determine the best location of future drill holes as well as provide information to extrapolate foundation conditions between existing drill holes. These data are useful for detecting irregularities in bedrock surface and at interfaces between strata. The cost of performing geophysical surveys is often less than the cost of drilling; therefore, judicious use of both geophysical methods and drilling can produce the desired information at an overall lesser cost. Although geophysical field work is relatively inexpensive, interpretation of results is difficult and specialized. For test results to be usable and reliable, correlations must necessarily be made locally with exploration data from borings.

Geophysical methods are best suited to prospecting sites for dams, reservoirs, tunnels, highways, canals, and other structures. Also, they have been used to locate gravel deposits and sources of other construction materials whose properties differ significantly from adjacent soils. Downhole, uphole, and cross-hole seismic surveys are used extensively to determine dynamic properties of soil and rock at small strains.

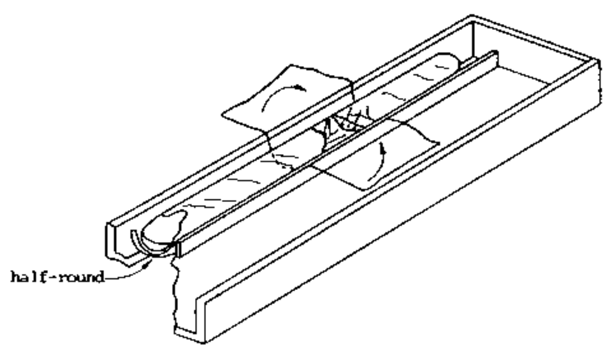
All geophysical techniques are based on detection of differences between properties of geologic materials. If such differences do not exist, geophysical methods will not be useful. These differences range from acoustic velocities to contrasts in electric properties of materials. Seismic methods, both reflection and refraction, depend on the difference in compressional or shear-wave velocities through different materials. Electrical methods depend on contrasts in electrical resistivities. Differences in density of different materials permit gravity surveys to be used in certain types of investigations, and contrasts in magnetic susceptibility of materials allow magnetic surveying to be used. Differences in magnitude of naturally existing electric current within the earth can be detected by self-potential surveys.



1. Remove upper split liner to expose fractured rock or shear zone.



2. Place "Saran" wrap over shear zone; then place half-round over top of core and wrap.



3. Take to core box, rotate liner, half-round and core 180° and place in core box; then wrap "Saran" wrap over top of core. An additional half-round may then be placed over the zone to protect it, or to write on. Shear zone may be lifted out of box as a unit if waxing of sample is desired.

Figure 2-48.—Use of a cardboard PVC half-round and plastic wrap to prevent disturbance and drying out of shear zone, special samples, or fracture zones.

Based upon detection and measurement of these differences, geophysical surveys can often be designed to gather useful data to assist engineers and geologists in performing a more complete geotechnical investigation for civil engineering structures. Geophysical methods constitute only another exploratory tool to geological investigations, and they must never be regarded as anything more than tools. These methods will not disclose more than a good set of boreholes and drill holes—and usually not as much—and they should not be used without specific and constant correlation with geologic information. A preliminary geologic investigation is essential before geophysical methods can be applied, since they require knowledge of certain general conditions of local geology. The most favorable condition occurs when high contrasts between geophysical properties exist, such as when

Figure 2-49.—Arrangement of cores in a core box to ensure proper identification of samples.

rock underlies a shallow surficial deposit and the physical characteristics of the two are markedly different. Each type of geophysical survey has its capabilities and its limitations.

Geophysical methods are now frequently used for preliminary investigations at potential damsites and at proposed locations of other types of water resources structures. The most extensive use of geophysical methods in the practice of civil engineering has been in the United States where the larger federal engineering organizations have used these methods regularly in preliminary exploration work. All the methods are subject to definite geological restrictions; rocks of essentially different physical character must be in contact, and the strata encountered must be fairly uniform with respect to their physical character. Low-density strata overlain by high density strata cannot be detected by any of the surface geophysical methods, although they can be detected by some of the borehole methods. Finally, all information obtained as a result of geophysical investigations must be studied and used only when properly correlated by a specialist with the maximum information available regarding local geologic conditions. Despite these qualifications and necessary restrictions, geophysical methods are a powerful and useful tool.

The use of geophysics has increased when characterizing hazardous waste sites. Large areas can be tested to reduce the number of conventional borings and monitoring wells. Six methods are commonly used for ground-water contamination studies:

- resistivity,
- electromagnetic,
- refraction and reflection,
- magnetic,
- ground-penetrating radar,
- borehole geophysics [43].

In many cases, water of differing chemistry or the presence of buried objects, such as tanks or drums, makes geophysical methods especially effective. For example, gravity surveys, which are not especially useful in civil engineering applications, may be useful at a site that contains buried storage tanks of unknown location. Expert systems have been developed to provide data on which forms of geophysics may be effective at a contamination site [44].

A list of geophysical methods commonly used follows. These methods for performing geophysical testing are discussed in more detail in the *Engineering Geology Field Manual* [20].

Surface Geophysical and Other Techniques

1. Seismic Refraction
2. Seismic Reflection
3. Shear-Wave Surveys
4. Surface Wave Surveys
5. Vibration Surveys
6. Electrical-Resistivity
   Profiling
   Soundings
   Dipole-dipole
7. Electromagnetic Conductivity
   Profiling
   Soundings
8. Ground Probing Radar
9. Self-Potential Surveys
10. Magnetic Surveys
11. Gravity Surveys

Borehole Geophysical and Other Techniques

1. Electrical Logging
   Spontaneous potential
   Single-point resistivity
   Multiple electrode arrays
   Micrologging
   Induction logging
   Borehole fluid resistivity
2. Nuclear Radiation Logging
   Gamma ray
   Gamma-gamma logging
   Neutron logging
3. Acoustic/Seismic
   Acoustic velocity
   Acoustic borehole logging
   Crosshole seismic
   Tomography
4. Optical Borehole Logging
   Television camera
   Film camera
5. Borehole Caliper Logger
6. Borehole Fluid Temperature Logger
7. Borehole Gravity Logger

Tables 2-6 and 2-7 show application and limitations for use of some of these techniques. Table 2-6 shows surface methods, and table 2-7 shows borehole logging methods.

Correlations have been made between rock rippability and seismic wave velocity. Figure 2-50 shows an example of such correlations for heavy duty ripper performance (ripper mounted on tracked bulldozer). Charts similar to that shown in figure 2-50 are available from various equipment manufacturers but must be used with extreme caution.

Each of the geophysical tests listed above is discussed in greater detail in either the *Design of Small Dams* [16] or the *Engineering Geology Field Manual* [20]. Two more references containing excellent discussions on geophysical methods are Hunt [45] and Legget and Karrow [46].

2-21. Field Testing Methods.—

a. General.—Quantitative data can be obtained during standard penetration testing, and several other field tests can be used to obtain information concerning in-place subsurface conditions when exploring foundations. These include:

- permeability tests,
- in-place density tests,
- penetration tests,
- *in situ* strength and modulus tests,
- hand tests.

b. Field Permeability Tests.—Approximate values of permeability of individual strata penetrated by borings can be obtained by making water tests in boreholes. Reliability of values obtained depends on homogeneity of the stratum tested and on certain constraints of mathematical formulas used. However, if reasonable care is exercised in adhering to recommended procedures, useful results can be obtained during ordinary boring operations. Open end tests and packer tests in boreholes are described in USBR 7310, although open end tests are often of lower reliability. A well permeameter test used for estimating canal seepage losses is described in USBR 7300. Using the more precise methods of determining permeability by pumping from wells with a series of observation holes to measure drawdown of the water table (aquifer testing), or by pumping-in tests using large-diameter perforated casing, requires special techniques. Detailed information on field permeability testing methods can be found in the *Groundwater Manual* [47].

c. In-Place Density Tests.—The sand replacement method is used to determine in-place density in a foundation, a borrow area, or a compacted embankment by

Table 2-6.—Tools and methods for subsurface investigations

Method	Principle and application	Limitations
Surface seismic refraction	Determine bedrock depths and characteristic wave velocities as measured by geophones spaced at intervals.	May be unreliable unless velocities increase with depth and bedrock surface is regular. Data are indirect and represent averages. Limited to depths of about 30 m (100 ft).
High resolution reflection	Determine depths, geometry, and faulting in deep rock strata. Good for depths of a few thousand meters. Useful for mapping offsets in bedrock. Useful for locating ground water.	Reflected impulses are weak and easily obscured by the direct surface and shallow refraction impulses. Does not provide compression velocities. Computation of depths to stratum changes requires velocity data obtained by other means.
Vibration	Travel time of transverse or shear waves generated by a mechanical vibrator is recorded by seismic detectors. Useful for determining dynamic modulus of subgrade reaction for design of foundations of vibrating structures.	Velocity of wave travel and natural period of vibration gives some indication of soil type. Data are indirect. Usefulness is limited to relatively shallow foundations.
Uphole, downhole, and cross-hole surveys (seismic direct method)	Obtain velocities for particular strata; dynamic properties and rock-mass quality. Energy source in borehole or at surface; geophones on surface or in borehole.	Unreliable for irregular strata or soft soils with large gravel content. Cross-hole measurements best suited for in-place modulus determination.
Electrical resistivity surveys	Locate fresh/salt water boundaries; clean granular and clay strata; rock depth; depth to ground water. Based on difference in electrical resistivity of strata.	Difficult to interpret and subject to wide variations. Difficult to interpret strata below water table. Does not provide engineering properties. Used up to depths of about 30 m (100 ft).
Electromagnetic conductivity surveys	Measures low frequency magnetic fields induced into the earth. Used for mineral exploration; locating near surface pipes, cables, and drums and contaminant plumes.	Fixed coil spacings limited to shallow depth. Background noise from natural and constructed sources (manufactured) affects values obtained.
Magnetic measurements	Mineral prospecting and locating large igneous masses. Highly sensitive proton magnetometer measures Earth’s magnetic field at closely spaced intervals along a traverse.	Difficult to interpret quantitatively, but indicates the outline of faults, bedrock, buried utilities, or metallic objects in landfills.
Gravity measurements	Detect major subsurface structures, faults, domes, intrusions, cavities. Based on differences in density of subsurface materials.	Not suitable for shallow depth determination but useful in regional studies. Some application in locating caverns in limestone.
Ground-penetrating radar	Locate pipe or other buried objects, bedrock, boulders, near surface cavities, extent of piping caused by sink hole and leakage in dams. Useful for high-resolution mapping of near-surface geology.	Does not provide depths or engineering properties. Shallow penetration. Silts, clays, and salts, saline water, the water table, or other conductive materials severely restrict penetration of radar pulses.

Table 2-7.—Geophysical methods and techniques for logging boreholes

Method	Principle and application	Limitations
Electrical logging	Several different methods available. Provides continuous record of resistivity from which material types can be deduced when correlated with test-boring data.	Provides qualitative information. Best used with test-boring information. Limited to uncased hole.
Neutron radiation logging	Provides continuous measure of natural moisture content. Can be used with density probe to locate failure zones or water bearing zones in slopes.	Data from neutron probe is limited to in-place moisture content values. Often differs from oven-dried moisture content and requires correction.
Gamma-gamma logging	Provides continuous measure of in-place density of materials.	Data limited to density measurements. Wet density usually more accurate than dry density.
Scintillometer (Gamma ray logging)	Provides measure of gamma rays. Used to locate shale and clay beds and in mineral prospecting.	Qualitative assessments of shale or clay formations.
Acoustic borehole imaging	Sonic energy generated and propagated in fluid such as air to water. Provides continuous 360° image of borehole wall showing fractures and other discontinuities. Can be used to determine dip.	Must be used in fluid-filled borehole unless casing is being inspected. Tool must be centered in the borehole. Logging speed is relatively low between 20 and 75 mm/s (4 and 15 ft/min). Images less clear than those obtained with borehole cameras.
Acoustic velocity logging	Can determine lithologic contacts, geologic structure, cavities, and attitude of discontinuities. Elastic properties of rock can be calculated. Compression (P-water) is generated and measured. Used almost exclusively in rock.	Borehole must be fluid filled and diameter accurately known. Penetration beyond borehole wall of about a meter or so. Geologic materials must have P-water velocities higher than velocity of the borehole fluid.
Crosshole seismic tests	Seismic source in one borehole; receiver(s) at same depth in second (or more) borehole(s). Material properties can be determined from generated and measured compression and shear waves. Low velocity zones underlying high velocity zones can be detected.	Borehole spacing is critical and should be >3 m and <15 m. Precise borehole spacing must be accurately known for data to be useful.
Borehole cameras	Borehole TV or film type cameras available. TV viewed in real time. Can examine cavities, discontinuities, joints, faults, water well screens, concrete-rock contacts, grouting effectiveness, and many other situations.	Requires open hole. Images are affected by water clarity. Aperture on film camera must be preset to match reflectivity of borehole wall materials.
Borehole caliper logging	Used to continuously measure and record borehole diameter. Identify zones of borehole enlargement. Can evaluate borehole for positioning packers for other tests. One to six arm probe designs.	Diameter ranges from about 50 to 900 mm (2 to 36 in). Must calibrate caliper against known minimum and maximum diameter before logging. Special purpose acoustic caliper designed for large or cavernous holes (dia.) 1.8 to 30 m (6 to 100 ft).
Temperature logging	Continuous measure of borehole fluid temperature after fluid has stabilized. Can determine temperature gradient with depth.	Probe must be calibrated against a fluid of known temperature. Open boreholes take longer to stabilize than cased holes. Logging speed 15 to 20 mm/s (3 to 4 ft/min).

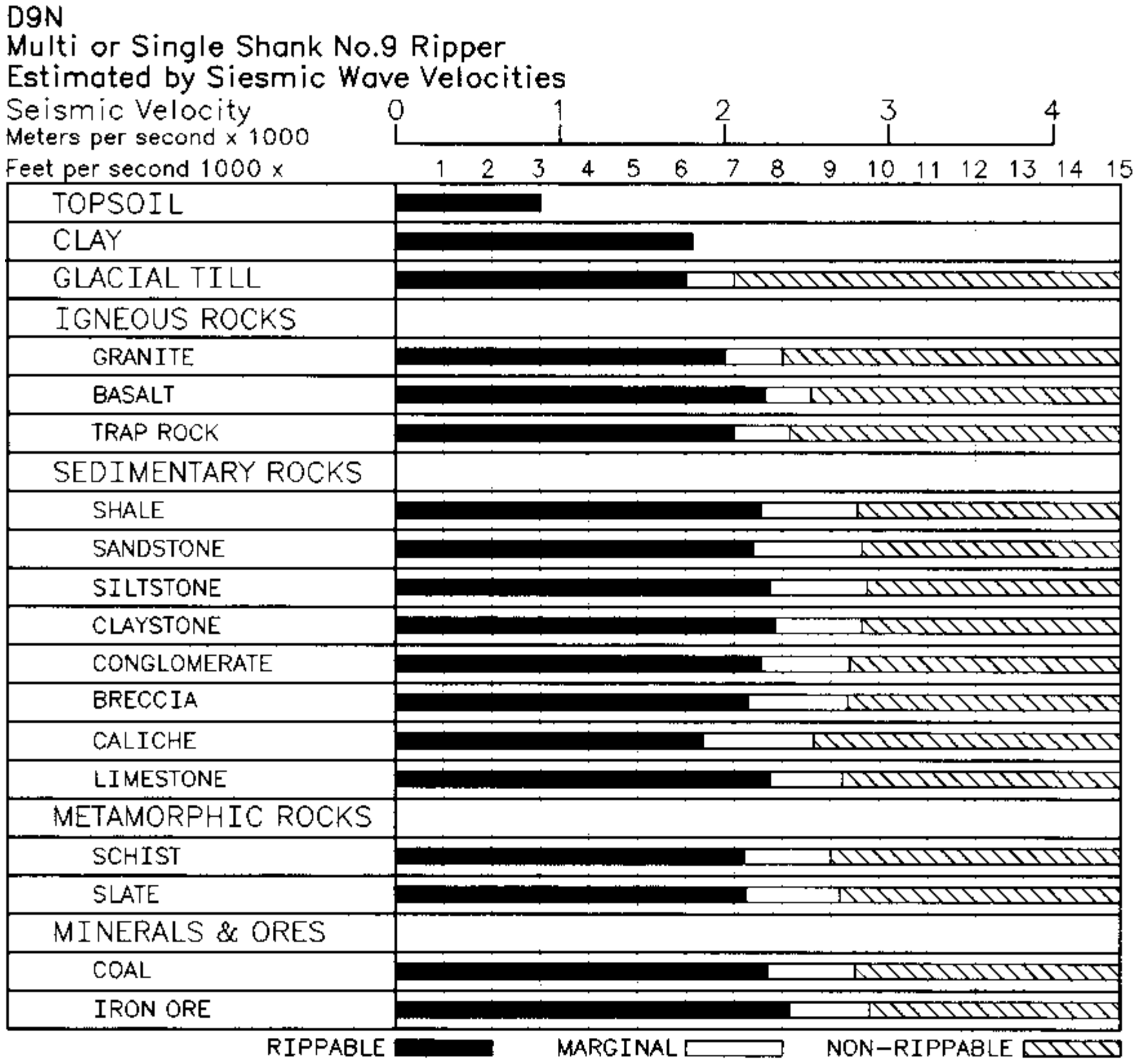


Figure 2-50.—Rock rippability as related to seismic P-wave velocities. (Caterpillar Tractor Co.)

excavating a hole from a horizontal surface, determining the mass of material excavated, and determining the volume of the hole by filling it with calibrated sand. Water content of soil excavated from the hole is determined, and in-place dry density is calculated. In-place density data are often used in calculations to determine shrinkage or swell factors between borrow area excavation and compacted embankment volumes. In-place density tests can be performed in shallow foundations to evaluate bearing capacity for small structures. Various devices using replacement methods (i.e., water, balloons and water, oil) have been used to measure the volume of the test hole. Indirect methods such as the nuclear moisture and density gauge (USBR 7230) are also used to determine in-place density, but the sand replacement method is most common. USBR 7205 and 7220 describe sand density methods to be used depending upon gradation of material being tested and consequent size of the required excavation. If the soil to be tested contains appreciable coarse gravel and cobbles, in-place density is normally determined by use of test pits with water replacement (USBR 7221). These large scale tests are expensive; normally, they only are performed when exploring critical coarse grained borrow materials and for construction control of rockfills and coarse shells of dams.

The determination of in-place dry density and water content of a fairly deep foundation is often necessary. In this case, the sand density method requires excavating a deep test pit or test trench to gain access to the soils to be tested. Excavation of deep test pits or accessible shafts can be hampered by the presence of ground water, and special safety precautions are required. Most deep test shafts are reserved for critical explorations of coarse and cohesionless soils. Mechanical drilling and sampling methods have mostly replaced deep test shafts. Mechanical drilling and sampling, if performed correctly, can provide reliable information on in-place moisture and density of deep foundation soils. Detailed procedures are given in USBR 7105 for determining density of undisturbed tube samples. These data can be reliably obtained in the field and are extremely useful for design purposes. If mechanical drilling and sampling is performed, it is strongly recommended that average tube density be determined.

d. Penetration Testing.—

*1. General.—*Data obtained from quasi-static or dynamic penetration tests can be used to estimate engineering properties of soils and to delineate subsurface stratigraphy. The most common tests employed are standard penetration and cone penetration tests [SPT and CPT tests (USBR 7015, 7020 and 7021)]. By evaluating resistance to penetration during the tests, engineering properties of compressibility and shear strength can be estimated. Also, test results can be compared and correlated to properties obtained from laboratory tests on undisturbed soil specimens. In many cases the number of undisturbed sampling holes may be reduced in favor of less expensive penetration tests to better interpret complex site stratigraphy.

*2. Penetration Resistance and Sampling of Soils.—*The SPT test consists of determining the number of blows *N*, of a 64 kg (140 lbm) hammer dropping 762 mm (30 in), required to produce 300 mm (1 ft) of penetration of a standard 50-mm (2-in) o.d. sampler into soil at the bottom of a prebored drill hole after the sampler is seated an initial 150 mm (6 in) (fig. 2-51). The *N* value can be used to estimate strength and compressibility of sands, consistency of clays, and bearing capacities. A disturbed sample is retrieved in the 35-mm (1-3/8-in) i.d. sampler barrel which allows for inspection, visual classification, and moisture content determination (fig. 2-52). Reclamation specifies a barrel with constant inside diameter of 35 mm (1-3/8 in), but most SPT barrels manufactured in the United States today are upset walled to 38-mm (1-1/2-in) diameter to accommodate liners. The difference in SPT *N* value can vary 10 to 30 percent between different barrels [48]. If critical studies are performed, the inside diameter of the barrel must be known so corrections can be made. Laboratory soil classification tests (USBR 5000) also can be performed on the soil sample. Normally, penetration tests are performed at 1.5 m (5 ft) intervals or with changes in soil strata. For critical investigations, such as foundations for dams, testing may be performed more frequently, but normally the minimum testing interval is 0.75 m (2.5 ft) to prevent disturbance of soil to be tested in the next interval. The penetration resistance test is widely used to evaluate foundation conditions for investigations requiring drill holes.

Penetration resistance obtained during field testing is inversely proportional to the energy delivered to the sampler system by the driving system. Research has shown that the wide variety of equipment available and in use in the United States results in large variations in energy delivery and consequently in *N* (blow count) values [48]. In USBR 7015, procedures require the use of safety hammers with the rope and cathead method. The drill rod energy delivered by safety hammers ranges from 60 to 70 percent of free fall hammer energy and is now considered the standard target range for SPT in the United States. Energy delivered by automatic and manual trip hammer systems is more reproducible, and possibly higher, because the influence of rope condition, method of rope release from the cathead, and friction in the mast sheave are all bypassed. Drill rod energy transmission ranges from 90 to 95 percent for those automatic trip hammers for which energy measurements have been made. Using automatic hammer systems enhances test reproducibility. These hammer systems can, however, only be approved for use after energy delivery and transmission characteristics are known. On critical programs, such as liquefaction studies, adjustment for higher energy of automatic hammers may be required [48].

Numerous studies have been performed that indicate the impossibility of obtaining a truly undisturbed sample of clean loose sand using thin wall tube and thin wall piston samplers [49]. Volume changes can occur in the clean sands during pushing because of the pervious nature of the material. As a result, there is increased reliance on empirical correlations to estimate engineering properties of sands. Figure 1-15 in chapter 1 shows results of Reclamation's research for estimating the relative density of medium to fine sands in relation to SPT blowcounts. Studies performed by the Corps of Engineers produced additional data on the factors affecting penetration resistance in sands. The Reclamation chart can be used with sufficient accuracy for evaluating foundation conditions for most small structures. Strength and compressibility can be estimated from knowledge of relative density.

Penetration resistance tests are also used to evaluate the liquefaction resistance of sands during earthquakes. The method consists of comparing site data to data from a compilation of case histories of liquefaction occurrence. Case history data are available for performance of silty

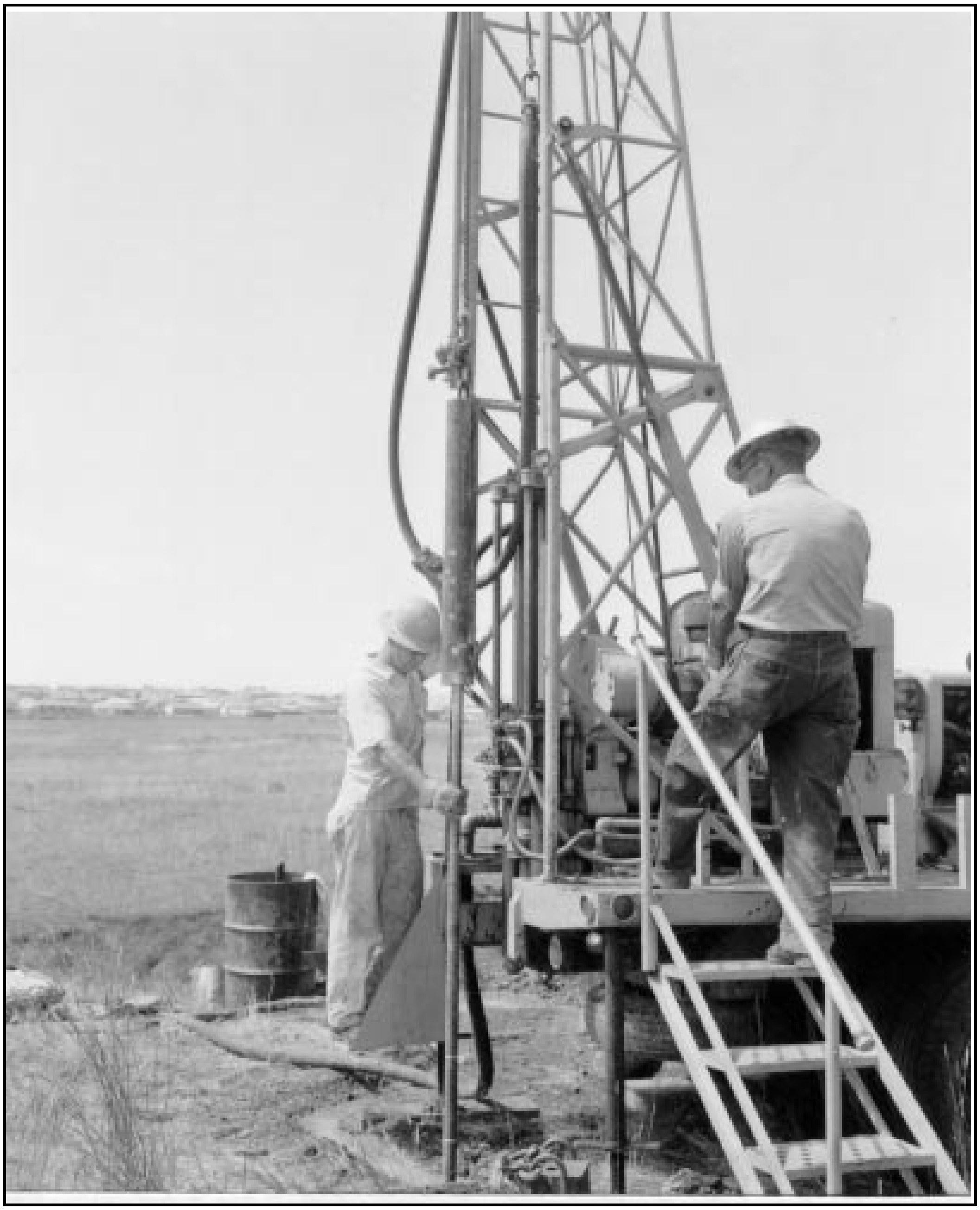
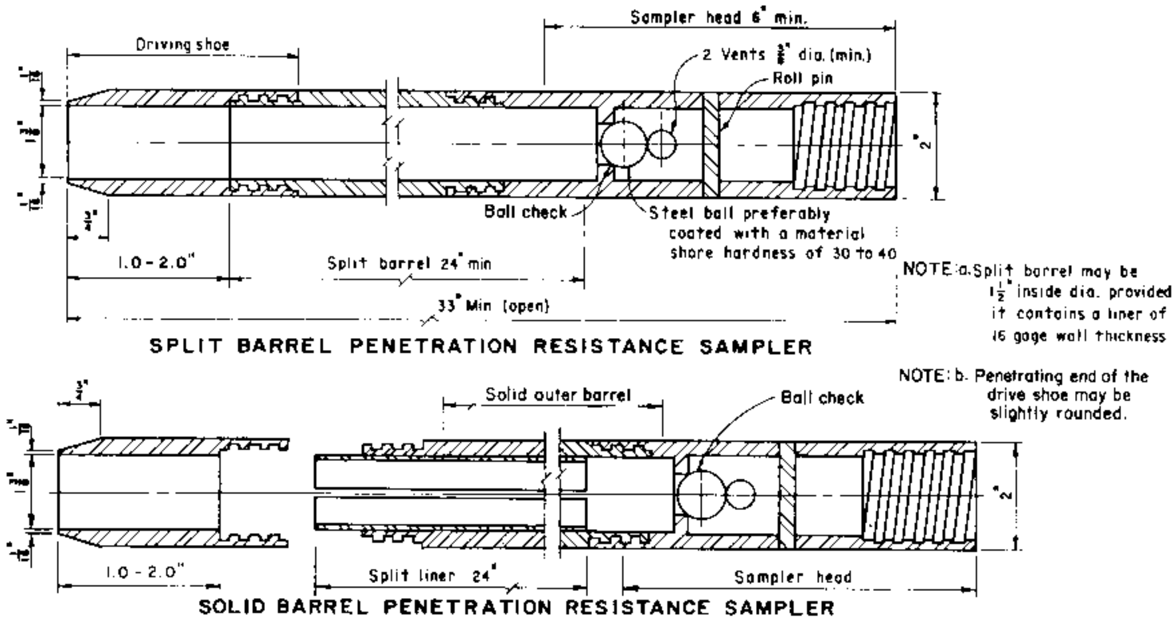


Figure 2-51.—Standard drilling equipment used to perform penetration tests.

sands during cyclic loading. Liquefaction testing is normally associated with drilling, testing, and sampling of loose clean sands and silty sands that must be carefully drilled. If procedures are not taken to stabilize the loose deposits, data will be unreliable. Normally, rotary drilling methods with drill muds are required to stabilize the boring. The primary cause of disturbance of sands is hydrostatic imbalance. To maintain hydrostatic balance, the boring must be kept full of drill fluid during drilling and by use of a pump bypass during removal of cleanout string and testing.

Penetration resistance data obtained with the 50-mm (2-in) diameter sampler are not reliable in gravelly soils. Research on penetration resistance of gravelly soils has been performed using larger samplers of 75 to 90 mm (3 to 3 1/2 in) o.d. along with larger hammers, but these tests are nonstandard. Recovery of coarse-grained soils and accuracy of blowcount data may be improved using the larger sampler



METRIC EQUIVALENTS

IN.	mm.	IN.	mm.
1/16 (16 gage)	1.5	2	50.8
1/2	12.7	3	76.2
3/4	19.0	6	152.4
7/8	22.2	24	690.6
1 3/8	34.9	33	838.2
1 1/2	38.1		

Figure 2-52.—Sampler requirements for penetration resistance testing.

barrels; however, the analysis of data obtained with this equipment and procedures should only be used with engineering supervision and oversight. If engineering properties of coarse-grained soils must be determined, it is recommended that accessible borings, in-place density tests, Becker penetration tests, and shear-wave velocity tests be performed.

Penetration resistance data can be used to estimate the relative consistency of cohesive soils as shown on table 1-1 (ch. 1). Moisture content data from SPT testing can be used in conjunction with laboratory data from undisturbed samples to evaluate engineering properties at critical sites. Vane shear, borehole shear, cone penetration, flat-plate dilatometer, and pressuremeter tests are better suited for determining undrained shear strength of medium to very soft cohesive soils than is the standard penetration test.

e. Becker Penetration Test.—

*1. Origin of Becker Penetration Test.—*The Becker penetration test is performed with the Becker hammer drill, a rugged and specially built hammer-percussion drill. The drill is used in geotechnical investigations for drilling, sampling and penetration testing in coarse-grained granular soils. The drill uses a double-acting diesel pile hammer to drive a specially designed double-walled casing into the ground. The casing comes in 2.4- or 3.0-m (8- or 10-ft) lengths and is available in three standard sizes: 140 mm (5 1/2 in) O.D. by 83 mm (3 1/4 in) I.D., 170 (6 3/4 in) O.D. by 110 (4 1/4 in) I.D., and 230 mm (9 in) O.D. by 150 mm (6 in) I.D. The main advantage of the Becker hammer drill is its ability to sample or penetrate cobbles and boulder deposits at a fast rate and provide a penetration test similar to SPTs. The heavy-walled Becker casing is strong and can break up and penetrate boulders and weak bedrock [50].

*2. Open-Ended Becker Drilling Method.—*The Becker casing can be driven open-ended with a hardened drive bit for drilling and sampling, in which compressed air is forced down the annulus of the casing to flush the cuttings up the center of the inner pipe to the surface. The continuous cuttings or soil particles are collected at the ground surface via a cyclone. At any depth, the drilling can be stopped, and the open-ended casing allows access to the bottom of the hole for tube sampling, standard penetration test or other in-place test, or for rock coring to be conducted. On completion of drilling, the casing is withdrawn by a puller system comprising two hydraulic jacks operating in parallel on tapered slips that grip the casing and react against the ground.

3. Close-Ended Becker Drilling Method.—The Becker casing can also be driven close ended, without using compressed air, to simulate the driving of a displacement pile. The idea is to drive the Becker casing close-ended like a pipe pile and use the recorded blow counts [blows per 0.3 m (1 ft)] to indicate soil density. The Becker denseness test, or the Becker penetration test (BPT), is generally less sensitive to gravel particle size than the SPT because of the larger Becker pipe [140 mm (5 1/2) O.D. and larger] compared to the SPT sampler [51 mm (2 in) O.D.], and has, therefore, been found to be useful as an indicator of density in gravelly soils. As a result, the BPT can be used for pile driveability and pile length evaluation, as well as for foundation design, usually through correlations with the SPT [51]. The BPT is also becoming accepted as a practical tool for liquefaction potential assessment in gravelly sites, again through correlations with the SPT [52, 53, 54].

4. Interpretation of BPT and SPT Data.—In order to make use of the large worldwide foundation performance data base currently available for the SPT, such as the SPT-based liquefaction data base, there is a need for reliable BPT-SPT correlations. Numerous attempts have been carried out in the past to correlate the BPT blow counts to the SPT N-values. Most of these correlations, however, have limited applications since they do not take into account two important factors affecting the BPT blow counts: the variable energy output of the diesel hammer used in the Becker system and the soil friction acting on the Becker casing during driving.

To overcome the variable hammer energy problem in the BPT, a method was proposed of using the measured peak bounce-chamber pressures at the top of the double-acting diesel hammer to correct the measured blow counts to a so-called "constant full combustion condition" [52]. The corrected BPT blow counts can be correlated to corrected SPT blow counts. Although an improvement over existing BPT-SPT correlations, this method still has serious limitations. The bounce-chamber, pressure-correction method cannot capture all the important variables affecting the BPT blow count, and the BPT-SPT correlation does not consider casing friction in the BPT [55].

f. Cone Penetration Tests.—Cone Penetration testing was developed in the Netherlands for rapid penetration testing of low density estuarine deposits. The procedure consists of measuring the resistance of a 60° apex angle cone, with 1000 mm^2 projected surface area, being pushed into the soil at a controlled rate of 20 mm/s. During most cone tests the force exerted on a cylindrical friction sleeve located behind the tip is also measured. Procedures for mechanical and electric cone penetration tests are given in USBR 7020 and 7021. Cone testing is performed for rapid delineation of site stratigraphy and estimation of strength, compressibility, pile capacity and liquefaction resistance.

With the introduction of electronic cone penetrometers, penetration resistance data can be collected at intervals as close as 20 mm (3/4 in) to provide an excellent record of subsurface stratigraphy. As shown on the example, figure 2-53, drained behavior in cohesionless deposits results in significantly higher cone tip resistance than in undrained clays. The resulting data trace accurately delineates bedding in layered deposits. The bottom layer, of the example figure, is a lacustrine deposit under a dam where seasonal gradational differences in the deposit are delineated with the cone. Additional sensors can be added to the cone body to obtain records of:

- pore pressure,
- cone inclination,
- temperature,
- electrical resistivity or conductivity,
- seismic wave velocity by use of geophones.

Using the piezocone, soundings can be stopped in sand layers and hydrostatic pressures can be determined.

Advantages of cone testing include the rapid rate of testing, ability to obtain excellent soil stratigraphy data, and repeatability of test data. Disadvantages of cone penetration testing are the inability to obtain a soil sample and to test firmer soils because of the large thrust required to push the cone into the soil. On favorable sites, having good access and moderate to soft soil conditions, production can reach 60 to 120 meters per day (200 to 400 ft/d) with resulting minimal exploration costs per meter of sounding. Error in test data is slight if testing is performed according to USBR 7020 and 7021. On investigations for significant structures, such as dams, the inability to obtain samples requires that site-specific correlation be performed with undisturbed sampling. If the cone is used, the number of expensive sampling holes may be reduced. Another disadvantage of cone penetration soundings is that backfill grouting of cone holes is difficult and requires specialized equipment. Backfilling may be required when sounding through water retaining embankments and hazardous waste sites.

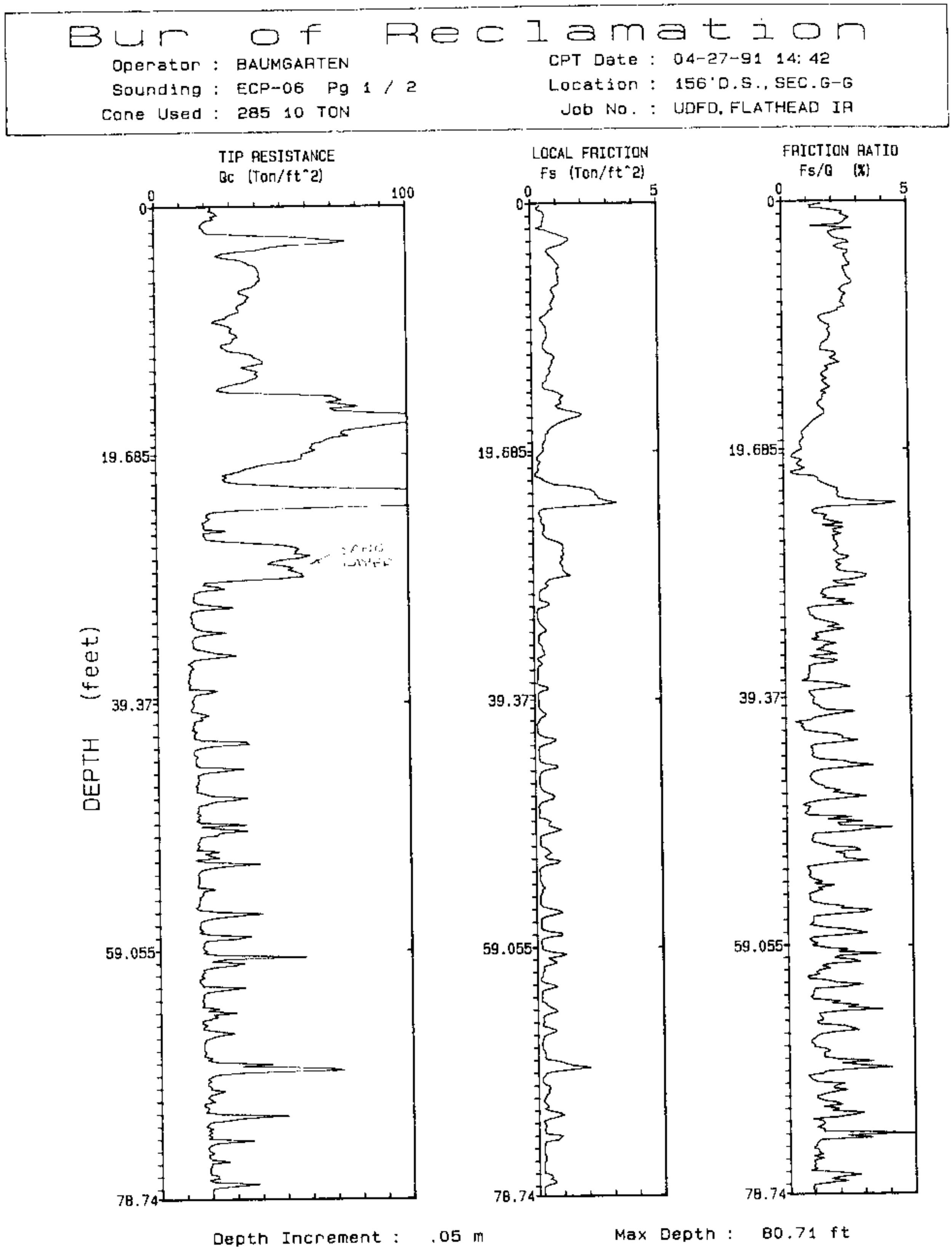


Figure 2-53.—Example of electronic cone penetrometer data.

Reclamation uses both mechanical and electrical cone adaptor kits for drill rigs, as well as an 18-metric-ton (20-ton), cab-mounted testing vehicle shown on figure 2-54. The all-wheel-drive, 18-metric-ton, gross-weight vehicle can advance cones through harder and/or coarser deposits and to greater depths than conventional drilling rigs, which can only develop 4.5 to 9 metric tons (5 to 10 tons) of reaction. Cone penetration testing is limited by thrust capacity and by durability of the rod and cone system. Lithified deposits and very firm to hard cohesive soils cannot be readily penetrated. Dense to medium dense gravel usually impedes penetration, but loose silty gravels have been penetrated. The enclosed truck-mounted cone testing rig allows for testing in difficult weather conditions. Data acquisition, reduction, and calibration equipment are all contained on board and allow for onsite data interpretation. A disadvantage of the large 18-metric-ton truck is access in soft ground surface conditions. If trafficability problems exist, all-terrain equipped drill rigs or trailer-mounted drill rigs using tie down augers for vertical reaction can be used. Conventional drill rigs used for cone testing should have a hydraulic feed system to ensure smooth, steady, quasi-static advance of the cone.

Primary use of cone penetration testing in Reclamation is for stratigraphic mapping. Cone testing is performed to acquire data in between conventional drilling and sampling holes. Correlations between cone data and soil classification, and strength and compressibility of sands and clay, have been developed for both mechanical and electrical cones [56, 57].

In clean sands, design methods for evaluating settlement of footings using cone data are accepted as one of the most reliable techniques. Reclamation has found that some degree of site specific data analysis is required to refine some of these correlations to achieve sufficient accuracy for design data purposes. For example, when evaluating undrained shear strength of clay, it is required that other field or laboratory tests be performed because the cone bearing capacity factor, *Nk*, varies widely. Recent researchers have focused on using dynamic pore pressures from the piezocone to refine these correlations. However, piezocone testing requires additional time and effort to prepare and calibrate the cone and prevent cavitation of the element. Cavitation of the saturated pore pressure measuring element is a problem when the sounding must first penetrate unsaturated zones or sands near the surface which dilate and cavitate the system.

g. Dynamic Penetrometers.—In Europe, dynamic penetrometers are frequently used for performing rapid soundings to develop site stratigraphy. Many different standards exist, depending on the country of use.

Procedures consist of driving a cone pointed penetrometer with repeated hammer blows and counting the number of blows to advance the point a given distance. Most dynamic penetrometer systems are portable and use automatic means to lift and drop the hammer, thereby reducing variability in the test method as well as experience and training required for operators. Dynamic soundings provide an economic means to perform investigations at close spacings. Most penetrometers can penetrate gravelly soils. Reclamation has not used dynamic penetrometers to a significant degree.

Figure 2-54.—Electronic cone penetrometer testing vehicle.

h. In Situ Strength and Modulus Tests.—General.—Many new methods of *in situ* load testing are being studied in an effort to improve prediction of engineering properties of soil and rock in place. The term "*in-situ*" is often used to describe these tests in geotechnical engineering and is used here to signify that the test actively loads the surrounding soil. Elsewhere in this manual, the term "in place" is used to describe in-place conditions.

In most of the *in situ* tests, loads are applied to the soil, and deformation or shearing resistance is measured. Most methods depend on carefully preparing a prebored drill hole to minimize borehole wall disturbance. In many cases, several different drilling techniques will need to be attempted prior to successful operation of the test. Testing specialists need to participate in the early phases of planning and testing. Even with increased cost of additional efforts in drilling, savings may be realized in the overall exploration program through reduction in laboratory or other *in situ* testing. The advantage of *in situ* tests is that the soil is tested at existing stress conditions; whereas in laboratory testing, detrimental effects of unloading (stress relief) and reloading must be considered. In soft weathered rocks and friable deposits, sometimes it is impossible to recover samples for laboratory specimens, and properties of these low recovery zones are often most critical to design of the structure. *In situ* tests offer the means by which engineering properties may be obtained in the absence of available samples for laboratory testing.

i. Field Vane Tests.—The vane method of testing is widely accepted for determining undrained shear strength of soft to very soft consistency clays. The test consists of pushing a four-bladed vane into the soil at the bottom of a borehole and measuring the rotational torque required to rapidly fail the soil. Equipment and procedures for performing the vane shear test are described in USBR 7115. Even though the vane shear test can be used to delineate stratigraphy, it may be more economical to perform cone penetration or flat-plate dilatometer testing for this purpose. In most vane shear testing, a remolded strength determination is also made to assess sensitivity of the clays. Determination of sensitivity of clays is difficult in the laboratory and is best determined using field vane shear testing.

Vane shear testing requires a rotary drill rig to drill and clean the boring and to press the vane into undisturbed soil. Reclamation has modified the **BX** casing housing of the vane and rods to act as a rotary drill string. During drilling, the vane is retracted inside the **BX** cutting bit. This modification reduces hole disturbance in very soft clays and precludes the need to withdraw the entire drill string and lower it again every time a vane test is performed. Determination of rod friction is easily accomplished using Reclamation's vane shear device which is equipped with a special vane slip coupling system.

j. Pressuremeter Testing.—Pressuremeter tests are performed to characterize the stress-strain behavior of soil and rock in place. Tests are performed by measuring the pressure and resulting volume change increments of a cylindrical expandable measuring cell which is inserted into a carefully prepared borehole (fig. 2-55). The measuring cell or probe is typically inserted into a prebored drill hole; but in some instances, it is advanced by self-boring or driving. Reclamation test procedures are still under development, but satisfactory procedures for interim use are given in ASTM procedure D 4719 [59].

Pressuremeter tests can be used to determine the in-place modulus of deformation of a wide variety of soils and rock for which undisturbed sampling and laboratory testing are difficult. The data are primarily used to evaluate anticipated structural settlements. Usually, testing is performed with pressure increments applied at 1 minute intervals, which results in undrained loading of fine-grained soils. Undrained strength of cohesive soils can be estimated from the limit pressure (pressure at infinite expansion), but requires extrapolation of test data. The pressuremeter is the only reliable method of determining the in-place friction angle of sands using stress-dilatency theory. However, use of a self-boring pressuremeter is required in saturated loose sands to minimize drilling disturbance.

Rock pressuremeters (dilatometers) with maximum pressure capability approaching 35.5 MPa (5,000 bf/in^2) can be used to evaluate deformation modulus in hard rock to soft rock and in areas of low recovery. Strength interpretation of both hard rock and softer rock is difficult, highly theoretical, and requires interpretations of stress-strain paths. Strength test data interpretation in cemented soils and soft or fractured rock is especially difficult. As a result of these difficulties, Reclamation still uses the rock dilatometer primarily to evaluate deformation modulus, and other field and laboratory tests are performed to determine strength.

Pressuremeter testing has been performed in gravelly soils by driving a slotted **AW** casing to protect the pressuremeter, but effects of driving displacements in gravel are not fully understood.

Pressuremeter testing in soils requires exacting execution of drilling to: (1) minimize disturbance of the borehole wall and (2) provide a borehole diameter within allowable tolerance. Test procedures require that the borehole be no

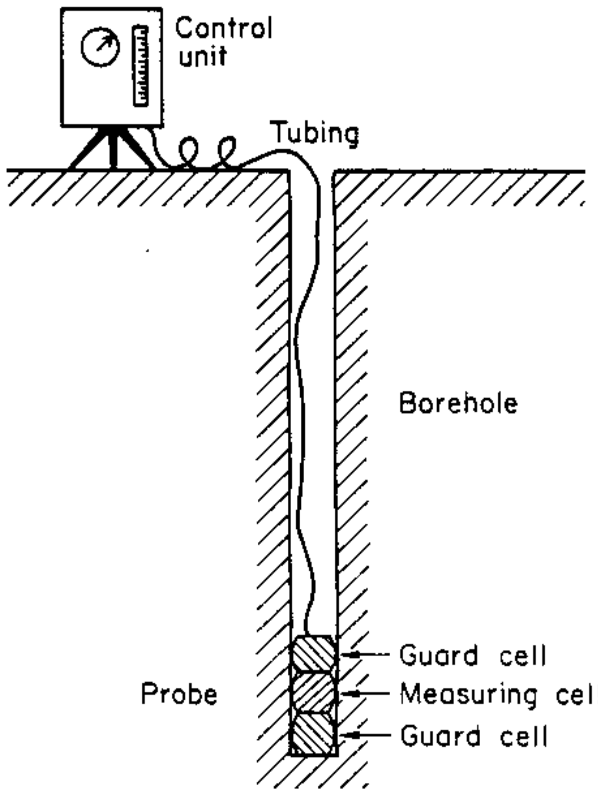


Figure 2-55.—Menard pressuremeter equipment. (Baguelin, Jézéquel, and Shields [58])

smaller than 97 percent of probe diameter or 20 percent greater than probe diameter when testing soils. Even tighter borehole diameter requirements are required for rock dilatometers. Since testing is typically performed in materials which are difficult to sample, a variety of drilling methods may need to be attempted to achieve success. In very soft clays and loose saturated sands the self-boring pressuremeter is recommended. Two self-boring pressuremeters are shown in figure 2-56. A recent trend in self-boring pressuremeters is using jetting instead of rock bitting for hole advancement. If jetting is used, the pressuremeter body should be equipped with fluid pressure sensors to ensure that circulation is maintained. Wall disturbance can be evaluated from the pressure-volume data. Testing duration can be up to 1 to 2 hours per interval. Although pressuremeter testing is one of the more expensive in-place tests, the testing is still performed at less cost than obtaining undisturbed samples and performing laboratory testing.

k. Soil and Rock Borehole Shear Testing.—Borehole shear testing is performed for rapid determination of shearing resistance. The test consists of lowering two diametrically opposed shear plates into a borehole, applying a normal force to seat the plates into the borehole sidewalls, and applying shearing force to the plates by vertically pulling on rods connecting to the ground surface. A schematic of the equipment is shown in figure 2-57. The soil borehole shear test was developed by Prof. Handy of Iowa State University. Later, under a Bureau of Mines research program, a rock borehole shear device was developed with capability for exerting normal forces of 35 MPa (5,000 lbf/in^2). Testing procedures for rock borehole shear devices are under development by Reclamation and the American Society for Testing Materials.

Borehole shear testing requires careful drilling to minimize disturbance to the borehole walls and to ensure appropriate borehole diameter. Usually, for rock borehole shear testing, **NX** diameter boreholes should not exceed 83 mm (3.25 in) in diameter. Shear stress and normal stress data can generally be obtained at three normal stresses at one location in the borehole within 30 minutes to 1 hour and results in significant cost savings over laboratory testing.

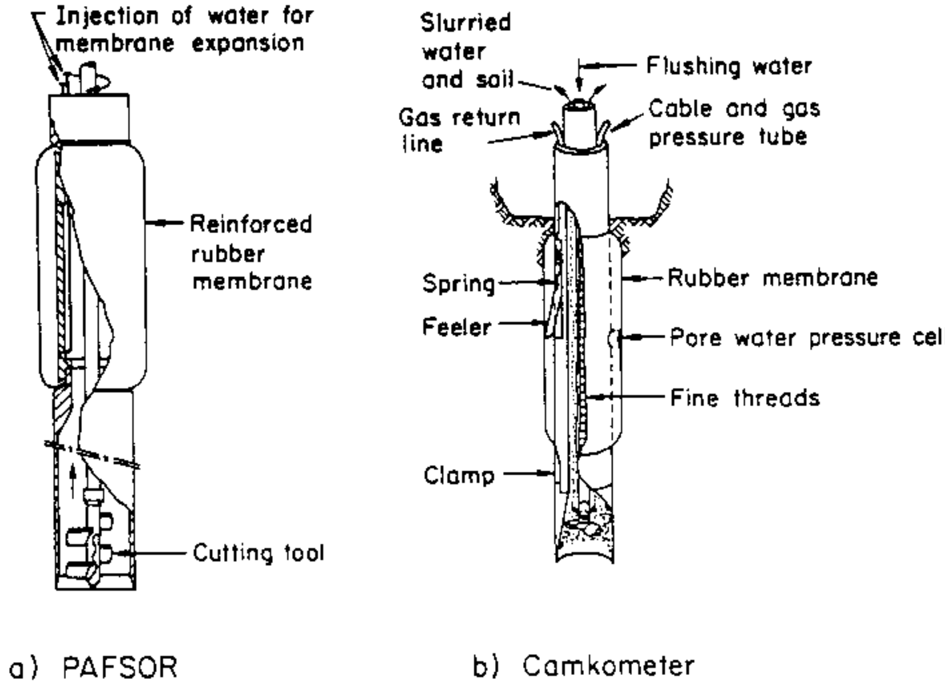


Figure 2-56.—Self-boring pressuremeters. (Jamiolkowski, Ladd, Germaine, and Lancellotta [60])

Strength interpretation of data from the borehole shear test in soils is complicated by the need to evaluate loading rate and resulting drainage conditions. A pore pressure sensor has been added to the shear plates to provide data on consolidation and drainage, so application of load can be evaluated. Borehole shear testing is not recommended for cohesionless soils or soft to very soft cohesive soils because of the difficulty in borehole preparation. Gravel particles in soils may result in invalid test results because of nonuniform stress distribution around the plates. Testing is most successful in firmer cohesive deposits, which are easily sampled for laboratory testing; as a result, testing in soils is not performed often.

Borehole shear testing of rock is economical and provides accurate test data [62]. It is a viable alternative to more expensive laboratory testing. Testing has been performed on a wide range of rocks including lightly cemented sandstones and claystones, and of coals to hard granites.

Laboratory shear strength tests tend to overestimate strength values, as only cores with sufficient length to diameter ratios are tested. Borehole shear testing permits testing of low sample recovery zones that are often the most critical for design. The normal force plates of the rock testing apparatus allow testing at three normal stresses at a single elevation in the borehole by rotating the shear head to an untested area of the borehole. Testing intervals as close as 50 mm (2 in) in elevation may be tested, depending on materials. The close spacing allows testing of thin seams and shear zones.

l. Flat Plate Dilatometer Test.—The flat plate dilatometer test (DMT) is performed to determine compressibility and shearing strength of a wide range of in-place soils [63]. The test requires advancing (by pushing) a blade-shaped penetrometer with an expandable membrane (fig. 2-58) into the soil using a drilling rig or cone penetration equipment. After pushing the dilatometer to the desired depth, the membrane is pneumatically expanded, and pressures are recorded at two positions of membrane deflection. Tests are normally performed at 200-mm (8-in) depth intervals. Testing is rapid and repeatable with simple equipment and operation. By using the expanding membrane, the instrument can more accurately determine soil compressibility than cone penetration testing and with similar economy in exploration costs. Testing procedures are under development by American Society for Testing Material, but are well established by more than 10 years of engineering experience [65]. The DMT testing can provide an estimate of soil type based on differences in the two deflection pressures. The pressure difference reflects drainage conditions influenced by permeability and coefficient of consolidation. With addition of thrust data to

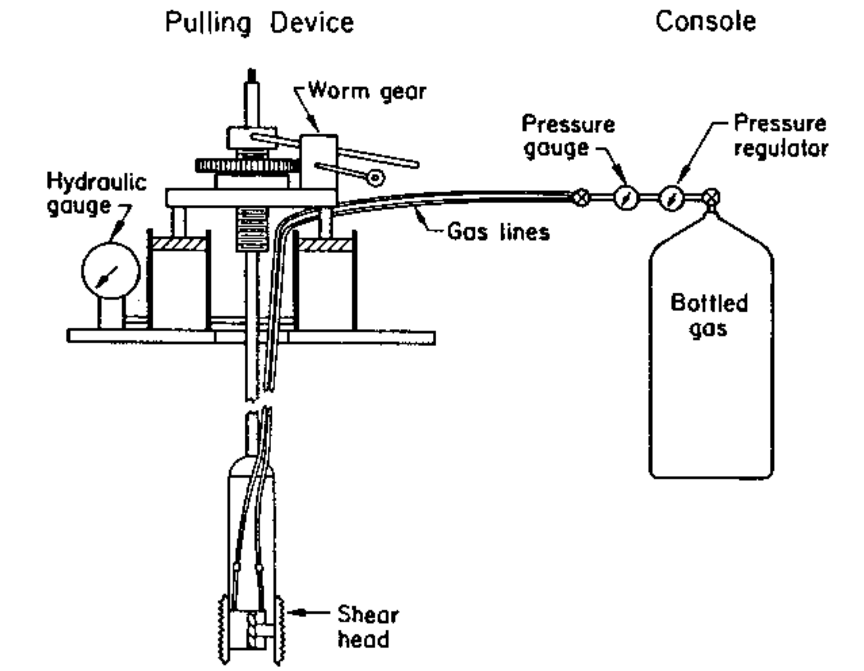


Figure 2-57.—Borehole shear device. (Wineland [61])

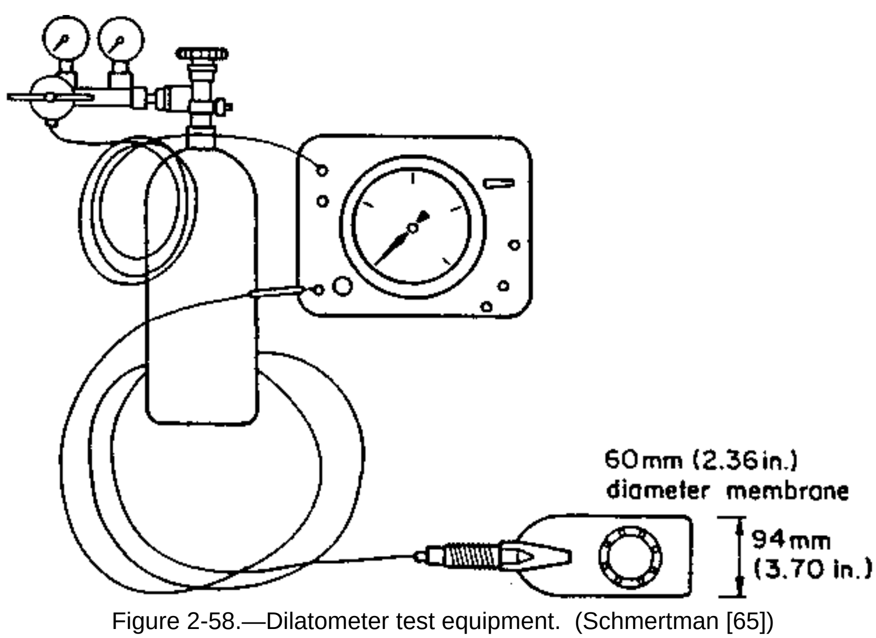


Figure 2-58.—Dilatometer test equipment. (Schmertman [65])

advance the dilatometer, the DMT provides data for defining subsurface stratigraphy similar to cone penetration testing.

The flat plate dilatometer provides data to accurately evaluate compressibility in normally consolidated clays and sands. Undrained strength in normally consolidated to lightly overconsolidated clays is equivalent to uncorrected vane shear strengths making the DMT an economical alternative or supplement to vane shear testing. DMT data can be used to estimate stress history and changes in stress history. Field studies have been performed in overconsolidated deposits to provide empirical correlations to engineering behavior. Prediction of friction angle in sands is performed using wedge penetration theory and requires measurement of thrust required to push the dilatometer into the ground. Since thrust is typically measured at the surface, rod friction limits depths in which friction angle can be accurately predicted.

Using 18-metric-ton (20-ton), truck-mounted, cone penetration testing equipment, the DMT can be pushed quasi-statically through soils with SPT *N* values less than 30. Stronger soils require preboring using conventional drilling equipment. The DMT testing is not suitable for testing gravelly soils. Testing rates of up to 50 meters per day (165 ft/d) can be obtained. Operation and maintenance of the testing equipment is simple.

m. Pocket Vane Shear Test.—The hand held vane shear testing device (Torvane) is used for rapid evaluation of undrained shear strength of saturated cohesive soils. It can be used on tube-type samples, block samples, or on sides of test pits in the field. Values obtained from these tests are sometimes useful to assist in planning laboratory or field investigations. The Torvane is designed for use on saturated, fine-grained cohesive soils. Torvane tests should always be performed in the ends of thin wall samples of soft clays in the field. The data regarding consistency is of great assistance in selecting laboratory tests to be performed on clay samples. The presence of coarse sand or gravel in test specimens could result in erroneous values of undrained shear strength of soil. If the test specimen contains coarse-grained material, the Torvane should not be used, and other shear strength measuring techniques should be performed. The device is shown on figure 2-59.

The Torvane device is a small vane shear meter capable of measuring shear strengths between 0 and 2.5 kg/cm^2 (2.5 ton/ft^2). The Torvane dial is marked with major divisions in units of 0.05 kg/cm^2 (0.05 ton/ft^2) to permit visual interpolation to the nearest 0.01 kg/cm^2 (0.01 ton/ft^2).

To perform the test, a flat area on the side of a test pit or undisturbed sample is selected. A vane size is selected which is appropriate for the material being tested. Several trials may be necessary to determine the correct vane size. Too small a vane will produce dial readings of insufficient magnitude to register on the dial head, while too large a vane will produce readings too great for the scale. A larger vane adapter is shown on figure 2-59. The vane is carefully pressed into the cohesive soil to the depth of the vane blades. The Torvane knob is turned while a constant normal load is kept on the device by finger pressure against the knob. A rate of rotation that causes failure in 5 to 10 seconds is recommended. After failure occurs, the dial head reading indicated by the pointer is read and recorded, and the shear strength is calculated. The procedure for performing this test is outlined in USBR 5770. Field data taken from sample tubes should be noted in the comments column of the undisturbed soil sampling data sheet 7-1656 (USBR 7105)

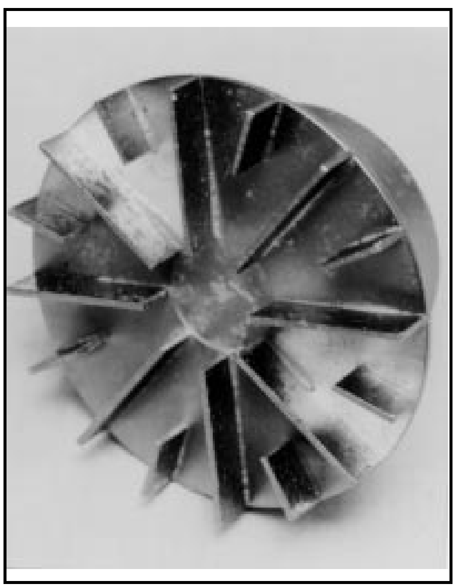
n. The Point-Load Strength Test.—Rock strength is an important property, and a suitable strength-index test is required. Simple "hammer and penknife" tests can be used; however, this approach seldom provides objective, quantitative, or reproducible results. The uniaxial (unconfined) compression test has been widely used for rock strength classification but requires machined specimens and, therefore is an expensive technique—essentially confined to the laboratory.

The point-load test is conducted in the field on unprepared rock specimens using simple portable equipment. Two types of point-load test machines are available as shown on figure 2-60. Essentially, the test involves compressing a piece of rock between two points. The point-load index is calculated as the ratio of the applied load to the square of the distance between the loading points. As illustrated on figure 2-61, the point-load test has a number of variations such as the diametral test, the axial test, and the irregular lump test.

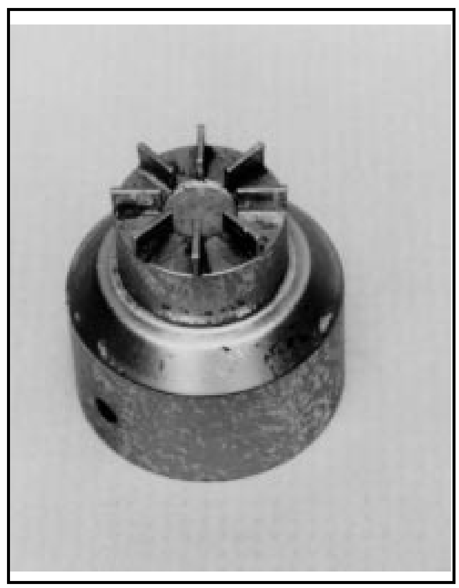
The strength at failure is expressed as a point load index, I_s.

(a) Top view.

(b) Side view. Note vane at bottom.



(c) Adaptor: 0 to 0.2 ton/ft^2.



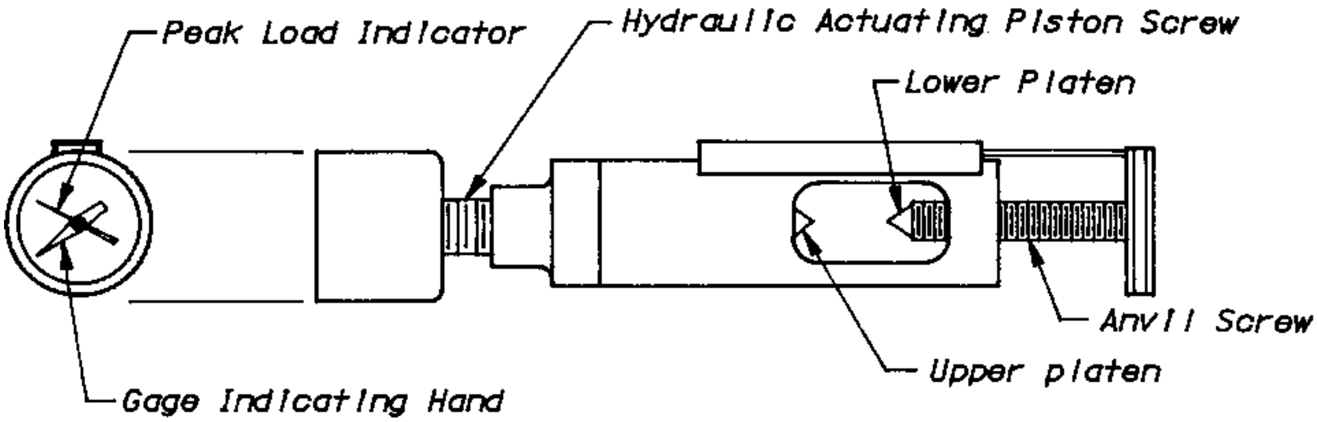
(d) Adaptor: 0 to 2.5 ton/ft^2.

Figure 2-59.—Torvane (pocket-vane shear meter) equipment.

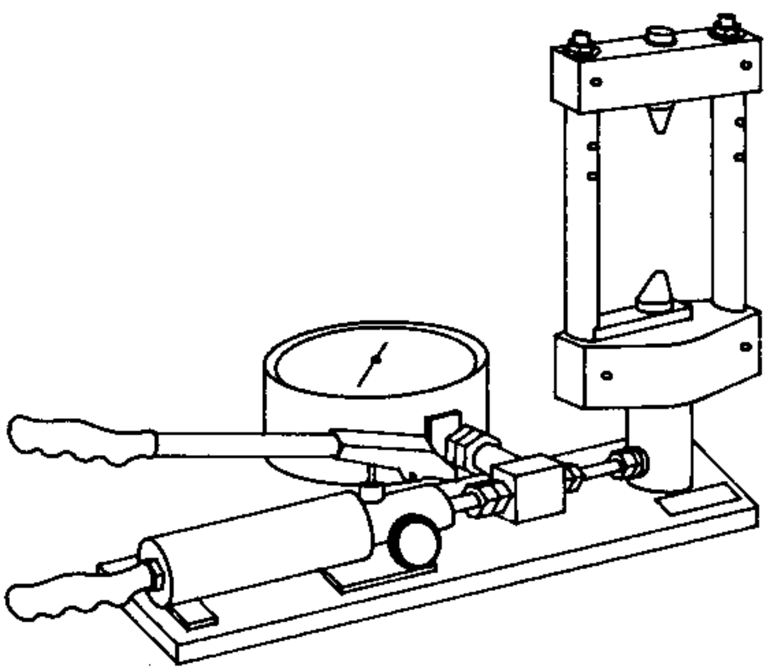
An approximate correlation exists between the point-load index and the uniaxial compressive strength, O_c , as shown on figure 2-62 and is given by: $O_c = 24\ I_s$.

In the above equation, the constant 24 is for a 54-mm (2.125-in)(NW)-diameter core. For other core sizes, values are: 20 mm (0.750 in)(RW) = 17.5, 27.0 mm (1.062 in)(AQ) = 19, 36.5 mm (1.432 in)(BQ) = 21, 47.6 mm (1.875 in)(NQ) = 23, and 63.5 mm (2.500 in)(HQ) = 24.5.

Reclamation experience indicates the above correlation may vary with geologic rock type as well as the condition of the core. On important projects, such as tunnels and pressure shafts, a site-specific correlation should be developed and the variability of the results evaluated.



(a)



(b)

Figure 2-60.—Point-load strength testing machines.

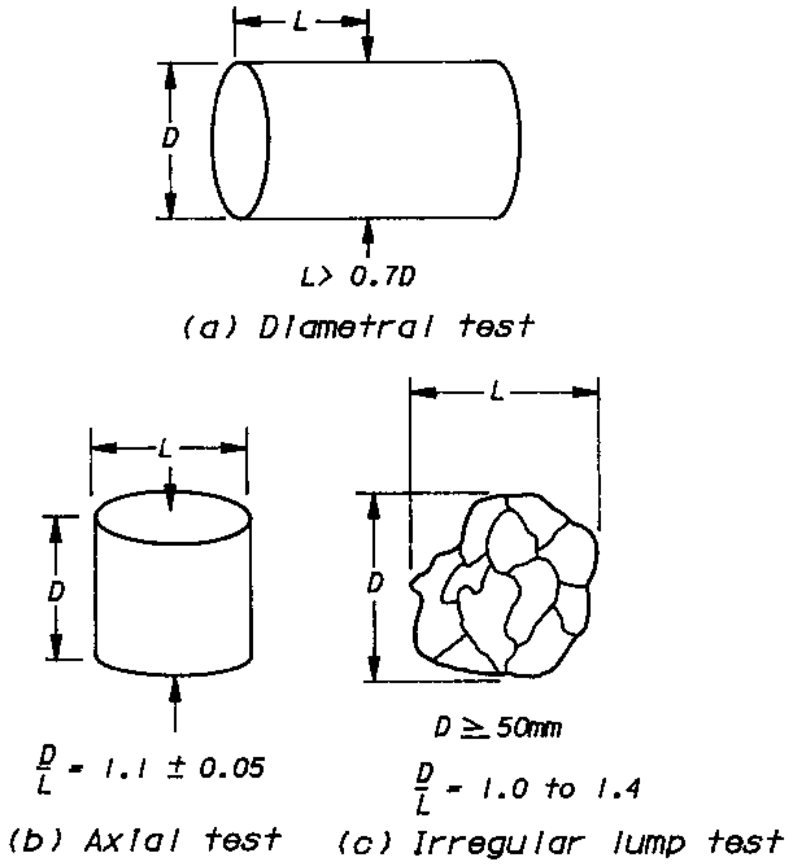


Figure 2-61.—Point-load strength testing variations (a) diametral test, (b) axial test, (c) irregular lump test.

To judge whether a point-load test is valid, the fractured pieces of core should be examined. If a clean fracture runs from one loading point indentation to the other, the test results can be accepted. However, if the fracture runs across some other plane, as might happen when testing schistose rocks, or if the points sink into the rock surface causing excessive crushing or deformation, the test should be rejected.

o. Portable Direct Shear Rock Testing Device.—Shear strength is an important factor for stability of a rock slope. The shear strength of a potential failure surface that may consist of a single discontinuity plane or a complex path following several discontinuities and involving some fracture of the intact rock material must be determined. Determination of reliable shear strength is a critical part of a slope design because relatively small changes in shear strength can result in significant changes in the safe height or angle of a slope. To obtain shear strength values for use in rock slope design, some form of testing is required. This may take the form of a sophisticated laboratory or in-place test in which all the characteristics of the in-place behavior of the rock discontinuity are reproduced as accurately as possible. Alternatively, the test may involve a simple determination depending upon facilities available, the nature of the problem being investigated, and the stage of the investigation.

A portable direct shear device for testing rock discontinuities is shown on figure 2-63. This machine was designed for field use; many of the refinements present on larger machines were sacrificed for simplicity and portability. A typical test can be performed in this device in about 15 to 30 minutes. As shown on figure 2-63(b), the test specimen is oriented and encapsulated in the shear box halves so the discontinuity is aligned within the shear plane of the testing device. Specimen size is limited to about 100-mm- (4-in-) diameter core or other shapes of about 100 mm (4 in) in dimension.

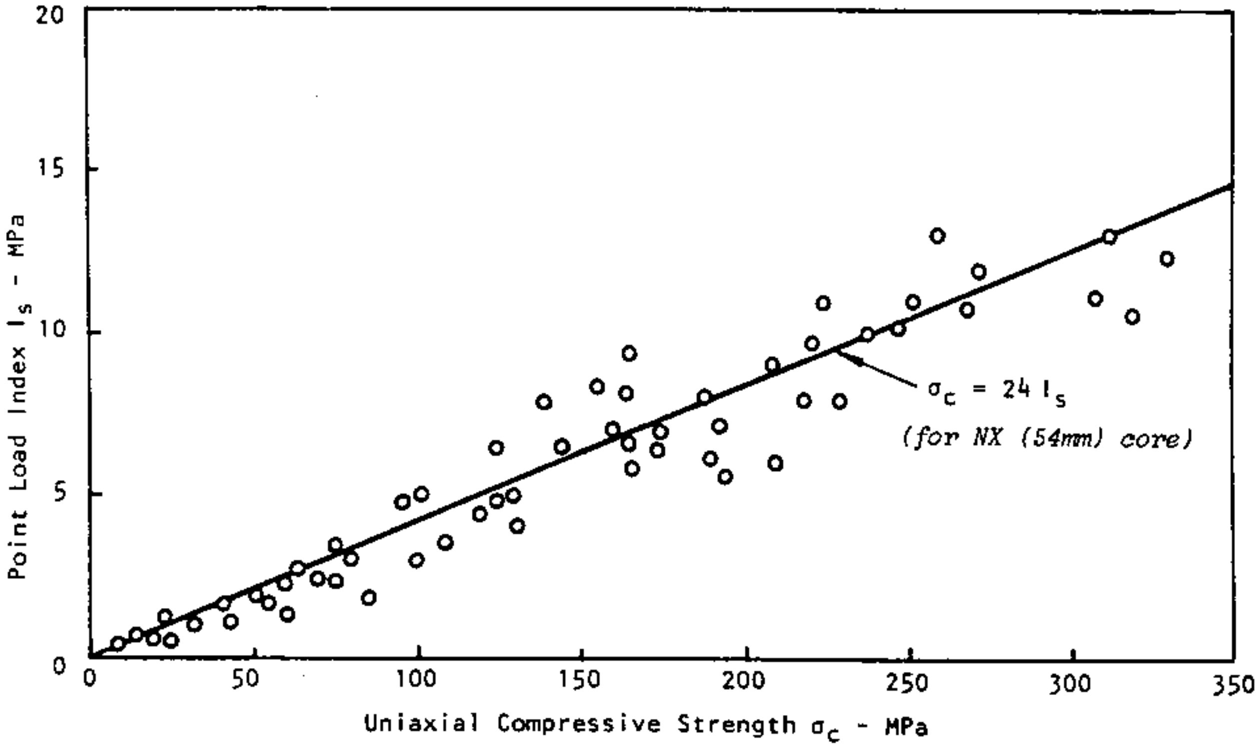


Figure 2-62.—Relationship between point-load strength index and uniaxial-compressive strength. 1MPa = 10.2 kg/cm^2 = 145 lb/in^2.

Stationary high pressure rock direct shear machines in the Earth Sciences laboratory are available for testing larger specimens. Reclamation has direct shear machines capable of testing 150- and 250-mm- (6- and 10-in-) diameter specimens at normal stresses of up to 700000 kPa (100,000 lbf/in^2).

p. Large-Scale, In-Place, Direct Shear Tests.—Direct shear testing of small rock cores may not produce accurate data because of scale effects. In many cases, it is desirable to test rock with discontinuities which are difficult to sample or which could be disturbed by the sampling process. If shear strength information is critical on large structures such as dams or powerhouses, performing large scale in-place shear tests may be necessary. These tests are expensive and difficult to perform. The test can be performed in accessible openings or adits. Figure 2-64 shows the arrangement for testing in an adit where the normal force is applied through reaction to the tunnel crown. Test specimens are normally 380 by 380 square by 250 mm (15 × 15 × 10 in) deep. Shear loads, approximating structure loading, are applied, and there is only one peak strength determination prior to sliding shear strength determinations. Normal stresses of up to 7,000 kPa (1,000 lbf/in^2) have been tested. A more detailed description for conducting these tests is presented in reference [66] and in the following section.

q. Uniaxial and Radial Jacking Tests in Rock.—Borehole dilatometer tests and tests performed on core samples may not produce representative results because they reflect behavior of only a small percentage of a rock mass. Modulus of deformation of laboratory specimens is usually higher than true rock mass behavior because joints and discontinuities are not included in the specimen. Borehole tests may include some discontinuities and result in lower moduli. The stress field from a large structure, such as an arch dam, may extend well into an abutment and may include many discontinuities which influence the behavior of the structure itself. To accurately design such structures, larger-scale tests have been developed. Uniaxial jacking tests such as those depicted in figure 2-65 can be performed in exploration adits. Uniaxial tests can be aligned to simulate thrust direction produced by the structure and to accommodate arrangement of jointing and discontinuities present in the rock. Uniaxial tests are always performed to provide data for the design of large concrete dams. The tests are major undertakings and are expensive. Radial jacking tests have also been developed to simulate "circumferential" in-place stresses for larger areas [67].

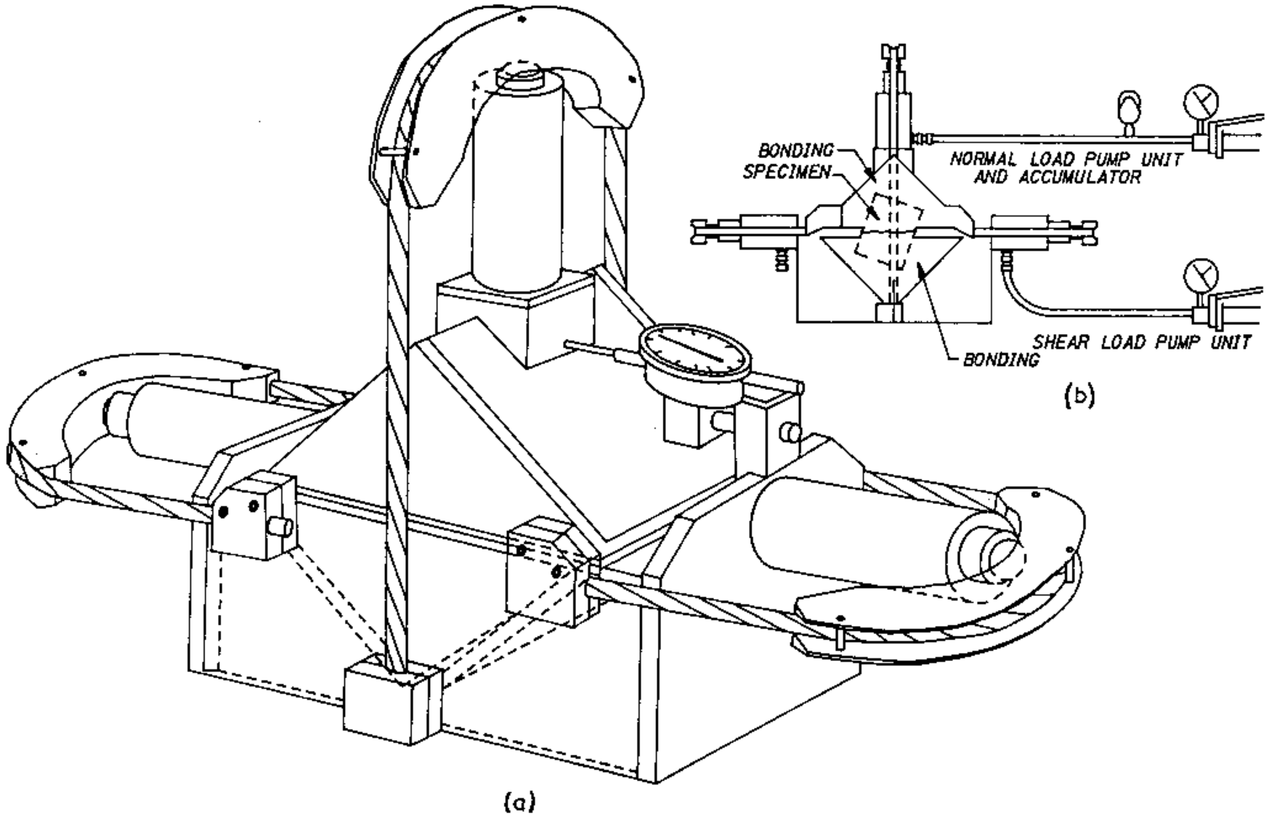


Figure 2-63.—Sketch of portable direct shear device.

Aluminum components are used to perform the uniaxial jacking test; and 0.9-m- (3-ft-) diameter hydraulic circular flat jacks are used for applying reaction to the column. Jacking pressure (simulated structure load on the rock) of up to 7,000 kPa (1,000 lbf/in^2) can be applied. Deformation is monitored in two **NX** size boreholes located in the center of the jacking pad. Seven-point retrievable steel rod extensometers are installed in each drill hole to monitor deformations at various depths from the rock face.

Locations of the extensometer anchors are such that both closing of surficial cracking and rock mass deformation can be evaluated. Usually, load increments are applied in 1,500 kPa (200-lbf/in^2) increments and sustained for 1 or 2 days, followed by complete unloading and evaluation of set [68]. The test site preparation is the most time consuming activity. Tunnel adits must be expanded to a minimum diameter to accommodate equipment. Test areas must be carefully prepared to remove any blast damaged material by using pneumatic chipping hammers and drills.

D. Recording and Reporting of Data

2-22. Maps.—Information which requires many pages of narrative can be shown on a map. Among the many varieties of mapping methods available, some suitable procedure can be found that will convey the necessary information clearly and easily.

*a. General.—*Three scale ranges for maps are commonly used. Maps ranging in scale between 1 and 10 miles per inch are usually suitable for showing the general area of the work; describing access and transportation facilities such as highways, railroads, rivers, and towns; and locating special types of material deposits such as riprap or aggregates. Map scales ranging between 5,000 feet per inch and 400 feet per inch often are used for more detailed information covering the immediate vicinity of the work; general geology of the area; reservoir areas; location of borrow pits; right-of-way lines; locations of

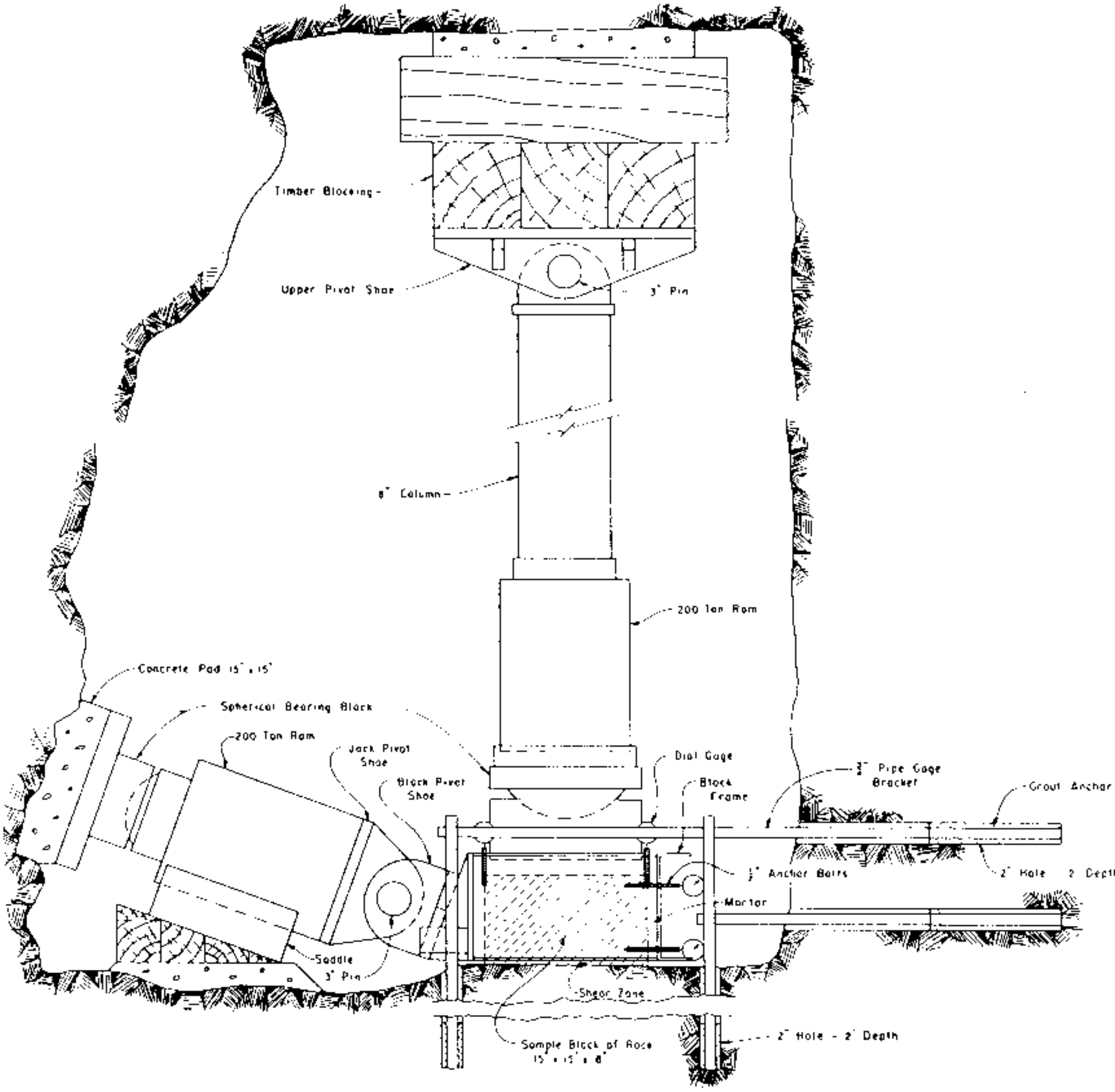


Figure 2-64.—Shear and sliding friction testing equipment, Auburn damsite.

roads, canals, and transmission lines; and similar information. To provide detailed information on a structure site, map scales ranging between 500 feet per inch and 20 feet per inch are commonly used; but long, low damsites with limited relief may be mapped at a scale of 1,000 feet per inch. Locations of small structures where local detail is important may be mapped on scales of 20 feet per inch. In selecting a scale, it is desirable to keep the ratio of ground measurement to map measurement as simple as possible; for example, the added detail that can be shown using a scale of 750 feet per inch may be less beneficial than the convenience of using a scale of 1,000 foot per inch. Also, the complete map should not be larger than may be conveniently spread on an ordinary table. The map's scale and a north arrow must always be shown. Further specific guidance is provided for proper scaling of various geologic maps in the *Engineering Geology Field Manual* [20] and *Engineering Geology Office Manual* [69].

State plane coordinate systems are preferred. All detail maps should be controlled by a coordinate system or other definite means of locating points on the ground. The grid lines should run true north and south, and east and west. If a local grid system is established, the origin of the system should be to the south and west of the area under consideration, and the displacement should be predominantly in one direction, so there will be a major numerical difference between the north and east coordinates of any point. If a damsite is involved, the displacement should be sufficient so the entire work area, including

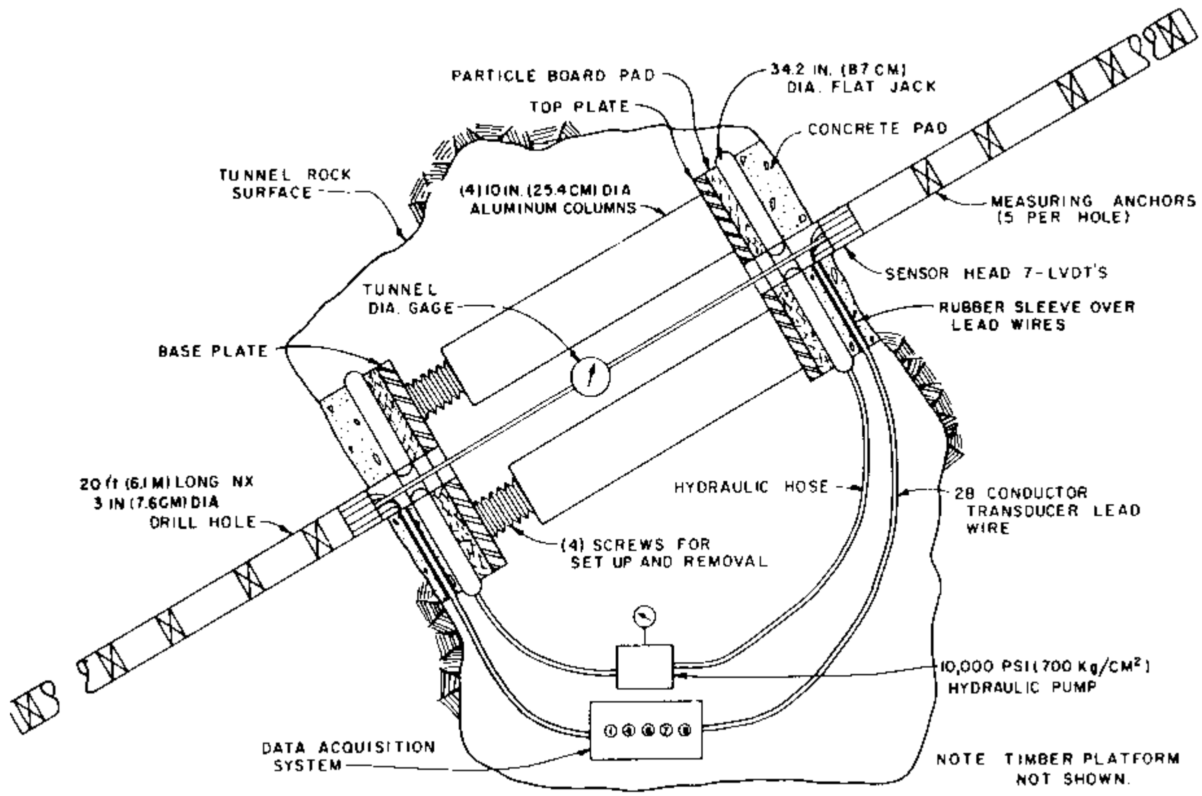


Figure 2-65.—Uniaxial jacking test.

borrow areas, plots within the northeast quadrant. The grid system should be referenced to public land surveys, triangulation stations, and other prominent permanent features in the area.

Variations in elevation are depicted on detail maps by use of contours. Contour intervals required may range from 20- or 25-foot intervals to 2-foot intervals, and, occasionally, 1-foot intervals depending on the map scale, the irregularity of the land surface, and the map use.

In general, contours should be sufficiently close together so that elevations between contours can be determined with confidence, but sufficiently separated so that a contour can be visually followed without difficulty. Elevations should always be referred to sea level on the basis of the nationwide survey system of the U.S. Geological Survey or U.S. Coast and Geodetic Survey. If an assumed datum is used for reconnaissance purposes, this elevation selected should not conflict with any known elevations.

Topographic maps are necessary in exploration of foundations and construction materials for hydraulic structures. The locations and elevations of exploratory holes, outcrops, and erosional features can be shown on the map, and landforms portrayed by contours can indicate, to some degree, the type of soil and subsurface geologic conditions. In the absence of topographic map coverage or where greater detail is needed, photogrammetric methods are used to produce maps of any desired scale and contour interval.

Commonly available general purpose topographic maps (e.g., USGS-type maps) are not detailed enough for site-specific needs. Topography for engineering exploration, design, and construction must be generated for the specific application. Horizontal scales of 1 inch to 50 or 100 ft and contour intervals of 1 to 5 ft are common. Site-specific maps are usually generated photogrammetrically using aerial photographs flown for the application, although small maps may be prepared using plane table or ground survey data. Topographic maps are generated from aerial photographs by viewing the photos in an analytical plotter. The plotter permits manual or automatic generation of lines of equal elevation at predetermined intervals and scales derived from stereo images of the photos.

The coordinate system used on a map to locate features depends on the needs or conventions of the project. State plane coordinates are usually used, but older projects may still use local coordinate systems set up for the specific project. The measurement units (U.S., metric, or degrees) are determined by the needs of the project.

Reference point accuracy is categorized in orders, depending on the need of the project. The locations of benchmarks vary in accuracy and may vary from precisely located geodetic benchmarks to locations scaled off a topographic map. The geodetic survey provides data on benchmark location, including those set by the USGS.

The map datum used for determining the map coordinate system is important because older maps generally use obsolete datums (e.g., NAD 27 used on most USGS quadrangles) and new maps or measurements use new datums (e.g. NAD 83). Mixing datums can produce apparent errors of several hundred feet that are often difficult to resolve.

Site topographic maps are used as base maps for geologic mapping as well as for foundation design and construction excavation maps. These site-specific maps are generally updated during construction to document actual conditions encountered. Updates are made by ground survey or are reflown on major jobs. Material quantities are often determined by successive topographic mapping as excavation proceeds.

When having specialized topographic maps made, several factors are important. The scale, contour interval, and area are the most important factors and are determined by the needs of the project. The map datum (generally NAD 27 or NAD 83) and the map medium are also very important. All topography should be provided on paper and digitally.

Site geologic maps are used for the design, construction, and maintenance of engineering features. These maps concentrate on geologic and hydrologic data pertinent to the engineering needs of a project and do not address the academic aspects of the geology. In addition to published topographic maps, the USGS has other information for mapped areas; for example, location and true geodetic potion of triangulation stations and elevation of permanent benchmarks established by the USGS.

The horizontal and vertical accuracy and precision of locations on a map depend on the spatial control of the base map. General base map controls are (in decreasing accuracy and precision):

(1) survey control or controlled terrestrial photogrammetry, mapped from survey-controlled observation points or by plane table or stadia;

(2) existing topographic maps. Control for these maps varies with scale. The most accurate are large-scale photogrammetric topographic maps generated from aerial photographs for specific site studies;

(3) uncontrolled aerial/terrestrial photogrammetry. Camera lens distortion is the chief source of error;

(4) Brunton compass/tape surveys. These surveys can be reasonably accurate if measurements are taken with care; and

(5) sketch mapping. Practice is needed to make reasonably accurate sketch maps.

The geographical positioning system (GPS) may provide adequate position locations depending on the required accuracy or precision.

The geographical positioning system is a system of satellites that provides positioning data to receivers on earth. The receiver uses the positioning data to calculate the location of the receiver on earth. Accuracy and type of data output depend on many factors that must be evaluated before using the system. The factors that must be evaluated are:

- Project requirements.—The location, accuracy, or precision needed by the project is a controlling factor in whether GPS is appropriate for the project. The actual needs of the project should be determined, being careful to differentiate with "what would be nice." Costs should be compared between traditional surveying and GPS.
- GPS equipment.—Different GPS receiver systems have different accuracies. Accuracies can range from 300 ft to inches (100 m to mm) depending on the GPS system. Costs increase exponentially with the increase in accuracy. A realistic evaluation of the typical accuracy of the equipment to be used is necessary, and a realistic evaluation of the needed, not "what would be nice," accuracy is important. Possible accuracy and typical accuracy are usually not the same.
- Data parameters.—The datum or theoretical reference surface to be used for the project must be determined at the start. USGS topographic maps commonly use NAD 27, but most new surveys use NAD 83. Changing from one datum to another can result in apparent location differences of several hundred feet (hundreds of meters).

The map projection is the projection used to depict the round shape of the earth on a flat plane or map. The most common projections used in the U.S. are the transverse Mercator and the Lambert conformal conic. State plane coordinate systems almost exclusively use one or the other. To use these state plane projections, location and definition parameters are necessary.

- Transverse Mercator.—The transverse Mercator projection requires a central meridian, scale reduction, and origin for each state or state zone.
- Lambert conformal conic.—The Lambert conformal conic projection requires two standard parallels and an origin for each state or state zone.

The coordinate system is the grid system that is to be used on the project. The state plane system is used by most projects, but latitude/longitude, universal transverse Mercator, or a local coordinate system may be used.

Standard American units or metric units should be selected as early in the project as possible. Conversions are possible, but converting a large 1-foot contour map to meters is no trivial matter.

Remember that when using several sources of location data, the reference datum must be known. Systematic differences in location data are generally due to mixing datums.

Location and general maps are oriented with north at the top of the page. Detail maps for water conveyance or storage structures are oriented for water movement to the top or to the right of the page. Railroad and highway location maps are oriented according to the practices of the organization involved. Every map has a north arrow.

Location maps should show all established transportation routes and communities adjacent to the area under consideration. Reservoir maps should show all major constructed fixed facilities including but not restricted to railroads, highways, pipelines, canals, telephone lines, and powerlines; buildings, mines, cemeteries, reservoirs, and wells. Also, the type and kind of plant cover should be shown. Detail maps, in addition to showing the above features, should show rock outcrops, talus, recognizable landslides, waterways, survey monuments and benchmarks; and section, township, and county lines.

For specifications, a map is required showing the extent to which right-of-way will be acquired for the structure involved. The map should show property lines and ownership of individual areas.

b. Site Geologic Mapping.—Engineering geologic mapping is done in two phases, mapping prior to construction based on study levels, and mapping during construction. In general, the following suggestions are for: (1) general mapping requirements for the job; (2) type of documentation needed; and (3) mapping requirements.

1. General Requirements.—Relatively detailed site geologic mapping studies generally are done for most structures or sites. Site mapping requirements are controlled by numerous factors, the most important of which are the type and size of structure to be built or rehabilitated, the phase of study (planning through operation and maintenance), and the specific design needs.

> Site mapping studies for major engineering features should be performed within an approximate 5-mile radius of the feature, with smaller areas mapped for less critical structures. These studies consist of detailed mapping and a study of the immediate site, with more generalized studies of the surrounding area. This approach allows an integration of the detailed site geology with the regional geology. The overall process of site mapping is a progression from preliminary, highly interpretive concepts based on limited data, to final concepts based on detailed, reasonably well-defined data, and interpretation. This progression builds on the previous step using more detailed and usually more expensive methods of data collection to acquire additional and better defined geologic information. Typically, site mapping is performed in phases: (a) preliminary surface geologic mapping; and (b) detailed surface geologic mapping, and construction geologic mapping. These phases are roughly equivalent to reconnaissance (or preliminary), feasibility, design, and construction geology mapping. All site mapping studies begin with preparation of a preliminary surface geologic map which delineates surficial deposits and existing bedrock exposures. The preliminary surface geologic map is then used to select sites for dozer trenches, backhoe trenches, and drill holes. These explorations provide data for the present and later mapping phases. Surface geologic maps are then reinterpreted based on the detailed surface and subsurface data. If required, detailed subsurface geologic data are also obtained from exploratory shafts and adits. The later mapping phases are generally the same but become more detailed and specific for the project.

2. Documentation.—Site data are documented on drawings (and associated notes) generated during the study. The drawings fall into two general categories—working drawings and final drawings. Working drawings serve as tools to evaluate and analyze data as they are collected and to define areas where additional data are needed. Analysis of data in a three-dimensional format is the only way the geologist can arrive at an understanding of the site geology, and it is critical these drawings be generated early in the study and continuously updated as the work progresses. These drawings are used for preliminary data transmittals. Scales used for working drawings may permit more detailed descriptions and collection of data that are not as significant to the final drawings. Final drawings are generated late in the mapping program after the basic geology is well understood. Although working drawings may be finalized, many times, new maps and cross sections are generated to illustrate specific data that were not available or well understood when the working drawings were made. These drawings serve as a record of the investigations for special studies, specifications, or technical record reports. Site mapping documentation is developed in phases: preliminary surface geologic mapping and detailed surface geologic mapping.

Preliminary surface geologic mapping.—The purpose of preliminary surface geologic mapping is to define the major geologic units and structures in the site area and the general engineering properties of the units. Suggested basic geologic maps are regional reconnaissance maps at scales between 1 inch = 2,000 feet and 1 inch = 5,280 feet (1:24,000 to 1:62,500) and a site geology map at scales between 1 inch = 20 feet and 1 inch = 1,000 feet (1:250 to 1:12,000). Scale selection depends on the size of the engineered structure and the complexity of the geology. Maps of smaller areas may be generated at scales larger than the scale of the base map to illustrate critical conditions. Cross sections should be made at the same (natural) scale (horizontal and vertical) as the base map's scale unless specific data are better illustrated at an exaggerated scale. Exaggerated scale cross sections are generally not suited for geologic analysis because the distortion makes projection and interpretation of geologic data difficult.

Initial studies generally are a reconnaissance-level effort, and the time available to do the work usually is limited. Initially, previous geologic studies in the general site area are used. These studies should be reviewed and field checked for adequacy, and new data should be added. Initial base maps usually are generated from existing topographic maps, but because most readily available topography is unsuitable for detailed studies, site topography at a suitable scale should be obtained if possible. Existing aerial photographs can be used as temporary base maps if topographic maps are not available. Sketch maps and Brunton/tape surveys or global-positioning-system (GPS) location of surface geologic data can be done if survey control is not available. Good notes and records of outcrop locations and data are important to minimize re-examination of previously mapped areas. Photography is a highly useful tool at this stage in the investigation, as photos can be studied in the office for additional data. Only after reasonably accurate surface geology maps have been compiled can other investigative techniques such as trenching and core drilling be used to full advantage. For some levels of study, this phase may be all that is required.

Detailed Surface Geologic Mapping.—The purpose of detailed surface geologic mapping is to define the regional geology and site geology in sufficient detail that geologic questions critical to the structure can be answered and addressed. Specific geologic features critical to this assessment are identified and studied, and detailed descriptions of the engineering properties of the site geologic units are compiled. Project nomenclature should be systematized and standard definitions used. Suggested basic geologic maps are similar to those done for preliminary studies, although drawing scales may be changed based on the results of the initial mapping program. Maps of smaller areas may be generated to illustrate critical data at scales larger than the base map's scale. The preliminary surface geology maps are used to select sites for dozer trenches, backhoe trenches, and drill core holes. As the surface geology is better defined, drill hole locations can be selected to help clarify multiple geologic problems. Detailed topography of the study site should be obtained if not obtained during the initial investigations. Data collected during earlier phases of investigations should be transferred to the new base maps, if possible, to save drafting time. Field mapping control is provided primarily by the detailed topographic maps and/or GPS, supplemented by survey control if available or Brunton/tape survey. If not, small scale aerial photographs of the site area flown to obtain detailed topography are useful in the geologic mapping.

2-23. Logging of Exploratory Holes.—

a. Location of Holes.—The initial holes drilled or excavated in an area are usually to clarify geological conditions. Therefore, the location is governed primarily by the geology; the final holes are drilled or excavated primarily for specific geology or engineering purposes and are located on the basis of the engineering structure to be built. Holes are also drilled or excavated both to establish the form and shape of a geologic unit and to examine the character of a geologic discontinuity. Although it is desirable to locate holes so as to satisfy as many of these requirements as possible, sometimes, these respective requirements are contradictory and separate holes are required. From an engineering standpoint, holes that bracket a condition are more desirable if other design requirements provide sufficient flexibility so the structure's location can possibly be moved to avoid an unfavorable condition. From a geologic standpoint and for those engineering situations where the questionable area cannot be avoided, holes in questionable areas are preferable.

Every hole drilled or excavated should be definitely located in space. The hole should be tied either to the coordinate grid system and have a coordinate location or be tied to a location in some other satisfactory manner such as stationing or section ties. The elevation at the top of each borehole should be established by survey or reliable global positioning system. Coordinates and elevation of an exploration hole or trench should refer to the center of the excavation. However, if more than one log is required to adequately describe the materials in a trench, the coordinates and elevations of each log should be supplied. The direction of the longitudinal axis of trenches should also be indicated. Any hole drilled or excavated should be logged for the full depth of hole. If, for any reason, a portion cannot be logged, the interval not logged should be recorded along with an explanation stating the reason for omission. The bearing and angle from horizontal of angle holes must be reported.

b. Identification of Holes.—To ensure completeness of the record and to eliminate confusion, test holes should normally be numbered in the order of excavation, and the series should be continuous through the various stages of the work. If a hole is planned and programmed, it is preferable to maintain the hole number in the record as "not drilled" or "abandoned" with an explanatory note rather than reuse the hole number elsewhere. However, it is permissible to move the location of holes short distances and to retain the program number where such moves are required by local conditions or by changes in engineering plans; but new coordinates and elevation should be established and recorded. When explorations cover several areas, such as alternative sites and borrow areas, a new series of numbers should be used for each site or borrow area. The favored practice is to begin excavation numbering of each new area explored at an even hundred. Recent practice has been to include the year of drilling in the hole number (i.e., DH-92-201).

Normally, test hole numbers are prefixed with a one-, two-, or three-letter designation to describe the type of exploration. The following prefix system is commonly used.

DH	Drill hole
AH	Auger hole
AP	Auger hole, power
TP	Test pit
TT	Trench
DT	Dozer trench
PT	Pitcher sampling hole
DN	Denison sampling hole
HS	Hollow-stem auger hole
CH	Churn drill hole
VT	Holes in which field vane tests are made
CPT	Cone penetration holes
MP	Pressuremeter holes
DM	Dilatometer holes
BS	Borehole shear test holes
SPT	Standard penetration test
BPT	Becker penetration test

In the above designations, DH is used to include not only rotary drilling, but also all the methods of hole advancement that produce core or relatively undisturbed samples, in contrast to the AH or AP holes that produce highly disturbed samples and CH holes that produce only cuttings and no samples. The CH designation should be used for all types of holes advanced by a chopping and washing action such as wash borings, jetting, or percussion drilling. The TP designation includes both hand- and machine-excavated test pits. Similarly, the T designation includes open trenches whether made by hand or machinery. Computerized logs may require a special prefix designation. Regardless of the prefix system used, somewhere in the data report and on the specifications drawings, a complete explanation of the prefix system used should be included.

c. Log Forms.—A log is a written record of data and observations concerning materials and conditions encountered in individual test holes and provides the fundamental facts on which all subsequent conclusions are based, including:

- additional exploration or testing,
- feasibility of the site,
- design treatment required,
- cost of construction,
- method of construction,
- evaluation of structure performance.

A log may:

- represent pertinent and important information that is used over a period of years,
- be needed to accurately determine a change of conditions with passage of time,
- form an important part of contract documents,
- be required as basic evidence in a court-of-law in case of dispute.

Each log should be factual, accurate, clear, and complete. In addition, logs can contain basic interpretations of the geologic nature of the materials encountered. Log forms are used to record and provide required information.

- A log should always include information on the size of borehole and on the type of equipment used for boring or excavating the hole. This should include:

  - the kind of drilling bit used on boreholes,
  - samplers used,
  - a description of the penetrating equipment or type of auger used,
  - method of excavating test pits, or
  - description of the machinery used.

- The location, elevation, and amount of material collected for test pit samples should be indicated on the logs.
- In boreholes, the length of core recovered should be computed and expressed as a percentage of each length of penetration of the core barrel (percent core recovery).
- Depth from which the sample was taken should be recorded.
- The logs should show the extent and the method of support used as the hole is deepened such as:

  - size and depth of casing,
  - location and extent of grouting if used,
  - type of drilling mud,
  - type of cribbing in test pits.

- Caving or squeezing material should be noted on borehole logs as this may represent a low strength or swelling stratum.

Various water levels should be recorded and water tests made at pertinent intervals (i.e., the following):

- Information on the presence or absence of water levels, and comments on the reliability of these data should be recorded on all logs.
- The date that measurements are made should be recorded, since water levels fluctuate seasonally.
- Water levels should be recorded periodically from the time water is first encountered and as the test hole is deepened.
- Upon completion of drilling, the hole should be bailed and allowed to recover in order to obtain a true water level measurement.
- Perched water tables and water under artesian pressure are important to note.
- The extent of water-bearing members should be noted, and areas where water is lost as the boring proceeds should be reported, since subsequent work on the hole may preclude duplicating such information.
- The log should contain information on any water tests made.
- Since it may be desirable to obtain periodic records of water level fluctuations in drilled holes, determination should be whether this is required before plugging or backfilling, and abandoning the exploratory hole.
- Hole completion data.

Detailed guidelines for logging exploratory holes are in the *Engineering Geology Field Manual* [20]. Numerous training manuals are available and should also be consulted so uniformity in logging and reporting data is maintained. These guides provide a common framework for presenting data. Some variation in log format is possible, because the information gathered should be tailored to project needs. A review of current procedures is presented in the following sections.

Examples of logs of two types of exploratory holes are discussed below:

1. Geologic Logs of Drill Holes.—Geologic logs of drill holes Log forms 7-1337 and 7-1334 (figs. 2-66 and 2-67), were developed for percolation and penetration resistance testing. These forms are suitable for logging all types of core borings which produce relatively undisturbed samples. The forms are often modified to present information for many other types of drilling and sampling. Drill hole logs are also prepared using several types of computer generated log forms. Regardless of how the form is configured, each must have five areas for presenting required information. The five required sections are:

- Heading block
- Left column for notes

- Center column for summary data of testing, sampling, and lithologic information
- Right column for describing physical condition and description of samples or material retrieved from the hole
- Bottom section for any explanation or notes required

Examples of a percolation test log and a penetration resistance test log are shown on figs. 2-66 and 2-67. These figures illustrate the type of information to be included in the log. The following outline shows the five sections of the log:

Heading: The top portion of the log provides space for identification information pertaining to project, feature, hole number, location, elevation, bearing, dip, inclination, start and completion date, and the names of persons logging and reviewing. *This information is mandatory.*

Drilling Notes Column (left column of drill log):

A. General Information
  1. Purpose
  2. Drill site and setup
  3. Drillers
  4. Drilling equipment (rig, rods, barrels, bits, pumps, and water test equipment)

B. Procedures and Conditions
  1. Drilling methods
  2. General drilling conditions and drillers comments
  3. Drilling fluid use, return, color, and losses
  4. Caving conditions
  5. Casing record
  6. Sample interval comments and any unusual occurrences during sampling
  7. Cementing record

C. Hole Completion and Monitoring Data
  1. Borehole survey data
  2. Water level data
  3. Hole completion, backfilling, and instrumentation details
  4. Reason for hole termination
  5. Drilling time

Summary Information (center column): As shown on the example logs, this portion of the log is used to summarize drilling, lithology, testing, and sampling results. The most common tests performed are percolation tests or penetration resistance tests for which the forms are designed. Center column is often modified to present results of other types of testing such as undisturbed sampling with in-place dry density determinations and in-place testing such as vane shear or borehole shear tests. Columns for noting core recovery, and locations and depths of lithologic contacts aid in visualizing geologic conditions and are included in the center portion of the log. Information on sampling intervals should include, at least, depths of sampling and, if space permits, graphic or tabular information on sample measurements such as in-place dry density. This information aids designers in selecting samples for testing.

Comments and Explanations (bottom portion): Notes as to abbreviations and other miscellaneous references such as deck elevations should be shown in this area. Dates and filenames, and dates of any revisions related to computer data bases should be recorded.

Classification and Physical Description (right column): An accurate description of recovered core is provided here and includes technically sound interpretations of nonrecovered material conditions.

- In areas of no recovery, it is frequently possible to infer conditions based upon drilling action and cuttings return.
- Rockbit intervals and characteristics of return materials should be described.
- Geologic interpretations to delineate major lithologic units are included in this column.
- Rock or soil cores or samples should be described as completely as possible.

Soil classifications are determined using the Unified Soil Classification System as described in USBR 5000 and 5005. Soil descriptions on test pits are discussed immediately following this subsection. Soil classifications and descriptions for the drill log's right column are the same as those for test pits, except for describing the in-place condition which is not included for soil cores. The Unified Soil Classification System (USCS) symbol for soil recovered from drill holes also can be shown in tabular form in the center column of the log.

Guidelines for logging rock cores are given in detail in chapters 4 and 5 of the *Engineering Geology Field Manual* [20]. In general, classifications include determination of a rock unit name based on lithologic characteristics followed by description of structural features of engineering importance. The suggested content for word descriptions is listed below:

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE: EXAMPLE ... PROJECT: ... STATE: ...
HOLE NO.: DH-123 ... LOCATION: Channel, right side ... Deck GROUND ELEV: 2139.2 / 2139.7 ... DIP (ANGLE FROM HORIZ.): -90°
COORDS. N. ... E. ...
BEGUN: 8-25-81 ... FINISHED: 9-9-81 ... DEPTH OF OVERBURDEN: -- ... TOTAL DEPTH: 134.3 ... BEARING: --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: See Notes ... LOGGED BY: F.J. Miller ... LOG REVIEWED BY: S.R. Cowlins

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Purpose of Hole:
Foundation investigation of small dam.

Location and Set up:
Located 25 ft rt of dam centerline, Sta. 3+85. Level pad, excavated approx. 1.5 ft of material for drill setup. All depth measurements from deck.

Drillers: L. Harding, F. Smith, and B. Green.

Drilling Equip.:
Skid-mounted Longyear 34, hydraulic feed, rotary drill. NW drill rods.

Drilling Methods:
0.0 - 3.0': 3-1/2" rock bit
3.0 - 7.3': NWD3 core barrel w/split-tube inner barrel; Nx side-discharge diamond bit, and clear water.
7.3 - 9.2 : 3-1/2" rockbit
9.2 - 134.3': NWD3 core barrel w/split inner barrel; Nx bottom discharge bit and clear water except as noted below.
63.6 - 79.1': used revert mud

Drilling Conditions and Drillers Comments:
0.0 - 2.0': Fast and smooth
2.0 - 10.3'; Very fast and smooth
10.3 - 29.8': Drilled moderately fast and rough
29.8 - 50.3 : Drilled slow and smooth
50.3 - 63.6': slow, rough, blocky
63.6 - 113.6': slow to fast, smooth to slightly rough
113.6 - 118.8': slow and rough, blocky
118.8 - 134.3': slow and smooth.

Caving Conditions:
0.0 - 10.0': heavy
11.5 - 18.5': light
5?.0 - 63.6': heavy
115.8 - 129.1': moderate.

TYPE AND SIZE OF HOLE: 3½ RB; NW D3; 3½ RB; NW D3

CORE RECOVERY (%): 0, 47, 0, 65, 98, 100, 100, 97, 100, 97, 95, 98, 100, 95, 86, 98, 97, 100, 100, 100, 100, 98, 100, 97, 100

PERCOLATION TESTS

Inflatable Nx packer (Packer No. 12) and 55 gpm Max volume FMC pump used for all packer tests

Depth (feet) From (P, Ca, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of test (min.)	Elevation (feet)
					2137.9
0.0 (casing to 8.0')	11.6	15	G	20	2128.4
0.0	20.0	18	G	20	
					2106.6
P 35.7	45.7	0.1	25	5	
					2101.6
P 44.8	54.8	0.0	25	5	2097.1
					2089.6
P 53.6	63.6	16.2	25	5	
		28.7	50	5	
		14.1	25	5	
P 63.6	73.6	0.6	25	5	2076.0
		1.1	50	5	
		0.4	25	5	
P 78.0	88.0	Packer did not seat			
P 83.7	93.7	0.2	25	5	
		0.3	50	5	2053.2
		1.5	100	5	
		0.4	50	5	
		0.1	25		
90.9	100.9	0.1	25	5	
		0.2	50	5	
		0.5	100	5	
		0.1	50	5	
		0.1	25	5	

DEPTH (FEET): 10, 20, 30, 40, 50, 60, 70, 80, 90

CLASSIFICATION AND PHYSICAL CONDITION

0.0-0.5': Deck
0.5-10.8': Quaternary Alluvium
10.8-63.2': Tertiary Volcanics
63.2-134.3: Jurassic Metamorphics

0.0-0.5': Deck
0.5-10.8': Quaternary Alluvium.
0.5-1.3': Residual Soil and Slopewash (Qc)
Classified from wash samples and surface exposures. Consists of approx. 75% red-brown lean clay (w/20% fine sand) and 25% Intensely Weathered. 0.1 to 0.2' soft, angular sandstone fragment.
1.3-10.8': River Channel Deposits (Qrs)
1.3-6.0': Clayey Sand. Approx. 50% coarse to fine, hard, subrounded sand; approx. 25% fine, hard, subrounded gravel; approx. 25% fines with medium plasticity, high dry strength, medium toughness; maximum size 50 mm; no reaction w/HCl. Firm, homogeneous, moist, reddish brown.
1.3-3.0': No Recovery. Rockbitted.
6.0-9.8': Silty Sand. Approx. 55% medium to fine sand; approx. 45% nonplastic fines, no dry strength. Max size, medium sand; reddish-brown. No reaction to HCl.
7.3-9.2': No Recovery. Rockbitted.
7.8-10.8': Poor Recovery. Recovered fine, hard, rounded to subrounded gravel.
10.8-63.2': Tertiary Volcanics.
10.8-32.6': Welded Rhyolite Tuff (Twr). Light gray, medium grained, 50% feldspar crystals and volcanic glass fragments to 3mm in microcrystalline ground mass, 10% oxidized mafic minerals and pyrite. Moderate; Weathered. Light rust stains surrounding many glass fragments; mafic minerals completely weathered; core locally leached with pits to 1/4" deep. Core scratches with light knife pressure, breaks across axis with heavy manual pressure to light hammer blow. Lightly Fractured. (100% joints). Core recovery in maximum lengths of 7.3' (excluding mechanical breaks) mostly 1.5 to 3.0'. Joint sets: (1) Dips 20-50°, spaced mostly 2.0 to 4.9', open, planar, with thin, red-brown clay coatings on rough joint surfaces; (2) Dips 10-15°, spaced mostly 1.0 to 1.3', planar, smooth. Present only in 30.6 to 43.2', interval.
32.6-49.6': Ash Flow Tuff (Tft). Pink (dry) to brick red (wet), w/25% gray pumice fragments to 0.1'. Moderately to Lightly Weathered. Core gouges with moderate knife pressure; breaks normal to core axis with heavy manual pressure. Moderately Fractures. (100%) joints). Core recovered in 0.7-1.0' lengths. Joint sets: (1) similar to set (1) of 10.8 to 32.6' interval; (2) Dips to 15-30°, subparallel to set (2) of 10.8-32.6' interval, spaced 0.4 to 2.0'.

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE: EXAMPLE ... PROJECT ... STATE ... SHEET 1 OF 2 ... HOLE NO. DH-123

GPO 1980 682-851

Figure 2-66.—Example geologic log of a drill hole.

STANDARD PENETRATION TEST HOLE NO. DH-400

SHEET 1 OF 3

PROJECT EXAMPLE FEATURE EXAMPLE AREA EXAMPLE STATE EXAMPLE

COORDS. N 10653 E 10003 GROUND ELEV 6742.5 ANGLE FROM HORIZ 90.0 DOWN

BEGUN 06-16-84 FINISHED 06-19-84 DEPTH TO BEDROCK 80.4 TOTAL DEPTH 80.4 BEARING

DEPTH TO WATER SEE NOTES LOGGED BY JOHN DOE REVIEWED BY JANE DOE

STANDARD PENETRATION TEST (DESIGNATION E-21, EARTH MANUAL)

BLOWS PER FOOT: 140 LB. HAMMER-30 IN. DROP (10, 20, 30, 40)

PERCENT CORE RECOVERY	NUMBER OF BLOWS	PERCENT MOISTURE
85	8	38.1
90	10	28.7
92	3	34.3
94	2	34.3
91	* 1	45.6
95	* 0	46.4
89	11	*27.7
85	21	18.4
85	28	*32.5
80	17	*28.5
95	* 25	21.3
88	14	*28.6
80	*50/1.3	*23.
40	*50/1.2	*24.
	*50/.4	
	*10/.1	

DEPTH SCALE (FEET): 10, 20, 30, 40, 50, 60, 70, 80, 90

CLASSIFICATION INTERVALS — ELEVATIONS (FEET): 6737.5, 6706.0, 6702.5

NOTES

PURPOSE:
FOUNDATION INVESTIG-ATION FOR PUMPING PLANT "X"

DRILL EQUIPMENT:
MOBIL B40-L TRUCK MOUNTED DRILL WITH BEAN 20 WATER PUMP

DRILLER:
JACK DOE

DRILLING METHOD:
0.0-80.4FT:
DRILLED WITH 2-7/8IN TRICONE ROLLER ROCK BIT USING BENTONITE AND WATER AS DRILLING FLUID WITH 50 SEC AVG FUNNEL VISCOSITY. PEFORMED SPT'S APPROX 5FT INTERVALS USING A 1-3/8IN ID SPLIT BARREL SAMPLER ON NW DRILL RODS WITH A 140 LBM DDM SAFETY HAMMER (TOTAL MASS 235 LBM) AND A 30IN DROP. NX CASING ADVANCED ON 5FT INTERVALS FOLLOW-ING SPT'S AND CLEANED OUT WITH ROCK BIT

SAMPLE INTERVAL NOTES:

15.0-16.5FT: DRILL STRING SANK THROUGH 0.2FT OF THE SEATING INTERVAL UNDER WEIGHT OF HAMMER AND RODS

20.0-21.5FT: DRILL STRING SANK 0.7FT THROUGH DRIVE INTER-VAL, 0.2FT UNDER RODS AND 0.5FT UNDER RODS AND HAMMER

25.0-26.5FT: DRILL STRING SANK 1.2FT THROUGH DRIVE INTER-VAL, 0.4FT-RODS, 0.8FT-RODS AND HAMMER. DRILLING DISTURBANCE NOT APPARENT

30.0-31.7FT: DRILL STRING SANK 1.7FT BELOW CLEANOUT DEPTH 0.5FT-RODS, 1.2FT RODS AND HAMMER. DRILLING DISTRUBANCE NOT APPA-RENT

35.0-36.5FT: UNEVEN PENETRATION. 2BLOWS IN 0.5-1.0FT INT., 9BLOWS INT. 0-1.5FT INT. PENE-TRATION DECREASED AT 1.1FT

45.0-46.5FT: UNEVEN PENETRATION-8BLOWS IN 0.5-1.0FT INT. 20 BLOWS IN 1.0-1.5FT INT. PENETRATION DE-CREASED AT 1.1FT

55.0-56.5FT: 0.4FT OF SLOUGH. SAMPLER VENT PORT PLUGGED WITH SAND POSSIBLE UNRELIA BLE N VALUE

65.0-66.3FT: REACHED 50BLOWS IN THE DRIVE INTERVAL AT 1.3 FT. 5 BLOWS IN SEATING INTERVAL. 25 BLOWS IN 0.5-1.0FT, 20 BLOWS IN 1.0-1.3FT. DRIVING IN GRAVELS

70.0-71.2FT: STOPPED-TEST AFTER 50BLOWS. POSSIBLE JETTING DIS-TURBANCE, 10 BLOWS IN

CLASSIFICATION AND PHYSICAL CONDITION

0.0 36.5 QUATERNARY LACUSTRINE SEDIMENTS (QL)

0.0-5.0: ROCK BIT INTERVAL; LEAN TO FAT CLAY WITH ORGANIC MATERIAL, VEGATATION AND ROOTS. DESCRIPTION BASED ON DRILLING CONDITION AND CUTTINGS RETURN

5.0-6.5: SPT SAMPLE
5.0-5.8: LEAN CLAY (CL): APPROX. 85% FINES WITH MEDIUM PLASTICITY, MEDIUM TOUGHNESS, HIGH DRY STRENGTH; APPROX. 15% PREDOMINANTLY FINE SAND; MAXIMUM SIZE, COARSE SAND; WEAK REACTION WITH HCL. - MOIST, TAN, FIRM, HOMO-GENEOUS. ROOTS AND ORGANIC MATERIALS PRESENT

5.8 6.5: FAT CLAY (CH): APPROX. 95% FINES WITH HIGH PLASTICITY, HIGH TOUGNESS, VERY HIGH DRY STRENGTH; APPROX. 5% PREDOMINANTLY FINE SAND; MAXIMUM SIZE, FINE SAND; WEAK REACTION WITH HCL. - MOIST, TAN, FIRM TO SOFT, HOMO-GENEOUS, SOME ROOTS PRESENT

6.5-10.0: ROCK BIT INTERVAL; SILTS AND CLAYS. DESCRIPTION BASED ON DRILLING CONDITION AND CUTTINGS RETURN

10.0-11.5: SPT SAMPLE
CLAYEY SILT (ML-CL): APPROX. 90% FINES WITH LOW PLASTICITY, LOW TOUGHNESS, LOW DRY STRENGTH; APPROX. 10% PREDOMINANTLY FINE SAND; MAXIMUM SIZE, FINE SAND; WEAK REACTION WITH HCL. - MOIST, GREY, SOFT, LAMINATED

11.5-15.0: ROCK BIT INTERVAL, LEAN TO FAT CLAYS BASED ON DRILLING CONDITION AND CUTTINGS RETURN

15.0-16.5: SPT SAMPLE
LEAN TO FAT CLAY (CL-CH): APPROX. 95% FINES WITH MEDIUM TO HIGH PLASTICITY, MEDIUM TO HIGH TOUGHNESS, VERY HIGH DRY STENGTH; APPROX. 5% PREDOMINANTLY FINE SAND; MAXIMUM SIZE, FINE SAND; WEAK REACTION WITH HCL. - WET, GREY-TAN, SOFT, LAMINATED

16.5-20.0 ROCK BIT INTERVAL; LEAN TO FAT CLAYS. DESCRIPTION BASED ON DRILLING CONDITION AND CUTTINGS RETURN

20.0-21.5: SPT SAMPLE
FAT CLAY (CH): APPROX. 100% FINES WITH HIGH PLASTICITY, HIGH TOUGHNESS, VERY HIGH DRY STRENGTH; WEAK REACTION WITH HCL. - WET, GREY, VERY SOFT TO SOFT, LAMINATED

21.5-25.0: ROCK BIT INTERVAL; FAT CLAYS, VERY SOFT, DESCRIPTION BASED ON DRILLING CONDITION AND CUTTINGS RETURN

25.0 26.5: SPT SAMPLE
FAT CLAY (CH): APPROX. 100% FINES WITH HIGH PLASTICITY, HIGH TOUGHNESS, VERY HIGH DRY STRENGTH; WEAK REACTION WITH HCL. - VERY WET, GREY, VERY SOFT, LAMINATED.

26.5-30.0: ROCK BIT INTERVAL; FAT CLAYS, VERY SOFT, BASED ON DRILLING CONDITION AND CUTTINGS RETURN

30.0 31.7: SPT SAMPLE
FAT CLAY (CH): APPROX. 100% FINES WITH HIGH PLASTICITY, HIGH TOUGHNESS, VERY HIGH DRY STRENGTH; WEAK REACTION WITH HCL. - VERY WET, GREY-BLACK, VERY SOFT, HOMOGENEOUS

31.7 35.0: ROCK BIT INTERVAL; FAT CLAYS, VERY SOFT-HIGH PLASTICITY, BASED ON DRILLING CONDITION AND CUTTINGS RETURN.

35.0 36.5: SPT SAMPLE
35.0-36.0: FAT CLAY (CH): APPROX. 100% FINES WITH HIGH PLASTICITY, HIGH TOUGHNESS, VERY HIGH DRY STRENGTH; WEAK REACTION WITH HCL. - WET, GREY-TAN, SOFT, LAMINATED. *MOISTURE - [illegible]

36.0 36.5: CLAYEY SAND (SC): APPROX. 70% FINE TO MEDIUM, HARD, SUBANGULAR SAND; APPROX. 30% FINES WITH HIGH PLASTICITY, HIGH TOUGHNESS, HIGH DRY STRENGTH; MAXIMUM SIZE, COARSE SAND; WEAK REACTION WITH HCL. - MOIST, GREY, DENSE, HOMOGENEOUS, FINES CONTENT IS HIGHER AT CONTACT. *MOISTURE - [illegible]

36.5 80.4: QUATERNARY ALLUVIUM (QAL)

36.5 40.0: ROCK BIT INTERVAL; CLAYEY TO SILTY

COMMENTS:

*-NEXT TO N VALUE, SEE NOTES COLUMN
*-NEXT TO MOISTURE, SEE CLASS DESCRIPTION COLUMN
- IN CLASS-DESCRIPTION COLUMN INDICATES DESCRIP-TION OF INPLACE CONDITION WILL FOLLOW

INT-INTERVAL
GRD-GROUND SURFACE

EXPLANATIONS:

BLOWS/FOOT — RECORD NUMBER OF BLOWS REQUIRED FOR ONE FOOT OF PENETRATION. IF 50 BLOWS RESULT IN LESS THAN ONE FOOT OF PENETRATION, RECORD DEPTH PENETRATED; THUS, 50/.4 INDICATES 0.4 FOOT PENETRATED WITH 50 BLOWS.

Figure 2-67.—Subsurface exploration—penetration resistance and log—example.

- Rock name —
- Lithologic descriptors — Such as texture, grain size and shape, color, porosity and other physical characteristics of importance.
- Bedding/foliation/flow texture — Such as thickness of bedding, banding, or foliation including dip and inclination
- Weathering/alteration — Use the standardized series of weathering descriptors. Also, describe other forms of alterations such as air slaking upon exposure to air.
- Hardness — Use standardized descriptors.
- Discontinuities — Describe shears, fractures, and contacts.

  - Note attitude, spacing, and roughness of bedding, foliation, and fractures.
  - Use standardized fracture density descriptors and characterize fracture frequency and Rock Quality Designation (RQD).
  - Describe shears and shear zones in detail, including all matrix materials present such as reworked material in the shear zone.

- Specific information for design. The empirical ground support (Q and RMR) calculations and the erodibility index utilize specialized parameters.—The need for these parameters should be determined at the start of explorations.

The recommended format for word descriptions (on boring logs) is one of the main headings describing lithologic groups, followed by indented subheadings and text describing important features of the core. Examples of classifications and physical descriptions for rock logging are shown in figure 2-66.

2. Log of Test Pit or Auger Hole.—As shown in figure 2-68, Form 7-1336-A is used for logging test pits, auger holes, and other excavations, such as road cuts, through surficial soils. Predominant use of this form is for construction materials investigations and investigations for line structures such as pipelines and canals. This form is suitable for all types of exploratory holes which produce complete but disturbed samples. It is used for logging auger holes where disturbed sampling is performed following USBR 7010. Other exploratory drill holes advanced by rotary drilling methods are reported on drill hole logs discussed in the previous subsection.

Detailed guidelines for logging test pits and auger holes are available to provide consistency in data presentation. Reclamation investigations of soils for construction require classification and description in accordance with the USCS as described in USBR 5000 or 5005. Guidelines for logging test pits are given in Reclamation training manuals [70] and in the *Engineering Geology Field Manual* [20]. Procedures for obtaining disturbed samples from test pits, trenches, accessible borings, and tunnels are given in USBR 7000. Procedures for obtaining disturbed samples from auger holes are given in USBR 7010.

The log form (fig. 2-68) has a heading area at the top and a remarks area at the bottom. The body of the log form is divided into a series of columns to include the various kinds of information required. Columns are for: classification group symbol; classification and description of material; and the percentage of particles larger than 75 mm (3 in) by volume.

Information under the heading is required. Spaces are provided for supplying information such as project, feature, hole number, location, elevation, dates started and completed, and the name of the person responsible. Depth to bedrock and to water table (free water) are important information and should always be reported. The date when water level was measured is very important information, as water levels can change seasonally. When this or any other information called for on the log cannot be obtained, the reasons must be stated on the log or on other supporting documents.

In the column for classification and group symbol, information such as group symbol, depth intervals logged, and samples taken are recorded. The system allows for modification of the group symbol with either prefixes or suffixes to further describe the soil (Annex X-5, USBR 5000). An example is shown on figure 2-68. The group symbol GW (well graded gravel) is modified by suffixes **scb** which indicate that sand, cobbles, and boulders are present to a significant extent in the sample. According to USBR 5005, the typical name for the strata is WELL GRADED GRAVEL WITH SAND, COBBLES, AND BOULDERS. While this system of abbreviations is useful for geologic drawings of borrow areas and trenches, considerable additional information is contained in the word descriptions, and decisions should not be based on group symbols alone.

In the right column of the log, the amount of oversize material [particles > 75 mm (3 in)] by volume is either estimated by visual inspection or calculated by determining the mass of the oversize. At this time, a detailed procedure for determining the mass of oversize has not been developed. In test pit operations, it may be advisable to screen all material from the pit and determine oversize

7-1336-A (1-86) Bureau of Reclamation	LOG OF TEST PIT OR AUGER HOLE	HOLE NO.

FEATURE
PROJECT
AREA DESIGNATION
GROUND ELEVATION
COORDINATES N E
METHOD OF EXPLORATION
APPROXIMATE DIMENSIONS
LOGGED BY
DEPTH WATER ENCOUNTERED 1/ DATE
DATE(S) LOGGED

CLASSIFICATION GROUP SYMBOL (describe sample taken)	CLASSIFICATION AND DESCRIPTION OF MATERIAL SEE USBR 5000, 5005	% PLUS 3 in (BY VOLUME) 3 - 5 in	5 - 12 in	PLUS 12 in
(GW)scb	0.0 to 7.4 ft WELL-GRADED GRAVEL WITH SAND, COBBLES, AND BOULDERS: About 70% coarse to fine, hard, subrounded gravel; about 30% coarse to fine, hard, subangular sand; trace of fines; no reaction with HCl.			
	TOTAL SAMPLE (BY VOLUME): 22% 3- to 5-inch hard, subrounded cobbles; 14% 5- to 12-inch hard, rounded cobbles; 2 percent plus 12-inch hard, subrounded boulders; remainder minus 3-inch; maximum dimension, 400 mm.	22	14	2
	IN-PLACE CONDITION: homogeneous, dry, brown			
	GEOLOGIC INTERPRETATION: alluvial fan			
7.4 ft				

REMARKS.

SOIL WITH MEASURED PERCENTAGES
OF COBBLES AND BOULDERS

1/ Report to nearest 0.1 foot

GPO 849-366

Figure 2-68.—Log of test pit or auger hole—abbreviated soil classification group symbols for soil with cobbles and boulders.

percentages by sorting over screens or templates and determining the mass of each size range. The procedures used for determining oversize by mass should be included in the construction materials or geologic reports. Once the mass is determined, it can be converted to volume by procedures described in USBR 7000. If oversize is determined by mass, the volumes are reported to the nearest percentage as shown on figure 2-68. If the volume of oversize is estimated visually, report percentages to the nearest 5 percent. If oversize is present, their presence is included in the log by adding to the typical name, and the particles are described as shown on the example of figure 2-68.

The "Remarks" section of the log is for comments on the overall excavation or drilling. Information includes:

- reason for stopping,
- details on equipment used,
- details on instrumentation,
- legend for abbreviations, and
- any miscellaneous comments.

Terminating statements similar to the following would be considered satisfactory:

- hole eliminated due to lack of funds,
- hole caved in (include depth to cave),
- depth limited by capacity of equipment,
- hole terminated at predetermined depth,
- encountered water, and
- unable to penetrate hard material in bottom of hole.

Material should not be described as bedrock, slide material, or similar interpretive terminology unless the exploration actually penetrated such conditions, and samples were collected to substantiate these conclusions. If rock is exposed at the base of a test pit, it can be logged according to guidelines in the *Engineering Geology Field Manual*.

The classification and description portion of the log can vary depending on the type of excavation, type of testing performed, and the materials encountered. For disturbed soils, such as materials from power auger holes or stockpiles, all the information is in a paragraph as follows:

- depth of strata,
- USCS classification group name,
- percentages and descriptions of the size fractions present (gravel, sand, and fines),
- maximum particle size,
- moisture, color, and odor,
- remarks, and
- reaction with dilute solution HCl acid.

If oversize is present in the logged strata, the next paragraph of word descriptions consists of describing the total sample by volume. The estimated percentages are given—along with a description of the cobbles and boulders—and the maximum size present. Noting the presence of oversize on test pit logs is extremely important. If only a trace or more of oversize is present, it must be included in the typical name. If the percentage of oversize (by volume) is less than 50 percent of the total volume, the characteristics of the oversize are described in a separate paragraph following description of the minus 75-mm (3-in) fraction. If the percentage of cobbles and/or boulders, by volume, exceeds 50 percent of the total material, the characteristics of the oversize are described in the first paragraph; the term cobbles and/or boulders is predominant in the typical name, and no classification symbol is assigned.

An additional paragraph is added to the word descriptions if access is available to evaluate the in-place condition of the soil in an undisturbed state. In this case, reporting of moisture, color, and odor is omitted from the first paragraph and replaced with a more detailed description in a following paragraph. In this additional paragraph, information includes the following items in order of presentation.

- consistency (fine grained soils only) — very soft, soft, firm, hard, and very hard
- structure — homogeneous, stratified, blocky, fissured, laminated, etc.
- cementation (coarse-grained soils only)
- moisture, color, and odor
- remarks

An additional example of in-place condition descriptions is given on figure 2-69.

Careful attention must be given to reporting when and where samples are taken and when laboratory testing was performed. If samples are taken, it should be noted in the left column of the log as shown on figure 2-69. A detailed description of how the sample was taken must be added to the word description. In many cases, disturbed sack samples are taken by cutting a vertical strip of soil through the interval being classified. If a sample is taken which does not match the full interval logged, describe the exact location of the sample.

In cases where laboratory testing is performed for classifying the soil, this information must be as shown on the test pit logs. The ability for a classification to identify

7-1336-A (1-86)
Bureau of Reclamation

LOG OF TEST PIT OR AUGER HOLE

HOLE NO. ____________

FEATURE Example	PROJECT
AREA DESIGNATION	GROUND ELEVATION
COORDINATES N ________ E ________	METHOD OF EXPLORATION
APPROXIMATE DIMENSIONS ________ ________	LOGGED BY
DEPTH WATER ENCOUNTERED 1/ ________ DATE ________	DATE(S) LOGGED

CLASSIFICATION GROUP SYMBOL (describe sample taken)	CLASSIFICATION AND DESCRIPTION OF MATERIAL SEE USBR 5000, 5005	% PLUS 3 in (BY VOLUME) 3 - 5 in	 5 - 12 in	 PLUS 12 in
CL three sack samples 4.2 ft	0.0 to 4.2 ft LEAN CLAY: About 90% fines with medium plasticity, high dry strength, medium toughness; about 10% predominantly fine sand; maximum size, medium sand; strong reaction with HCl. IN-PLACE CONDITION: Soft, homogeneous, wet, brown. Three 50-lbm sack samples taken from 12-inch-wide sampling trench for entire interval on north side of test pit. Samples mixed and quartered.			
(SC)g block sample 9.8 ft	4.2 to 9.8 ft CLAYEY SAND WITH GRAVEL: About 50% coarse to fine, hard, subangular to subrounded sand; about 25% fine, hard, subangular to subrounded gravel; about 25% fines with medium plasticity, high dry strength, medium toughness; maximum size, 20 mm; weak reaction with HCl. IN-PLACE CONDITION: homogeneous except for occasional lenses of clean fine sand 1/4 inch to 1 inch thick, moist, reddish-brown. 12- by 12-inch block sample taken at 6.0 to 7.0 ft depth, at center of south side of test pit.			

REMARKS:

TEST PIT WITH SAMPLES TAKEN

1/ Report to nearest 0.1 foot

GPO 849-366

Figure 2-69.—Log of test pit or auger hole—in-place conditions and sampling.

and describe soils is learned under the guidance of experienced personnel and by systematically comparing laboratory test results with visual classifications. On new projects, laboratory tests are sometimes used to "calibrate" the visual classifier when working with unfamiliar soils. However, visual classification is based upon total material observed and that laboratory classifications must be performed on representative materials for comparison with the visual information. In many cases, borrow area investigations will have a certain proportion of laboratory classifications to visual classification as verification. However, on large projects, it is not economical for all the classifications to be based solely upon results of laboratory testing.

When the soil classification is based upon laboratory data, this must be clearly stated and distinctly noted on the log. Particle-size percentages are reported to the nearest 1 percent, and information on Atterberg limits, coefficients of uniformity and curvature are reported on the test pit log. Under the left column group symbol, the group symbols are noted as being obtained through laboratory classification. If laboratory classification tests are performed, and results of visual classification are also reported on the log, laboratory data are presented in a separate paragraph. Figure 2-70 illustrates combined reporting of visual and laboratory classifications. If the laboratory classification based upon USBR 5000 differs from the visual classification, both group symbols are shown in the left column. Do not change the visual classification or description, based upon laboratory data, because the visual description is based upon a widely observed area. If the typical name is different, this is shown in the laboratory test data paragraph.

In-place density tests are often performed in test pits and trenches in borrow areas to determine shrink-swell factors. In-place density tests are also performed in pipeline investigations to evaluate soil support for flexible pipe. In most cases, laboratory compaction tests (USBR 5500, 5525, 5530, and 7240) are performed depending on the material and requirements of the investigation. The compaction tests are used to determined relative density or percent compaction (USBR 7250, 7255). The in-place density and percent compaction or relative density must be reported in the paragraph on in-place condition on the log. In-place density tests such as the sand cone or nuclear gage normally represent a smaller interval than the interval logged. For in-place density tests, the interval is measured or estimated and reported on the log.

On some features, using standard USCS group symbols and typical names may make it difficult to distinguish materials important to construction. For example, topsoil is normally stripped and stockpiled in construction operations. In this case, the log should clearly show the interpreted strata thickness and heading for topsoil present. An example follows:

Classification symbol	Description
TOPSOIL	0.0 to 2.6 ft TOPSOIL—would be classified as ORGANIC SOIL (OL/OH). About 90% fines with low plasticity, slow dilatency, low dry strength, and low toughness; about 10% fine to medium sand; soft, wet, dark brown, organic odor, roots present throughout strata, weak reaction with HCl.

Other material types that may be important to distinguish for design and construction operations may be drill pad, gravel road surfacing, mine tailings, and fill or uncompacted fill. If a certain soil feature requires a special description for investigations, they should be determined at the beginning of the exploration program. The exploration team should consider these requirements and advise investigators of the special cases.

The USCS was developed to classify naturally occurring soils; it was not intended for classifying lithified or manmade materials. Problems can occur by classifying a partially lithified material such as a weathered rock or shale as a soil. An example would be the use of power-auger holes in shales. If the power auger grinds up the shale into a soil-like material, problems could arise when excavating during construction. It is important to note that the visual classification was performed on materials after processing. Other materials such as claystones, processed aggregates, and natural materials (e.g., shells) are examples of materials where the group symbol and typical name from USCS should not be used without careful notation. Examples of logs for these materials are shown on figure 2-71. The USCS typical name is enclosed within quotation marks as the classification of material after processing.

Geologic interpretations should be made by or under the supervision of a geologist. The geologic interpretation is presented in a separate paragraph. An example is shown on figure 2-68. Factual geologic data can be provided in construction materials reports.

Large machine-dug test pits or test trenches may require more than one log to adequately describe variation in

7-1336-A (1-86)
Bureau of Reclamation

LOG OF TEST PIT OR AUGER HOLE

HOLE NO. ____________

FEATURE ____________ PROJECT ____________
AREA DESIGNATION ____________ GROUND ELEVATION ____________
COORDINATES N ________ E ________ METHOD OF EXPLORATION ____________
APPROXIMATE DIMENSIONS ____________ LOGGED BY ____________
DEPTH WATER ENCOUNTERED 1/ ________ DATE ________ DATE(S) LOGGED ____________

CLASSIFICATION GROUP SYMBOL (describe sample taken)	CLASSIFICATION AND DESCRIPTION OF MATERIAL SEE USBR 5000, 5005	% PLUS 3 in (BY VOLUME) 3-5 in	5-12 in	PLUS 12 in
GP (visual) GW (lab classif) three sack sample 3.2 ft	0.0 to 3.2 ft POORLY GRADED GRAVEL WITH SAND: About 70% coarse to fine, hard, subangular gravel; about 30% coarse to fine, hard, subangular sand; trace of fines; maximum size, 75 mm; no reaction with HCl. IN-PLACE CONDITION: homogeneous, moist, brown LAB TEST DATA: Sample had 64% gravel, 34% sand, 2% fines, Cu = 24, Cc = 1.8 Laboratory classification is WELL-GRADED GRAVEL WITH SAND. Three 50-lbm sack sample taken for testing from 18-inch-wide sampling trench for entire depth interval on east side of trench. Material mixed and quartered to get sample.			
CL (lab classif) one sack sample 7.6 ft	3.2 to 7.6 ft LEAN CLAY: About 90% fines with medium plasticity, high dry strength, medium toughness; about 10% predominantly fine sand; maxiumum size coarse sand; no reaction with HCl. IN-PLACE CONDITION: Firm, homogeneous, moist, yellowish-brown. LAB TEST DATA: 86% fines, 14% sand, LL = 36, PI = 19 One 40-lbm sack sample taken for testing from 12-inch-wide sampling trench from 4.7 to 6.8 ft depth.			

REMARKS:

REPORTING LABORATORY CLASSIFICATION IN ADDITION TO VISUAL CLASSIFICATION

1/ Report to nearest 0.1 foot

GPO 849-356

Figure 2-70.—Log of test pit or auger hole—combined laboratory and visual classification.

7-1336 A (1-86)
Bureau of Reclamation

LOG OF TEST PIT OR AUGER HOLE

HOLE NO. ____

FEATURE ____ PROJECT ____
AREA DESIGNATION ____ GROUND ELEVATION ____
COORDINATES N ____ E ____ METHOD OF EXPLORATION ____
APPROXIMATE DIMENSIONS ____ LOGGED BY ____
DEPTH WATER ENCOUNTERED 1/ ____ DATE ____ DATE(S) LOGGED ____

CLASSIFICATION GROUP SYMBOL (describe sample taken)	CLASSIFICATION AND DESCRIPTION OF MATERIAL SEE USBR 5000, 5005	% PLUS 3 in (BY VOLUME) 3 - 5 in	5 - 12 in	PLUS 12 in
SHALE CHUNKS 12.6 ft	11.2 to 12.6 ft SHALE CHUNKS: Retrieved as 2- to 4-inch pieces of shale from power auger hole, dry, brown, no reaction with HCl. After slaking in water for 24 hours, material identified as "SANDY LEAN CLAY (CL)" - About 60 percent fines with medium plasticity, high dry strength, no dilatancy, medium toughness; about 35 percent fine to medium sand; about 5% gravel-size pieces of shale			
CRUSHED SANDSTONE Bin No. 3	Bin No. 3 CRUSHED SANDSTONE: Product of commercial crushing operation; "POORLY GRADED SAND WITH SILT (SP-SM)" - About 90% fine to medium sand; about 10% nonplastic fines; maximum size, medium sand; dry, reddish-brown; strong reaction with HCl.			
CRUSHED ROCK NE Stockpile	NE Stockpile CRUSHED ROCK: Processed from gravel and cobbles in Pit No. 7; "POORLY GRADED GRAVEL (GP)" - About 90% fine, hard, angular gravel-size particles; about 10% coarse, hard, angular sand-size particles; maximum size, 19 mm; dry, tan; no reaction with HCl.			
BROKEN SHELLS 3.2 ft	0.0 to 3.2 ft BROKEN SHELLS: Natural deposit of shells; "POORLY GRADED GRAVEL WITH SAND (GP)" - About 60% gravel-size broken shells; about 35% sand and sand-size shell pieces; about 5% fines.			

REMARKS:
MATERIALS OTHER THAN NATURAL SOILS

1/ Report to nearest 0.1 foot

GPO 849-366

Figure 2-71.—Log of test pit or auger hole—materials other than natural soils.

materials found in different portions of the pit or trench. The initial log of such pits or trenches should describe a vertical column of soil at the deepest part of the excavation, usually at the center of one wall of the pit or trench. If this one log will not adequately describe variation in the different strata exposed by the pit or trench, additional logs should be prepared for other locations within the test excavation to provide a true representation of all strata encountered in the test pit or trench. In long trenches, at least one log should be prepared for each 15 m (50 ft) of trench wall, regardless of uniformity of material or strata. A geologic section of one or both walls of long test trenches is normally required to describe variation in strata and material between log locations. When more than one log is needed to describe the material in an exploratory pit or trench, coordinate location and ground surface elevation should be noted for each point for which a log is prepared. A plan geologic map and geologic sections should always be prepared for test trenches that encounter bedrock in structure foundations.

Sketches of test pit walls are useful to describe variability of materials. Figure 2-72 shows a sketch depicting location of the pit wall which was logged. This figure is attached to the test pit log for reference. Additional examples can be found in the *Engineering Geology Field Manual* [20]. Photographs of test pit walls are valuable inclusions in geologic design data and construction materials reports.

2-24. Subsurface Sections.—Guidelines for preparing geologic drawings and sections are presented in the *Engineering Geology Field Manual* and *Engineering Geology Office Manual* [20, 68]. These guides contain detailed instructions for developing geologic drawings. The five primary objectives of geologic sections are to:

- Compile and correlate surface and subsurface geologic data.
- Present geologic interpretations.
- Save the user time by concisely and accurately displaying pertinent geologic conditions.
- Graphically show in two or three dimensions the subsurface conditions which cannot be determined with ease from a geologic map, particularly those interpretations which may be significant to engineering planning or design.
- Indicate probable structure excavation limits and show geotechnical considerations or treatment.

Geological sections are used to show interpreted subsurface conditions in geological reports, materials reports, and design data for dams, canals, and other project features. Location of sections should be chosen so as to present conditions described in the best possible way. Cross-valley sections are generally more informative than a series of sections parallel to the valley. Also, sections should cross physical features at right angles as nearly as possible. A clear differentiation should always be maintained between factual and interpretive data. A system where lines range from dotted to solid is recommended. In this system dots represent purely hypothetical interpretation, a solid line represents fact, and dashed lines define the degree of reliability of intermediate data according to length of dash. The cross section should always show the name of the person who made the interpretation and the date the interpretation was made.

2-25. Sampling.—Samples of soil and of rock are collected for visual examination so that a log of the test hole may be prepared for preservation as representative samples in support of the descriptive log, for testing to determine index properties, and for laboratory testing to determine engineering properties.

Requirements for undisturbed samples are given in USBR 7100 and 7105.

When drilling core holes, the total material recovered as core is collected and stored in core resealable boxes. In addition, samples of soil should be collected and placed in sealed pint jars or resealable plastic bags to preserve the natural water content representative of each moist or wet stratum. Samples representative of the various types of material found in the area under investigation should be collected as the work progresses. Samples should be 100 to 150 mm (4 to 6 in) long and must be representative of that material found in the area, particularly as to the degree of alteration. If a wide variation in material quality exists, samples representative of the ranges of the material should be collected.

When exploring for materials in borrow areas and in foundations where substantial quantities occur that potentially may be used in embankment construction, samples should be collected representative of each stratum in a volume sufficient to provide 35 kg (75 lb) of material passing a 4.75-mm (No. 4) sieve to be used for testing for engineering properties. Only material larger than 75 mm (3 in) should be removed from a sample, and the percentage of plus 75 mm removed should be reported. However, in some cases, larger samples are required for tests on total material. If the entire test hole appears to be in uniform material, samples from the upper, middle, and lower one-third of the test hole should be collected. USBR 5205 should be referred to for size of sample required for a particular test.

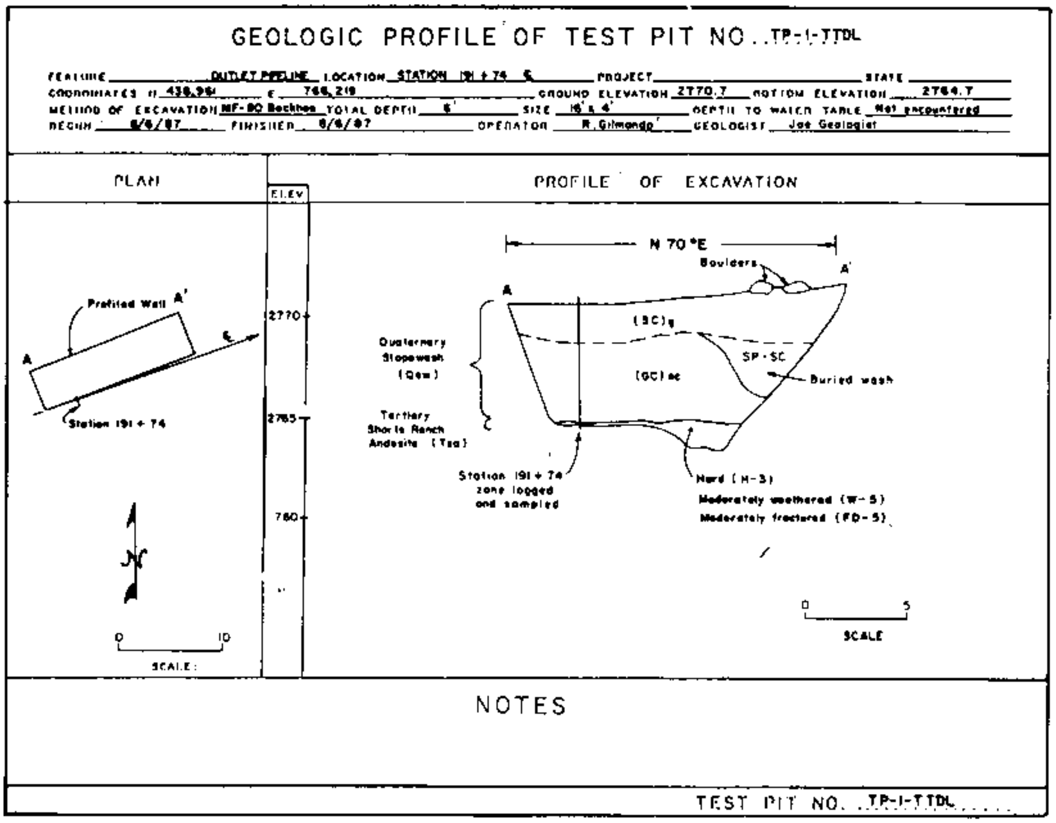


Figure 2-72.—Geologic interpretation in test pit (geologic profile).

When investigating riprap sources, samples consisting of three or four pieces of rock totaling at least 275 kg (600 lb) representative of the source should be collected. Collecting samples of concrete aggregates is covered in the *Concrete Manual* [20]. Collecting samples of blanketing material, filter material, and ballast should conform to requirements for collection of borrow material for embankment construction.

From the set(s) of samples collected described above, samples are selected for performing index and engineering properties tests. Care should be taken to preserve sufficient samples to substantiate logs of exploratory holes and for representative samples to be transmitted.

Samples collected in the process of routine exploration generally are not satisfactory for testing associated with determining properties of in-place soil or rock. For this purpose, samples of material unaffected by seasonal climatic influence are collected from large-diameter boreholes [100 to 150 mm (4 to 6 in) in diameter minimum] or from the bottom of open pits. Borehole samples should be 300 to 600 mm (12 to 24 in) long, and open-pit samples 250- to 450-mm (10- to 18-in) cubes. Every effort should be made to preserve such samples in as nearly an in-place natural condition as possible. Procedures for collecting and preserving this type sample are described in USBR 7100 and 7105.

2-26. Reports.—

a. General.—Guidelines for preparing geologic design data reports are presented in the *Engineering Geology Office Manual* [69]. This guide contains detailed instructions concerning required information for inclusion in geologic design data reports.

The results of every investigation should be presented in a report. In the reconnaissance stage for a small structure, a letter report describing in general terms the nature of problems associated with the investigation, the extent of the investigation, and the conclusions reached may suffice. As an investigation proceeds, additional data are collected and evaluated. As the various stages of investigation proceed, previously assembled material is incorporated in a progress report. During preparation of this progress report, previous reports should be examined; those questions which have been answered should be noted in the report together with the resolution or, if unanswered, should be carried over for future consideration. The final report should either answer all questions raised in the past or discuss why a positive solution cannot be reached within the scope of an investigation.

Every investigation should contain:

- A statement of the purpose of the investigation
- The stage for which the report is being prepared
- The kind of structure contemplated or the type of study
- The principal dimensions of the structure

The following features pertaining to foundations and earthwork should be included in all reports.

b. Foundations and Earthwork.—Foundation data should reflect recognition and consideration of the type and size of the particular engineering structure and the effect on, or relationship to, the structure of the significant characteristics of foundation materials and conditions at the particular site.

The general regional geology should be described. The description should include major geological features, names of formations found in the area, their age, their relationship to one another, their general physical characteristics, and seismicity.

- A description and interpretation of local geology should include:

  - physical quality and geologic structure of foundation strata,
  - groundwater and seismic conditions,
  - existing and potential slide areas, and
  - engineering geologic interpretations appropriate to the engineering structure involved.

- Geologic logs of all subsurface explorations should be included in the report.
- Combine geologic map plotted on the topographic map of the site showing surface geology and the location of geologic sections and explorations.
- Supplement the above map by geologic sections showing known and interpreted geologic conditions related to engineering structures.
- Photograph pertinent geologic and topographic features of the terrain, including aerial photographs for mosaics, if available, are valuable additions to the report.

Engineering data on overburden soils within the foundation of the proposed structure should be shown by detailed soil profiles and reported as follows:

- A classification of the soil in each major stratum according to the Unified Soil Classification System.
- A description of the undisturbed state of the soil in each stratum.
- A delineation of the lateral extent and thickness of critical, competent, poor, or potentially unstable strata.
- An estimate, or a determination by tests, of the significant engineering properties of the strata such as density, permeability, shear strength, and compressibility or expansion characteristics; and the effect of structure load, changes in water content, and fluctuations or permanent rise of ground water on these properties.
- An estimate or a determination of the corrosive properties and sulphate content of the soil and ground water as affecting the choice of cement for use in structures.

For data on bedrock, the following are required:

- A description of the depth to and contour of bedrock, thickness of weathered, altered, or otherwise softened zones, and other structural weaknesses and discontinuities.
- A delineation of structurally weak, pervious, and potentially unstable zones and strata of soft rock and/or soil.
- An estimate or a determination of the significant engineering properties of bedrock such as density, absorption, permeability, shear strength, stress, and strain characteristics; and the effect of structure load, changes in water content, and fluctuations or permanent rise of groundwater on these properties.

c. Construction Materials Data.—As part of the design data for earth dams, an earth materials report is required containing a list of:

- available impermeable soils,
- permeable soils,
- sand for filters, and
- rock for riprap and rockfill.

Occasionally, a list is required for canals and other large structures when appreciable quantities of these materials are required. Sometimes, similar reports for smaller quantities of special materials are needed. The principal items to be covered in the earth materials report are:

- A grid map showing the topography of the deposit and of the structure site and the intervening terrain if within a radius of 3.2 km (2 mi). The location of test holes and trenches should be shown using standard symbols.
- Ownership of deposit.
- Brief description of topography and vegetation.
- Estimated thickness of the deposit, including variations. Drawings showing subsurface profiles along grid lines should be included.
- Areal extent of the deposit.
- Estimated quantity of the deposit.
- Type and thickness of overburden.
- Depth to water.
- Accessibility to the source.
- General description of rock.
- Amount of jointing and thickness of bedding of rock strata.
- Spacing, shape, angularity, average size, and range of sizes of natural boulder deposits.
- Brief description of shape and angularity of rock fragments found on slopes and of the manner and sizes into which the rock breaks when blasted.
- Logs of all auger test holes and exposed faces of test trenches or pits.
- An estimate, or determination by tests, of the pertinent index and engineering properties of soils encountered. The amount of testing should be limited in the feasibility stage but should be more detailed for specifications.
- Photographs, maps, and other drawings are helpful and desirable for the record of explorations.

In most cases, information gathered for the earth materials report of an earth dam, for final design and specifications, may not be of sufficient detail to permit development of a plan for using available earth materials to the best advantage. However, explorations should be sufficient to assure that sufficient materials required are available. As soon as funds are available for constructing the dam, additional studies can be performed.

The primary purposes of a detailed study are to determine the depth of borrow pit cuts, the most efficient distribution of materials to be placed in the embankment, and the need for addition or removal of moisture. In most cases, it is desirable to add moisture to dry, impervious borrow materials before excavating. Studies should include an analysis of moisture conditions in each borrow area from which plans may be developed for irrigating the areas. If materials in borrow areas are too wet for proper placement, plans for draining these areas may be based upon results of detailed studies. Seasonal variations of water content, variation of water content with depth, and rate of water penetration are items requiring consideration.

Detailed investigations are also desirable for canals and structures where large quantities of required excavations and borrow are involved. In any case, sufficient preconstruction explorations should be made to know where specified types of materials are to be obtained and where all materials are to be placed.

Information on concrete aggregates should be reported according to instructions in the *Concrete Manual* [20]. Information on sources and character of acceptable road surfacing materials, if required, should be given in the construction materials report. Reference should be made to results of sampling and analysis of materials, including previous tests. Figures 2-73 and 2-74 are example forms for summarizing field and laboratory tests on embankment materials which accompany preconstruction reports.

Form 7-1465 (9-58)
BUREAU OF RECLAMATION

SUMMARY OF FIELD AND LABORATORY TESTS FOR EMBANKMENT MATERIALS REPORT
IMPERMEABLE TYPE MATERIALS

Feature
Project
Date of report
Borrow area

FIELD: Identification of exploration (prefix and number)	Location of exploration (coordinates)	Composite depth of sample tested, or depth of density test (ft)	Field identification (unified soil classification group symbol)	Max particle size (inches)	Oversize percentage of volume: Over 5 inches	Over 3 inches	FIELD AND LABORATORY: Specific gravity plus No. 4 fraction	Density in place minus No. 4 fraction: Dry density (lbs per c.f.)	Water content % of dry wt	LABORATORY: Mechanical analysis minus 3" fraction % of dry weight: % passing No. 4 sieve	% passing No. 200 sieve	Characteristics at standard laboratory compaction: Proctor max dry density (lbs per c.f.)	Optimum water content % of dry wt	Proctor needle (lbs per sq in)	Specific gravity minus No. 4 fraction	Percolation settlement test, before saturation: Dry density as placed (lbs per c.f.)	Water content as placed (% of dry wt)	Applied pressure (lbs per sq in)	% settlement of placed sample	Saturated: Proctor needle (lbs per sq in)	Water content (% of dry wt)	% settlement of placed sample	Percolation rate (ft per yr)
TP-200	N 202,000 - 10,000 E	10-21.0	GW-GC	6	3.0	7.0	2.54	—	—	41.1	10.0	126.1	12.3	1200	2.69	123.1	10.3	20	3.1	900	14.3	3.1	0.37
AH-400	N 177,000 - 6,000 E	1.5-21.5	CL	[illegible]	0	0	—	—	—	92.3	84.0	112.2	19.1	600	2.73	109.8	17.1	100	4.1	450	22.6	4.1	0.012
TP-20[illegible]	N 202,000 - 10,500 E	5.0	GC	3	0	0	—	114.0	8.2	40.2	11.0	125.6	12.8	1100	—	—	—	—	—	—	—	—	—
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24

NOTES

Column 1 TP-200 and AH-400 are in different borrow areas and would ordinarily be on different sheets
3 Depths are from ground surface. Topsoil should not be included in the sample.
4 Classification is for composite sample
5 Maximum size of particle encountered in depths shown in column 3
6 7 Obtained by weighing rock, assuming or measuring specific gravity and measuring volume of hole or sampling trench.
8 When determined in laboratory.
9 10 See field density test procedure. Tests required for coarsest, finest, and average material for each proposed borrow source
13 14 15 Proctor compaction test
17 18 Use optimum water content and maximum dry density (columns 13 and 14)
19 Use 100 psi for manifestly impervious soils, 20 psi for soils of doubtful imperviousness.
20 Determined from settlement gage readings after consolidation under load is practically complete, and before saturation
21 Average of 3 readings at end of test.
22 Obtained by drying and weighing of sample
23 Determined from settlement gage reading at end of test, with load (column 19) held constant

Sheet ... of ...

Figure 2-73.—Summary of field and laboratory tests for embankment materials report—impermeable-type materials.

Form 7-1530 (9-58)
BUREAU OF RECLAMATION

SUMMARY OF FIELD AND LABORATORY TESTS FOR EMBANKMENT MATERIALS REPORT
PERMEABLE TYPE MATERIALS

Dam
Project
Date of report
Borrow area

FIELD: Identification of exploration (prefix and number)	Location of exploration (coordinates)	Composite depth of sample tested, or depth of density test (ft)	Field identification (unified soil classification group symbol)	Maximum particle size (inches)	Oversize percentage of volume: Greater than 5-inch	5-inch to 3-inch	LABORATORY: Soil gradation percentage of dry weight: 1½-inch to 3-inch	No. 4 to 1½-inch	Smaller than No. 200	Specific gravity: Plus 3-inch	Minus 3-inch	Maximum dry density lbs. per c.f. minus 3-inch	Minimum dry density lbs. per c.f. minus 3-inch

Sheet ... of ...

Figure 2-74.—Summary of field and laboratory tests for embankment materials report—permeable-type materials.

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Chapter 3

CONTROL OF EARTH CONSTRUCTION

A. Principles of Construction Control

3-1. General.—In many types of engineering work, structural materials are manufactured to obtain certain characteristics; their use is prescribed by building codes, handbooks, and codes of practice established by various engineering organizations. However, for earth construction, the common practice is to use material that is available locally rather than specifying that a particular type of material of specific properties be secured. Likewise, a variety of procedures exists by which earth materials may be satisfactorily incorporated into a structure. When earth is the construction material, personnel in charge of construction control must become familiar with design requirements and must verify that the finished product meets the requirements.

Design of earth works must allow for an inherent range of earth material properties. For maximum economy, tolerance ranges will vary according to available materials, conditions of use, and anticipated methods of construction. A closer relationship is required among the operations of inspection, design, and construction for earthwork than is needed in other engineering disciplines. Construction control of earthwork involves not only practices similar to those normally required for structures using manufactured materials, but also the supervision and inspection normally performed at the manufacturing plant. In earthwork construction, the processes by which an acceptable material is produced are performed in the field.

Structures designed by the Bureau of Reclamation (Reclamation) are usually built by a contractor. The basis of the contract is a set of specifications that has a schedule of items of work. Through information provided in specifications, the contractor proposes prices for performing the items of work that, when accepted, become a part of the agreement between the Government and the contractor. The primary functions of the construction control organization are to:

- Ensure the structure is built according to specifications.
- Report and document any changed conditions.
- Certify to what extent the items of work have been completed.
- Test for compliance with requirements.
- Determine payment due the contractor.

Although the Government prepares plans and specifications defining work to be performed, once a contract is signed, the Government representatives have no right to change contract requirements, and the contractor has no right to change unit prices. A condition may exist in the field that is different from what was anticipated to exist during preparation of the specifications. If changes to designs are necessary, a contract modification is agreed upon by both the contractor and the contracting officer.

Specifications requirements for earthwork construction may be grouped into two types: those requirements based on performance and those requirements based on procedures. This distinction must be clearly recognized. The requirement for earthfill in a dam embankment is usually based on both a minimum procedure and on performance. If the two requirements prove to be incompatible, the performance is adjusted to obtain compatibility. The desirability of a performance-type requirement is recognized; however, the present state of knowledge of soil behavior, the complexity of specifications required, and the extensive testing requirement make this type of specification very expensive for some types of noncritical earthwork construction.

When developing specifications, it is very important to include the assumptions, confidence, and uncertainties inherent in the design. This allows both the contractor and contract administration personnel a clearer understanding of the expectations of the designer and the inherent risks involved in the contract. It has been Reclamation practice to make all existing data available to bidders either by incorporation of the data into specifications or by referencing reports and making them available for review. In the past, it was Reclamation's practice to develop a separate document called "construction considerations" which included the designers' assumptions, uncertainties, and requirements for successful construction. Reclamation has recently attempted to include some of these considerations directly in the specifications. An example would be to include the reasoning for excavations shown on certain levels in the foundation. Acceptable foundation conditions should be given along with the reasoning and the geologic interpretations providing the basis for the excavation limits. Also, Reclamation gives prebid briefings to prospective contractors. During these briefings, it is helpful to show a video of preconstruction excavations or to actually perform excavations such as test pitting for the prospective bidders.

Often, Reclamation performs rather extensive design investigations; however, they are performed to collect design data and may not be related to constructability. Given the nature of the earth and its complex geology, it is not economical to attempt to perform extensive investigations to alleviate any change of condition. Contractors are encouraged to perform prebid investigations and investigations as construction progresses, as necessary, to avoid surprises during construction.

Specifications requirements also may be divided into two groups: definite requirements and those qualified by the phrase "as directed." The undesirability of the latter requirement is recognized and is avoided whenever possible. However, the "as directed" type of requirement is used in establishing minor dimensions in areas where investigations sufficient to establish such dimensions are not justified and when new conditions are encountered for which requirements have not been established.

Where the "as directed" requirement refers to dimensioning, either maximum and minimum dimensions or an average dimension is noted on the design drawings. If maximum and minimum dimensions are stated, the usual practice is to excavate to the minimum dimension; and where visual examination indicates the excavation is still within inferior material, excavation up to the maximum dimension is continued, if required, to reach a satisfactory foundation.

Where the "as directed" requirement refers to a new condition, the contractor and construction engineer should exercise joint effort to establish practical performance limits or to establish procedures that will produce satisfactory results where performance cannot readily be defined. Some experimentation is desirable, but this aspect should be small compared to the total requirement.

3-2. Inspection.—The adequacy of construction is determined by visual examination, by measurements, and by testing. Inspection determines whether the requirements of plans and specifications are being satisfied; it does not determine what the requirements should be. An inspector must become familiar with the specified requirements for work to be inspected. In determining whether work satisfies the requirements, an inspector should also be familiar with how the requirement is defined. For earthwork construction, dimensional requirements and quality requirements are evaluated. A requirement may be defined as a condition to be achieved from which certain deviations (plus or minus) are tolerable, or it may be defined as a limiting condition from which deviation is allowable in only one specified direction. Both methods are in common use. Although a procedure is specified for some types of work (notably, earthfill in dam embankments), inspection may be made on the basis of satisfying a performance requirement. Basically, inspection determines only whether work is acceptable or not acceptable, but it also is desirable to determine the *extent* to which work is acceptable or unacceptable.

An inspector should be familiar with the various safety and health regulations for construction activities. Inspectors should stay fully informed on progress of work and on future programmed work.

To facilitate construction control, relationships between engineering properties test results and index test results are established wherever possible. In addition, visual observations of soil characteristics are correlated with both index and engineering properties. When noticeable differences exist between acceptable and unacceptable work or materials, sufficient testing is required to confirm that differences persist. As these differences become less obvious, the amount of testing should be increased. In any event, testing should be sufficient to provide adequate quality control and to furnish the necessary permanent records.

It is impractical to completely test all the work performed. The usual procedure is to select samples of work or materials for testing that are representative of some unit of work or material. Accuracy of such procedures depends on:

- the relationship of sample size to size of unit it represents,
- procedures used for sample selection, and
- frequency of sampling.

For most earthwork construction performed for Reclamation, the ratio of sample size to unit of work or material represented is small, so special sample selection procedures are used. Since the principal objective is to ensure adequate work, samples are selected at random with a minimum recommended frequency.

3-3. Field Laboratory Facilities.—The primary purpose of a field laboratory is to perform testing of construction materials. Test data serve as bases for determining and ensuring compliance with specifications, for securing maximum benefit from materials being used, and for providing a record of materials placed. Physical properties testing of earth materials during investigation stages may be performed in the field laboratory or in the Denver Office laboratories—or a combination used.

The size and type of field laboratory are dependent on magnitude of the job and on types of structures. When a laboratory building needs to be constructed, requirements usually will conform to one of the designs shown on figure 3-1 or a suitable modification. The Type C arrangement shown on figure 3-1 is appropriate for major earthwork projects. This design was used for Palisades Dam, Idaho, which contains about 10.7 million m^3 (14 million yd^3) of embankment and 115,000 m^3 (150,000 yd^3) of concrete. It was used at Trinity Dam, California, which has about 23 million m^3 (30 million yd^3) of earth embankment and 76,500 m^3 (100,000 yd^3) of concrete. The laboratory shown on figure 3-1 as Type B is applicable for projects where facilities are required for both earth and concrete testing of moderate scale. This type laboratory was used at Heart Butte Dam, North Dakota, which has about 1,070,000 m^3 (1.4 million yd^3) of earthwork and 7,600 m^3 (10,000 yd^3) of concrete. The small mobile or stationary laboratories shown as Type A on figure 3-1 are suitable for small projects, as a satellite laboratory on a large earthwork project, or on projects divided into several work divisions. They may be truck-mounted or trailer units. An example of a mobile laboratory is shown on figure 3-2.

At times, the type of laboratory suitable for a particular job may be difficult to determine. The main factors in determining the size and number of laboratories are size of contemplated work, concentration of work, and complexity of materials to be tested. In most cases, concrete and earth materials testing are combined into the same building. However, for small satellite control laboratories, separate facilities for earthwork control may be desirable. When earthwork is concentrated at one location—as for a dam—the project laboratory can be erected near the worksite so that laboratory facilities are immediately available for necessary control work. When earthwork is spread out over long distances, as in the cases of canal and road construction, testing facilities in addition to those at the main project laboratory must be provided near the work. Some projects employ a utility-type vehicle or an equivalent equipped with the necessary testing equipment as shown in figure 3-2. Other projects have used small skid-mounted buildings. As work progresses, these buildings are towed or hauled to new locations. Other projects have used large portable boxes where equipment can be stored. For testing work in very dry or rainy weather, sheltered facilities are advantageous so soil tests can be made without objectionable soil moisture changes.

To ensure acceptable accuracy of test results, not only must equipment be maintained to a known accuracy, but the laboratory testing environment requires efficient control. Laboratory areas, where permeability and hydrometer analyses are performed, should be maintained at a comfortable, uniform temperature with a variation in temperature not to exceed ± 3 °C (± 5 °F); otherwise, test results may be erratic and questionable. Usually, the most satisfactory method of temperature control is a central forced air system where air is heated or cooled (as required), filtered to remove dust, and returned through ducts to the various parts of the laboratory. Laboratory areas receiving heat from other sources such as concentrated sunlight, radiators, ovens, or hotplates should be avoided for tests where temperature change would adversely affect test results. For example, a hydrometer analysis would be adversely affected by a heating radiator if a constant temperature water bath is not used.

Some types of balances are sensitive to air currents. These balances must be either properly shielded or located in areas where air currents (from the heating or ventilating system) do not interfere with their proper operation. If possible, a high relative humidity should be maintained in areas where undisturbed soil samples are stored.

Operations that produce dust, such as sieving, processing, or pulverizing, should be conducted in an area separate from the main laboratory; an adequate system should be provided for removing dust from the atmosphere either by filtration or exhausting to the outside. Noisy operations such as sieving, compacting, and maximum index density testing should be conducted in rooms separate from the main laboratory because of their adverse effect on personnel and on other laboratory operations. When exposed to noise levels above 85 decibels for over 15 minutes per day, proper hearing protection must be worn. This noise level is commonly attained during the sieving of soils containing gravel.

3-4. Laboratory Data.—Data forms generated in daily operations of field laboratories are important technical and legal documents; they must be properly handled and processed as described in USBR 9300: Checking, Rounding, and Reporting of Laboratory Data.[1] Following correct procedures produces uniformity and consistency in data reporting.

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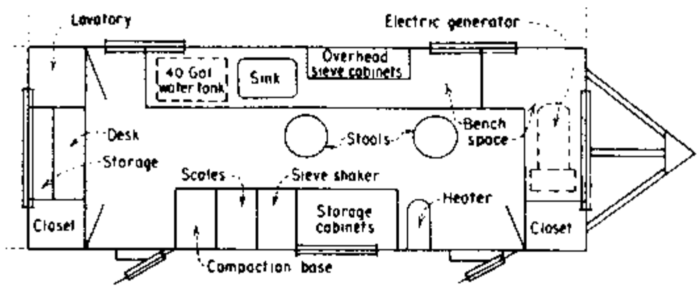
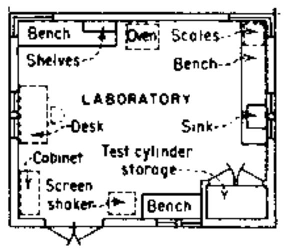
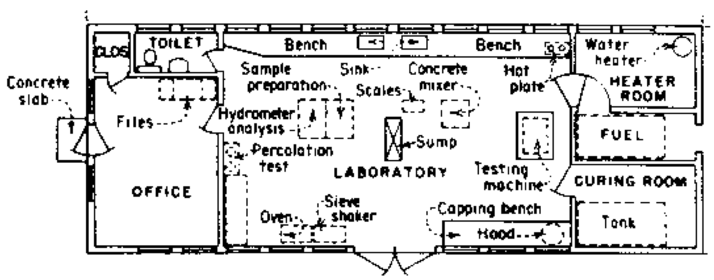
[1] 9300: Checking, Rounding, and Reporting of Laboratory Data.



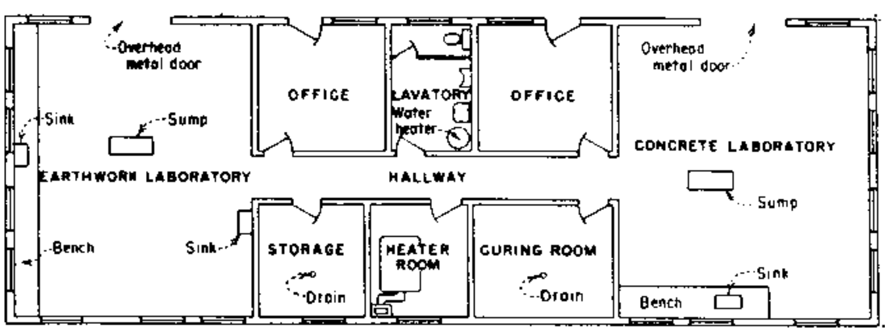
(TYPE A)
(MOBILE)



(TYPE A)
(STATIONARY)



(TYPE B)



(TYPE C)

Figure 3-1.—Examples of floor plans for field control laboratories.

a. Semimobile trailer unit used as a field laboratory.

b. Type V laboratory—vehicle equipped for on-site earthwork control tests.

Figure 3-2.—Typical field laboratories.

Proper data recording means not only writing a measured or calculated number in the correct location—with the appropriate number of digits—but also completing the test identification information on each data form. Observations made during the test that might have an effect on test results must be noted. Erasures are not allowed on data forms. If a value has been erroneously recorded or calculated, a line is drawn through the incorrect number, and the correct value is written above or next to the original value.

All data calculated or transferred from another source must be checked; the checkmarks are shown on the form. It is important that the one who checks data uses the same steps or method of computation as the original data reporter. Numbers calculated by "chain" computations (with a calculator or computer) may result in different values than those obtained when values are rounded for each entry and the rounded value is used in subsequent calculations.

When a numerical value is to be rounded to fewer digits than the total existing digits, rounding should be done as follows:

When the first digit dropped is:	The last digit retained is:	Examples of rounded numbers:
< 5	unchanged	2.44 to 2.4
> 5	increased by 1	2.46 to 2.5
exactly 5	increased by 1	2.55 to 2.6
5 followed only by zeros	increased by 1	2.5500 to 2.6

The same policy (rules) applies when rounding a number with many digits to a number with few digits. A computer or calculator may display the answer to a computation as 10 digits, and the answer is to be recorded to 2 digits. For example, the number 2.3456789 would be rounded to 2.3; the first digit dropped would be the 4; other examples are:

2.49999 to 2.5	2.55555 to 3
2.49999 to 2	2.50000 to 3
2.55555 to 2.56	

The above examples of exactly 5 or 5 followed by zeros are rounded differently than indicated in some references. These references indicate the number is to be rounded to the closest even number. That is, 2.50000 would be rounded to 2 and not 3. Unfortunately, calculators and computers do not follow this rule and always round up. Recognizing the universal use of calculators and computers, the policy as stated should be followed.

Data sheets, equipment literature, and calibration records must be filed for easy and ready retrieval any time during construction.

3-5. Reports.—A record of construction operations should be maintained as they are indispensable when repair or modification of the structure is required in the future. Also, a record is necessary when claims are made by either the contractor or the Government that work required or performed was not according to the contract. Recorded data are beneficial in improving engineering knowledge and practices for future work. Basic documents of the construction record are:

- Plans and specifications
- Adopted modifications that were considered to come within the terms of the contract
- Amendments made to the contract as extra work orders
- Orders for work changes
- Contractual protests
- Results of:
  - tests,
  - measurements of work performed, and
  - contract earnings.

To ensure that a proper record of construction is developed and available, various periodic reports are required. By reviewing these reports, supervisory personnel can determine whether proper performance is being achieved or whether deficiencies or misunderstandings exist. Necessary corrections can be made quickly on the basis of such reports. The progress report permits coordination of various operations required for servicing a contract to be performed in a timely and efficient manner.

The amount of reporting required varies according to function and degree of supervision.

- Reports are made of every test performed in the laboratory and in the field.
- Inspectors make daily reports concerning *adequacy*, *progress*, and *comments* on decisions. These daily reports may be of vital importance in subsequent actions.
- Administrative personnel make monthly reports on quantity of work performed, contract earnings, safety, employment records, and various other statistical information as required.

- Intermediate and supervisory personnel summarize these basic data periodically and make frequent informal and monthly formal reports, as required, and they include reports of all decisions made on controversial matters.

Every construction project submits a monthly progress report (referred to as the "L-29" report), "Construction Progress Report, Concrete Construction Data, and Earthwork Construction Data." This report covers the progress of current construction activities and the structural behavior of completed or partially completed features in which observations are being made. Also, it provides a continuous record of events and data for future reference.

Specialized reports on grouting, pile driving, instrumentation, soil-cement construction, etc., are required as needed [1]. These "technical" or "specialist" reports contain:

- Tabulations of test results
- Statements of progress
- Amount of work performed
- Any questions concerning interpretation of requirements or test results
- Descriptions of abnormal conditions or methods that affect either quality or quantity of work accomplished

Copies of all these reports are sent to the central office for immediate review by specialists so assistance or advice concerning performance of work or testing can be provided promptly if necessary.

Upon completing an earth dam embankment or other major works, a *Final Construction Report* (summarizing the work accomplished) should be prepared. In special cases, additional summaries or reports may be required.

The narrative summary of these reports should be restricted to matters of special importance or interest relating to technical control exercised in earthwork construction. All summaries should describe any difficult or unusual experiences encountered during the reporting period. Types of information desired are exemplified in the following list. Of the items listed, only those that warrant reporting should be included, such as:

- Description of foundation conditions encountered and methods, procedures, and equipment used in: preparing the foundation, stripping, excavating, dewatering and unwatering the foundation, placing piling or caissons, consolidating soils in place, etc.; also, methods for protecting the foundation surface prior to placing materials such as installation of drains, wells, filters, geosynthetics, concrete blankets, sealers, etc. Report unusual or unexpected foundation conditions encountered and methods of treatment.
- Describe preparation, clearing and stripping, irrigation, and excavation operations in borrow pits, required excavations, quarries, or other sources of earth materials destined for the construction. List methods used for adding or removing moisture, separating, excavating, and hauling. Indicate sources and delivery means of imported materials such as filter sand, lime, cement and pozzolans, etc. Also, describe borrow area drainage and restoration procedures. Report any difficulties or unusual events and remedies.
- Describe placement operations for embankments or other earthfills, earth linings, structure and instrumentation backfill, pipelines, and other earth structures. The narrative shall include descriptions of: equipment used (particularly the rollers); other compaction or consolidation equipment; methods and equipment for separating, scarifying, mixing, blending, and controlling moisture on the fill; finishing surfaces; placement of riprap, rock blankets, filters, cover materials, and gabion installations; final cleanup, landscaping, and seeding. Include the quantities of earth materials placed in each fill area, length of pipe backfilled, volume of soil-cement placed, etc. Also include the elevations reached for the various zones of major structures. Report unusual or difficult experiences or conditions.
- Where necessary to clarify subjects emphasized in the narrative section, suitably captioned photographs, drawings, and sketches should be attached to the summaries. Additional media such as films, video, etc., may be useful.

B. Earthwork

3-6. General.—Differences in design requirements and construction procedures have led to classifying fill construction into three types: (1) embankments, (2) linings and blankets, and (3) backfill. Usually, division lines between fill types are not distinct. Embankment construction applies primarily to laterally unsupported fills built on top of the natural ground surface; however, refill of cutoff trenches or key trenches is included as embankment construction— especially regarding construction control. Lining and blanket construction applies mainly to relatively thin sheets of fill spread over an area of natural ground, excavated surfaces, or embankment. Backfill refers to refill of excavations below the ground surface or earth placed in confined spaces and against rigid structures.

Based on the amount and kind of work required, each of these groups is further divided into several types. Because of the indistinctiveness of definition, where two or more different types of construction are required involving separate contractual pay items, it is customary to establish arbitrary boundaries for the different types of work. Separation may be definite, or an overlap may exist. The lines of distinction will vary from job to job, so reference to the particular specifications involved is required in all cases.

3-7. Embankment.—Engineering properties of soil can be changed—and often improved—by:

- Selecting
- Controlling moisture content
- Mixing
- Stabilizing using various admixtures
- Compacting

a. Types of Embankment.—In the order stated, each of those operations is successively more expensive to perform. Although engineering properties are improved and made more uniform, some of these operations may not be justified for all structures.

In construction, any of the following types of embankment construction may be specified:

- dumped fill,
- hydraulic fill,
- selected fill,
- equipment-compacted embankment,
- rolled earthfill,
- vibratory-compacted embankment,
- blended earthfill,
- modified soil fill.

b. Dumped Fill.—The simplest construction operation of moving material from excavation and depositing it in a fill to lines and grades is called dumped-fill construction. This type of embankment is used to construct minor roads, canals, and laterals when necessary engineering properties can be developed in the available soil without special effort. Although selection and distribution of material are not specifically required, dumped fills should be free of tree stumps and other organic matter such as trash, sod, peat, and similar materials. Rocks, cobbles, and similar material should be distributed throughout the section and not nested or piled together. For the fill to be reasonably uniform throughout, materials should be dumped in approximately horizontal layers. "End dumping," a process by which fill material is pushed off the edge of the fill and allowed to roll down the slope, is objectionable because it fosters material segregation and should be avoided wherever possible. Dumped fill is often further classified as dragline-placed fill, truck-dumped fill, or scraper-placed fill. If traffic over the fill occurs during construction, either by construction equipment or otherwise, it should be routed to distribute compaction as much as possible.

On canal construction, a further requirement is to place the finer and more impervious material on the water side of the embankment. On road work, the gravelly material should be in the top of the fill, and large rock should be well buried. Inspection consists of visual examination to ensure that the above requirements are satisfied and that dimensional requirements are attained. Laboratory testing of dumped fill construction is not required except for record purposes.

c. Selected Fill.—Commonly used selected fill types include selected impervious fill, selected sand and gravel fill (pervious fill), rockfill, and riprap. Selected fill is a dumped fill constructed of selected materials; occasionally, compaction is required. This type of construction is widely used where one engineering property is more important than others, and soil with a specific property can be secured by selective excavation. Accordingly, selected fill may be specified to satisfy one of several engineering properties. Construction and inspection requirements vary according to the engineering property being emphasized.

The selected impervious fill type of embankment is used in canal construction when selective excavation of the canal prism will provide a superior structure with little extra effort. Excavation from borrow pits and short hauls may be required, but benefits derived solely from selection usually do not warrant the cost of securing a better material from a more distant source.

Selected sand and gravel fill is specified to improve stability, to prevent frost heave, to provide improved wearing surface on roadways, to prevent wave erosion of underlying embankment, and to provide for removal of seepage water without "piping." Benefits derived from this type of construction often justify securing satisfactory material from a considerable distance, and sometimes processing to improve gradation is warranted. In choosing material for sand and gravel fill, gravel is the more important component, and a well-graded, rather than a poorly-graded, material is preferred. Other characteristics may be required for filter materials. Visual inspection should be confirmed with an occasional gradation test.

Riprap and rockfill are used primarily for surface protection. Riprap is used to protect against erosion by flowing water and from ice and wave action and to protect against rain and surface runoff. Rockfill is used to protect against rain and minor surface runoff and to provide stability to a fill structure; to some extent, it is used as a substitute for sand and gravel fill in dams. Durability and gradation are important requirements for riprap. These are also desirable characteristics of rockfill, but the tolerance ranges are much broader.

d. Equipment-Compacted Embankment.—Situations arise where selected fill does not produce an adequate structure, and the addition of compactive effort by routing of equipment will produce an acceptable fill. This type of construction is mostly used on canal embankment and road construction. Specifications for construction of these features usually do not require a definite degree of compaction. Control of moisture in the material may or may not be required. Reference should be made to pertinent specifications. Separate pay items may be used for addition of water. The engineer in charge determines the amount of water to be added largely on the basis of volume of fill to be constructed and the nature of materials being used. Sometimes, visual inspection is supplemented by laboratory tests.

e. Rolled Earthfill.—Improving engineering properties to the maximum practical extent by selection, compaction, moisture control, and special processing is generally justified in construction of earth dams and in canal embankments. A failure could result in the loss of life or substantial property damage, and conditions could develop that would cause excessive water loss or expensive maintenance costs. Where pertinent, procedures and equipment are specified to ensure development of desirable engineering properties to the maximum practical extent.

Materials selection will have been based on preliminary investigations; that is, borrow areas will have been designated. Preliminary investigations should have disclosed the nature of the average materials and the probable range of variation. Field personnel in charge of construction operations should review preconstruction investigation results, perform additional exploration and testing (if necessary), and learn to recognize the kinds of material acceptable for the adopted design. Since earth dams and major canal embankments are designed to accommodate materials available in the vicinity, the materials used for various purposes will differ from site to site. If, as a result of more detailed exploration or in the process of construction, materials are encountered whose characteristics differ appreciably from those anticipated, the construction methods and procedures may have to be changed.

For embankment dams, a specific moisture range and density are generally specified. In specifying compaction, requirements will include the general type of roller to be used, the thickness of lifts, and the number of passes. These requirements are based on extensive experience and statistical data and will produce a satisfactory fill when placed using good construction procedures. Specifications include requirements for removal of oversize particles and that the material be homogeneous in texture; that is, free from lenses or pockets of material differing in gradation or classification from the average material.

Soil moisture content must be controlled to obtain maximum benefit from compaction. Specifications require water content be uniform throughout the layer to be compacted and that the layer be as close as possible to the water content that results in most efficient densification of material to be compacted. For the specified compactive effort, for earthfill in dams, this water content is often slightly less than optimum water content as determined by the laboratory compaction test (see USBR 5500).[2]

[2] 5500: Performing Laboratory Compaction of Soils—5.5-lbm Rammer and 18-in Drop.

Inspection is performed to determine whether the:

- Material is uniform and free from oversize particles.
- Compaction equipment complies with specifications and is maintained in working order.
- Thickness of lifts and number of passes are according to specifications.
- Moisture is uniform within the layer.

The inspector should determine that moisture content is correct by performing manual tests to observe how the material compacts in the hand and to confirm this by tests using USBR 7240: Performing Rapid Method of Construction Control.[3] Inspection observations will be supported with field and laboratory tests—USBR 7205, 7220, and 7240[4]—to determine the degree of compaction and variation of water content from optimum, and USBR 5600 to determine permeability.[5]

f. Vibratory-Compacted Embankment.—Compaction by vibration is usually specified in the construction of earth dams to improve the engineering properties of strength and consolidation in comparatively permeable soils; that is, clean sands and gravels (GW, GP, SW, and SP). However, several other methods of compaction will produce satisfactory densification when permeability is not a concern. Some of these methods are specified both for highway and airport construction. If Reclamation forces are required to supervise the construction of highways or airports where maximum densification of sand and gravel soils is required, specifications of the agency concerned will be followed, and instructions necessary for field control will be issued.

In constructing an earth dam, the contractor may propose some method other than vibratory compaction to densify permeable embankment materials. For those accepted proposals, letter instructions covering construction control will be issued as required.

In selecting filter and drain material to be compacted by vibration, emphasis will be to eliminate soil contamination by excessive amounts of fines [minus 75 µm (minus No. 200 sieve size) material]. As a result, filters and drains are designed with less than 5-percent fines. Field inspection, therefore, will be directed toward removing overburden fines, avoiding silt and clay lenses or pockets, and preventing excavation into less permeable materials below the sand and gravel deposit. Thickness of placed embankment layers is commonly adjusted to avoid a requirement for removal of oversize material, and maintaining soil uniformity does not require the attention that impervious fill construction requires. However, coarser material should be placed toward the outer slopes. Where vibratory compaction is specified, the material source will be designated, minimum size tractor or vibratory roller described, lift thickness and number of passes enumerated, and moisture requirement defined.

Compaction by a vibratory roller or track-type tractor depends on the vibration produced by the equipment in operation. A secondary benefit results both from size and mass; for example, thicker layers can be compacted with larger equipment. Speed of the vibratory roller is considered beneficial in that the effect of increased vibration with high speeds more than compensates for any detrimental effects of short period application. Maximum compaction of sands and gravels is obtained by vibration when the soil is either completely dry or thoroughly wetted without being saturated. The wet condition is specified because satisfactory density is more readily obtained, and field tests are more likely to be reliable with wet compaction. However, under certain conditions, dry compaction has been permitted. In these circumstances, extra compactive effort is necessary in lieu of thorough wetting. To secure a thoroughly wetted material, excavation of materials from below water table has been permitted in some instances. This procedure is successful when small-size excavation equipment combined with a relatively slow placement rate is used. With large capacity equipment and a rapid placement rate, drainage of excess moisture may be too slow to permit satisfactory compaction.

Inspection consists of noting that proper quality of material is used and that specified thickness of lifts and number of passes are obtained. Water content is sufficient if free moisture appears in the equipment tracks immediately following passage. With proper water content, the compacted fill will appear firm and solid. If the fill remains soft, moisture is too great; if the fill is fluffy, water content is insufficient.

g. Blended Earthfill.—Circumstances occur in which two individual materials do not have adequate engineering properties but, when combined, produce a satisfactory material. Other cases arise in which a material having inferior properties may be combined in a mixture with

[3] 7240: Performing Rapid Method of Construction Control.

[4] 7205: Determining Unit Weight of Soils In-Place by the Sand-Cone Method.

7220: Determining Unit Weight of Soils In-Place by the Sand Replacement Method in a Test Pit.

7240: Performing Rapid Method of Construction Control.

[5] 5600: Determining Permeability and Settlement of Soils 8-in (203-mm) Diameter Cylinder.

another material of adequate engineering properties to increase the quantity of satisfactory material available. When the materials to be blended occur as strata (one above the other) in the same borrow pit, excavation by shovel or wheel excavator can readily blend them together with little extra effort or expense (see fig. 3-3). In this case, construction control will require maintaining the height of cut to obtain the necessary proportions of each type material and to ensure that the two materials are being blended. With dragline excavation, satisfactory mixing can be accomplished, but greater attention must be given to the operation because the tendency will be to remove material in horizontal cuts. Several types of excavating equipment, such as wheel excavators and belt loaders, have excellent capabilities for blending vertical cut material (see fig. 3-24).

When scraper excavation is used, the difficulty in attaining a satisfactory mixture is increased; frequently, supplemental mixing is required on the fill by plows, discs, rippers, or a blading operation. Excavation is performed by making a slanting cut across the different types of materials; care must be exercised to effect the proper proportions of each type of material in each scraper load. Loading the materials in an upslope direction usually gives a more effective mix of materials than loading in a downslope direction. A common practice is to spread the materials in half-lift thicknesses to improve uniformity and minimize requirements for mixing on the fill.

The cost considerations limit the procedure for blending materials from separate sources to special situations such as blanket and lining construction. First, one of the materials is stockpiled upon the other so excavation can be made across the two materials, or the materials are placed in thin layers on the fill; then, they are mixed by blading, plowing, or similar procedures before being compacted. Mixing machines may be specified to obtain an optimum mixture for earth linings.

h. Modified Soil Fill.—A fill using a soil that has been modified by addition of minor quantities of a selected admixture prior to compaction is called a modified soil fill. The most commonly used additives are lime and cement. The use of modified soils should be considered in lieu of replacing poor soils with selected material from a distant source. For example, to construct a switchyard fill, lime can be used to modify expansive characteristics of a clay soil.

Usually, conventional methods are used to spread and compact the modified fill. In addition, some type of equipment must be adapted to distribute and mix the additive. This equipment requires initial calibration and periodic checks during construction to ensure that the proper amount of additive is being applied. Special tests of the modified soil may be required to verify the amount of additive, and visual observations should be made to ensure that the additive is uniformly distributed. Construction control testing of placement moisture and density is required.

Inspection requirements will be comparable to those required for concrete or other specialized materials, and extensive testing should be anticipated. Considerable experimentation may be required to develop suitable construction procedures.

i. Hydraulic Fill.—Most methods of embankment construction all involve control of water content. Some situations require placement of fill material under conditions of excess water content. These situations may involve excavation and transportation of the material by use of flowing water. The more common procedures involve only the placement of material in still water or by the process of washing material into place or densifying it with a stream of water.

Terminology for the different types of hydraulic construction is not standardized. In general, the term "hydraulic fill" is applied to the complete operation of excavation, transportation, and placing by flowing water. When the hydraulic operation is confined to placing, it is described as semihydraulic construction. The term "puddled fill" is applied to special types of semihydraulic fill, and the term "sluiced fill" is used for material washed into place.

There are several severe limitations to hydraulic fills which may make them undesirable for embankment construction. Uniform granular or cohesionless hydraulic fills, without active densification measures, will result in very low degrees of compaction. For undensified cohesionless soils, the typical relative range is 30 to 60 percent. There have been numerous examples of severe distress to hydraulic fills, such as embankments, quay walls, and tailings piles under earthquake loadings [2]. One such example is earthquake-induced liquefaction experienced at Lower Van Norman Dam during the 1971 San Fernando earthquake. Puddled, sluiced, or hydraulically placed, fine-grained soils will be weak and highly compressible. Many years may be required before fine-grained soils are improved by consolidation. In large embankments, the use of high water content, fine-grained soils may result in high pore pressure generation and reduced shear strength resulting in unacceptable deformation or shear failures. Many hydraulic

Figure 3-3.—A deposit of impervious material which overlies a deposit of pervious material.

fill structures were built prior to the 1930s. Due to the limitations listed above and the advent of modern compacting equipment, hydraulic fill lost popularity after the 1930s [3].

Economical use of hydraulic fill construction depends on: (1) availability of a material that is readily sorted by the action of flowing water into a pervious material zone and an impervious material zone, (2) a large volume operation, and (3) a source of inexpensive power. This combination of conditions happens so rarely that control procedures for hydraulic fill construction have not been developed for Reclamation operations. In the event that hydraulic fill or semihydraulic fill is specified, special instructions will be issued by the Denver Office engineering staff.

Sometimes, puddled-fill construction is used where climatic conditions make moisture control operations impractical, for backfill around pipelines and structures, and in lieu of special compaction against rough and irregular surfaces. The puddling process consists of depositing the soil in a pool of water, stirring the soil-water mixture until a fairly uniform slurry is developed, and then allowing the soil to settle out of the mixture. Puddling is used mainly for soils of low permeability, such as silts and sandy lean clays, where high density is not necessary. Inspection is by visual examination to establish that the proper type of material is used and to ensure that thorough mixing is accomplished. Puddled fill has many undesirable properties. The slurry can consolidate over time, and unacceptable deformations and stress distributions can occur. The slurry is weak, and upon continued loading, high pore pressures can be generated, resulting in further loss of strength. These undesirable properties make the puddling method unacceptable for many structural backfill applications. Puddled clay core dams were often constructed in older dams in the United Kingdom, where wet clay was healed by foot or equipment traffic into thin central clay cores. The wet clay had very low hydraulic conductivity. Many of the dams have performed well, but there are concerns about stress distributions, cracking of shells, and hydraulic fracturing.

Sluiced-fill construction involves working a pervious material into place by a washing operation produced by the flow of a high-velocity stream of water. The method is used to wash sand and gravel or rock fines into the voids of a rock mass such as a rockfill. In lieu of tractor or vibratory rollers, the method may be used for compacting sand and gravel fill. Vibratory-compaction construction is considered superior to sluiced-fill construction; hence, the latter is usually restricted to areas inaccessible to equipment. Inspection consists of visual examination to determine that appropriate materials are used and that the washing operation is properly performed; that is, the materials are actually moved in the process. When sluiced-fill construction is used in lieu of vibratory- compaction construction, relative density tests are required to ensure adequate densification.

Compacted fill is sometimes specified to be compacted in a thoroughly wet condition using equipment such as pneumatic rollers, vibratory rollers, surface vibrators, or tractors. Soils placed by this method must have a relatively high permeability for proper densification. A density requirement is specified and is based on relative density.

3-8. Linings and Blankets.—

a. General.—In earth construction, many situations require constructing a relatively thin layer or blanket of selected material. These categories include:

- Layer of riprap on the upstream slopes of earth dams
- Sand and gravel blanket under riprap
- Rockfill blankets or blankets of gravel or topsoil on the downstream slope of dam embankments
- Filter blankets under the downstream portions of dam embankments and under the floors of concrete structures
- Impervious blankets on the floors of reservoirs
- Channel linings for canals
- Linings and covers for hazardous waste disposal areas

Base courses and surfacing on roads or highways, and ballasting for railways, are considered to be in this category.

A common requirement of these types of blankets is proper selection; that is, locating and using suitable materials. Sources of material that have established characteristics and in ample quantity for the various blanketing requirements should always be determined before construction.

A requirement of all blankets is careful placement. Requirements may vary widely according to the type and location of blanket placement, but uniformity and thickness are extremely important in every case. On the basis of material used, blankets may be divided into four types:

(1) Rock.
(2) Sand and gravel.
(3) Silt and clay (impervious).
(4) Topsoil.

b. Rock Blankets.—The rock fragments used in rock blankets should be essentially equidimensional, angular, well graded from a maximum dimension equal to blanket thickness to about one-tenth of blanket thickness—and sound, dense, and durable. Elongated and thin slabs of rock are undesirable, but rock fragments whose minimum dimension is about one-third to one-fourth of the maximum dimension are not objectionable. Although angular rock is the most desirable, subangular and subrounded cobbles and boulders are commonly used. While rounded cobbles and boulders have been used as riprap in certain cases, their use is more appropriate for blankets on flat surfaces. Testing and placement of rock blankets are discussed in more detail in section 3-24 on riprap.

Grading of riprap and other large stone can be performed in accordance with ASTM procedure D-5519-94 [4]. The procedure allows for determination of grading by volume, mass, or a combination of volume and mass. On critical projects, it may be necessary to specify testing and provide for testing facilities such as concrete pads and scales. Generally, blasted rock that has an appreciable quantity of fragments near the maximum size requirement will possess a satisfactory gradation. The condition to avoid is an excess of small-size fragments. Fine material such as rock fines, sand, and rock dust, in a volume not to exceed the volume of the voids in the larger rock, may not be objectionable, depending upon the purpose of the blanket.

Soundness, denseness, and durability requirements may vary somewhat according to usage. The highest quality rock is required for protection of slopes in stilling basins. The soundness requirements for riprap on dams may vary somewhat according to reservoir fetch and operating characteristics; rock size, to some extent, can be substituted for high density. Protection of downstream slopes of dams can be accomplished with any rock that does not break down appreciably by normal exposure to weather. Shale is about the only rock that is unacceptable for downstream slope protection of dams.

c. Sand and Gravel Blankets.—The characteristic of sand and gravel mixtures to allow passage of water, while at the same time preventing passage of soil grains, is extensively used in the design of water retaining structures. The properties of resistance to displacement by flowing water, resistance to wear from vehicular traffic, and maintenance of strength and limited volume change over a large range of water contents, make sand and gravel useful in providing surface protection to canal banks, roads, and working areas in the various facilities of irrigation and power projects. The wide range in gradation possible in sand and gravel mixtures, together with the wide range in structural materials to be protected, results in a wide range of acceptability for the materials used for sand and gravel blankets. However, blankets under the downstream portion of an earth dam embankment must be designed as a filter.

Pervious blankets under concrete structures usually are made as thin as possible. This condition makes processing of material from natural deposits almost a necessity so as to produce the specified gradation of material. Two-layer filter systems are frequently used.

Sand and gravel used as base courses and surface courses on roads, highways, and switchyards are normally processed material. Materials used for surface courses differ from pervious blankets for other uses in that a certain amount of clayey material is considered desirable to bind the material together and, in some instances, is actually added to improve the quality of surface courses.

Pervious blankets used to prevent erosion in canal channels require a material predominantly in the gravel range. Appreciable quantities of sand are undesirable, and, in some instances, removal of fine sand may be required. A sand layer may be required between a coarse gravel layer and the subgrade to prevent movement of fine subgrade soils through the gravel. Materials for a pervious gravel blanket and materials for an impervious blanket may be combined to provide an erosion-resistant impervious blanket. Usually, filter criteria are not important in impervious blankets.

Crushed rock may be used for any of the above-described purposes. Crushed rock is a processed material that can be manufactured to fulfill specifications requirements.

Sand and gravel or crushed rock blankets usually are densified by equipment compaction using vibrating equipment or operating smooth rollers. They may be placed either dry or thoroughly wetted. Inspection is commonly by visual examination, but gradation and relative in-place density tests may be required in some cases.

d. Impervious Blankets and Linings.—Impervious soils are used on reservoir floors and in canal channels to reduce seepage. Material for these purposes must be:

- Impermeable
- Free from shrinking and swelling characteristics
- Resistant to erosion from flowing water
- Stable when placed on the sides of canals and reservoirs

Probably the best material for these purposes is a well-graded gravel with clay (GW-GC), which offers both impermeability and excellent erosion resistance. A clayey gravel (GC) material or a silty gravel (GM) material is also suitable. Materials in other soil groups can be used according to the engineering use chart, table 1-4 (ch. 1). When satisfactory soils are not available, processing soil by blending or protecting the fine-grained soil with a blanket of sand and gravel is necessary.

Blankets and linings to be permanently underwater need not meet erosion resistance and shrinking and swelling criteria. Blankets and linings placed on essentially horizontal surfaces need not possess high stability characteristics. However, an impervious blanket beneath a reservoir may require a filter beneath it to prevent piping of the blanket material into the foundation.

Thickness of impervious blankets is usually controlled by placement procedures. However, at high reservoir heads, thickness greater than the minimum required for construction operations may be required. Also, where some swelling or shrinkage is anticipated, thickening impervious blankets may be desirable so an effective thickness of blanket (unaffected by swelling or shrinkage cracks) is maintained. For canal linings, thickness generally is varied depending on design requirements; availability of lining material may affect thickness, and construction methods may need to be revised to attain desired engineering properties.

Construction and inspection requirements are similar to those for rolled earthfill and blended earthfill.

e. Topsoil Blankets or Zones.—Topsoil blankets or zones sometimes are specified for downstream slopes of dams so slopes can be seeded to protect the underlying zone against wind and rain erosion. Topsoil for restoring borrow areas or other areas—where existing topsoil has been destroyed or removed—is specified to fulfill ecological and environmental requirements.

Normal topsoil sources are materials stripped from required excavation for the dam and appurtenant structures and approved stripping from borrow areas. Generally, it is not feasible to borrow topsoil. Consequently, suitable materials from stripping, which would otherwise be disposed of as waste material, should be selected during excavation and, if necessary, stockpiled for later use.

Normally, specifications require a 0.3-m (1-ft) normal thickness for topsoil blankets; thus, areas requiring topsoil should be brought to within 0.3 m of the prescribed final cross section at all points and finished smooth and uniform before topsoil is applied. Topsoil should be evenly placed and spread over the graded area and compacted in two layers.

Occasionally, for dams requiring topsoil, it will be advantageous and economical to specify a thin topsoil zone for downstream slope protection. A prime advantage is that excavated topsoil can be placed concurrently with the rest of the embankment—eliminating the requirement for stockpiling. When a topsoil zone is specified, construction and inspection requirements are similar to those for rolled earthfill.

In most cases, seeding of topsoil on downstream slopes is required; thus, topsoil should be selected from required excavation or from a stockpile so the most fertile soil is obtained. Topsoil must be free of excessive quantities of large roots, brush, rocks, and other objectionable matter. Topsoil should not be placed when the subgrade is frozen or in a condition otherwise detrimental to proper grading and seeding.

3-9. Backfill.—

a. General.—Backfill is earthfill in confined spaces, such as refilling operations about concrete structures. Refilling operations associated with embankment, construction work adjacent to the concrete structure and in other confined areas may be called special compaction rather than backfill. Backfill operations may be divided into three categories: (1) backfill, (2) compacted backfill of clayey and silty soils, and (3) compacted backfill of cohesionless free-draining soils.

b. Compacted Backfill.—Compacted backfill covers two types of materials and compacting operations. The *first* is compaction of clayey and silty materials of low permeability. These soils are used for backfilling structures when seepage is to be limited or if drainage is not required. Normally, this type of backfill is compacted by tamping rollers when space is available or by power tampers in confined areas. Generally, the soils are compacted at optimum moisture content and to a given percentage of the laboratory maximum density.

The *second* type of compacted backfill uses cohesionless free-draining soils of high permeability. These soils are used around or under pipelines and structures when free drainage is desired or when a bedding or a foundation of low compressibility is required. The materials used must be free-draining sandy and gravelly soils. When high stability and low settlement are design requirements, this type of backfill is often preferred over compacted backfill of clayey and silty soils because of the ease and economy of placing—particularly in confined areas. For example, when good support is required under and around concrete pipe, the material can easily be made to flow and can be compacted to high density in the critical area beneath the pipe and between the pipe and trench surfaces—providing proper procedures are used for wetting and vibrating the material. A fill of clean sands or gravels is specified often for use under pumping plants and other structures when necessary to improve the bearing capacity of soft foundation soils. This improvement is accomplished by removing the soft materials and replacing with selected backfill to spread the load onto lower strata. Also, cohesionless free-draining soils are preferred as backfill adjacent to some structures when excessive surface settlements are undesirable or when space is limited.

Generally, compaction of these soils is accomplished by pneumatic rollers, vibratory rollers, tractors, surface vibrators, or internal vibrators.

Proper selection of materials is particularly important when compacting using saturation and internal vibration. Excessive amounts of fines will fill the voids between coarser particles and will prevent drainage. Table 3-1 provides information for preliminary selection of soils suitable for compaction using saturation and vibration.

Table 3-1.—General suitability of soils for compacted backfill by saturation and internal vibration

Soil type	Limitation[1]
GW, GP, SW, SP	All soil types are suitable. Fines in these soils are limited to less that 5 percent by definition.
GW-GM, GW-GC GP-GM, GP-GC	May or may not be suitable, depending on gradation and plasticity.
SW-SM, SP-SM, SP-SC	Test section may be required. Fines are limited to 5 to 12 percent by definition.
SM, SC	Normally unsuitable.

[1] Fines are particles smaller than U.S.A. Standard 75 µm sieve (No. 200).

3-10. Excavation.—Materials for embankment or lining construction are obtained either from required excavations or from borrow pits. Materials from required excavations should be incorporated into embankment or lining construction to the maximum extent possible. When materials from required excavations can be used, the same practices should be followed as in borrow-pit excavations.

Organic material such as plant growth and decaying vegetable matter should be removed from the surfaces of borrow pit within borrow areas before initiating excavation. Depth of stripping varies according to the nature of ground cover—from 50 to 75 mm (2 or 3 in) for prairie grasslands to 600 to 900 mm (2 or 3 ft) in valley bottoms and forested areas. The removal of all material containing grass and tree roots is not necessary. The fine hair roots of grass and tree roots less than 6 mm (1/4 in) in diameter are normally considered not sufficiently detrimental to justify wasting material containing minor amounts of this matter. Excavating an area greater than required, in one construction season, is inadvisable because of recurring weed growth. The minor amount of weed growth that may develop during the latter part of a construction season on a stripped area is usually considered unimportant.

Occasionally, deposits of impervious soil are covered with a layer of boulders; often, sand and gravel deposits are covered with a layer of silt and clay. Such a deposit is shown on figure 3-3. A thorough materials investigation for an earthfill dam resulted in the most economical benefit to use the upper clay-silt-sand portion of the deposit for the impervious zone and the lower silt-sand-gravel portion for the pervious zone, thus eliminating the need for another borrow source with its subsequent additional development costs. When impractical to salvage upper layers, they are classed as stripping and wasted even though the layers may be thicker than is normally considered applicable to stripping.

In highway and canal construction, where the normal method of excavation is by scraper, a lower limit to the depth of cut is not required. In all other excavations, cut depths less than 0.2 to 1.5 m (4 to 5 ft) are avoided for economic reasons unless deficiency of material dictates smaller cuts. Except in situations where separation between two different types of material is desired or in which controlled blending is required, maximum depths are usually limited by the range of excavating equipment. However, the maximum depth of cut may be limited by the presence of bedrock or hardpan and, except for sand and gravel excavation, by location of the water table. In stratified borrow areas, excavation must be made so that every load delivered to the point of use contains a mixture of the full designated depth of cut. Extra mixing on a stockpile or on the fill may be required.

C. Foundations

3-11. General.—If conditions in soils supporting a foundation could be completely described as a result of subsurface investigations and designs were prepared accordingly, field inspection—during construction—would be reduced to merely determining that dimensional requirements of designs were satisfied. However, even a thorough investigation program leaves the majority of foundation soils and rock unexplored.

Because of the variety of conditions that may be encountered in natural soil deposits, a complete set of guidelines for judging adequacy of foundation soils is impossible to provide. Individual judgment based largely on experience must be used. The first objective in developing experience is to enhance one's ability to discriminate between sound and unsound foundation soils. In practice, this means classifying foundation soils as adequate, inadequate, and questionable for the type of structure being built. Initially, most cases seem to fall in the questionable category; but with increasing experience, the number of cases decrease.

To supplement judgment, test procedures have been developed for evaluating foundation soils. Such tests should have been performed during the investigation program; comparison by means of index tests and visual examination between tested and untested areas will usually suffice to establish adequacy. Additional tests may be required during construction.

Conditions for which adequacy of foundation soils should be established include bearing capacity, stability, ettlement, uplift, deterioration, and permeability. The degree to which each of these items is important depends on the nature of the proposed construction, character of foundation soils, and structural characteristics of the foundation.

3-12. Bearing Capacity.—Foundations for rigid structures are usually evaluated on the basis of bearing capacity. Bearing capacity in this sense involves both shear strength of the soil and its consolidation characteristics. Bearing capacity may be determined in the laboratory by shear and consolidation tests on undisturbed samples or approximated in the field from in-place dry density tests, standard penetration tests, vane shear tests, cone penetration tests, borehole shear tests, or pressuremeter tests.

The design of rigid structures should include the type of foundation treatment required based on information available from the investigation program. Where foundation soils appear to be the same as those disclosed at locations of the investigations, they may be assumed to be equally competent. If soil conditions are disclosed that are observably different from those disclosed by previous investigations, the lateral and vertical extent of this different condition should be determined before construction continues.

One should be aware that a pocket or lens of harder or firmer material in foundation soils can be equally as detrimental as softer deposits. Therefore, common practice is to specify that if such material is encountered, it will be removed to some depth and be refilled with compacted soil to provide a more uniform soil deposit at foundation grade. This requirement should include beds, layers, or lenses of indurated gravel or sand, shale, indurated clay, hardpan, caliche, and other erratic cemented deposits such as calcareous or siliceous material. The most desirable soil deposit for a structural foundation has a uniform character, and a moderate amount of additional excavation to develop such a condition is usually justifiable.

3-13. Stability.—Excavation for a foundation can be divided into two parts, the base and the side slopes. Except for structures such as chutes of canals and spillways, the load of the structure rests on the base of the excavation and settlement usually limits development of full shear resistance of foundation soils. Consequently, stability of the base of an excavation may not present a problem. However, foundation excavations adjacent to bodies of water or directly above water bearing strata should be investigated for loss of support caused by artesian pressure.

For most construction, cut slopes should be determined by practicing geotechnical engineers using data available from the investigation program together with appropriate analyses and design experience. Federal, State, or local requirements must be considered in the selection of excavated slopes. Use of slopes not previously analyzed or pre-established should be discouraged because of the many variables involved in determining stable slopes such as soil type, ground-water conditions, depth of cut, and intended use. Both experience and engineering analyses are required for this determination.

In determining stable slopes, a great deal of information can be gained from examining previously excavated slopes and natural slopes in similar soil in the same geographic area. Stable slopes in sand and gravel are independent of height of cut, but are very dependent on local ground-water conditions. Likewise, safe cuts in saturated clay are dependent on cut slope height and relatively independent of slope. Because most soils are combinations of sand, gravel, and clay, safe cuts will be controlled both by height and by slope. Evaluation of natural slopes should consider that the safety factor against failure is variable and that any increase in slope (i.e., placing fill at the top of a slope or excavating at the toe of the slope) results in reducing the safety factor. Therefore, a natural slope should be steepened as little as possible.

The presence of water has a significant effect on slope stability. This can be in any combination of seepage, rainfall, or water being conveyed or stored. An example of failure of a natural slope (landslide) is shown on figure 3-4. Excessive moisture was allowed to enter the soil, and removing soil at the toe of the slope for constructing a stilling basin may have triggered the slide. Where an excavation cuts across the ground-water table, sloughing may occur. To overcome this condition, the usual practice is to remove ground water from the area before excavation by any combination of well points, deep wells, or sump pumps. Figure 3-5 shows well points and sump pumps used in trench excavation; figure 3-6 shows removal of ground water using drainage ditches and sump pumps. Damage to a slope may result from high hydraulic gradients produced by seepage. Where pumping is not practical, excavations may be accomplished by making repeated cuts, which result in gradually lowering the hydraulic gradient as ground water is drained away. The success of any method is dependent on a thorough knowledge of ground-water behavior and of local soil conditions.

Where permanent cut slopes intersect the ground-water table, some type of treatment is usually necessary to prevent slope failure or sloughing. Required treatment involves moving the free surface of the water table back from the face of the slope or controlling the water as it exits the slope. This may be accomplished either by drilling drainage holes into the slope, excavating and placing drains in the slope, or protecting the surface of the slope with free-draining materials such as a gravel or rock blanket on geosynthetic drainage materials. Unless the potential for seepage is high, a rock or gravel blanket will usually suffice. A blanket 0.3 to 0.6 m (1 to 2 ft) thick placed on a geosynthetic fabric or granular filter could be used in such cases that are discovered during construction because cost

Figure 3-4.—Landslide (natural slope failure).

Figure 3-5.—Cutoff trench at Davis Dam in Arizona.

Figure 3-6.—Cutoff trench at Twin Buttes Dam in Texas.

is relatively small. Where material in the slope is well graded and free draining, a blanket will develop naturally in many situations, and special treatment may not be needed.

Rainwater damage commonly takes the form of surface erosion. Preventive measures may include:

- Divert surface runoff away from slope faces with small collector ditches a short distance back from the intersection of the slope with the ground surface.
- Install surface sodding or blanket with a thin layer of gravel or rock on a layer of geosynthetic fabric.
- Construct ditches to carry water off laterally from the slopes.
- Steepen the slopes, as long as the slope remains stable.

However, when rainwater can infiltrate foundation soils to produce a rise in ground-water levels and resultant seepage, both the sodding and slope steepening procedures may actually be detrimental and, therefore, should not be used in conjunction with a seepage condition without an adequate analysis of the situation.

Erosive action from flowing water in a canal or wave action in a reservoir is potentially the most severe type of water damage. Protection involves flattening of slopes or protection with gravel or rock blankets placed on appropriate granular filters or geosynthetic materials depending on the severity of the potential damage. Situations exist where sloughing of slopes is not particularly important such as in borrow pits within the reservoir area. Such slopes should be flattened sufficiently to reduce the hazard.

Situations may arise where selection of slopes is controlled by criteria other than stability alone. For example, some state highway departments use cut slopes closely approximating natural slopes as a method of drift control for sand and snow. Some state highway departments require flattened slopes for grass cutting equipment. In some canal work, 2h:1v slopes are specified (depending upon the types of materials) instead of the more conventional slopes to minimize maintenance and to provide more stable channels. Sometimes, slopes are flattened in connection with earth dams and high canal fills to distribute stresses caused by high embankments.

3-14. Settlement and Uplift.—If a structure is not on bedrock or supported by means of piles, piers, caissons, or walls, the structure must be capable of withstanding some vertical movement in the foundation soils. The usual method for reducing such movement to within acceptable limits is by spreading the base or footings of the foundation so that unit pressures are reduced.

Uplift of structures may occur from several causes such as hydrostatic forces, wetting of expansive soils, and frost action. Structures and canal linings must be protected against excessive upward movement. For light structures, such as chutes and lined canals, protection against hydrostatic uplift must be provided. This is commonly accomplished by underdrains or drainage blankets of sand and gravel and/or geosynthetic drainage materials. Where possible, these drains will discharge into adjacent drainage channels or into the structures. For certain lined canals, this may require flap valves on outlets into the canal. Special care should be taken to keep the drains open and the drainage medium free from contamination.

Expansive soils may cause uplift upon wetting, particularly when the loads are light, when the initial in place dry density is high, and when the initial moisture content is low. Usual methods for reducing movement of rigid structures are by using long friction piles or belled caissons attached firmly to the structure, by increasing unit footing pressures, or by removing expansive soils and replacing with nonexpansive soils to a depth sufficient to control movement. Flexible canal linings such as earth linings or geomembrane linings are used in preference to rigid canal linings on expansive soils. In some cases, the structure load is less than that of the original earth removed, and elastic rebound may occur. This is true, particularly when saturated expansive clays are involved. In some cases, where structural loads are small, as in canal linings, conditioning the expansive clay is possible by prewetting the foundation and maintaining a moist condition so as to minimize future expansion.

Frost action may also cause undesirable heaving and uplift. The susceptibility of soil to frost action depends on: (1) soil type, (2) temperature conditions, usually expressed in terms of a "freezing index" (a summation of the degrees below freezing multiplied by the days of below-freezing temperatures), and (3) water supply. Uplift, from the development of ice lenses in foundation soils, may cause damage to such works as concrete canal linings, spillway apron slabs, and other lightly loaded structures. Frost action can also cause a decrease in the dry density in compacted earth canal linings [5]. All soils, except those classified as GW, GP, SW, or SP, may be susceptible to frost action in cold climates and with water available. The susceptibility of gravelly and sandy soils varies with the amount of fines. Silts and clays, with a plasticity index of less than 12, and varved silts and clays are particularly susceptible to frost action.

Collapsible/low density soils can result in sudden settlement of a foundation upon wetting (see sec. 3-17). This condition is particularly severe in very arid portions of the Western United States, where these types of soils were usually deposited as Loess or as quickly deposited materials on the outer limits of alluvial fans. More than 1 m (3 ft) of settlement is common in widespread areas of collapsible/low density soils. The usual treatment for collapsible/low density soil areas is to prewet the foundation areas for weeks or months in order to precollapse the soils at depth. The best method of prewetting is through ponding (preferably with wick drains); however, other methods such as sprinkling, water injection, or dynamic compaction can be effective. Explorations and analyses need to be performed in areas of potentially collapsible/low density soils to determine the areal extent, depths, and percentages of collapsible/low density soils that can be expected.

Subsidence due to long-term fluid withdrawal is also an area of potential settlement concern. Generally, subsidence occurs at such a slow rate that foundations are not immediately affected. However, the long-term use of a structure, such as a canal, can be severely impacted. Again, extensive explorations and analyses need to be completed in order to mitigate the effects of subsidence on the long-term operation of a structure.

3-15. Deterioration.—The surface of excavated slopes in both soil and rock should be preserved in their natural state as much as possible. A cover of soil should be maintained over the excavated surface until final cleanup, and the structure or foundation should be placed immediately. Drying of the excavated surface should be avoided. Some indurated clays and shales will dry and crack when exposed to the air and then turn into a soft slurry upon rewetting. When it is impossible to place the structure or foundation immediately after such surfaces are exposed, a spray-on cover of pneumatically applied mortar or other approved material has been satisfactory in some situations.

3-16. Permeability.—Water retention structures, particularly reservoirs and canals, depend on foundation soils for a part of their water barrier. If access to a water-bearing stratum exists, either through the process of excavation or from natural conditions, some corrective action is indicated. However, no soil formation is water-tight, and some water loss can always be expected. Movement of water from a reservoir or canal may result in piping or excessive water loss. These problems can be reduced by: (1) controlling the velocity of flow; (2) controlling the volume of flow; and (3) controlling exit conditions. The treatment required may or may not be the same for problems (1) and (2).

Damage to foundation soils resulting from water movement may be caused by gradual removal of soil particles either by solution action or by mechanical movement. Solution action is a problem only when foundation soils contain substantial amounts of soluble minerals.

The existence of solutioning in a foundation can be a serious problem. Whether such a condition involves primarily bedrock or soil solutioning can produce sinkholes in surficial soils. These sinkholes are often small in extent; and unless positive evidence is apparent that they are watertight, sinkholes should be stripped to sufficient depth and filled with compacted impervious soil when they are found along canal alignments. In reservoir areas and dam foundations, further explorations and more positive methods of treatment may be necessary. A sinkhole is shown on figure 3-7. Similarly, in the excavation process, when bedrock surfaces are exposed (i.e., contain solution channels, open holes, or cracks), such openings should be sealed with concrete or blankets of compacted impervious soil. An extensive grouting program may be required to seal the bedrock surface.

In limiting water loss at dams, as a rule of thumb, at least 95 percent of the pervious cross-sectional area of the soil foundation must be blocked. However, where a soil foundation consists of zones (or strata) of differing permeability, and 1 zone is more than 10 times as permeable as another, blocking of the more pervious zone is usually effective in reducing water loss even though it may be only a small part of the total permeable area. When water conservation is required, a permeable strata only partially excavated may be unacceptable. The strata may need to be completely cut off by continuing the excavation down to a lower elevation so that the entire permeable zone will be blocked, or by cutting the strata completely off by use of a cut off wall, or other similar means.

Figure 3-7.—Typical sinkhole.

Similarly, on canals, any lined reach should be extended to cover the entire reach of a pervious stratum. Consideration should be given to fully lining (or using pipe) all constructed water conveyance systems. Thorough justification is required for using an unlined waterway. A study considers:

- Value of water saved
- Operation and maintenance cost
- Required drainage
- Right-of-way
- Allowable velocities
- Structure costs
- Cost of various types of lining correlated with other conditions
- Amount of tillable land removed (for canal alignment and because of seepage)

The unit value assigned to each item varies with local conditions; therefore, the amount of water loss allowed before lining a canal may vary considerably in different areas.

In constructing long canals, explorations are often insufficient to reveal fully all pervious strata that will be intersected. Where pervious canal reaches are encountered, an impervious lining should be considered provided that: (1) suitable material is available, and (2) slope and right-of-way limitations will permit. If space limitations, availability of suitable impervious materials, or other considerations indicate a requirement for another type of lining, or for a more expensive lining, appropriate authorities should be consulted.

3-17. Unsuitable Foundation Soil Conditions.—

a. General.—Materials normally considered unsuitable for foundations include:

- Organic matter such as topsoil, swamp deposits, or peat
- Low density materials such as loose deposits of silt, sand, loess, talus deposits, spoil piles, and dumps
- Some clays classified as highly plastic, active, sensitive, or swelling
- Soils in a soft and saturated condition

Whether a soil is deemed unsuitable depends upon several factors. Most important is the type of structure being contemplated. Another important factor is the geographical location of the intended structure. For example, some soils may be determined to be unsuitable in one climate, while the same soils in another climate may be deemed undesirable or even suitable depending upon the climate and the availability of suitable soils. In other words, the shortcomings of a soil can often be overcome through prudent design.

For most structures, unless the deposits of unsuitable materials prove extensive, avoiding these materials for use in structural support may prove economical and satisfactory. The more common methods to avoid using unsuitable materials include:

- Removal and replacement with compacted select material
- Full penetration with piles, piers, caissons, or walls
- Displacement; that is, remove a mass of material equal to the mass of the structure

If feasible, the structure can be relocated to provide better soil conditions. Treatment with additives may also be effective for some soils.

b. Topsoil.—As in borrow deposits, the usual practice is to remove topsoil from below all foundation elements. Topsoil in this sense is the surface layer of material containing decaying vegetable matter and roots. Removing all soil containing fine hairlike roots is unnecessary but is required of the rather heavy root mat. On prairie topsoil, which contain light grass cover 50 to 75 mm (2 or 3 in), stripping will suffice. Agricultural lands are stripped to the bottom of the plowed zone, usually 150 to 250 mm (6 to 10 in). Valley bottom lands and brush-covered land commonly require stripping up to 0.6 or 0.9 m (2 or 3 ft) for removal of inadequate material. Forest land requires removal of stumps, resulting in stripping requirements of 1 or more meters.

c. Swamp Deposits.—Marshlands, river backwaters, lakebeds, and flood flow deposits are included in this category. Foundation soils formed by these deposits contain appreciable quantities of vegetable matter, clay, silt, or sand lenses deposited in water.

Shallow deposits should be removed; however, deposits may be deep, and their depth and extent should be established before adopting a method of treatment. Relocation is recommended to avoid such deep deposits, particularly where roads, canals, pipelines, or transmission lines are involved. For dams, powerplants, and similar large structures, such materials may be excavated and wasted or used for topsoil; for bridges, pumping plants, and medium-size structures—where a water barrier is not required—fully penetrating piles are commonly used. Sometimes, adequate structural performance can be obtained by excavation to firmer material and placing and compacting a pad of select material.

d. Silt and Sand.—While dense silt and sand deposits generally will be satisfactory, low density deposits of these materials may be undesirable. Usually, such low density materials are found as surficial deposits occurring as loess, sand dunes, sandbars in stream channels, alluvial fans, and deltas at the head of reservoirs and lakes. The surface layers of soil subject to frost action are low density in some areas.

Low density, poorly-graded sand, silts, and gravels that are saturated and located in an seismically active region are highly susceptible to the phenomenon of liquefaction when disturbed by vibration. Many slope and bearing capacity failures have occurred during earthquakes because of liquefaction in low density materials.

Loose sands and wet silts will consolidate quite readily when loaded and may not require special treatment when used for flexible structures such as road fills and canal banks up to 6 m (20 ft) high. However, dry, low density silty soils require removal or prewetting to support a structure. Figure 3-8 shows cracking and settling of a canal bank in dry, low density silt after filling the canal.

Foundation soils composed of low density materials and wet silts must be removed for earth dams and for fills higher than 6 m (20 feet). For foundation soils of dry, low density silt, either removal or consolidation by ponding is required as shown on figure 3-9. Consolidation of saturated sands can be facilitated by draining water from the soil.

Rigid structures usually are not founded on loose silts and sands. These materials are either removed or consolidated, or footings or piles for the structures are extended to firmer material. Normally, designs will show the method to be used.

Where loose sands or silts occur in foundation soils of water retention structures, such as canal banks and dams, lining or cutoffs are indicated. Permeability and the value of water are usually the basis for determining the need for treatment. A long canal may require treatment for a soil condition that would not need treatment for a short lateral construction.

Figure 3-8.—Cracking and settling of canal bank in dry, low-density silt.

Figure 3-9.—Ponding dry foundation of Trenton Dam in Nebraska.

e. Talus and Spoil Piles.—The term "talus" describes the buildup of soil and rock debris at the base of a slope. Generally, such deposits consist of low density materials and seldom provide adequate support for any structure more important than a simple roadway. The usual treatment is to remove the talus. Excavations into talus deposits are likely to be unstable if cut slopes are appreciably steeper than the natural slope of the talus deposit.

The processes used for disposal of excess or unsatisfactory materials in spoil piles are similar to the processes by which talus deposits developed—so the two have similar characteristics. Spoil piles and talus deposits should not be used for support of any structure without thorough investigations, testing, analysis, and design.

f. Highly Plastic Clays.—Clays that create foundation problems have one or more of the following characteristics:

- plasticity indexes greater than 25 percent,
- colloid content greater than 20 percent,
- high sensitivity, and
- shrinkage and swelling characteristics as noted by their tendency to shrink and crack when drying.

Also, a relationship appears to exist between moisture content and strength of clays. Criteria for identifying expansive clays are given in chapter 1.

g. Soft or Saturated Soils.—Almost any soil foundation is reduced in quality when it contains large amounts of water. An apparently firm dry foundation soil may become unstable when saturated. Foundation soils subject to saturation may be protected by either preventing water from entering the soil through use of drains to divert surface runoff or by removing water that reaches the soil using underdrains, drainage blankets, or relief wells. Structures whose foundation soils will be permanently underwater are designed against hydrostatic uplift.

Because of the difficulty in preparing and inspecting soils located beneath the water table. Typically, specifications contain provisions that require dewatering of foundation soils before excavation. However, some work in water is generally necessary when constructing cofferdams, pump sumps, and drains. These structures should be kept away from planned excavation surfaces so the surfaces will not be damaged by the construction or by the action of dewatering facilities. Saturated materials can be damaged by permitting an upward flow of ground water into the excavation or by permitting heavy equipment traffic in the area.

3-18. Foundation Improvement Technology and Dam Rehabilitations.—Undesirable foundation conditions can sometimes be alleviated by treatment in place. New technologies have been developed to remediate foundation problems [6, 7]. As part of the safety of dams program, Reclamation has had to re-examine many of its older structures. These examinations have revealed foundation concerns at many structures including excessive seepage and possible piping; collapse of dry, low density abutment soils; and potentially liquefiable soils during earthquake shaking. Many dams have been rehabilitated using ground improvement methods [8]. These ground improvement techniques are also applicable to foundations for new structures. This section contains a brief discussion on several ground improvement methods, and their applicability to new or existing foundations is given. For more details on these technologies, consult references provided.

Each method of ground improvement has unique advantages and disadvantages relative to the ground conditions requiring improvement. In many cases, differing methods can be combined to overcome disadvantages of the methods and benefit from the positive aspects of each method. For example, at Steinaker Dam, Reclamation desired to treat dirty sands and silts with dynamic compaction. Wick drains were used in combination with the dynamic compaction to effectively treat those soils. Another example is Jackson Lake dam where a block treatment approach was used. Dynamic compaction was used for shallow treatment under the center of the dam, while mixed-in-place, soil-cement columns in a honeycomb pattern at depths up to 90 feet were placed under the toes of the embankment. The use of the two combined methods resulted in satisfactory ground improvement to preclude liquefaction failure.

a. Slurry Walls.—Slurry walls can be used to provide positive cutoff for new dams or can be installed to alleviate seepage problems in existing dams. A deep trench is excavated with various types of equipment, depending on the depth and consistency of the soil or rock. The trench is supported with bentonite or biopolymer slurry. Backfill materials can range from soil/cement/bentonite (SCB) mixtures to structural grade concrete. Often, slurry wall backfill is designed with strength and deformation properties similar to those of the surrounding soils. Material with soil consistency can be excavated in long trenches with backhoes, draglines, or clamshells (see fig. 3-10). Clamshells can be used in a wider range of deposits and can be used to excavate "cells" or "panels." Material with rocklike consistency can be excavated with chisels or rock mills with hardened cutterwheels. Rock mills are most often used to make cells or panels for cutoff walls (fig. 3-11). Reclamation has applied slurry wall technology to existing dams and new dams. An interesting application of slurry wall technology for a new dam was performed at Diamond Creek Dike, where an embankment was built with a sand core which was later cut with a slurry wall for installation of a SCB backfill to provide the water barrier or "core" zone in the dike. This construction method was selected due to a shortage of clay borrow and to accelerate embankment construction.

Design considerations for slurry wall applications should include determination of the geologic and geotechnical conditions, establishment of the hydraulic conditions, requirement of the design flow per unit surface area, determination of the wall size and geometry, and design and determination of the properties of the backfill material.

Quality control issues with slurry walls include verticality, caving, foundation contact and connection of panels or cells, permeability, compressive strength, and voids or windows [9]. Additional quality control issues include excavation and repayment balance, width and depth measurements, and cleaning of foundation contacts. During construction of slurry walls, sounding devices can be dropped into the slurry to sound the width of the trench to determine if caving has occurred. Fresh SCB backfill material cylinders may be cast for compressive strength testing. The heat of neutralization test can be used to check cement contents [10]. Samplers can be used to check for clean foundation contact conditions prior to backfilling. Drilling, water testing, and drill coring programs are often employed to check backfill quality. However, if the backfill material is weak (less than 200 lb/in^2 (1400 kPa), core recovery may be difficult and precautions must be made to avoid wall fracture. Cross borehole tomographic P wave imaging can be used to look at wall integrity and guide investigations.

b. Fill Loading with Sand or Wick Drains.—Fill loading has long been used to consolidate soft silty and clayey deposits for embankments. The time required for consolidation can be determine by consolidation theory. In past practice, vertical sand drains were constructed to accelerate the process. Recently, synthetic wick drains have been developed to replace sand drains. The wick drains are plastic drainage elements enclosed in a filter fabric. Wick drains are installed by vibratory pile driving equipment.

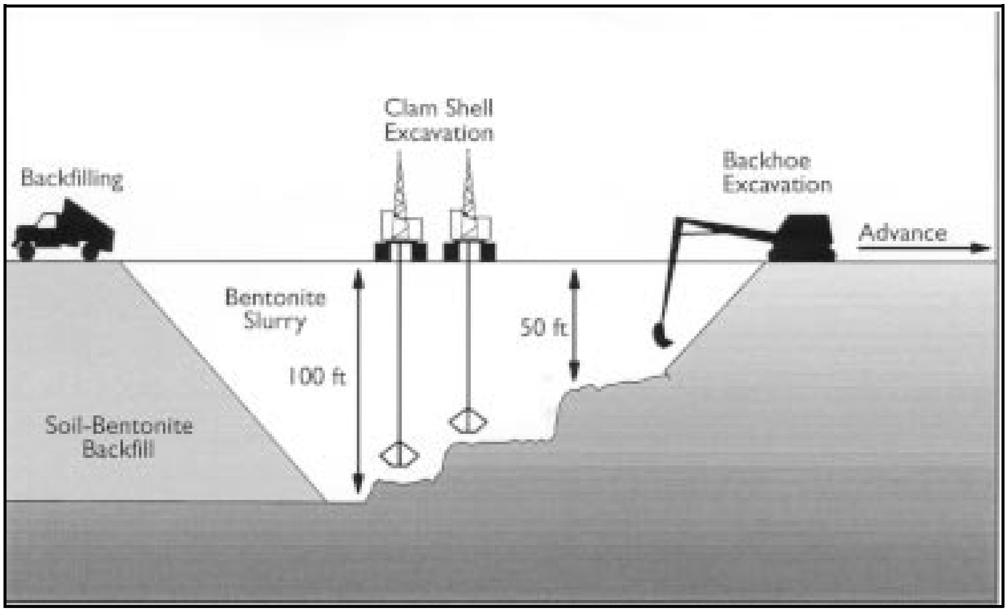


Figure 3-10.—Schematic drawing showing slurry wall construction methods—backhoe and clamshell excavation and surface backfill method. (Hayward Baker, Inc.)

These drains are also a useful complement to other densification methods such as dynamic compaction.

Design considerations for wick drain application include establishing the geologic/geotechnical site conditions, determination of the driving and installation methods, testing of undisturbed samples for consolidation rates and strengths, determination of proper spacings, depths, and types of drain elements, and determination of the required foundation properties and fill loading times.

Quality control testing depends on the application. For fill loading to improve soft clays, piezometers are often installed to measure excess pore pressure dissipation over time. Field strength tests, such as vane shear or lab tests, can be used determine strength gain over time. When drains are combined with other ground improvement methods, there may be no quality control test needs other than assurance of proper placement.

c. Dynamic Compaction.—Dynamic compaction can be a relatively low cost method for increasing the densities of foundation soils. Dynamic compaction has been used to treat low density, potentially liquefiable and collapsible soils. Weights (steel or concrete) of up to 35 tons are repeatedly dropped from heights of up to 30 m (100 ft) in a pattern across a site to create high intensity surface impacts (fig. 3-12). Cranes are generally modified from standard configuration to accommodate the heavy weights used to perform dynamic compaction. Weights are dropped in patterns and sequences until densification is judged to be sufficient There are a number of case histories documenting the use of deep dynamic compaction to remediate seismically sensitive embankment foundations [11].

Effective treatment depth is usually limited to depths between 9 and 10 m (30 and 35 ft), depending on foundation soil types, ground-water conditions, site geometry, weight drop patterns, equipment, and equipment sequencing. Dynamic compaction is typically used at sites with nonplastic materials. The potential for effective treatment becomes less for sites having soil horizons with high fines contents (greater than 20- to 30-percent fines). Effective treatment of sites having soils with high fines content can be achieved by installation of wick drains or other drainage features prior to deep dynamic compaction.

Design considerations include determination of the geologic/geotechnical conditions, evaluation of water control for working pad, procuring backfill material, determination of the drop weight, height and pattern, and determination of monitoring parameters such as penetration resistance testing.

Quality control issues include crater depths (mapping), surface heave or settlement monitoring, hammer energy, pore water pressure buildup, and vibration monitoring. Control testing for dynamic compaction can include load testing, piezometric monitoring, penetration testing, shear wave velocity determinations, and soil sampling for evaluation of engineering properties.

Figure 3-11.—Slurry wall construction using panel excavation and tremie placement—view of a hydromill and crane with 75-cm (30-inch) x 3-m (10-ft) clamshell—Twin Buttes Dam modification.

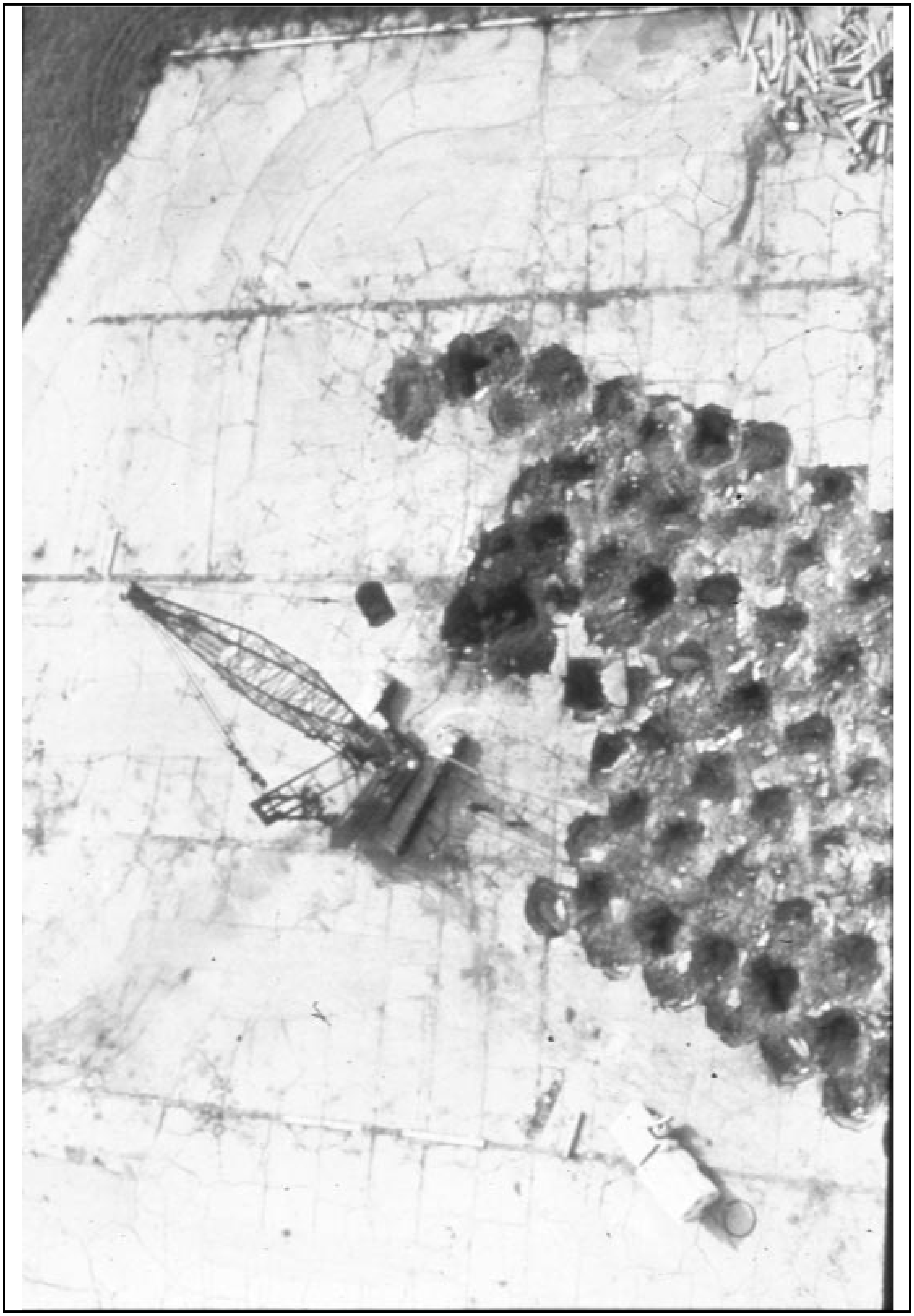


Figure 3-12.—Photograph of dynamic compaction showing crane, weight, and impact pattern (Hayward Baker, Inc.)

d. Vibratory Densification.—There are many forms of vibratory densification which can be used to improve foundation conditions. These include vibro-compaction or -replacement, vibrating probes (vibrating rods with small diameter columns), and blasting.

1. Vibro-Compaction/Replacement (Also Known as Vibroflotation and Stone Columns).—Loose foundation soils at some damsites may be densified for low to moderate costs by inserting a type of vibrating tube or probe into the foundation at a variety of depths and locations across the site (fig. 3-13). Water and/or air jets may be used to aid insertion of the vibratory probe and densification of surrounding soils. The effectiveness of these vibro-compaction methods depends upon soil type (gradation) and depth, spacing of treatment, vibration levels, backfill added (if any), and pore pressure response during treatment. In general, these methods are effective in treating cohesionless soils with less than 20 percent fines [7]. For soils higher in fines content, stone columns are used to improve the soil.

In the vibro-replacement method, sand and/or gravel can be introduced into the foundation during the compaction process, which displaces and intrudes into soils around the probe, resulting in densification of foundation soils. Introduction of sand or gravel can also improve pore pressure dissipation and improve shearing resistance of the treated zone. The probe is repeatedly withdrawn and inserted at the point of treatment to maximize foundation improvement. This process results in construction of a "column" in the foundation, the characteristics of which vary with the nature of the treated materials and those introduced during construction. Columns are typically constructed in triangular or rectangular grid patterns, at center-to-center spacings of 1.5 to 3 m (7 to 10 ft), and are generally 1 m (3 ft) or greater in diameter.

Vibro-compaction and replacement methods using water can have substantial containment and/or cleanup costs during the construction, compared to dry methods. It may be necessary to handle large quantities of excess mud, water, silt, sand, and gravel, depending on the final construction methods specified, foundation and construction material types, ground-water conditions, and site surface characteristics.

2. Vibrating Probes (Small Diameter Vibratory Columns/Drains; i.e., TerraProbes, Vibrorods, Vibro-wing).—Small diameter [less than 0.5 m (1.5 ft)] columns or drains may be constructed in loose soils, densifying them through displacement, vibration, and the introduction of sands or gravels in the process. In addition to densifying surrounding soils, the small diameter columns that result also improve pore pressure dissipation and provide some increase in shearing resistance. Columns are usually spaced on close centers, generally less than 1 m (3 ft), in rectangular or triangular patterns. This method is moderate to high cost when compared to other foundation improvement methods. As the depth of required foundation increases to around 21 m (70 ft) or greater, this method may become more cost effective as equipment limitations begin to impact the effectiveness of other methods.

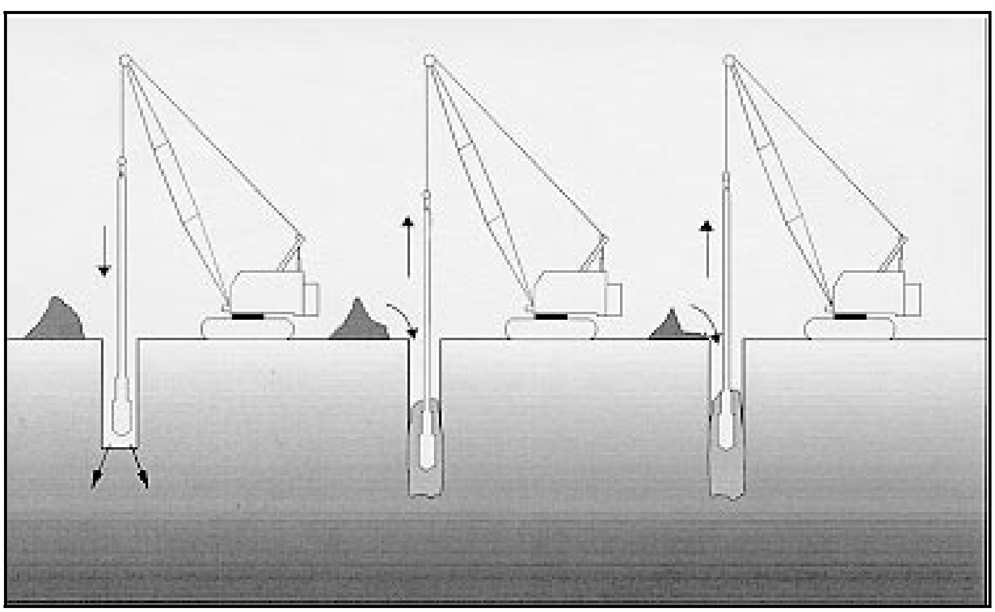


Figure 3-13.—Schematic drawing of vibro-replacement—backfill can be added by top feed (illustrated) or bottom feed methods. (Hayward Baker, Inc.)

There is little data available to evaluate the performance of dam foundations remediated by this method. There no are data from testing of posttreatment hydraulic conductivity in treated foundation materials versus embankment performance during earthquake shaking. Posttreatment testing of dam foundation densities has also not been completed. Therefore, this method should generally be considered as an economical secondary defensive design feature, counting primarily on the columns to improve pore pressure dissipation during an earthquake. Series of columns can be used adjacent to areas treated by other methods to prevent high pore pressures from adjacent untreated areas from adversely affecting the treated zone. This method has been used in the design and construction of seismic remediations for Mormon Island Auxiliary Dam near Sacramento, California. Typical control tests for verification are penetration resistance tests, shear wave velocity, and undisturbed soil sampling.

Design considerations for vibratory ground improvement methods should include determination of the geology and geotechnical properties, establishment of ground improvement requirements, selection of optimum vibratory ground improvement method(s), evaluation of ground settlement, determination of backfill properties and location of backfill sources, and establishing baseline data for quality control testing.

Quality control issues for vibratory ground improvement include column locations, spacing and verticality, energy input required, and quantities of backfill placed.

Acceptance testing for vibratory ground improvement is primarily based on penetration resistance testing, standard penetration tests, cone penetration tests, or Becker penetration tests. Improvement in shear wave velocity can be determined. Undisturbed samples can be procurred to evaluate density and moisture changes. Stress levels and modulus changes can be evaluated with a flat plate dilatometer or pressuremeter. It is wise to never base acceptance on a single test. Acceptance should be based on average properties, not minimum or maximum properties.

3. Blasting.—Blasting, when used as a foundation improvement method, densifies soils as a result of shock waves, vibration, limited liquefaction, displacement, remolding and settlement in the treated soils. This is a relatively low cost foundation improvement method but results in nonhomogeneous soils may be erratic. It is also difficult to assess any potential damage that may occur to, or beneath, an existing embankment or an appurtenant structure. The use of blasting as a foundation improvement method is much more likely to be practicable for sites prior to construction of a new embankment. An example of blasting at the Jebba damsite is given in the references [12]. This method has been used in ground improvement projects where rubble and large boulders preclude the use of other vibratory methods.

Design considerations for blasting should include determination of geology and geotechnical properties; evaluation of charge size, burial depth, charge spacing, number of coverages, and surface settlements; evaluation of required performance parameters; evaluation of blasting; and vibration safety requirements.

Typical verification testing includes piezometric measurements, penetration resistance tests (SPT, CPT, BPT), shear wave velocity determinations, and undisturbed sampling. Settlement is usually monitored.

e. Grouting and Soil Mixing.—Numerous technologies exist for grouting and soil mixing in ground improvement. The most common of these technologies include compaction grouting, jet grouting, and soil mixing [13].

1. Compaction Grouting.—This method of soil densification utilizes a low slump mortar grout (usually 0 to 50 mm (0 to 2 in), that is injected under high pressures (2.1 to 4.8 mPa, 300 to 700 lb/in^2), to form a series of grout bulbs that displace and compact loose foundation soils between and adjacent to the bulbs.

The cost of compaction grouting, when compared to other methods of foundation improvement, is moderate to high. The relative cost of compaction grouting becomes more economical when treating identifiable thin lenses or layers where treatment of competent overlying layers is not required, or in small areas adjacent to structures.

Compaction grouting is often used in tunneling and underpinning operations for settlement and deformation control. The method is rarely used in the construction of earth dams but has been used to remediate older dams. Prior to selection of compaction grouting for ground improvement associated with dam embankment foundations, the following issues need to be considered:

- Compaction grouting may not be practical on the downstream side of an embankment since a series of impermeable grout bulbs, while not interconnected, might impair drainage in the downstream direction. This could adversely impact embankment stability during static or dynamic loading.
- For remedial seismic designs of embankments that might use compaction grouting as a method of

foundation treatment, it is recommended that compressive strengths and shearing resistance provided by grout bulbs should be carefully evaluated. How the individual grout bulbs respond in treated dam foundation materials during strong seismic shaking is presently not well understood.

Design considerations for compaction grouting should include definition of the geotechnical parameters and structure interaction considerations, location of the compaction zones, definition of densification parameters, estimation of grout volumes required, and development of a plan for sequencing along with monitoring programs.

Quality control testing includes monitoring of grout consistency (slump and compressive strengths), pressures and volumes injected, and measurement of ground heave. On critical programs, site-specific evaluation of grout bulb effectiveness may be made. After installation, settlement is monitored and the improvement in soils adjacent to the columns can be monitored with penetration tests (SPT, CPT). Increases in stiffness and horizontal stresses can be monitored by flat dilatometer and pressuremeter tests.

2. Jet Grouting and Deep Soil Mixing.—Jet grouting is performed in foundation soils by high pressure, and high velocity jets of water and/or air, which "excavate" the materials, inject them, and mix them with some type of stabilizer (fig. 3-14). This procedure can be used to construct columns of improved soil in a wide variety of foundation soil types (gravels, sands, silts, and some clays). Columns and panels can be interlocked to form walls, to cutoff/reduce seepage through a foundation, or form cells within a potentially liquefiable foundation. The cost of jet grouting can be relatively high when compared to costs of other foundation improvement methods, depending on the design goals of foundation remediation and the distribution and type of soil to be treated.

Deep soil mixing methods can also be used to construct columns, walls, or cells of fairly precise final dimensions, in materials ranging from sand to clay. This method utilizes hollow stem auger flights and mixing paddles, which are advanced into the soil, while grout is pumped through the tip of the auger. The auger flights and mixing paddles mix the grout with the surrounding soils during penetration to design depth and withdrawal. The cost of deep soil mixing relative to that of other foundation improvement methods is moderate to high.

Jet grouting or soil mixing methods may introduce impermeable elements into the dam foundation that may adversely affect embankment stability (especially on the downstream side of a dam). In several cases, these methods have been used as cutoff wall elements for dams. In seismic strengthening applications, the stress-strain compatibility of the columns with encapsulated or adjacent foundation soils is poorly understood (use of compressive strengths or shearing resistance should be closely scrutinized in any analysis). Finally, ground motions reaching an embankment may be amplified through these stiffer elements, resulting in stronger shaking of the dam.

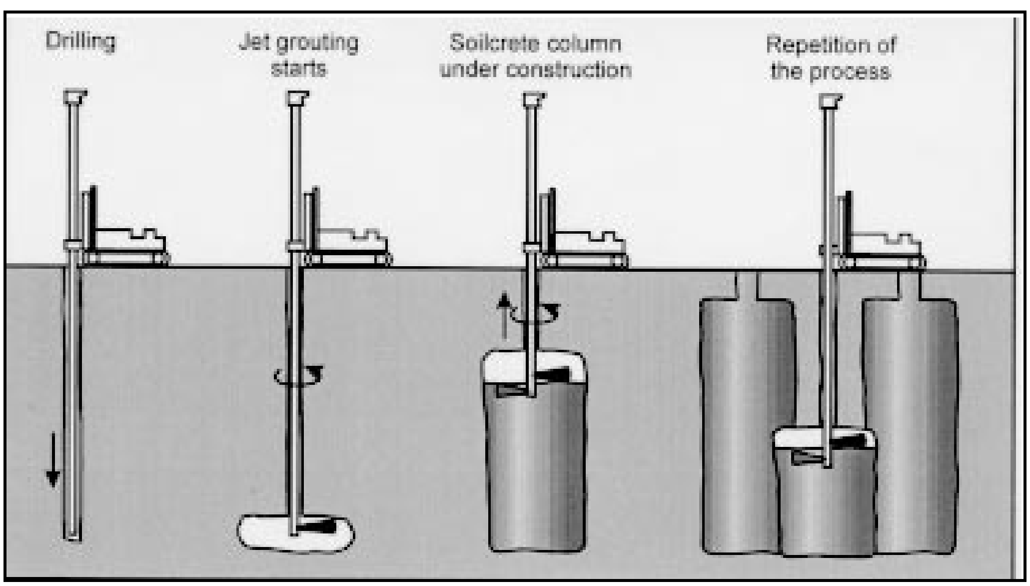


Figure 3-14.—Schematic drawing of jet grouting process showing construction of columns. (Hayward Baker, Inc.)

Design considerations for jet grouting and soil mixing include determination of the range of soils to be treated, required stabilizing additive dosage through trial mix design studies, reasonable strengths and permeability that can be obtained, treatment areas and depths and volumes, and required performance parameters and quality analysis and control monitoring plans.

Typical field testing consists of either waste or fresh column sampling of the soil additive mixture. Waste sampling can provide insight as to volume of the treated area and conservative assessment of in situ characteristics. Core samples are often procurred from cured columns, and compressive strength is evaluated. Water testing can be performed in boreholes, but there are often problems with proper location of the holes for reliable tests. Records of treatment methods (columns installed, volumes injected) are maintained to monitor production and document construction.

D. Embankment Dams

This section provides information according to Reclamation practices in the early 1990s. Clearly, the designer acquires expertise as knowledge and experience develops, technology advances, and investigation and construction practices improve. Reclamation publishes a series: *Design Standards No. 13—Embankment Dams;* it is periodically updated and reflects the latest technology. *Design Standards No. 13* should be consulted for detailed information on the aspects and components of embankment dams [14].

3-19. Foundation Treatment.—

a. Design Features.—An embankment dam is often the most feasible type of structure for a particular damsite because of relatively poor foundation conditions that are unfavorable for rockfill or concrete dam construction. Embankment dams are frequently founded on alluvial deposits that consist of layers of varying thickness of coarse sand and gravel, silts, or clays before bedrock is reached. Deeply weathered or faulted rock may also be encountered in embankment dam foundations. Foundations of thick deposits of windblown silts and fine sands present especially difficult problems. Foundation design must consider three main problems: (1) seepage, (2) stability, and (3) control of deformation.

Cutoff trenches, cutoff walls, or combinations of trenches and walls are most commonly used in foundations of permeable materials. These cutoffs are designed to stop the flow of seepage water or to lengthen the path of percolation under the dam. Blankets of impervious material that extend upstream from the impervious zone of a dam on the reservoir floor and cover all or part of the abutments are sometimes used for lengthening the percolation path.

Cutoff features are typically located at or upstream of the centerline of the crest of the dam beneath the impervious zone of the embankment. The location and design of the cutoff feature should provide an adequately long percolation path through both the embankment and the cutoff feature. Where the embankment is founded on or near bedrock, "curtain" grouting and "blanket" grouting are usually necessary to extend the cutoff into rock. Closing of joints and other openings in bedrock is usually necessary and is accomplished by injecting grout (under pressure) to form a curtain cutoff. Comparatively shallow low-pressure grouting ("blanket grouting") may be used to seal near surface fractures in the rock beneath the impervious foundation area.

Two general types of cutoff trenches are (1) sloping-side trenches and (2) vertical-side trenches. Depth, location, and type of cutoff trench used depend on the nature of foundation materials and on their permeability. Common practice for foundations, with bedrock at a shallow to moderate depth, is to excavate a sloping-side trench located at or upstream from the centerline of the crest of the dam. For trenches located upstream, the trench centerline parallels the centerline of the dam crest across the valley floor but converges toward the crest centerline as it extends up the abutment so that the cutoff is fully beneath the impervious zone of the embankment. When bedrock or an impervious stratum is relatively close to the surface, the cutoff trench may be widened to the full width of the impervious zone of the embankment. Sometimes, the entire embankment is founded on bedrock when the top of bedrock is located at a shallow depth. Where bedrock cannot be reached economically, the cutoff excavation is extended either to a stratum of relatively impervious material or to a depth that will provide the desired length of the seepage path. Care must be exercised when analyzing a dam foundation to ensure that liquefiable materials are not left in the foundation.

Sloping-side cutoff trenches are excavated using front-end loaders, shovels, backhoes, draglines, or scrapers. The excavation is backfilled with selected impervious materials equivalent to the impervious zone in the dam embankment and compacted in the same manner as rolled earthfill embankment material. Properly designed filters and transitions must be provided between the downstream face of the cutoff and alluvium (or rock surfaces) to preclude any piping into foundation alluvium (or bedrock). Rock surfaces must be shaped to provide gradually changing surfaces and must be treated to seal fractures ([14], ch. 3).

Vertical-side trenches may also be used as cutoffs. In some instances, the depth of a sloping-side cutoff trench may be extended by continuing the excavation as a narrow trench with vertical side walls. Vertical-side cutoff walls require special design and construction techniques where the two types of trenches are connected or where the wall connects to the base of the impervious zone. Sometimes, vertical-side cutoff walls are used near the upstream toe of a dam or in conjunction with upstream impervious blankets in which case they must be connected to the impervious zone of the dam. Use of cutoff walls has increased in recent years because of improved technology and construction techniques. There are basically three types of vertical-side cutoff walls: (1) soil-bentonite slurry-trench cutoff wall, (2) concrete cutoff wall, and (3) cement bentonite/plastic concrete cutoff wall (see sec. 3-18a).

In addition to forming cutoffs, concrete walls are sometimes used to treat certain geologic features such as faults or fractured zones. Particular geologic features may require placing short, independent wall sections at certain locations on the abutments or within the river channel. Concrete cutoff walls may be required:

- When bedrock is hard, but intensely fractured and irregular
- For bedrock dipping steeply, especially on the abutments
- Across shear or fault zones
- Across weathered zones

Careful consideration must be given to foundation contacts in the design of an embankment. Any continuous void spaces caused by lack of intimate contact between the impervious portion of the embankment and the foundation may result in dangerous seepage. Overhanging rocks and undesirable material should be removed; slopes should be flattened and smoothed to permit embankment materials to bond properly to the abutments. In thin core dams, the impervious section may be widened at the points of contact with the abutments.

Foundation grouting is a process of injecting suitable grouting materials into the underlying foundation through specially drilled holes of seal or fill joints, bedding planes seams, fissures, or other openings. Grouting may be done with neat cement (the most common method), mixtures of cement and sand, or cement mixed with bentonite or other clay. In special cases, chemical or bituminous grouting may be used. The adopted program may consist of grouting along a specified line or multiple lines to create a deep, relatively impermeable curtain; comparatively shallow or "blanket" grouting over wider portions beneath the impervious zone of the foundation area; or both. Every foundation presents a unique grouting situation that depends on composition and nature of the material and on the geologic structure. Figure 3-15 is a geologic section of an abutment of a dam showing a grout curtain.

In foundations of unlithified permeable materials, the path of percolation can be increased by constructing a blanket of compacted impermeable material upstream of the dam. Blankets are usually used when cutoffs to bedrock—or to an impervious layer—are uneconomical because of excessive depth. Sometimes, they are used in conjunction with partially penetrating trenches designed to increase the length of the path of percolation. The topography just upstream from a dam and the availability of impervious materials are important factors in the design of blankets. The length of upstream extension is determined by desired reduction of volume of seepage and of hydraulic gradient. In zoned embankments, the blanket must extend beneath the upstream shell as a continuation of the central, impervious section. Blankets may be constructed to cover either or both abutments or particular abutment areas. The blanket thickness will vary with permeability of the material and with the head of water. Filters may be necessary to prevent piping of the impervious blanket layer into the foundation.

Methods for improving foundation stability include:

- Removal of unsuitable material
- Preconsolidation by dewatering or by irrigation and loading
- Adequate drainage of the downstream toe by means of drainage blankets and filters, toe drains, and relief wells to avoid piping and to reduce hydrostatic pressures
- Construction of stabilizing fills to prevent displacement of the foundation
- In place densification

The minimum requirement for foundation stripping is removal of vegetation, sod, topsoil having high organic content, and other unsuitable material that can be removed

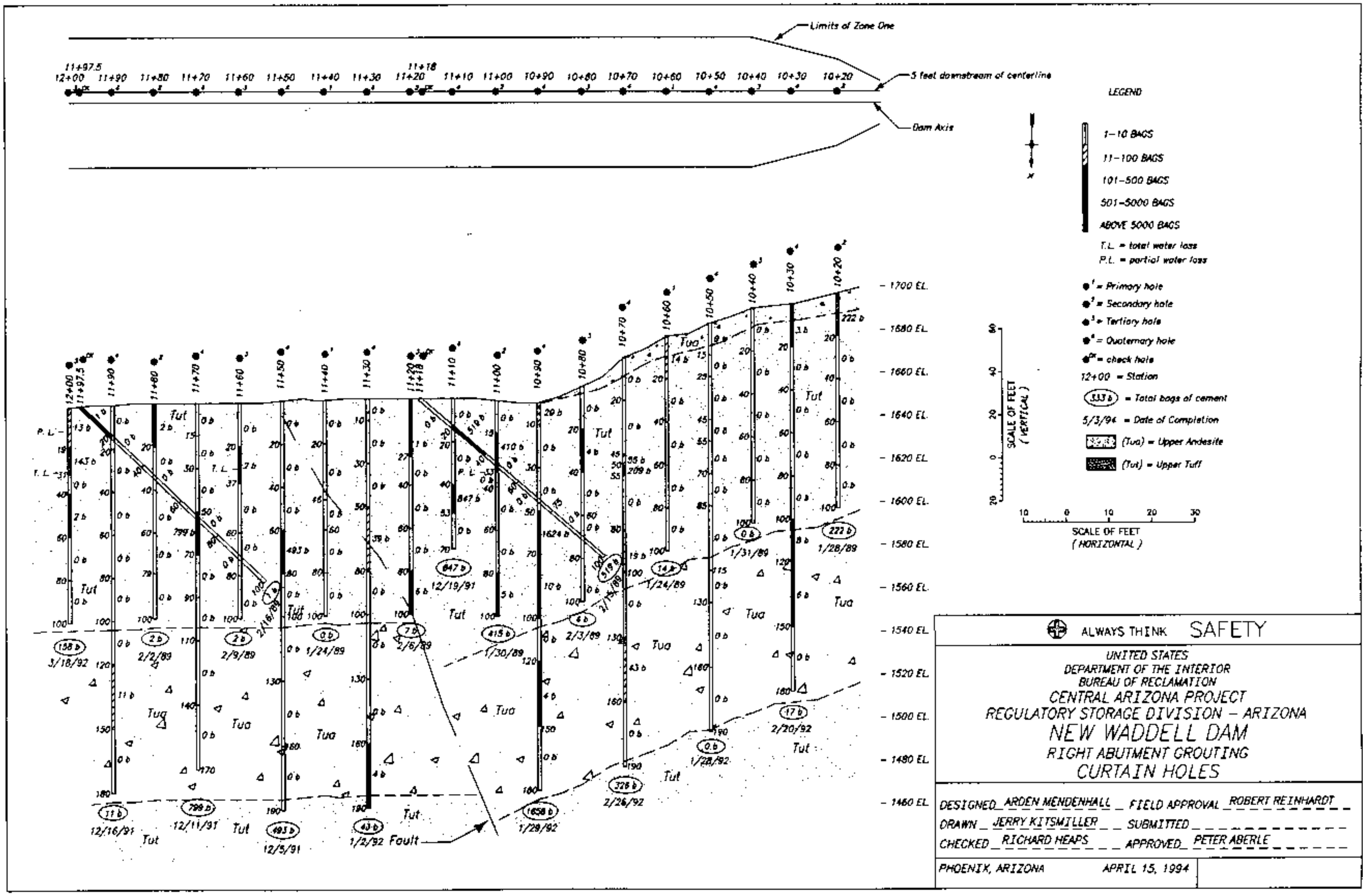


Figure 3-15.—Curtain grouting in right abutment of New Waddell Dam in Arizona.

by opencut excavation. In cases where overburden is comparatively shallow and composed of soft clays, loose fine sands and silts, or extremely pervious sands and gravels, or liquefiable materials, the entire foundation area of the dam may be stripped to bedrock.

In special cases, water may be added to the foundation by irrigation to cause consolidation. Dry loessal (wind deposited) soils and other low density silty soils have been successfully treated by this method. Sometimes, the entire foundation area is saturated and preloaded to cause consolidation and stabilization of otherwise unsuitable foundation materials when the cost is less than the alternative of excavating and replacing the material with selected borrow material. Usually, such treatment is accomplished by staged construction of the embankment.

Loose, fine sand deposits below ground-water table have been successfully consolidated by deep compaction techniques such as compaction piles and dynamic compaction—usually in conjunction with dewatering. Sometimes, these procedures are used to treat liquefiable sands and silts (see sec. 3-18).

The purpose of dewatering is to permit construction in the dry and to increase the stability of excavated slopes. Clean, coarse to medium sands can be readily dewatered. Clean, fine sand can be dewatered with some difficulty. Dirty, fine, medium, and coarse sands are difficult to dewater. Silts and clays require extraordinary methods to dewater. Usually, dewatering consists of drains, drains with sumps, deep wells, and wellpoints either alone or in combinations for maximum effectiveness [15]. Dewatering shall maintain a sufficiently low water table to allow for satisfactory excavation and backfill placement. Dewatering systems must be carefully designed to ensure the adequacy of the system. If sufficient effort is not made in dewatering, foundations can be damaged, excavation safety can be jeopardized, and costly construction delays can occur. Reclamation has taken a proactive approach of designing their own dewatering schemes in certain critical situations to avoid costly construction delays.

Preventing all seepage under a dam is not possible. Inevitably, water will escape somewhere in the vicinity of the downstream toe. When foundation materials in that area consist of or contain appreciable quantities of fine sands, silts, or clays that may be carried away—by water escaping under pressure—a filter or drainage blanket is provided. The filter is constructed of selected materials designed to satisfy filter criteria. The filter allows the necessary passage of water but prevents movement of fine soil particles from the foundation. A filter design accounts for grain sizes of foundation soils and head loss. Two, or more, layers of different sizes of sand and gravel may be required to prevent piping and to provide adequate drainage capacity. Another method to relieve hydrostatic pressures near the downstream toe of an embankment or high pressures beneath impervious foundation layers in the vicinity of the downstream toe is by installing vertical relief wells. The purpose of the wells is to relieve excess water pressure in the foundation, and thereby prevent formation of boils and heaving or piping.

Toe drains are commonly installed in a trench along the downstream toe of a dam as part of the filter or drainage blanket to collect and convey water to measuring devices. Perforated concrete, high density polyethylene, or other types of pipe may be used for the drains. Beginning with small diameter pipe laid along abutment sections, the lines progressively increase in size; with lines of maximum diameter being placed across the valley floor. Toe drain pipe must be laid in a trench of adequate depth and grade to be effective. Drainpipe is encased in a filter (usually sand) and drain material (usually gravel) to prevent piping of surrounding material into the drain. Access points should be installed along the drainpipe and outlet line for inspection and maintenance and, if desirable, at other locations for observation and measurement of the amount of seepage water and detection of any piping. The function of drains is to allow uninhibited escape of seepage water, thereby preventing saturation and consequent reduction of stability of the downstream embankment and foundation of the dam. Toe drains, placed sufficiently deep in the foundation keep the ground-water level low, preclude high exit gradients, and prevent formation of boggy areas near the downstream toe of a dam.

To make foundations of low strength materials suitable as support for a dam, stabilizing fills are often used to provide mass at the upstream, downstream, or both toes of a dam. These fills act to restrain foundation materials from displacing under load by increasing the normal effective stress across potential failure surfaces. The depth and extent of these fills depend on the properties of the foundation materials. Stabilizing fills are generally contiguous and integral with embankments.

b. Specifications Provisions.—During construction, diversion and care of the river (stream, or surface runoff) is an important factor in the design of an embankment dam. Figure 3-16 shows a temporary diversion channel through an embankment dam. Specifications require that the contractor's plan for diversion be approved, or the diversion plan should be provided in the specifications.

Figure 3-16.—Temporary diversion channel through Bonny Dam, Colorado.

Underwater excavation for the foundation of a rolled embankment dam is not permitted. The contractor is required to unwater the foundation in advance of excavating an embankment cutoff trench. The unwatering is required to be accomplished in a manner that will:

- Prevent loss of fines from the foundation
- Will maintain stability of the excavated slopes and bottom of the cutoff trench
- Will result in all construction operations being performed in the dry

Dewatering is to be continuous until the excavation has been filled, except that in filling the cutoff trenches, the ground water may be permitted to rise within 1.5 m (5 ft) of the top of the compacted embankment after at least 3 m (10 ft) of fill have been placed in the dry. This provision enables the contractors to remove the majority of well points if they use that type of dewatering process. Usually, it is impractical to prevent water from seeping out along the contact of the cutoff trench slopes with the foundation. Hence, some surface drains are needed to ensure that backfill is placed in the dry. Open-jointed or perforated pipe drains surrounded by gravel and leading to sumps can be used to dry up the foundation contact. Placement of drains that will be left in place should be very carefully controlled and must be meticulously filled with grout to avoid leaving any potential leakage paths. The layout of these drains should be carefully planned so that the contact of the impervious zone is never penetrated, transversely, by more than one-third of its width. The distance between the ends of transverse drains from opposite sides of the impervious zone should be no closer than one-third the width of the pervious contact.

The requirements for excavation of embankment foundation provide that the area to be occupied by the embankment shall be excavated to a sufficient depth to remove all topsoil, rubbish, roots, vegetable matter of every kind, and all other perishable or objectionable material that might interfere with proper bonding of the embankment with the foundation. Earth foundation surfaces are to be prepared by leveling and rolling in the same manner as specified for placing embankment materials. Rock foundation surfaces are to be cleaned of all loose and objectionable materials. Foundation surfaces of shale and other materials that tend to slake or loosen on contact with air are to be cleaned just before placing earthfill materials or protected from moisture change. All foundation surfaces are to be moist but free from standing water. Figure 3-17 shows a foundation being prepared by excavation and water jetting to remove loose rock and to smooth the impervious foundation prior to placing earthfill.

c. Control Techniques.—The weak points in embankment dam construction are generally found within the foundation at the foundation contact with the embankment and at the contact of the natural ground surface with placed embankment. Construction of foundation seepage control and stability features included in the design must be carefully supervised by the inspection force to ensure compliance with specifications. Dewatering methods used in connection with the excavation of cutoff trenches and for stabilizing foundations should be carefully checked to verify that fine material is not being washed out of the overburden materials because of improper performance of drainage wells. To avoid creation of a "live" bottom due to upward flow of water, continuous pumping may be required.

Where overburden is stripped to firm material, the foundation surface (including all pockets or depressions) should be carefully cleaned of soil or rock fragments before placing the embankment. This may require handwork water or compressed air-water cleaning or both. Cleaning is particularly important beneath the impervious zones. Surfaces that disintegrate rapidly on exposure should be covered immediately with embankment material or an approved protective covering. If stripping operations indicate the presence of unstable or otherwise unsuitable material, test pits should be excavated for further exploration, and generally this material will require removal.

Before placing the first layer of impervious zone material on an earth foundation, the foundation surface should be leveled, moistened, and compacted. Foundation surfaces of firm material should be moistened, but standing water should not be permitted when the first lift is placed. An earth foundation surface may require scarification by disks or harrows to ensure proper bonding. When the foundation can be penetrated by tamping roller feet, scarification is not usually necessary. Where the surface would be damaged by penetration of the tamping-roller feet, a thicker layer of earthfill may be permissible for the first compacted layer, or the first layer may be compacted with rubber-tired (pneumatic) rollers. However, the first layer should not exceed a thickness that will prevent attainment of the specified density and should never exceed 400-mm (15-in) loose lift. When thicker lifts are placed, additional roller passes are usually required to ensure that proper compaction is attained. Special compaction methods should be used in pockets that cannot be compacted by the specified roller instead of permitting unusually thick first lifts to obtain a uniform surface for compaction. Figure 3-18 shows compaction of earthfill (zone 1) near the foundation contact with a pad foot roller at Reclamation's Jordanelle Dam, Utah.

On steep, irregular rock abutments, wetter-than-optimum material may be necessary or desirable to obtain complete and proper bond. The rock surface should be properly moistened. Rock abutments should be shaped by excavation or dental concrete to allow compaction by pneumatic tire equipment or rollers directly against the contact. The fill surface should be sloped or ramped to about 1v:6h near the foundation surface to direct the force of the pneumatic roller wheel towards the foundation surface. Tamping by hand operated equipment should be avoided. If absolutely unavoidable, when hand-held compactors are used, care must be exercised to ensure that bond is created between successive layers of material. This may require light scarification between lifts of tamped material.

3-20. Compacted Earthfill.—

a. Design Considerations.—Rolled embankment dams vary considerably in proportion and physical characteristics of soils from which they are constructed. Some dams contain essentially one kind of material and are called homogeneous impervious structures, while others are constructed of zones of sand, gravel, rockfill, and miscellaneous soils in a variety of quantities and locations within the fill, in addition to an impervious zone. The impervious materials include clays (CL and CH), clayey sands or gravels (SC or GC), and silty clays (CL-ML). Occasionally, more pervious materials such as silts (ML) and silty sands or gravels (SM or GM) are used in the impervious zone. This impervious zone of earthfill, usually designated zone 1 in Reclamation dams, provides the water barrier and is common to all rolled embankment dams. Homogeneous dams are virtually nonexistent in modern day dam engineering—for dams of any size. Internal filters and drains are always provided except for extremely small dams.

In controlling construction of impervious earthfill zones, the following design criteria must be satisfied:

(1) The material must be formed into an essentially homogeneous mass free from any potential paths of percolation through the zone or along the contacts with the abutments or concrete structures.
(2) The soil mass must be sufficiently impervious to preclude excessive water loss through the dam.

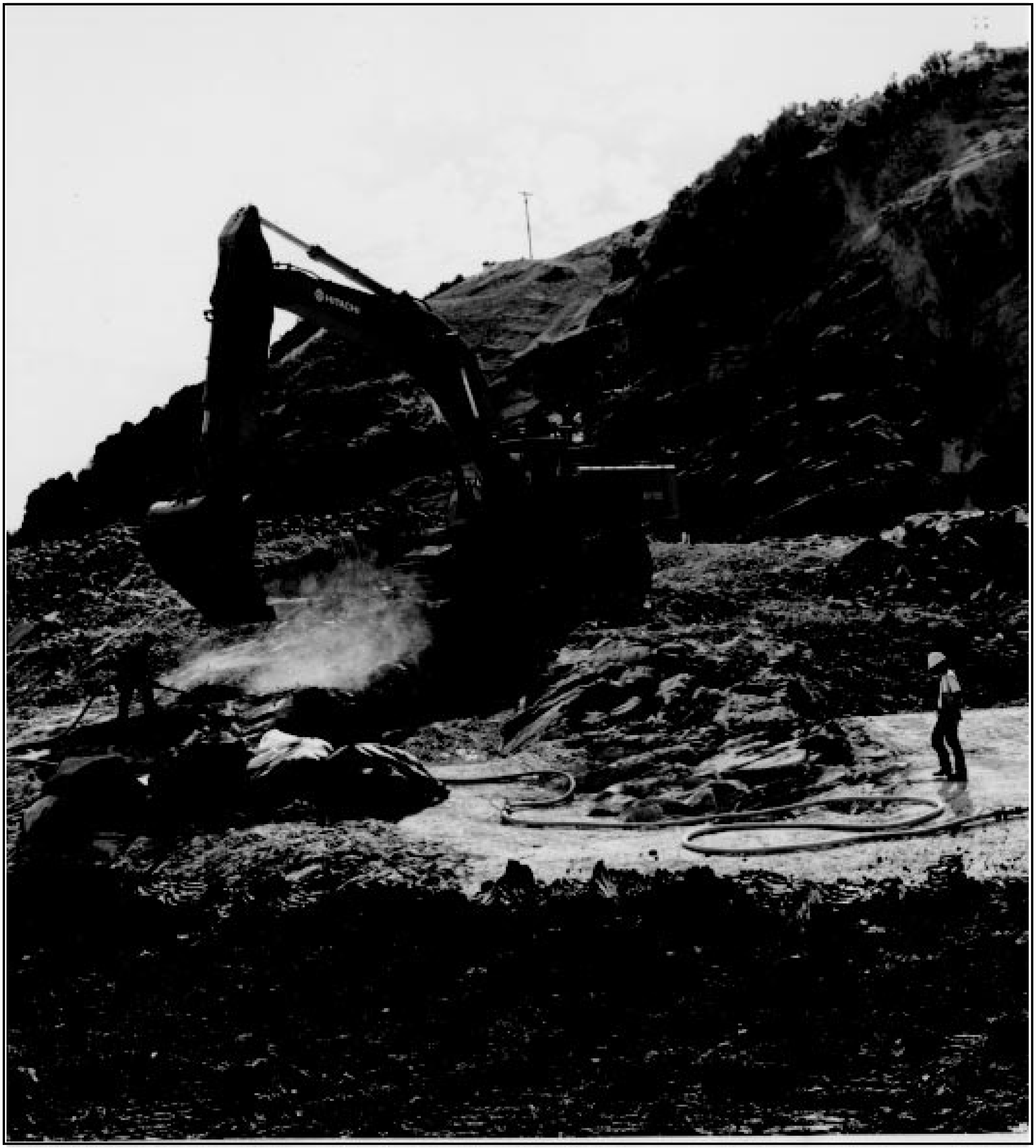


Figure 3-17.—Foundation preparation for the impervious (zone 1) foundation at Jordanelle Dam, Utah. The workman near the backhoe is using a high pressure mixture of air and water. The backhoe has a metal plate welded across its bucket teeth to provide a smooth edge for scraping. Inspector in right center, and impervious fill shown in foreground (July 1990).

Figure 3-18.—Earthfill (zone 1) compaction by a model 925 C Caterpillar pad foot roller. Also, notice filter upstream of zone 1 being compacted by a vibratory roller. Filter is over weathered fracture zones in the foundation rock (July 1990).

(3) The material must not consolidate excessively under the weight of superimposed embankments.
(4) Differential settlement that would cause shear planes must be avoided.
(5) The soil must develop and maintain its maximum practicable shear strength.
(6) The material must not consolidate or soften excessively upon saturation.

These design objectives are achieved by proper use of equipment and methods of borrow-pit conditioning, excavating, placing, compacting, and foundation shaping.

To satisfy design criteria of imperviousness, low consolidation, and resistance to softening, a high density is desirable. However, to control development of pore pressure—which would reduce shear strength—and to avoid formation of slickensided layers, which may become paths of percolation, material that is too wet must be avoided. Based on knowledge of available materials gained from exploration and testing, the designer will specify moisture control requirements near laboratory optimum moisture content. The inspector should observe the fill placement operation to ensure that specifications requirements are satisfied and that an adequate fill is obtained. The plan of control of the impervious earthfill for high dams must be a compromise that will result in a proper balance among all the design criteria. Experimentation with different moisture-density control procedures may be necessary to obtain proper deformation properties of earthfill (zone 1). For example, the California Department of Water Resources compaction control procedure was used at New Waddell Dam, Arizona to obtain deformation properties slightly stiffer than standard Reclamation laboratory tests would have yielded.

b. Specifications Provisions.—The specifications for open cut excavation require that all suitable excavated materials shall be used for permanent construction. Materials suitable for earthfill must be excavated separately and in the same manner as those fill materials excavated from borrow areas. Provisions are made for direct placement of materials in the embankment or for stockpiling.

The specifications designate which borrow areas to use for earthfill material and provide Government control for both location and extent of borrow activity to obtain suitable materials. Designated borrow pits within a borrow area are required to be cleared and stripped of unsuitable material and are to be irrigated or drained to obtain the proper water content before excavation. The contracting officer designates the depth of cut in all borrow pits; the contractor's operations must be such that earthfill material delivered on the embankment is equivalent to a mixture of materials obtained from an approximately uniform cutting from the face of the borrow-pit excavation. When necessary, separation of oversized material from the earthfill is specified.

The specifications prohibit placing frozen soil in the embankment or placing unfrozen material upon frozen embankment. The maximum allowable difference in elevation between zones during construction is specified. The Government controls all openings through the embankment, limits the slopes of bonding surfaces, and requires careful preparation of sloping surfaces before additional material is placed. Slopes of bonding surfaces in earthfill material usually are limited to 4h:1v or flatter. Each load of embankment material must be placed in the location designated by the Government.

Specifications provide for placing earthfill only on dewatered and prepared foundations. All cavities within the foundation area are to be filled. Materials to be used for earthfill are described. In placing earthfill, distribution and gradation of material must avoid lenses, pockets, streaks, or layers that would prevent homogeneity of the fill. If the earthfill material contains only occasional cobbles or rock fragments, provisions are made to remove such cobbles or rock fragments over 125 mm (5 in) from the fill before rolling. When an appreciable percentage of oversize is known to exist in the source of earthfill material, a separation plant is specified, as shown on figure 3-19. When a separation plant is required, separation may be specified on either the 75- or 125-mm (3- or 5-in) size. Earthfill placement is to be in approximately horizontal layers not more than 150 mm (6 in) in compacted thickness. Fill surfaces that are too dry or too wet are required to be reconditioned before additional materials are placed.

Moisture control provisions of earthfill require uniformity of water content throughout the entire layer before and during compaction. Water content of a soil is limited to a range based on design considerations. In general, the average water content will be near the standard Reclamation laboratory optimum moisture content or, in some cases, near optimum of an alternative moisture-density control test such as the California Department of Water Resources compaction test. The contractor is required to perform all operations necessary to obtain the proper water content by irrigating or draining borrow pits, by performing supplementary sprinkling on the fill, and by working the material in the borrow area or on the fill prior to compaction.

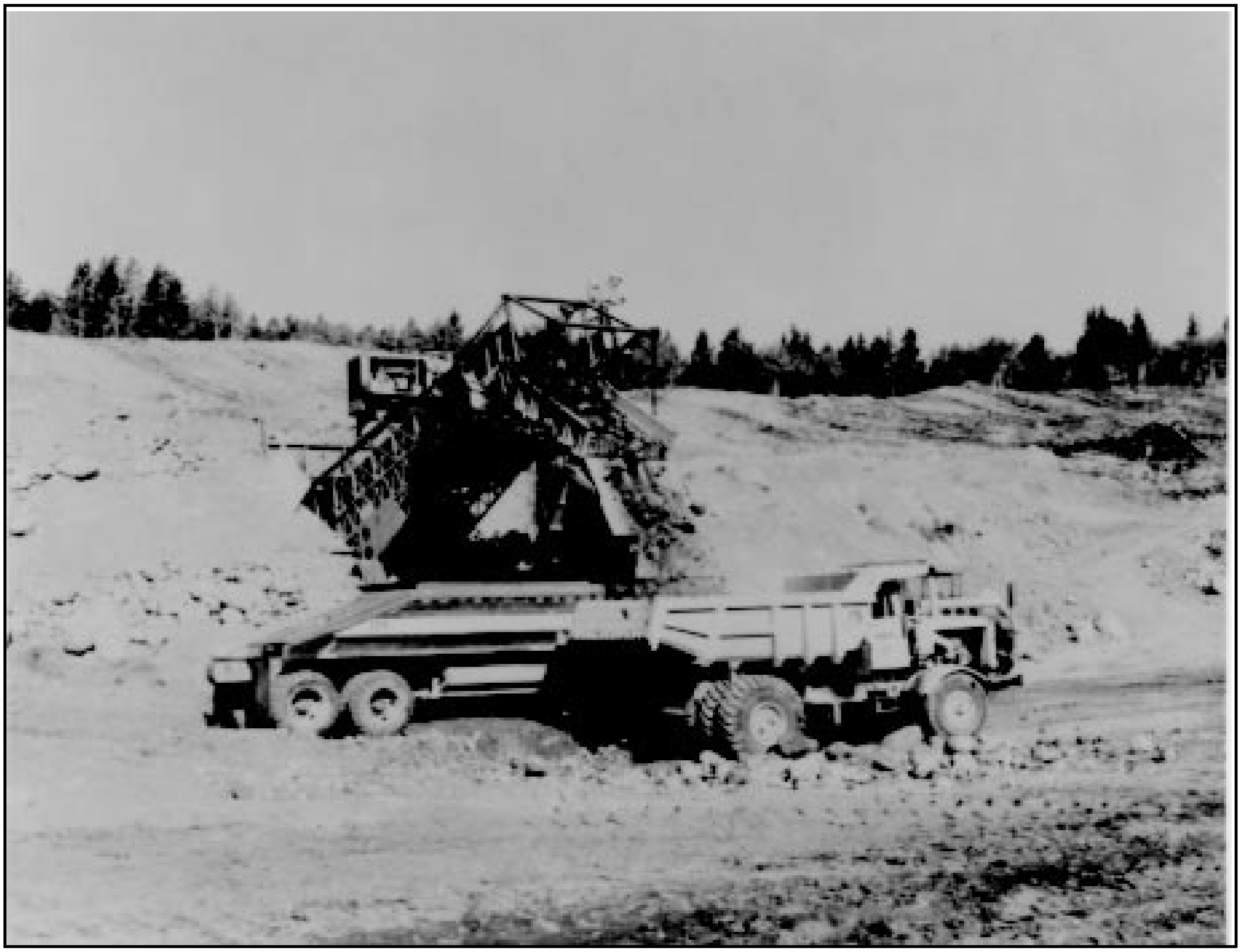


Figure 3-19.—Separation plant at Meeks Cabin Dam in Wyoming.

Tamping (sheepsfoot) rollers are generally required for compacting earthfill. However, many other acceptable rollers have recently been developed and used. If a roller other than the specified roller is proposed, a test fill should be constructed to ensure that the proposed roller will perform adequately. Normally, the tamping roller is still specified. The size, mass, feet arrangement, and safety features of the drums are specified. The number, location, length, and cross-sectional area of the tamping feet are described. The contractor is required to keep the spaces between the tamping feet clear. The number of passes of the roller on each layer of material is specified. Provisions are made for situations where materials are too dry or too wet. Where rolling with a specified roller is undesirable or impractical, provisions are made for special compaction by tampers or other equipment and for procedures that will obtain the same degree of compactness as the rolling process. Figures 3-20 and 3-21 show the operations of placing, spreading, conditioning, and compacting earthfill.

The specifications provisions for control of placement water content and compaction must be implemented by establishing procedures to ensure their attainment. Prior to construction of earthfill, all information available from field and laboratory investigations is analyzed and reviewed in light of design considerations to formulate tentative control procedures. Special limiting moisture tests are performed on representative samples of materials to be used in high embankment dams for determining placement conditions that will prevent development of excessive pore-water pressures during construction and for settlement of the loaded soil when saturated by water from the reservoir. Based on field and laboratory investigations and on experience with similar soils, the Denver Office will recommend placement moisture limits and minimum density for representative types of materials.

c. Control Techniques.—Although fill conditions are approximated by laboratory tests, it is always necessary to

Figure 3-20.—Earthfill operation on impervious zone at Reclamation's McPhee Dam, Colorado. Selfpropelled tamping compactor and disk plow in the foreground. Dart and Caterpillar belly dumps hauling and placing impervious fill in the center of the photo. Contact between filter/drain chimney can be seen on right side (July 1983).

check planned control against actual results obtained on the embankment and then to make any required adjustments. This is done by careful observation and analysis of operations, control test results, and embedded instruments, especially during the early stages of earthfill construction.

In structures where unusual conditions are present, the specifications may include provisions for the contractor to construct one or more test sections of earthfill material. The purpose of a test section is to determine the most effective excavating, processing, placing, and compaction procedures for representative soils under jobsite conditions. By varying placement procedures within certain limits, by exercising rigid control over the relatively small volume of the section, and by maintaining complete, accurate records of tests, the most applicable procedures for the rolled earthfill portion of embankment may be determined during the initial stages of construction. Results of field density tests made on test sections will provide the necessary information for establishing construction control procedures consistent with design requirements. Figure 3-22 shows a test section under construction using a pad foot roller to compact zone 1 at New Waddell Dam, Arizona.

After proper placing procedures have been determined from the embankment test section or from initial placing operations when test sections are not required, construction can proceed at full scale. Throughout construction, the contractor's operations must adhere strictly to limits and conditions set forth in the specifications and to

Figure 3-21.—A view of McPhee Dam under construction. Upstream is to the left. Zoning can be clearly seen. From the left: gravel and cobble fill shell; impervious zone 1; zone 2 filter material; zone 2a drainage material; and downstream gravel and cobble fill shell.

recommendations made by the design engineering staff. A complete record of all operations must be kept so the most efficient methods may be perfected. Relationships between laboratory compaction curves and field roller curves obtained by analysis of control test results are shown for three dams on figure 3-23.

The most important variables affecting embankment construction are:

- Selection of materials in the borrow area
- Distribution of materials on the embankment to obtain uniformity
- Placement water content, and uniformity of this moisture throughout the spread material
- Water content of borrow material
- Methods for correcting borrow material water content if too wet or too dry
- Roller characteristics
- Number of roller passes
- Thickness of layers
- Maximum size and quantity of rock in the material
- Condition of the surface of layers after rolling
- Effectiveness of power tamping in places inaccessible or undesirable for roller operation

To maintain control of earthfill construction, an adequate number of inspection and laboratory personnel are essential. For each shift, at least one inspector should be on the embankment and one technician in the laboratory. Borrow


Figure 3-22.—Pad foot roller on a test fill of zone 1 material at New Waddell Dam, Arizona (1990).

operations on small jobs may be inspected by the embankment inspector. On a large project, an inspector is required at the borrow pit, especially when moisture control is critical. On large projects, additional inspectors and technicians may be required in all areas.

The borrow-pit inspector selects the areas in the borrow pit to be excavated, determines the depths of cut, and specifies the zone of the embankment where a particular material should be placed according to an approved distribution plan. The inspector checks adequacy of any mixing or separation methods performed by the contractor and, as required, the inspector cooperates with the contractor in determining the amount of moisture to be added (or removed) to the borrow pit to attain proper water content in the materials prior to placement. Moisture content determinations of borrow-pit materials should be made well in advance of excavation so that corrective measures can be taken promptly.

The difference between laboratory optimum water content and borrow-pit water content can be determined by USBR 7240: Rapid Method of Construction Control.[6] As a rule, to adjust to the final moisture content in the borrow pit is impractical because of potential changes in water content caused by (1) rain, (2) mixing or separation operations, and (3) evaporation. However, an effort should be made to have the excavated material as close as possible to the desired water content before delivery to the embankment. In some cases, specifications will require that no more than 2- to 3-percent water can be added on the fill.

The foregoing inspection principles also are applicable to structural excavations where materials are to be placed directly in the embankment or are to be first processed or stockpiled. Figure 3-24 shows the cut face of a borrow pit where mixing of soils is accomplished during excavation; figure 3-25 shows another method of excavating that produces a good mixture from a layered deposit. Also, figure 3-3, shows a cut face of a borrow pit in which controlled excavation allowed material from the same pit to be used for two separate embankment zones—pervious and impervious.

The embankment inspector should determine the location and elevation of tests made on the embankment and record the location of the contractor's operations. Horizontal control, or station and offset, should be established. Vertical control can be benchmarks and hand levels and stadia rods used to establish elevations on the fill. When

[6] 7240: Performing Rapid Method of Construction Control.

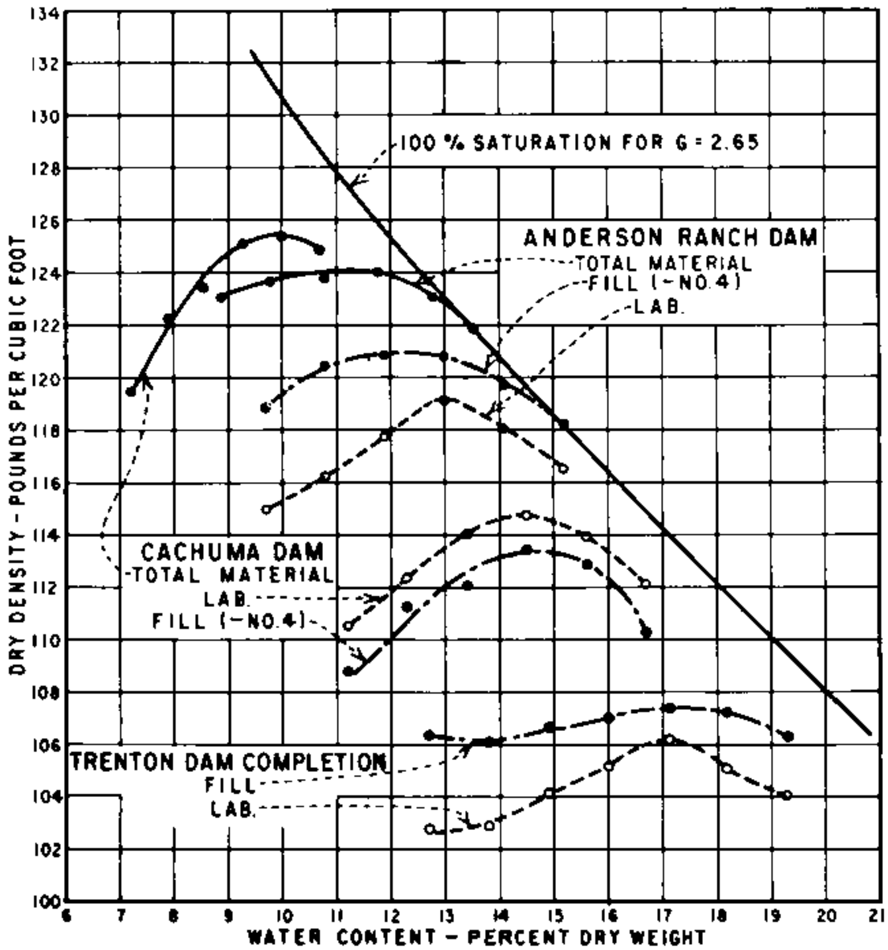


Figure 3-23.—Average field and laboratory compaction curves.

materials are brought onto the embankment, the inspector must verify placement in the proper zone, as indicated by the approved distribution plan. Lines of demarcation can be painted on rock abutments or marked by colored flags.

After materials are placed in the correct location, the embankment inspector determines if the material contains the proper amount of moisture before compaction. The rapid method of construction control (USBR 7240) will determine whether supplementary water is required. When materials arrive on the embankment too dry, water can be added by sprinkling before, during, or after spreading. The contractor's operations in sprinkling and mixing water with soil will vary. However, regardless of the method used, the proper water content must be uniformly distributed throughout the layer before compacting.

Another important inspection task is to determine the thickness of the compacted layer. A layer spread too thick will not have the desired density for given conditions of compaction. For initial placing operations, the test section will have determined the proper spread thickness of a layer which will compact to specified thickness; this specified compacted thickness should never be exceeded. A method for determining average thickness of placed layers is to plot (daily) a cross section of the fill at a reference station(s). The inspector's report for that day will contain the number of layers placed at that section; hence, average thickness can be determined.

When oversize rock content is greater than about 1 percent, removal of oversize rock from earthfill embankment material is best accomplished prior to delivery of the soil on the embankment. Smaller amounts of oversize rock can be removed by handpicking or, under favorable conditions, by various kinds of rock rakes. Generally, oversize rock, which had been overlooked before rolling, can be detected by the inspector during rolling by observing a bounce when the roller passes over a hidden rock. The inspector should be sure that the contractor removes the rock from the fill.

Figure 3-24.—A Holland loader excavating a vertical face mixing silt and fine sand deposits for impervious fill at Reclamation's Calamus Dam, Nebraska (Sept. 1983).

The inspector must ensure that each lift receives the specified number of roller passes. An oversight in maintaining the proper number of passes may lead to undesirable low densities. Insistence upon orderliness of fill procedures and the establishment of routine construction operations will help to achieve the required number of roller passes.

The final check on the degree of compaction is determined by field density tests (USBR 7205, 7220)[7] and the rapid method of construction control (USBR 7240).[8] The rapid method should be checked periodically against values obtained using the standard laboratory compaction of soils test procedure (USBR 5500).[9] If the degree of compaction of material passing the 4.75-mm (No. 4) sieve is at or above the minimum allowable percentage of laboratory maximum dry density and the water content is within allowable limits, the embankment is ready to be scarified and moistened, if necessary, to secure a good bond between layers before the next layer is spread. Time required to obtain results of the field density test by the rapid method should be about 1 hour or less.

[7] 7205: Determining Unit Weight of Soils In-Place by the Sand-Cone Method.

7220: Determining Unit Weight of Soils In-Place by the Sand Replacement Method in a Test Pit.

[8] 7240: Performing Rapid Method of Construction Control.

[9] 5500: Performing Laboratory Compaction of Soils—5.5-lbm Rammer and 18-in Drop.

Figure 3-25.—Two model 992C Caterpillar front end loaders loading zone 4 (gravel and cobble fill) from borrow into a model 540 Dart belly dump truck. This type of loading operation, with steep face of borrow area, can be used to get a well mixed fill from a layered deposit. Jordanelle Dam, Utah (August 1990).

Mechanical tamping—used around structures—along abutments, and in areas not accessible to rolling equipment, should be carefully observed and checked through use of frequent density tests. The procedure to be followed for mechanical tamping depends greatly on the type of tamper used. Some factors affecting density are:

- Thickness of layer being tamped
- Time of tamping
- Water content of the material
- Mass of tamping unit
- Condition of tampers
- Air pressure (if air tampers are used the manufacturer's recommended pressures should be maintained)

The embankment inspector must be alert for areas of low density and have them remedied by sprinkling, scarifying, removing, or rerolling, as required. An important function of inspection is to determine when and where to make field density tests. These tests should be made:

- In areas where the degree of compaction is doubtful
- In areas where embankment operations are concentrated
- For every 1,500 m^3 (2,000 yd^3) of embankment when no doubtful or concentrated areas occur
- For representative tests of every 25,000 m^3 (30,000 yd^3) of earthfill placed
- For "record" tests at all embedded instrument locations
- At least once during each work shift

Areas of doubtful density are sometimes detected by the inspector's observations. Insufficiently compacted locations may include the following:

- The transition between areas of mechanical tamping—or special compaction—and rolled earthfill embankment along abutments or cutoff walls
- Areas where rollers turn during rolling operations
- Areas where too thick a layer is being compacted
- Areas where improper water content exists in a material
- Areas where less than specified number of roller passes were made
- Areas where dirt-clogged rollers are being used to compact the material
- Areas where oversize rock has not been removed from the fill
- Areas where materials with minor amounts of frost have been placed, or were placed at nearly freezing temperatures
- Areas that were compacted by rollers that have possibly lost part of their ballast
- Areas containing material which differ substantially from the average

The number and location of field density tests should be such that the extent of the doubtful area is determined. All tests made in areas of doubtful density should be identified by the letter D.

When an embankment operation is concentrated in a small area, i.e., if many layers of material are being placed one over the other in a single day, tests should be made in this area in every third or fourth layer to ensure the desired density is attained. In such an area, all tests should be identified by the letter C.

Even if areas of doubtful compaction do not exist or tests are not required because of other reasons, at least one field density test should still be made for each 1,500 m^3 (2,000 yd^3) of compacted embankment, and it should be representative of the degree of compaction being obtained. Such tests should be identified by the letter R.

Regardless of the number and purpose of other field density tests, for large dams, one test is required for each 25,000 m^3 (30,000 yd^3) of earthfill placed; the tests should be representative of the conditions of the fill. Percolation settlement tests and specific gravity tests are made on the same sample; the tests should be identified by the letter R ("record"). Additional material must be excavated adjacent to the field density test hole to obtain enough soil to make these additional tests.

"Record" tests are made at all instrument installations and are recorded separately.

Test pits should be excavated periodically during construction to visually check the fill for:

- Uniformity
- Laminations
- Contact areas
- Moisture content
- Density
- Interface zones

The effect of rolling and of superimposed loading is evaluated by making field density tests as the test pit is excavated. The degree of uniformity of moisture content with depth is found by testing samples from various depths. Visual inspection indicates whether successful bonding of layers had been accomplished. As shown in figure 3-26, undisturbed samples should be taken periodically for laboratory testing to determine engineering properties and so comparisons can be made to values used in the design. The pit should be photographed. During a seasonal shutdown period, test pits, as shown in figures 3-26 and 3-27 should be excavated in the compacted embankment to determine the net result of the season's operations. Any unusual conditions observed in the embankment test pit hole should be reported.

3-21. Compacted Pervious Fill.—

a. Design Considerations.—Permeable materials are used in rolled embankment dams in internal filters and drains and transition zones to control seepage and piping and in outer shells to provide high strength to support the impervious core. Usually, pervious fill is free-draining cohesionless sand and gravel containing less than about 5 percent fines (material passing the 75-µm (No. 200) sieve). Four criteria must be satisfied in controlling construction of zones of sand and gravel:

Figure 3-26.—Block sample from test pit in zone 1 at McPhee Dam, Colorado. Final saw cut being made beneath the sample with a chain saw equipped with special tungster carbide bit. Also shows area where an in place density test was taken 600 mm upstream of the block sample (1983).

- The material must be formed into a homogeneous mass free from large voids.
- The soil mass must be relatively free draining depending upon its intended use.
- The material must not consolidate excessively under the mass of superimposed fill.
- The soil must have a high angle of internal friction.

If chimney drains containing filter and drainage layers are not used (recently, chimney drains almost always are a requirement), homogeneity of the shell is important to ensure distribution of the inevitable seepage from the impervious zone throughout the length of the dam since concentration of seepage water into a few channels may induce dangerous piping. The permeability of the permeable zone must be much greater than the impermeable zone. In addition, permeability of the free-draining zone should be high enough to preclude development of construction pore-water pressures in this zone. The requirements of low consolidation and high shear strength for a permeable zone can be obtained by properly compacting the materials. Because excessive pore-water pressure is impossible to develop in pervious free-draining materials, the maximum practicable compaction of such materials is desirable.

b. Specifications Provisions.—Specifications on opencut excavations provide that suitable materials shall be used for permanent construction. Provisions are made for temporarily stockpiling such materials. When permeable

Figure 3-27.—Test pit at the interface of sand filter and gravel drain, Jordanelle Dam, Utah (September 1990).

soils are to be obtained from borrow pits, provisions for compacted earthfill materials apply equally for these soils, except that irrigation of borrow pits is not required.

Specifications prohibit use of frozen soil and do not allow placing unfrozen material on frozen embankment. These provisions are applicable to all kinds of soils. Provisions for maximum allowable difference in elevation between adjacent zones control placement of permeable material. Control of openings through embankments, slopes of bonding surfaces, and preparation of bonding surfaces apply equally to all zones within the embankment. Distribution of material within a permeable zone is controlled by the inspection staff.

Written directives are in the specifications pertaining to permeable material used as a zone in the embankment. The material will be described and its source designated. Allowable thickness of the layer will be specified, either in the form of compacted thickness or placed thickness. Vibratory steel-drum rollers in the mass range of 4,500 to 9,000 kg (5 to 10 tons) are the best equipment for compacting pervious clean coarse-grained soils. Drum-drive self-propelled vibratory rollers may be effective on fine uniform sands when other vibratory rollers are not. Rubber-tire rollers may be used if they can produce the required densities. At times, track-type equipment are used effectively in rough areas or in confined areas where a vibratory roller cannot operate effectively, for example, in compacting a horizontal drainage layer on an irregular foundation. The contact pressure of the equipment should be at least 62 kPa (9 lbf/in^2) and should operate at a speed which imparts greatest vibration to the fill. Track-type equipment should be discouraged for compacting drainage blankets and filter drains because of potential breakdown of the soil particles.

Sometimes, alternate methods of compaction are permitted by the specifications. In such cases, a minimum relative density requirement is specified.

c. Control Techniques.—Workability of a permeable soil is reduced considerably by the presence of even small amounts of silt or clay. The contractor's operations in the borrow pits and on the fill must be such that contamination of the pervious soil is minimized. As the material is brought on the fill, it must be directed to the proper zone. Within pervious shell zones, individual loads should be directed so coarser material will be placed toward the outer slopes. This does not apply to filter or drain material in chimney drains or blankets.

When compacted thicknesses are specified, the required thickness of the loose layer, which will result in the specified final thickness, must be determined by the inspector during initial construction stage. Since in-place density will be checked by relatively few tests after satisfactory placing procedures have been established, the required thickness of the loose layer must be maintained within close limits throughout construction. The specified thickness of the compacted layer is usually great enough to accommodate the maximum size of rock encountered in the borrow. In cases where cobbles or rock fragments occur that are greater in size than the specified thickness of layer, provisions are made for special embedding, for removal to outer slopes of the pervious zone, or for removal to other zones. The inspector must ensure that provisions for disposal of oversize rock are followed.

After material has been placed, spread to the desired thickness of lift, and oversize cobbles or rock fragments removed, the next important step is application of water. Thorough and uniform wetting of materials during or immediately before compaction is essential for best results. The best method for adding and distributing water on the fill should be determined during initial placement. Relaxing requirements for thorough wetting may result in densities far below the minimum requirement—even with great compactive effort. Different pervious materials require different amounts of water for thorough wetting and best compaction. In general, adding as much water to the material as it will take is desirable. Too much water cannot be added to an extremely pervious soil; however, permeable soils containing small quantities of silt or clay may become temporarily boggy if an excessive amount of water is used. For these soils, care must be exercised in wetting the soil; the contractor's operations must be carefully controlled to avoid creation of temporary boggy conditions.

Vibratory compaction units should be checked frequently to ensure they are operating at a frequency which produces the highest possible density. For cohesionless materials, the frequency of vibration generally should fall between 1,100 and 1,500 vibrations per minute. If the track-type equipment is used, operating at the highest practicable speed is desirable during the compaction operation. High speed is conducive to greater vibration and aids compaction. When inspecting compaction made by track treads, ensure that the operator makes complete tread coverage of the area to be compacted before making the second and subsequent passes.

During initial placing operations, relative in-place density tests (USBR 5525, 5530, and 7250)[10] and gradation analyses (USBR 5325 and 5330)[11] are recommended at a frequency of one test to represent each 750 m^3 (1,000 yd^3) placed. After placement procedures have proved satisfactory, and unless significant changes in gradation occur or doubtful conditions occur, one relative density test for every 7,500 m^3 (10,000 yd^3) of material placed will suffice. In the event of significant gradation changes in borrow materials, increased frequency of field tests may be needed to ensure satisfactory compaction of the variable materials.

3-22. Semipervious Fill.—Semipervious materials silty sands or gravels (SM or GM). Sands with dual symbol classifications such as SW-SM, SW-SC, SP-SM, or SP-SC [i.e., sands having as high as 12 percent passing the 75-µm (No. 200) sieve (5 percent is the usual upper limit for a material to be classified as pervious)] may have the characteristics of semipervious material even though such materials may be allowed by the specifications in embankment zones designated "pervious fill." Generally, compaction curves indicating adequately defined optimum water contents and maximum dry densities can be developed using standard compaction tests on impervious and semipervious materials.

Also, in some dams, outer embankment zones composed of coarse-grained sands and gravels, which contain appreciable amounts of fines (i.e., > 5 percent), are sometimes designated as "pervious zones." Compaction equipment, procedures, and control for such materials can be either those presented for pervious fill or those presented for impervious

[10] 5525: Determining the Minimum Index Unit Weight of Cohesionless Soils.

5530: Determining the Maximum Index Unit Weight of Cohesionless Soils.

7250: Determination of Percent Relative Density,

[11] 5325: Performing Gradation Analysis of Gravel Size Fraction of Soils.

5330: Performing Gradation Analysis of Fines and Sand Size Fraction of Soils, Including Hydrometer Analysis.

fill. The decision to use which control method depends on whether the soil behaves like a pervious material or more like an impervious material.

3-23. Rockfill.—Embankments with large rockfill zones are becoming more common. This is primarily because of:

- Necessity for using sites where rock foundation conditions are unsuitable for concrete dams
- Suitability of modern construction equipment to handle rock
- Increasingly higher dams being constructed
- Economic benefit that is obtained by maximum use of rock from required excavation.

Rock that does not easily break during handling, transportation, and compaction is considered sound and results in a pervious fill. Such sound rock is desirable for rockfill dams, or rockfill zones, especially if the source is from required excavations such as a spillway.

a. Sound Rock in Rockfill Dams.—

*1. Specifications.—*The specifications for pervious rockfill sections generally require that rock be sound, well graded, and free draining without specifying gradations; but, the maximum permissible size is specified together with lift thickness. Normally, rock fill is placed by dumping from trucks; bulldozers spread the material to the desired lift thickness. The required placement and spreading operations should avoid segregation of rock sizes. Type of roller, lift thickness, and number of passes are specified, preferably based on results of test fills.

*2. Placement Operations.—*When rock is dumped on the fill surface and pushed into place by a bulldozer, the fines are moved into the upper part of the lift, thereby creating a smoother working surface for the compacting equipment. If a layer of fines of significant thickness is produced that might seal off the upper part of the lift and prevent distribution of fines throughout the lift, the rock may be dumped directly in place.

All oversize rock must be removed before compaction. Usually, this is done with bulldozers fitted with special "rock rakes." Oversize rocks are often pushed into a specified zone in the outer slopes. Sometimes, excessively large rocks are hauled off to be dumped elsewhere or they are broken with a drop weight or explosives and used in the rockfill or riprap zone. Another common way of breaking oversize rock is with hydraulic chisels or spliters. Oversize rocks should never be allowed to accumulate along the contact slope of a closure section.

Close inspection is required to ensure that the material does not contain excessive fines. Excessive fines can cause excessive postconstruction settlements when the reservoir is filled. Specifying a limited amount of fines in the specifications is difficult; consequently, this is rarely done. However, the design office should provide guidance to field personnel based on results from rock-test fill studies which will aid in determining if excessive fines are present. The design and construction staff must be alert for material variations that could result in undesirable changes in gradation of the material brought to the embankment. If this occurs, the contractor should be informed so a change can be made in quarrying techniques.

*3. Compaction.—*Current practice is to use smooth steel-wheel vibratory rollers to compact all sound rockfill in comparatively thin lifts with minimal application of water. The lift thickness specified is dependent on size and type of rock and on type of compaction equipment used; usually, thickness is determined from results obtained during construction of a test fill. Less desirable rocks may break down in varying degrees upon excavation, handling, and compaction. These unsound rocks may degrade because of:

- Lack of induration
- Structure (such as thin bedding)
- Degree of fracturing
- Weathering
- Other physical and chemical properties

Some examples of material to be avoided are:

- Shales
- Mudstones
- Siltstones
- Claystones
- Chalk
- Earthy limestones
- Other sedimentary rocks which may be poorly cemented

Ordinarily, sound rocks that are quite weathered and very fractured may not be acceptable for rockfill zones because of their susceptibility to breakdown caused by weathering and fracturing.

Because of general unpredictability of breakdown of most types of rocks, use of test quarries, test fills, and laboratory tests for designing and constructing rockfill dams or zones is widespread. Generally, the lift thickness specified will be no thicker than 60 cm (24 in) unless test fills show that adequate compaction can be obtained using thicker lifts. The maximum particle size allowed in the lifts should not exceed 90 percent of the lift thickness.

Scarification of compacted lift surfaces is not necessary and should not be allowed because it disturbs the compacted mass.

b. Unsound Rock in Rockfill.—Formerly, use of soft or unsound rocks has been dictated by availability of large quantities from required excavations. The main concern about these materials is the tendency to weather and soften with time when exposed to air and water within the embankment. However, dams with large portions of embankments composed of unsound rocks attained adequate shear strength, and the rock experienced insignificant breakdown after placement when the rock was placed in random and semipervious zones. Where unsound rocks will constitute a significant structural portion of a fill, their properties and the best methods for compaction should be determined by means of a test embankment constructed during design studies.

Some rockfills composed of unsound rocks have been compacted by first rolling the loose lift with a heavy tamping roller equipped with long spike or chisel-type teeth ("shale breaker") and then compacting the lift with conventional tamping or rubber-tire rollers.

3-24. Riprap.—Riprap is the commonly specified material for slope protection. Properly graded riprap, placed to provide a well-integrated mass with minimum void spaces so that underlying bedding cannot be washed out, provides excellent slope protection. The primary factors that govern successful construction are:

- Loading from the quarry to provide a good mixture of different sizes within the required gradation in each load. Proper production from the quarry requires that blasting operations or processing produce proper rock sizes and that loading operations are properly inspected. Gradation testing is often specified. Grading of riprap and other large stone can be performed in accordance with ASTM procedure D-5519-94 [a] (see sec. 3-8b). Gradation tests are best performed at the placement site by sampling several trucks. Hauling vehicles should be weighed for determination of tonnage hauled.
- The rock should be placed on the slope to provide uniform distribution of different sizes without segregation to provide a rock mass without large voids. Placement should be accomplished by placing loads along the slope against previously placed riprap; this reduces size segregation which otherwise occurs if loads were dumped in separate piles. Dumping rock at the top of the slope into a chute should never be allowed because this results in segregation. If dumping is done from trucks it is usually necessary to winch load haulers down the slope to the placement location. Dumping should proceed along horizontal rows and progress up the slope; loads should not be dumped to form rows up the slope. If large rock is specified, a crane with an orange-peel or clam shell attachment operating from a platform built on the slope can be used. Recently, large backhoes have been used to place riprap; experienced equipment operators can do a good placement job. In using backhoes, riprap must be placed as the embankment is constructed because riprap placement must stay within about 5 m (15 ft) of the top of the fill. The process of end dumping without manipulation is generally not sufficient for good placement. Reworking with backhoes or other acceptable equipment will be required. Close visual inspection or actual measurement is required after dumping and spreading riprap to determine the adequacy of the distribution of different sizes. Reworking may be minimized by exercising care when loading to ensure that each truck load has the proper amount of each size rock (i.e., the proper gradation). Placement should result in a low void ratio, on the order of 0.35 or smaller. The void ratio over a placement area can be estimated by using mass tickets from each truck and the area of placement. If the volume of an area is considered a solid volume, then the void ratio is determined by proportion of the solid volume and the actual mass. For example, assume an area of 100 by 100 by 3 feet. Assume the density of riprap as 160 lb/ft^3 (or use the specific gravity of the rock). The solid volume is then 160x100x100x3 = 4,800,000 lb = 2,400 tons. If the mass tickets equal 1608 tons, the void ratio is (2,400-1,680)/2,400 = 0.33. This method of void ratio ensures a tight structure but does not ensure good distribution of grain sizes. Strict enforcement of specifications is required, especially during early stages of riprap placement to ensure a well-graded mass without large voids.
- A bedding layer (or layers) designed as a filter must be provided between the riprap and embankment material to protect the embankment material from erosion by wave action and to provide a stable base for riprap. Many riprap failures have occurred because bedding material was not large enough to preclude washing out through riprap interstices by

wave action. Removal of any bedding causes settlement and dislodgement of overlying riprap, which further exposes bedding and embankment material to direct wave action. Good practice is to place rock spalls or crushed stone of like size between the bedding and the riprap if they are available from quarry or required excavation. The term "spalls" refers to the finer materials resulting from rock excavation for materials such as riprap. Spalls must be durable fragments of rock, free of clay, silt, sand, or other debris. Spall gradation varies and must be specified for each particular job. End dumps with spreader boxes provide the best means of placing bedding and spalls.

3-25. Miscellaneous Fills.—Dam embankments on very plastic foundations may require toe berms to improve stability. Excavations for dam foundations or for appurtenant structures often produce material unsuitable for, or in excess of, requirements for the structural zones of a dam. Such excavated material can be used in toe berms of the dam. In localities where good quality riprap is expensive, fill materials from structural excavations have been used to flatten the upstream slope of the dam to permit use of lesser quality riprap and, in some cases, no riprap at all. In a few cases, excess low quality material from required excavation has been used in a noncritical zone—in the downstream portion of a dam—merely to replace material which otherwise would have had to be borrowed at greater expense.

Permeability of miscellaneous fills is not important in the design; they may not specifically require compaction. However, full use should be made of compaction obtainable by routing hauling and placing equipment on placed layers of the material. Sometimes, the nature of available materials or the design may require that some compactive effort other than routing of equipment be used. For example, sheepsfoot rolling has been used to break up fairly large pieces of soft rock to avoid excessive settlement. Compaction should be specified if the miscellaneous fill is to function as an impervious blanket or is within the limits of the embankment.

For miscellaneous fills, specifications should describe the materials to be used and state their sources. Thickness of layers will be specified, and the requirement for routing construction equipment over each layer will be stated. The specifications will also state if any moisture or special compaction requirements are needed.

Inspection of miscellaneous fills is entirely visual. Ordinarily, control tests are not made on such fills. A few fill density tests may be required for record purposes, and instruments may be placed to obtain performance characteristics. When inspecting miscellaneous fills, it is important to ensure that specified lift thickness is not exceeded; also make sure that hauling equipment never concentrates on a roadway but is routed over the entire breadth of the placed layer.

3-26. Instrument Installations.—The behavior of many embankment dams is monitored during construction and throughout operation by instrumentation placed in the embankment, in the foundation, and in or on appurtenant structures.

a. Instruments.—Data obtained from the instruments are used to assess performance of the structures by comparing the results to design assumptions and to normal performance expectations of embankment dams and their foundations. Besides visual inspections, performance monitoring with instrumentation is used to ensure safe, long-term performance of dams.

The various types of instrumentation installations can be grouped into five categories:

- Devices to observe pore-water pressures in the embankment and in the foundation
- Devices to observe earth pressures in the embankment
- Internal and external devices that provide data on deformations and displacements of the embankment and foundation
- Devices for monitoring seepage flows
- Vibration recording devices to indicate the result of transient stresses on the embankment

Type and quantity of instruments installed at a dam will vary with:

- Design
- Purpose
- Foundation conditions
- Embankment materials
- Embankment size

Detailed information concerning types of embankment dam instruments can be found in the *Embankment Dam Instrumentation Manual* [1].

b. Installation of Embankment Dam Instrumentation.—Work required for installing the instruments is usually performed by the contractor under provisions of the specifications for construction or modification of a dam. In special cases, however, Government forces may perform all or part of the work. The *Embankment Dam Instrumentation*

Manual provides detailed information concerning methods for installing embankment dam monitoring equipment.

c. Inspection.—Inspectors are required to inspect and test all the equipment, to oversee all installation work, and to obtain instrument readings during construction. Close inspection is essential to ensure correct installation of equipment and proper operation of the instrumentation.

d. Record Tests.—Record tests of embankment and foundation materials at internal instrument installations are necessary to obtain data on soils adjacent to the instruments. One field density test should be made at each unit or element of displacement or deformation monitoring device in earthfill material. Two test samples should be obtained from soil near the element or unit as well, one in the rolled embankment near the element and one in specially compacted material placed around the unit. When all or part of a dam is founded on highly compressible materials, representative undisturbed samples should be obtained in the foundation at the locations of the vertical movement and baseplate installations unless suitable samples have been obtained previously. Tests are not required when these devices are placed on a rock foundation unless specifically requested. Tests at the elements or units should be designated "record rolled," "record tamped," or "undisturbed," depending on the material being tested.

One field density test is required near each embankment earth pressure cell and piezometer tip. This test should be made in the rolled fill at the piezometer tip location prior to excavating the offset trench for the tip for closed-system piezometers. Each hole drilled for foundation piezometers should be logged throughout its length, and a record sample of the material should be obtained at the depths where piezometers will be located.

In addition to field density tests on embankment materials and in-place density tests on foundation materials, the following four tests should be performed on the samples taken:

1. Gradation analysis
2. Specific gravity
3. Liquid and plastic limits
4. Percolation-settlement

This laboratory testing may be performed during seasonal shutdowns when workload permits. Sufficient material must be obtained while making the density tests so that additional tests can be made. Before testing, foundation samples should be sealed to prevent moisture loss. Special care should be exercised to attain desired accuracy in all record tests. Embankment field moisture and density conditions must be duplicated for the percolation-settlement test and in-place conditions must be maintained for foundation materials. During and after construction, the dam designer may request that some record test samples or other special samples (core, blocks) be sent to special laboratories for testing and that the remaining samples that are not moisture sensitive be permanently stored at the damsite, in accordance with Reclamation guidelines. All stored samples should be properly marked and referenced to the pertinent density test.

Each container should be identified as follows:

- Name of dam.
- Identification symbols.
- For samples taken at piezometer tips, the letter P and the number of the tip (as shown on installation drawings) should be used (e.g., "Heron Dam, P-14") identifies the sample.
- For samples taken at porous-tube piezometers, the symbol PTP is used.
- For samples taken at crossarm units of a vertical movement installation, e.g., "Heron Dam, X-A-8-R" and "Heron Dam, X-A-8-T" represents, respectively, *record rolled* and *record tamped* material adjacent to the eighth crossarm unit installed in installation A for the dam.
- Likewise, as above, the symbols "Trinity Dam, H-B-6-1-R" and "Trinity Dam, H-B-6-1-T" represent samples taken from excavation and backfill of the trench for plates associated with a horizontal movement device.
- The symbol BP can be used for samples from foundation settlement (baseplate) installations.

These identifying symbols should be marked on the container in addition to identification normally required on record samples.

Record samples obtained from the location of foundation piezometers should contain representative materials taken from the location of the tip of the piezometer. Quantity of sample will depend on drilling equipment used to excavate the holes.

3-27. Records and Reports.—The inspector must make daily, periodic, and final reports covering construction activities work.

a. Daily Reports.—The daily reports should record the progress of construction, provide pertinent information for the inspector (who is beginning a shift) about shutdowns, construction difficulties, and tests in progress. These reports will also furnish data for use in compiling the construction progress report. The form of a daily report will vary to suit the requirements of each job, but all information in the periodic earthwork progress report should be based on day-to-day records. A consistent, systematic method is desirable for identifying field density tests made on the embankment. Each test should be designated by the date, shift, number on that shift, and purpose; e.g., "8-2-92-a-2-D" defines:

- The field density test made August 2, 1992
- The 1st workshift, the 2nd test made on that shift
- The purpose was to check an area of doubtful compaction

The above legend is as follows:

a = First shift
b = Second shift
c = Third shift
D = Doubtful area
C = Concentrated area
R = Representative

Results of tests made daily on the embankment are reported in the progress report of embankment construction on the appropriate forms.

b. Periodic Progress Reports.— Adequate technical control of construction is an important and essential function of field forces on an embankment dam project. Complete records of all data pertaining to methods and procedures of achieving satisfactory control must be maintained as a normal part of the inspection. *Summary of Earthwork Construction Data* reports are required of each project, for each month during the working season that embankment is being placed, except in special cases additional summaries or reports may be required. This report is a part of the Construction Progress Report (L-29). Copies of the *Summary of Earthwork Construction Data* report may be required for review by design team members. A *Summary of Earthwork Construction Data* report consists of:

1. Narrative and Photograph.—The narrative shall consist of a description of earthwork operations during the reporting period. Important features of normal embankment operations to be included in the narrative of initial reports are outlined below:

Borrow-pit operations (each borrow area):
- Description of material
- Equipment used
- Natural water content
- Method of adding moisture
- Depth of cut
- Stratification
- Mixing, separating
- Transporting

Embankment operations (each zone):
- Equipment used
- Spreading, mixing
- Method of adding moisture
- Maximum size of rock fragments
- Method of removing oversize rock fragments
- Compaction method
- Thickness of layers

To avoid repetition, many items reported in the above outline should be described only once. Thereafter, only significant changes in materials or in methods and procedures should be described. Any difficulty or unusual conditions encountered should be fully described. The narrative should clarify any results reported on the various forms that require special mention, and it should report pertinent information not covered by the forms.

Photographs should be taken far enough in advance to ensure no delay in submitting the reports and should include:

(a) A view from an abutment point showing the extent of operations on the dam during the period of the report. The camera position should be located sufficiently high so that location can be used until the dam is complete.
(b) An overall view showing the extent of operations in each major source of material.
(c) Closeup views of each operation such as dumping, spreading, adding moisture, disking, rolling, or mechanical tamping. Photographs of these operations need be submitted only once. Thereafter, only photographs showing different or unusual procedures should be submitted.
(d) Closeup views of typical borrow-pit cut banks. These should be submitted once.

2. Completion of form 7-1352.—This form should include results of field and laboratory tests made during the reporting period for all materials controlled by USBR 7240: Performing Rapid Method of Construction Control,[12] or by laboratory compaction tests (USBR

[12] 7240: Performing Rapid Method of Construction Control.

5500 or 5515).[13] The specific gravity of the minus 4.75-mm (-No. 4) material and the percolation-settlement tests need be performed and reported only for every 30,000 m^3 (30,000 yd^3) of material placed. The latter is a minimum requirement; variations in material may require more frequent tests.

3. Completion of form 7-1582.—This form should include results of field and laboratory tests made during the reporting period on all materials controlled by percent relative density.

4. Computer software programs are available which summarize results of in-place density, rapid construction control, and relative density tests, and that generate forms similar to 7-1352 and 7-1582. Reclamation field laboratories currently use the computer program PCEARTH for statistical compilation of earth construction data. This program provides for statistical compilation and reporting of compaction control data by rapid compaction and relative density test methods. The program also allows for compilation of physical property data from borrow investigation, foundation acceptance, and aggregate processing activities. Modules are also available for soil cement and soil cement slurry (flowable fill used in pipe embedment) test data. This program can be obtained for a small fee from the Earth Sciences laboratory at the Technical Service Center.

5. Roller Data.—Figure 3-28 shows a suggested tabulation of data for tamping rollers. Manufacturer's specification data sheets are also acceptable. Such data should be obtained for each roller used on the embankment to ensure compliance with specifications. This information should be reported only once for each roller, except when modifications are made to the roller.

c. Final Embankment Construction Reports.—Upon completion of embankment construction, a report should be prepared summarizing work accomplished. Final Construction Reports are required to be prepared in duplicate and submitted to the Denver Office for all major works, including dams. A detailed summary of embankment construction need not be transmitted separately but should be included in the Final Construction Report. A suggested outline for the embankment portion and related construction requirements of the report follow. Items in the outline not applicable to a particular project should be disregarded, and variations or additions should be made to suit individual job conditions.

1. General.—

(a) Location and purpose of structure.
(b) Description of dam and appurtenant works, including dimensions and quantities involved.

2. Investigations.—

(a) Foundation explorations. -
(1) Description of foundation conditions.
(2) Itemized summary of foundation explorations showing number, depth, and coordinate location of: drill holes, test pits, shafts, and tunnels.
(3) Description of any special tests and methods of taking undisturbed samples.
(b) Construction materials. -
(1) If a preconstruction earth materials report were prepared, a general description of materials investigation should be prepared and reference should be made to the previously prepared report. However, a complete and detailed description of additional investigations made subsequent to the preconstruction earth material report is required.
(2) If the preconstruction earth materials report was not prepared, a complete and detailed description of material investigations of borrow areas and rock sources with test results and quantities available shall be included in the Final Construction Report.

3. Construction history (include photos within text.—

(a) Required excavations, except borrow areas. -
(1) Stripping of foundation(s).
(2) Cutoff trench(s).
(3) Grout cap trench.
(4) Miscellaneous excavations.

The construction history shall include discussion on depth, quantities, dewatering methods, disposition of materials excavated, and type of equipment—including number used and rated capacity.

(b) Foundation drilling and grouting. -
(1) *General.*—Include discussion on type of equipment used and capacity. For the grouting plan, show location and depth of holes.
(2) *Pressure grouting.*—Include discussion of: grouting procedures, type of equipment and

[13] 5500: Performing Laboratory Compaction of Soils—5.5-lbm Rammer and 18-in Drop.
5515: Performing Laboratory Compaction of Soils Containing Gravel.

PROJECT ______ DAM DATE ______

EMBANKMENT ROLLER DATA

	Roller No.	1	2	3	4	5	6
(a)	Make of roller						
(b)	Number of drums						
(c)	Length of drums						
(d)	Diameter of drums (outside)						
(e)	Knobs (k) sheepsfoot (sf), or square (sq)						
(f)	Number of horizontal rows of feet						
(g)	Number of feet per row per drum						
(h)	Total number of feet per drum (f) x (g)						
(i)	Length of feet						
(j)	Dimensions of bottom of feet						
(k)	Area of bottom of feet						
(l)	Weight of roller (empty)						
(m)	Ballast capacity (all drums)						
(n)	Weight of roller as used						
(o)	Ballast used (material)						
(p)	Weight of roller ÷ total area all feet						
(q)	Cleaners (yes or no)						
(r)	Type of frame (rigid or oscillating)						

Figure 3-28.—Form for recording embankment roller data.

pressures used, grout take and results obtained, and any special problems and methods used to obtain satisfactory results.

(c) Borrow area operations. -

(1) *General*.—Discussion of plan and use of specified material sources and any problems in borrow areas not anticipated at the time specifications were issued.

(2) *Borrow areas*.—Detailed report of: initial conditions of borrow pits, methods of excavating, mixing, screening, and transporting of material. Include: type, size, and rated capacity of all equipment used—with special emphasis on any unusual requirements for blending or excavating procedures. Also, note: borrow pit yield, methods of controlling moisture, disposition of oversize material, and material from stripping operations.

(d) Embankment operations. -

(1) *Zones*.—Discussion to include: construction methods and sequence, material source, and any variation from the approved materials distribution plan and reasons therefore. For riprap and rockfill, record gradation.

(2) *Placing*.—Description of methods used including: scarifying, spreading and leveling, rolling, power tamping, removal or reworking, and any other part of the work considered peculiar to the placing operation.

(3) *Equipment*.—Discussion of type, capacity, size, number used, efficiency, and suggested improvements (if any) for all equipment used in placing embankment.

(4) *Testing*.—Performance of field laboratory, including number of testing personnel and number and type of tests performed should be summarized. Discussion of control tests, such as moisture content and density, and any special tests should include: procedures in making tests and reporting methods, extent of testing, adequacy of tests, and difficulties encountered.

4. Embankment Dam Instrumentation.—Brief description of systems installed such as piezometer tips, horizontal and vertical movement crossarms, and embankment and structure settlement measurement points.

(d) Embankment Dam Instrumentation Reports.—A monthly report, separate from other construction reports, is required during construction of the dam to present progress of placement of embankment dam instrumentation. This report should include:

- A short narrative on progress of work
- As-built information
  a. Reports of problems or abnormalities during installation work
  b. Readings made during the preceding period
  c. Representative photographs showing current details of the work

Within 6 months after completion of instrument installations, a Report on Embankment Dam Instrumentation is required and should include the following:

- Document materials and equipment actually used in instrumentation installations.
- Records of preinstallation testing of equipment.
- Drill logs for holes in which instruments were installed.
- Record test information.
- Problems or abnormalities that occurred during instrument installations.
- Deviations or modifications from specification requirements relative to instrument installations.
- Contractor claims that arose relative to instrumentation work.
- Photographs as available.
- Complete as-built information.
- Comments, criticisms, and suggestions so that future instrumentation design work can be improved.
- All instrumentation readings taken during construction.

Following completion of the dam, instrument readings shall be taken in accordance with the current Schedule for Periodic Readings (L-23) or with the reservoir filling criteria.

3-28. Control Criteria.—The concept of limiting the moisture condition for placement of compacted earthfill in dam embankments has been used by Reclamation for many years. The idea of limiting the magnitude of pore-pressure buildup in cohesive soils during construction, by controlling the placement moisture, has been a major consideration since 1946. The relationships among moisture control, pore-water pressure potential, and stability of the structure are well established and are generally recognized as a basis for an upper placement moisture limit.

Localized shear planes may develop under certain conditions of placement moisture and compactive effort. Using Reclamation procedures for compaction of earthfill, these shear planes occur when the soil is at optimum water content or wetter when construction pore pressures normally increase rapidly with increased moisture content. A typical example of laboratory tests on one soil is shown on figure 3-29. The net effect of development of localized shear planes would be a significant loss of shear strength; hence, placement moisture should be so controlled as to avoid any possibility in developing local shear planes.

The phenomenon of loss of effective strength of dry, low-density soil as the water content is increased is also well known. This reduction in strength may result in appreciable volume change of a loaded soil, either in the foundation or in the compacted embankment. In the case of embankment dams, wetting the foundation or the embankment—following construction—does not occur uniformly. Hence, such volume changes usually will result in differential settlement and may result in cracking of the embankment. Many failures of small embankment dams are attributed to cracking due to differential settlement. Placement moisture should be maintained in a range between the specified upper and lower moisture limits so that the soil will not exhibit additional consolidation upon saturation, will not develop excessive pore pressures, and will not develop shear planes under field compactive effort. Generally, this range is from about 2 percentage points dry to 1 percentage point wet of optimum moisture content.

High shear strength, low consolidation, and permeability are all related to dry density obtained in a fill. Design is usually based on impervious or core material being placed at or slightly below average laboratory maximum dry density of the minus 4.75 mm (-No. 4) fraction. An average *D*-value (ratio of dry density of fill to laboratory maximum dry density, expressed as a percentage) near 100 percent is expected for most soils containing less than about 30 percent plus 4.75 mm material. For gravelly materials containing more than about 30 percent plus 4.75 mm material, an adjusted field density of the control fraction less than laboratory maximum dry density of the minus 4.75 mm fraction may be expected during construction and is considered in design.

Overall quality of the embankment being placed can be evaluated by statistical analysis of test results. A simple frequency distribution analysis of the variables w_o-w_f and *D*-value, as outlined on table 3-2, can be maintained throughout construction to aid engineers and inspectors in their efforts to control placement within recommended or specified limits. Figures 3-30 and 3-31 show worksheets designed for such an analysis. Forms of this type, on which data from each test are entered as results become available, should be maintained in the laboratory or in some other suitable place where they

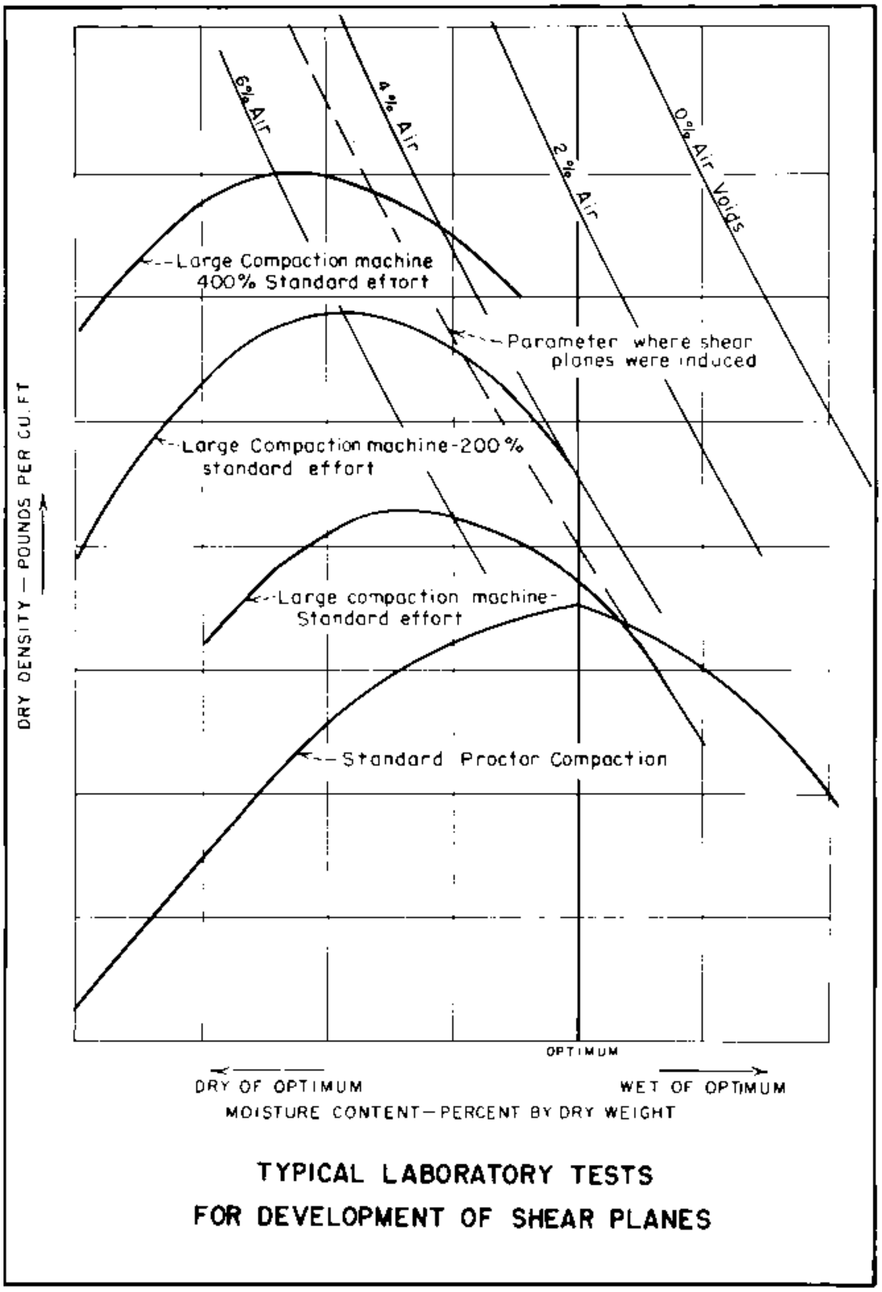


Figure 3-29.—Typical laboratory tests for the development of shear planes.

will be readily available for inspection. By means of this analysis, the frequency distribution of test results is obtained, and statistical values such as the mean, the standard deviation, and the percentage of tests falling outside specified limits can be determined. Computer software programs are available which calculate statistical information on test data such as average *D*-value, average w_o-w_f, percent of accepted tests, and several other parameters. Reclamation's computer program PCEARTH can readily perform such statistical compilations and generate distribution graphs as shown on figures 3-30 and 3-31.

Various other criteria for quality control have been proposed. Table 3-2 lists suggested limits of density and moisture control based on Reclamation's experience with earthfill dams. In the absence of instructions to the contrary, criteria given in this table may be used.

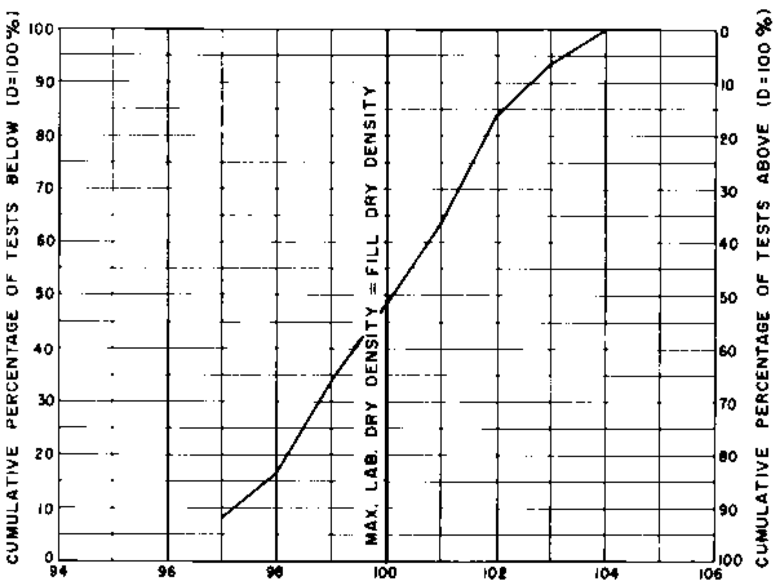
Example DAM ZONE 1

$D = \dfrac{\text{FILL DRY DENSITY}}{\text{MAXIMUM LABORATORY DRY DENSITY}} \times 100$ (MINUS NO. 4 MATERIAL)

D	F (PREV)	THIS PERIOD: FREQUENCY OF OCCURRENCE	THIS PERIOD: F	THIS PERIOD: CUM. F	THIS PERIOD: CUM. %	TO DATE: F	TO DATE: CUM. F	TO DATE: CUM. %
93.0 - 93.9								
94.0 - 94.9								
95.0 - 95.9								
96.0 - 96.9		IIII	4	4	8			
97.0 - 97.9		IIII	4	8	16			
98.0 - 98.9		卌 IIII	9	17	34			
99.0 - 99.9		卌 II	7	24	48			
100.0 - 100.9		卌 III	8	32	64			
101.0 - 101.9		卌 卌	10	42	84			
102.0 - 102.9		卌	5	47	94			
103.0 - 103.9		III	3	50	100			
104.0 - 104.9								
105.0 - 105.9								
TOTALS			50					

	PREV.	THIS PERIOD	TO DATE
Average max. lab. γ_D (PCF)		117.1	
Average fill γ_D (PCF)		117.1	
Mean variation from max. lab. γ_D (PCF)		0.	
Average rock content (% of plus No. 4 by dry weight)		2.6	

PERIOD OF REPORT ______ TO ______
TESTS 9-26-70-0-1R TO 10-24-70-0-1R



$$D \text{ (IN PERCENT)} = \frac{\text{FILL DRY DENSITY}}{\text{MAXIMUM LABORATORY DRY DENSITY}} \times 100$$

Figure 3-30.—Statistical analysis of test results for density control of fill.

Example DAM ZONE 1

OPTIMUM WATER CONTENT MINUS FILL WATER CONTENT ($w_o - w_f$) IN PERCENT OF DRY WEIGHT

		F (PREV.)	THIS PERIOD: FREQUENCY OF OCCURRENCE	THIS PERIOD: F	THIS PERIOD: CUM. F	THIS PERIOD: CUM. %	TO DATE: F	TO DATE: CUM. F	TO DATE: CUM. %
w_f IS BELOW OPTIMUM	3.3-3.7								
	2.8-3.2								
	2.3-2.7		卌 I	6	6	12			
	1.8-2.2		卌	5	11	22			
	1.3-1.7		卌 I	6	17	34			
	0.8-1.2		卌 卌 I	11	28	56			
	0.3-0.7		卌 卌	10	38	76			
+0.2 TO -0.2			卌 II	7	45	90			
w_f IS ABOVE OPTIMUM	0.3-0.7		III	3	48	96			
	0.8-1.2		I	1	49	98			
	1.3-1.7								
	1.8-2.2								
	2.3-2.7		I	1	50	100			
	2.8-3.2								
	3.3-3.7								
TOTALS				50					

	PREV.	THIS PERIOD	TO DATE
Average optimum water content		13.7	
Average fill water content		12.8	
Mean variation from optimum water content		0.9	

PERIOD OF REPORT ________ TO ________
TESTS 9-26-70-a-1R TO 10-24-70-a-1R

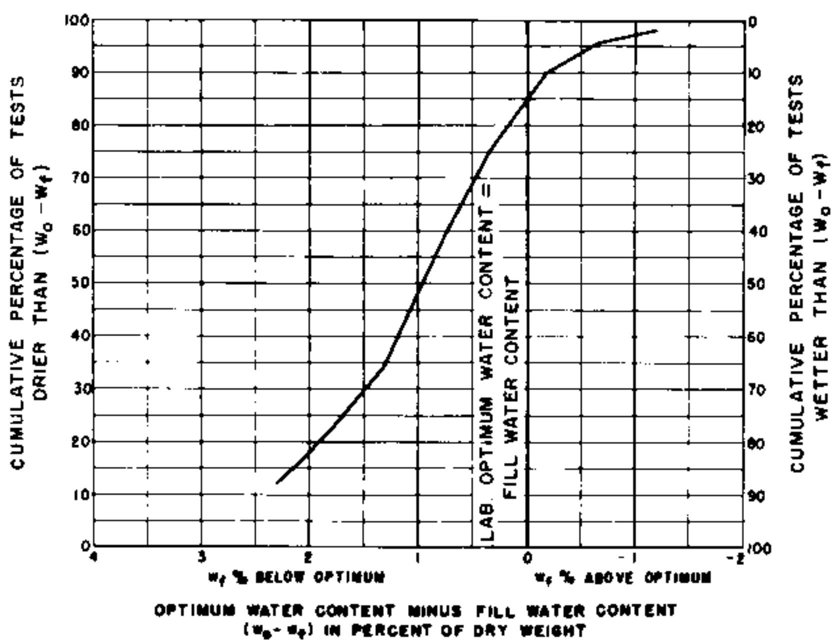


Figure 3-31.—Statistical analysis of test results for moisture control of fill.

Table 3-2.—Criteria for control of compacted dam embankments

		Percentages based on minus 4.75 mm (-No.4) fraction					
		15 m (50 ft) or less in height			15 m (50 ft) or greater in height		
Type of material	Percentage of plus 4.75 mm (+No.4) fraction by dry mass of total material	Minimum acceptable density	Desired average density	Moisture limits, w_o-w_f	Minimum acceptable density	Desired average density	Moisture limits, w_o-w_f
Cohesive soil: Soils controlled by the laboratory compaction test	0 to 25 percent	D = 95	D = 98	-2 to +2	D = 98	D = 100	2 to 0
	26 to 50 percent	D = 92.5	D = 95	-2 to +2	D = 95	D = 98	Note[2]
	More than 50 percent[1]	D = 90	D = 93	-2 to +2	D = 93	D = 95	
Cohesionless soil: Soils controlled by the relative density test	Fine sands with 0 to 25%	D_d = 75	D_d = 90	Soils should be very wet	D_d = 75	D_d = 90	Soils should be very wet
	Medium sands with 0 to 25%	D_d = 70	D_d = 85		D_d = 70	D_d = 85	
	Coarse sands and gravels with 0 to 100%	D_d = 65	D_d = 80		D_d = 65	D_d = 80	

The difference between optimum water content and fill water content of dry mass of soil is w_o-w_f , in percent.
D is fill dry density divided by laboratory maximum dry density, in percent.
D_d is relative density as defined in chapter I.

[1] Cohesive soils containing more than 50 percent gravel sizes should be tested for permeability of the total material if used as a water barrier.
[2] For high embankment dams, special instructions on placement moisture limits will ordinarily be prepared.

E. Canals

3-29. Design Features.—Canals are constructed channels of widely varying capacities used to convey water for irrigation, power, domestic uses, or drainage. Distributory channels conveying water from the main canal to farm units are called laterals and sublaterals. The design capacity of an irrigation canal or lateral is based on maximum daily consumptive use rate downstream of the section under consideration, including allowance for seepage losses. Water loss from seepage is a major problem in canal design. On a typical irrigation project, about 20 to 40 percent of water diverted into the canals and laterals can be expected to be lost by seepage unless lining is provided. Conversely, water seeping into a canal can (under favorable conditions) be used to augment the water supply. Also, inflow seepage is the means by which subsurface drainage is accomplished for open-ditch drainage systems.

Canal construction involves a variety of earthwork problems. Because of the great longitudinal extent of the work, many different kinds of earth foundation and material conditions are encountered, all of which must be treated differently. Thus, some reaches of a canal may require only uncompacted embankments made of unselected materials excavated from the prism, while other sections may require careful control of cuts in borrow areas, strict moisture control, and compaction of earth in the form of linings or entire embankments. The following paragraphs summarize major design features related to earthwork in canal construction.

Canals are classified as unlined or lined, depending on the extent to which special provisions are made to prevent seepage. An unlined canal or lateral is an open channel excavated and shaped to the required cross section in natural earth without special treatment of the subgrade. Construction of compacted embankments at locations where the canal water surface is above natural ground is considered a normal seepage control measure in unlined canal construction.

The cross section of a canal is selected to satisfy the most desirable criteria for bottom width, water depth, side slopes, freeboard, and bank dimensions, and to facilitate operation and maintenance. The ratio of bottom width to depth ranges from 2:1 for the smallest laterals to 8:1 for large canals; side slopes on unlined canals range from 1.5h:1v to 2h:1v, or flatter where unstable soils are encountered. Design of canals is based on development of a steady-state seepage condition from the normal high water surface, followed by rapid drawdown of the water surface. Dimensions of the banks and the soils of which they are composed must ensure stability under these extreme hydraulic conditions.

The water section of a canal may be entirely in cut, entirely in fill, or partially in each, depending on economics of the route, necessary gradient, structure design, and requirements of safety and water distribution. If the water section is partially or entirely in fill, compacted embankments may be used to prevent excessive amounts of seepage.

Requirements for compacting an embankment under compacted earth lining depend upon (1) height of fill, (2) earth construction materials used, and (3) method of fill construction. These requirements are reflected in the specifications.

Operation and maintenance roads are usually provided on the lower bank of large canals. In some cases a maintenance berm may be needed on the upper bank. In deep cuts, berms are often used to reduce bank loads, to prevent sloughing of earth into the canal, and to lower the elevation of the operating road for easier maintenance. Berms also provided between deep cuts and adjacent waste banks. Even where operating roads are not planned, canal and lateral banks are smoothed by blading to allow power mowers and other power equipment to control vegetation.

A lined canal or lateral is one in which the wetted perimeter of the section is lined or specially treated. Materials used to line canals include:

- Unreinforced or reinforced concrete
- Pneumatically placed mortar
- Buried geomembranes
- Compacted earth
- Earth treated with additives (stabilized soils)

Various types of asphalt, brick, stone, and other materials have been used in the past.

The great need to reduce water losses in canals means a continuing search for effective low-cost linings. In addition to water savings achieved by lining canals, other advantages include:

- Prevention of water logging adjacent land
- Lower operation and maintenance costs
- Less danger of canal failures
- Reduced storage with less diversion requirements

In the case of hard-surface linings, further advantages include:

- Possible smaller canal sections and structures
- Less right-of-way
- Higher permissible velocities
- Steeper gradients

Side slopes on earth-lined canals are 2h:1v or flatter, and on hard-surface linings usually 1.5h:1v.

Lined canals can be built in either cut or fill or partially in each; the economic advantages of balanced cut and fill must be weighed against special treatment of fills to support the lining properly in a canal. The lining above the water surface varies with height and depend upon:

- Flow capacity
- Water velocity
- Alignment curvature
- Inflow of storm water

In addition, height increases with water surface elevation caused by:

- Canal checks
- Checks in pumping plant forebays
- Surge waves
- Wind action

Various types of earth linings have been used by Reclamation to reduce water loss [16]. The simplest method is silting (sediment sealing), in which fine-grained soils are dispersed into the water by hydraulic or mechanical means and are deposited over the wetted perimeter of the canal. Although it is effective in reducing seepage, silting is not considered a permanent seal because the silt layer is easily destroyed. Sometimes, thin, loose earth blankets of fine-grained soils have been used as linings. Such linings are inexpensive and are temporarily quite effective, but they erode easily unless protected by a cover layer of gravel. Bentonite applied as a buried membrane, or mixed with permeable-type soils, and buried geosynthetic or asphalt membranes have been used; but, such linings must be protected by gravel or soil cover. When bentonite is used for the membrane, a high-swell bentonite (sodium montmorillonite) is required. Acceptance criteria for this material are contained in the specifications.

Compacted earth linings 150 or 300 mm (6 or 12 in) thick usually satisfy design requirements from a seepage standpoint; but, they require gravel cover or protection against erosion and are relatively expensive because of difficulty in compacting them on side slopes [17]. Of the earth-type linings, heavy or thick compacted earth linings have proved most desirable because:

- Conventional construction equipment and methods can be used.
- Wider variety of soils may be used without erosion protection.
- Lining is not easily destroyed by cleaning operations.
- Excellent seepage control can be achieved [16].

Heavy compacted earth lining is placed in the canal bottom in 150-mm (6-in) compacted layers to a depth of 300 to 600 mm (1 to 2 ft) and on the canal slopes to a horizontal width of 0.9 to 2.4 m (3 to 8 ft). In areas of potentially serious frost action, a minimum lining thickness of 600 mm is recommended. Sheepsfoot rollers are normally used for compaction. When a thick compacted lining is placed on a 2h:1v slope about 750 mm (2 1/2 ft) of fully compacted soil—normal to the 2:1 slope—is obtainable for an 2.4 m (8 ft) horizontal width. In some instances, layers of compacted fill on the side slopes are sloped downward toward the canal to provide additional width needed for wider rollers. The slope of the layers cannot be so steep as to cause movement or segregation of the earth lining materials during placing or compacting. Figure 3-32 shows a typical canal section with compacted earth lining. Figure 3-33 shows a thick compacted earth lining under construction, and the completed canal is shown on figure 3-34.

A wide range of impervious soils has been used successfully for compacted earth linings. The most desirable soils combine the most favorable properties of gradation, plasticity, and impermeability.

- Best results are obtained from a WELL-GRADED GRAVEL WITH SAND AND CLAY (GW-GC). Good stability and erosion resistance characteristics are provided by the well-graded, sandy-gravel fraction as well as by the cohesive binder. The fines content should be high enough to reduce seepage (12-20 percent).
- CLAYEY GRAVELS (GC) are next in quality.
- WELL-GRADED SAND WITH CLAY (SW-SC) follows the above.
- CLAYEY SANDS (SC) is next in quality.
- SILTY GRAVELS (GM) have been used with success.

The best materials contain just enough fines to make the soil impervious and have sufficient gravel for erosion resistance. Of the fine-grained soils, a LEAN CLAY (CL) is desirable due to its imperability and workability.

A good cohesive binder usually means the soil must have significant plasticity as measured by the plasticity index, PI. However, some low PI (7 to 12 percent) earth linings have performed reasonably well [18]. Additional side slope protection (gravel or riprap) is always required on the outside bank of curves and downstream of in-line structures and sometimes on straight reaches.

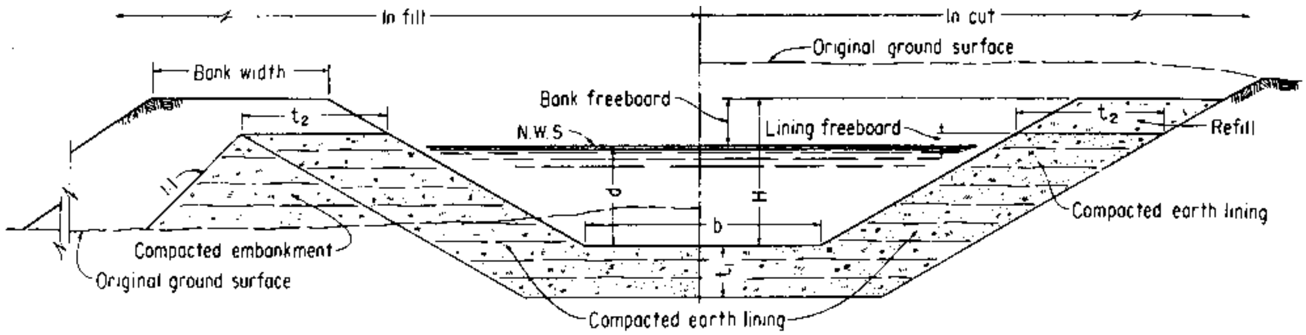
The gravel cover layer sometimes used for erosion protection of earth linings may vary from 150 to 300 mm (6 to 12 in) thick. Pit-run sand and gravel up to 75 mm (3 in) in size and containing a small amount of fines has been found satisfactory. In some instances, placing a 150-mm-thick sand and gravel filter layer on an open foundation structure has been necessary to ensure that loss of earth lining material would not occur. A WELL-GRADED GRAVEL WITH SAND (GW) with a maximum size of 75 mm is used [17].

Usually, riprap or coarse gravel is required to protect wetted earth surfaces in the vicinity of structures where high water turbulence may occur. The material will vary in size and thickness—depending upon degree of protection required—but should be graded from the smallest to the largest particle size.

The type of material considered suitable for covering buried membranes depends on grade and velocity in the canal and on steepness of side slopes. Cover materials must be stable soils when saturated. Side slopes are normally 2h:1v or flatter. In some cases, earth cover (sandy, silty, or clayey soils) is adequate; in other cases, where erosive forces are high, gravelly soil or gravel cover is required. Sometimes, earth cover is placed directly on the membrane, and a gravel cover is placed over the earth [19]. The total thickness of cover is usually from 150 to 300 mm (6 to 12 in). The canal subgrade must be relatively smooth—without abrupt breaks or protruding materials—and open cobble structure must be filled with a suitable finer filter material. Cover materials must be carefully placed so the membrane is not punctured or does not slip. Shear strength in the form of friction between the different layers of material must be great enough to prevent sliding of one material on the other. Laboratory direct shear tests can be used to determine the interface friction between different membrane and earth cover materials. Installation of a buried geosynthetic membrane lining is shown on figures 3-35 and 3-36.

3-30. Specifications Provisions.—

a. General.—The following paragraphs summarize various standard specifications provisions used by Reclamation for earthwork in canal construction. Their



TYPICAL EARTH LINED SECTION

TABLE FOR EARTH LINING

d	t_1	t_2
2.0' or less	1.0'	3.0'
>2.0' to 4.0'	1.5'	4.0'
>4.0' to 6.0'	2.0'	6.0'
Over 6.0'	2.0'	8.0'

NOTE

If lining material requires a protective cover of gravel or riprap to prevent scour or erosion, excavation shall be extended to provide for the designated thickness of lining plus the gravel or riprap cover.

Figure 3-32.—Typical canal section and compacted earth lining.

purpose is to clarify and to relate the different types of work. However, this discussion should not be considered a substitute for the published specifications for any particular job. Each set of specifications includes only those standard provisions that apply to the local situation; in addition, modifications are made to fit each job.

b. Subgrades and Foundations for Embankments and Compacted Earth Lining.—Provisions for canal excavation require the contractor excavate to sections shown on the drawings. If, because of undesirable bottom or slope conditions, changes can be ordered for removal of unsuitable material and for refilling as necessary.

Provisions for treatment of embankment foundations cover required items of work like stripping, scarifying, and compacting the foundation surface. Stripping includes removing all material unsuitable for embankment foundation. Necessary clearing of right-of-way, grubbing of stumps and other vegetable matter and their roots, and burning or other disposal of the material are required of the contractor.

As required, excavation for canals will allow slopes of excavations and embankments for canals to be varied during construction. Access ramps must not be cut into canal slopes below the proposed water level in unlined canals or below the top of the lining in lined canals. Above the canal section, rock excavation need not be finished and will be allowed to stand at its steepest safe angle. In unlined sections of canal excavated in rock, sharp points of rock extending not more than 150 mm (6 in) into the water prism are sometimes permitted. Earth slopes are to be finished neatly to lines and grades, either by cutting or by constructing earth embankments.

Surfaces under all canal embankments, except for rock surfaces, are required to be scored with a plow to make furrows not more than 900 mm (3 ft) apart and not less than 200 mm (8 in) deep. For compacted embankments, the entire foundation surface is scarified or disked to a depth of not less than 150 mm (6 in) in lieu of scoring. Foundations for embankments are stripped of unsuitable material.

c. Earth Embankments and Linings.—In specifications for construction of earth embankments and linings, general provisions are made to:

- Adhere to the dimensions shown on the drawings
- Provide allowance for settlement
- Grade the tops of canal banks
- Distribute gravel, cobbles, and boulders uniformly throughout the fill

Figure 3-33.—Placing and compacting thick earth lining in Welton-Mohawk Canal in Arizona.

- Prohibit use of frozen materials and frozen working surfaces for canal embankments and built-up canal bottoms

Provisions are made for limiting the amount of backfill or compacted backfill that can be placed about structures that are built prior to excavation of the canal and construction of canal embankment. Using all suitable materials from excavations for construction of fills is required. Excess material from excavation is used to strengthen canal embankments or is wasted. Material, from cut not needed to strengthen adjacent embankment and excavated material in excess of requirements and unsuitable material, is deposited in waste banks on Government right-of-way. Stockpiling of suitable materials for use in earth linings or earth cover for geomembrane linings may be required.

Where canal excavation at any section does not furnish sufficient material for required fills, the contractor is required to borrow materials from areas designated by the Government. Adequate berms are left between embankment toes and adjacent borrow pits with 1-1/2h:1v slopes in the pits unless otherwise specified. Borrow pits may require drainage by open ditches.

Specifications for earth canal embankments and linings require that construction operations result in an acceptable and uniform gradation of materials to provide for low permeability and high stability when compacted, and that all rock larger than maximum size specified [usually 75 or 125 mm (3 or 5 in)] be removed before compaction. These requirements are particularly important in lining construction because permeability is controlled by a relatively thin layer of compacted soil. When blending fine-grained and coarse-grained soils (from different sources) is required to secure the lining material desired, specifications usually require that the fine soil be placed first, followed by the coarse soil, after which the two are blended. This procedure is used so that


Figure 3-34.—Completed thick earth lining in Welton-Mohawk Canal in Arizona.

coarse lenses will not be left at the bottom of any finished layer. Another requirement is that blending be accomplished by a mixing machine built for the purpose of blending soils or by equipment capable to produce comparable results and specifically approved for the work. Weight of mixing equipment is important because light-weight equipment rides upward and is incapable of accomplishing proper blending.

An item covering excavation and refill of inspection trenches is often provided in the specifications. This provision allows proper inspection of materials, and blending and compaction operations can be made from time to time, as required, to ensure an acceptable lining product.

Sometimes, provisions are made for compacting clayey and silty subgrade soils of low permeability for the base of a compacted earth lining in the canal bottom. This base is considered equivalent to about one 150-mm (6-in) compacted layer in the canal bottom.

Construction of fills for canals in the forms of earth embankments, linings, backfill, and refill is covered by a number of specifications paragraphs. Table 3-3 summarizes pertinent provisions and shows relationships among the different types of fills.

d. Riprap, Protective Blankets, Gravel Fills, and Gravel Subbase.—Riprap or a coarse gravel cover may be required:

- On canal side slopes
- At in-line structures
- On the outside banks of curves
- In places where wind-generated waves may cause erosion
- In areas where water velocity is otherwise high

Figure 3-35.—Geomembrane being placed over prepared subgrade.

Figure 3-36.—Placement of a protective earth cover over a geomembrane.

Table 3-3.—Materials and placing requirements for various fills used in canals

Type of construction and where used	Material	Placement requirements
Embankment not to be compacted. For lined and unlined canals.	Not specified	Approximate horizontal layers. If built by excavating and hauling equipment, travel is routed over the embankment. If built by excavating machinery, thickness of layer is limited to depth of material as deposited by excavating machine. Water may be required, either added at site of excavation or by sprinkling of banks. Working may be required to secure uniformity of moisture and materials.
Embankment to be compacted. For lined or unlined canals.	Suitable material from required excavation or borrow; remove plus 125-mm (+5-in) material; gradation must be acceptable.	Prepare foundation; if clayey or silty soils, 150 mm (6 in) compacted thickness at uniform optimum moisture; tamping rollers, 95-percent compaction. If cohesionless and free-draining cohesionless and free-draining soils, horizontal layers 150 mm (6 in) compacted thickness if rollers or tampers are used; 300 mm (12 in) compacted thickness if tractor treads or other heavy vibrating equipment is used; thorough wetting; 70-percent relative density.
Loose earth lining. For lined canals.	Impervious soil from canal excavation or borrow; uniform mixture required; depth of cut in borrow areas to be designated; uniform cutting of the face of the excavation; no plus 75-mm (+3-in) rock; may require blending.	Not specified.
Compacted earth lining. For lined canals.	Impervious soils from canal excavation or borrow; uniform mixture required; depths of cut in borrow areas to be designated; uniform cutting of the face of the excavation; no plus 75-mm (+3-in) rock; may require blending.	Prepare foundation; if clayey or silty soil, 150 mm (6 in) compacted thickness at uniform optimum moisture; tamping rollers, 95-percent compaction.
Refill above lining. For lined canals.	Selected from excavation or borrow.	Not specified.
Loose backfill. About structures.	Approved material from required excavation or borrow.	Amount can be limited by the contracting officer; method of placement not specified, but subject to approval.
Compacted backfill. About structures.	Selected clayey and silty material; from required excavation or borrow; no plus 75-mm (+3-in) rock.	Horizontal layers; compacted thickness at uniform optimum moisture shall not exceed 150 mm (6 in) or two-thirds the length of roller tamping feet, whichever is the lesser; compacted thickness shall not exceed 160 mm (4 in) where compaction is performed by hand or power tampers; 95-percent compaction.
	Selected cohesionless free-draining material; from required excavation or borrow; no plus 75-mm (+3-in) rock.	Horizontal layers 150 mm (6 in) compacted thickness if rollers or tampers are used; 300 mm compacted thickness if tractor treads or surface vibrators are used; and not more than the penetrating depth of the vibrator if internal vibrators are used; wetting by hoses, flooding, or jetting is required; 70-percent relative density.

Riprap should be hard, durable, sound rock that is predominantly angular and equidimensional. Reclamation determines the durability of riprap by petrographic examination and freeze-thaw durability testing [20]. If a source of unknown quality is being considered, representative samples of the material should be sent to the Denver laboratories for testing. A guide for evaluating potential riprap sources is presented in ASTM D 4992 [21]. Properly graded riprap will provide long-term, stable protection. Gradation of rock particles should be well distributed between the maximum and minimum sizes permitted so placement will result in a firm mass with minimum void spaces. (See sec. 3-24.)

A bedding layer of sand and gravel is normally required between riprap and the embankment or lining. If the bedding is designed as a filter, it will provide resistance to erosion from runoff or wave action while serving as a stable base for the riprap ([14], chs. 5 and 7). A geotextile may be used as a filter, but a layer of sand and gravel is still required between the riprap and the geotextile to protect the geotextile.

For some canals, a coarse granular cover can be used as a beach belt or as a complete cover for an earth lined canal. This gravel protection not only protects the banks against erosion and wave action, but also reduces maintenance for weed control and improves the appearance of the canal. Coarse gravel cover for a canal lining should:

- Have a maximum particle size between 75 and 150 mm (3 and 6 in)
- Have between 5 and 50 percent of particles passing a 4.75-mm (No. 4) sieve,
- Contain 10 percent or less fines passing a 75-mm (No. 200) sieve [22].

Gravel fills or gravel subbases can be pit-run gravel obtained by the contractor from approved borrow sources on Government right-of-way or from sources provided by the contractor. The material should be less than 75 mm (3 in) in size, reasonably well-graded, and contain a minimum amount of fines. Sometimes, gradation limits are specified so there is a reasonably good distribution of particle sizes from the coarsest to the finest without a major deficiency of any size or group of sizes. Uncompacted gravel fill is placed in uniform layers to the required thickness. If compaction is specified, it is accomplished according to the provision for compacting cohesionless free-draining materials.

Provisions for one-course gravel surfacing (of prescribed thickness), for operating roads and parking areas, require material consisting of sand and gravel from which all rock having a maximum dimension of more than 37.5 mm (1-1/2 in) have been removed. Material from gravel sources is selected to include a sufficient quantity of natural cementitious or binding material such that the surfacing will bond readily under the action of traffic. The desirable gradation conforms to ASTM D 1241 for Type I, Gradation C, surface-course material that satisfies the requirements of note 4 in D 1241 [23]. This gradation produces a good distribution of all particle sizes with 8 to 15 percent fines to provide imperviousness and binding material. These fines are recommended to be in a limited range of plasticity such that the PI is from 4 to 9, and the liquid limit, LL, is less than 35. Surfaces to receive the gravel surfacing should be bladed or dragged to secure a uniform subgrade. Surfacing material is placed and spread uniformly on the prepared subgrade to required depth and dimensions and moistened if required. Geosynthetic fabrics may be used between the subgrade and surfacing material to improve performance of the surfacing material. Usually, minimum compaction is required such as routing hauling and construction equipment over the entire width of surfacing to distribute the compacting effect of equipment to the best advantage. However, where imperviousness and wearing surface are important, the higher degree of compaction usually specified for roller compaction is required.

3-31. Control Techniques.—

a. General.—Recognition of the importance of controlling earth placement in embankments and canal linings has increased concurrently with knowledge of factors affecting stability of such structures. Not only must the inspector be thoroughly familiar with provisions of the specifications, but one must have a good understanding of design and construction principles involved and of field and laboratory tests used for control. The inspector should:

- Obtain all available information on design assumptions
- Obtain letters of instruction, laboratory reports, other reports, and specifications
- Check whether field conditions revealed are compatible with assumptions made on the basis of preconstruction investigations

The principles are equally applicable to foundations, whether under earth or concrete structures.

b. Subgrades and Embankment Foundations.—The subgrade and embankment foundation for a canal may consist of rock or soil. Canal sections excavated in rock should be inspected to determine whether open structure or fissures exist that will cause excessive seepage or piping which would require canal to be lined. Rock foundation surfaces on which compacted earth is to be placed should be moistened before placing the first layer of earth, but standing water should not be allowed.

When canal sections are excavated in soils, or embankments are constructed on soil foundations, greater care is required for inspection than for rock subgrades and foundations because of the nature of the material. In addition to noting possible seepage or piping conditions, the inspector must identify and locate conditions that could lead to large settlement and slope instability. During investigations for design of a canal, it is not economically possible to predetermine all conditions which may be encountered. Therefore, during excavation or stripping operations, the inspector should call to the attention of supervisors the necessity for:

- Additional exploratory holes
- Field penetration tests
- Permeability or density tests
- Other sampling and tests

to determine the extent of any materials of doubtful suitability for the foundation of embankments, for cut slopes, or for canal subgrades.

Some canal specifications require that reaches of canal to be lined shall be determined immediately in advance of construction by exploratory excavations, such as bulldozer or backhoe trenches dug by the contractor. In such cases, the inspector must be familiar with the permeability characteristics of soils encountered, making use of in-place permeability tests as required.

The inspector should examine canal subgrade soils on canal slopes and bottom with respect to possible future seepage and erosion. If high seepage losses are anticipated or if soils may be anticipated, to erode badly under proposed operating conditions, the inspector should call these facts to the attention of supervisors. If, upon further consideration, these conditions appear to be critical, advice from the designers should be sought. Lined sections in areas of high ground water must be protected against uplift by using drainage systems, and these are normally specified. If such areas previously unknown are observed during construction, provisions should be made for installing pressure relief devices. Similarly, when high ground water is encountered near silty or fine sandy subgrade soils of concrete-lined sections in freezing climates, attention should be directed to this fact so frost-action preventive measures can be instituted, if necessary.

If fine-grained soils are to be placed as a sublining under concrete or membrane linings, or will be used for earth linings, the natural subgrade should be inspected for open voids as in cobbly soils or fissure formations into which the fine soil might erode. In such cases, the inspector should call the conditions to the attention of supervisors so filter blankets or geosynthetic filters can be installed.

The inspector should make certain that all organic matter and any soils that may become unstable upon saturation, such as highly organic soils, loose silts, fine sands, and expansive clays, are removed or properly treated for embankments and canal linings to the extent necessary to provide a safe, stable foundation or subgrade under operating conditions. Specifications requirements for stripping and scoring earth foundations for all canal embankments, and special treatment for preparation of foundation surfaces for compacted embankments and earth linings, are important details that require careful visual inspection during construction. Determination of depth of stripping requires experience and good judgment because of compromise between what would be desirable from a design standpoint and excessive cost. Good bond between the foundation and first layer of fill is achieved by moistening the foundation rather than using a very wet embankment layer.

Foundation control information should be included in the earthwork portion of the monthly construction progress report (L-29). If field density tests are made, the data can be reported along with the compacted earthwork control data (forms 7-1581A and 7-1581B), if properly identified. Other test data, logs of holes, and observations can be included in the narrative portion of the report. The forms above are examples of how results of field and laboratory tests performed in conjunction with construction control are reported. Form 7-1581A is used when the materials are controlled using USBR 7240: Performing Rapid Method of Construction Control,[1] or by USBR 5500 and 5515 laboratory compaction of soils.[2] Form 7-1581B is for materials controlled using USBR 5525, 5530 (minimum and maximum index density), and USBR 7250: Determination of Percent Relative Density.[3] Other forms, such as those

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[1] 7240: Performing Rapid Method of Construction Control.

[2] 5500: Performing Laboratory Compaction of Soils─5.5-lbm Rammer and 18-in Drop.

5515: Performing Laboratory Compaction of Soils Containing Gravel.

[3] 5525: Determining the Minimum Index Unit Weight of Cohesionless Soils.

5530: Determining the Maximum Index Unit Weight of Cohesionless Soils.

7250: Determination of Percent Relative Density.

generated by computer program PCEARTH (see sec. 3-27), can be used as long as all pertinent information is shown. Information on current computer software programs is available and can be obtained from the Denver Office.

c. Earth Embankments and Linings.—Depending upon design requirements, earth embankments for canals and laterals may consist of impervious or pervious soils placed loose, partially compacted by equipment or well compacted by rollers, or a combination of these. Earthwork control inspection of loose embankments consists only of making sure that proper types of materials are being used, that material is uniformly placed, and that layers are of proper thickness when layer thickness is specified. Normally, the finest available material is placed on the canal prism side, and coarser materials are placed toward the outside of the embankment. When specifications require partial compaction by routing equipment, inspection requirements noted above are applicable, and the inspector should make sure that hauling and spreading equipment is routed so as to provide the most uniform coverage and best compaction. In some instances, uniform moistening of partially compacted fill material is specified. Normally, the water required is that amount which will provide a total moisture equivalent to optimum moisture content for the compactive effort being applied. Standard laboratory optimum moisture content may be used. Density tests are not taken for fill placed in this manner.

Control of compacted impermeable soils in embankment and compacted earth lining consists of:

- Inspecting materials used
- Checking amount and uniformity of soil moisture
- Maintaining thickness of layer
- Determining the percentage of standard laboratory maximum dry density of the fill

Using the available logs of explorations and careful observation of excavations for the canal, appurtenant structures and borrow areas will be useful in selecting the best soils for use in compacted embankments and earth linings. In borrow areas, the depth of cut can be regulated to obtain high quality and uniformity of soil. Deposition of each load of material on the fill is directed to control uniformity. The thickness of loose lift required to result in a 150-mm (6-in) compacted layer must be determined and regularly checked.

Blending of two or more materials is sometimes specified to produce suitable soils for earth linings. Most commonly, fine soil from borrow is added to pervious coarse soils obtained from excavation to decrease permeability. Coarse soils from borrow may be added to soils from excavation to improve erosion resistance for lining or cover purposes. Proportions of soils to be blended are determined earlier by laboratory testing. The inspector has responsibility to ensure that materials are properly proportioned and mixed. Normally, excavation materials, to be used for one of the blended materials, are required to be completely removed from the section and to be placed in the lining layer to the depth required for proper proportioning. The correct depth of fine soil is placed first, followed by the correct depth of coarse soil; then, blending is allowed.

Generally, blending is specified to be done by a machine designed for mixing soils or by other equipment which obtains comparable results. Therefore, the inspector must ensure that equipment being used provides a uniform blend of soils for the total layer depth. This is done by frequently digging holes through the blended layer before compaction to observe uniformity of the layer after blending. When soils of different colors are blended, color of the mixture at different depths in the layer is a useful index indicating effectiveness of the mixing operation. Compaction should not be allowed to proceed until a satisfactory blend has been attained. Inspection trenches should be requested frequently at the beginning of a blended lining contract, and at lesser frequency as the job progresses, so results of the contractor's blending and lining operations can be visually observed.

Cobbles larger than specified, usually 75 or 125 mm (3 or 5 in), must be removed before compaction. This is especially important in all types of compacted earth linings where a relatively thin section is relied upon for seepage control. Pockets of cobbles provide certain access for water loss and piping. Often, cobbles are buried in the layer being placed and are difficult to detect or to remove. Therefore, if the percentage of oversize material is large, some means should be provided for removing this material at the point of excavation.

The inspector is responsible for controlling water content of the soil to that which is "optimum for compaction" and to ensure that moisture is uniform throughout the layer to be compacted. If the soil is several percentage points dry of desired moisture content, it is more efficient to add most of the water at the location of excavation with only supplemental sprinkling after the layer has been spread. After sprinkling, mixing is required to produce a uniform moisture condition throughout the layer before compaction can proceed. Moisture uniformity should be evaluated frequently by digging holes in the loose layer just before compaction. Unless otherwise specified, optimum moisture requirements must be enforced for canal embankments and linings even though required density can be obtained at other moisture conditions. Adverse settlement and permeability properties may result if placement moisture is too low; and adverse shear strength properties may result if placement moisture is too high. Therefore, the inspector must be prepared to request application or removal of moisture, as required.

Control of pervious material includes visual inspection of material for free-draining characteristics and uniformity of material. Thickness of layer is controlled, depending on type of compaction. A thickness of:

- Not more than 150 mm (6 in) after compaction is specified if smooth or pneumatic-tire rollers are used
- Not more than 300 mm (12 in) after compaction is specified if vibrating rollers or tractor treads are used
- Length of vibrator if internal vibrators are used

Each layer must be thoroughly wetted during the compaction operation for all types of compaction.

Adequacy of compaction and moisture control of impervious or pervious soils is controlled by field density test USBR 7205 or 7220 in conjunction with USBR 7240 Performing Rapid Method of Construction Control for clayey and silty soils,[4] or the relative density tests USBR 5525, 5530, and 7250 for pervious sand and gravel soils.[5] Unless otherwise specified, minimum acceptable density is 95-percent laboratory maximum density for the minus 4.75 mm (-No. 4) fraction of clayey and silty soils and 70-percent relative density for the minus 75-mm (-3-in) fraction of pervious sand and gravel soils. For soils that are borderline between silty and clayey soils controlled by the compaction test and pervious sand and gravel soils controlled by the relative density test, control is based on either 95-percent laboratory maximum density or 70-percent relative density, whichever produces the highest in-place density. Table 3-1 is a guide for determining materials for which the relative density test is applicable.

Inasmuch as the adequacy of compaction is specified in terms of soil density achieved, the inspector is responsible to arrange or perform sufficient tests to ensure adequacy of compaction for acceptance purposes. At the beginning of any work, a considerable number of tests are required to ensure construction operations are producing required results. Then, the number of tests required is that necessary to ensure that specifications requirements are being satisfied. Because of the widespread operations of canal work, the number of field density tests required for adequate control cannot be positively stated. The following is a guide for the minimum number of field density tests:

- For all types of earthwork, one test for each work shift
- For canal embankments, one test for each 2,000 m^3 (2,000 yd^3)
- For compacted canal linings, one test for each 1,000 m^3 (1,000 yd^3)
- For compacted backfill or for refill beneath structures:

  i. Hand tamped (mechanical tamping), one test for each 200 m^3 (200 yd^3)

  ii. Roller or tractor compacted, one test for each 1,000 m^3 (1000 yd^3)

- One complete permeability settlement test should be made in the laboratory for each 10 density tests for compacted canal linings, impervious embankments, and impervious backfill.

When field density tests are performed in relatively narrow compacted earth linings where equipment travel is essentially along a constant route, density tests should not be taken in the tractor or wheel tracks. At such locations, density may be considerably higher than the average density of normally rolled lining.

Horizontal and vertical control must be available so the inspector can adequately locate each field density test. Usually, locations are defined in terms of station, offset from canal (or structure) centerline, and elevation above bottom grade. The inspector is responsible for locating these tests so a complete representative record of finished work is available. Companion laboratory tests by the rapid method of construction control for clayey and silty soils and relative density tests for free-draining sand and gravel soils should be made for each density test. A laboratory compaction test or a relative density test is required for each control field density test so the percent compaction or relative density, respectively, can be computed.

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[4] 7205: Determining Unit Weight of Soils In-Place by the Sand-Cone Method

7220: Determining Unit Weight of Soils In-Place by the Sand Replacement Method in a Test Pit

7240: Performing Rapid Method of Construction Control

[5] 5525: Determining the Minimum Index Unit Weight of Cohesionless Soils

5530: Determining the Maximum Index Unit Weight of Cohesionless Soils

7250: Determination of Percent Relative Density

F. Pipelines

3-32. Design Features.—Reclamation uses buried pipelines for pumping plant discharge lines, irrigation laterals, and distribution systems for domestic, municipal, and industrial deliveries. These systems may be constructed using one or more types of pipe such as:

- Reinforced concrete pipe
- Reinforced concrete cylinder pipe
- Steel pipe with a number of combinations of coatings and linings
- Pretensioned concrete cylinder pipe
- Ductile iron pipe
- Fiberglass pipe
- Polyvinyl chloride (PVC) pipe
- High-density polyethylene (HDPE) corrugated pipe

Usually, the type of pipe is selected by the contractor from several acceptable types described in Reclamation specifications.

Buried pipe is a structure that incorporates both the properties of the pipe and the properties of the soil surrounding the pipe. Structural design of a pipeline is based on certain soil conditions; construction control is important to ensure these conditions are satisfied. Two basic types of pipe are used—rigid and flexible. Pipe of 250-mm (10-in) nominal inside diameter and smaller can be considered either rigid or flexible, but Reclamation designs this size pipe to be relatively independent of soil conditions. Installation requirements for pipe are different for each condition: rigid, flexible, and 250-mm diameter and smaller. Rigid pipe must be supported on the bottom portion of the pipe. Flexible pipe must be supported both on the bottom and on the sides of the pipe.

Rigid pipe includes:

- Reinforced concrete pressure pipe
- Ductile iron pipe 500 mm (20 in) and less in diameter
- Reinforced concrete cylinder pipe

Rigid pipe is designed to transmit the backfill load on the pipe through the pipe walls to the foundation beneath the pipe. The pipe walls must be strong enough to carry this load. A concentrated load at the top *and* bottom of the pipe is the worst possible loading case. If load can be distributed over a larger area on the top and at the bottom of the pipe, the pipe walls do not have to be designed as strong as for a concentrated load. Normally, soil backfill load is well distributed over the top of the pipe. However, proper support must be constructed on the bottom of the pipe to distribute the load. Proper soil support on the bottom of the pipe is necessary to maintain the grade of the pipe and to prevent unequal settlement.

Flexible pipe includes:

- Steel pipe
- Pretensioned concrete cylinder pipe
- Ductile iron pipe over 500 mm (20 in) in diameter
- Fiberglass pipe
- PVC pipe
- HDPE corrugated pipe

Generally, steel pipe is lined with cement-mortar, either in the factory or in place after installation. The coating can be cement-mortar or some type of flexible coating such as polyethylene tape wrap.

Flexible pipe is designed to transmit the load on the pipe to the soil at the sides of the pipe. As load on the pipe increases, the vertical diameter of the pipe decreases, and the horizontal diameter increases. The increase in horizontal diameter is resisted by the soil at the sides of the pipe. Adequate soil support on the sides of the pipe is essential for proper performance of the pipe. The side-soil support must be strong enough to carry the applied load without allowing the pipe to deflect more than the allowable amount for the specific type of pipe. Proper soil support on the bottom of the pipe is necessary to maintain the grade of the pipe and to provide uniform support.

Pipe 250-mm (10-in) nominal inside diameter and smaller do not require any compaction of the soil placed around the pipe. While pipe less than 250 mm may be a type otherwise considered rigid or flexible, Reclamation designs pipe of this size to be strong enough to withstand the backfill load without any special soil support. Figure 3-37 shows typical installation requirements for pipe 250-mm diameter and smaller. Pipe 250-mm diameter and smaller includes PVC, steel, and ductile iron.

3-33. Specifications Provisions.—Typical trench details for buried pipe are shown on figure 3-38. The specifications describe type of soil and compaction requirements for the foundation, bedding, embedment, and backfill.

The foundation is the in-place material beneath the pipe. If the foundation is unsuitable, a minimum of 150 mm (6 in) of

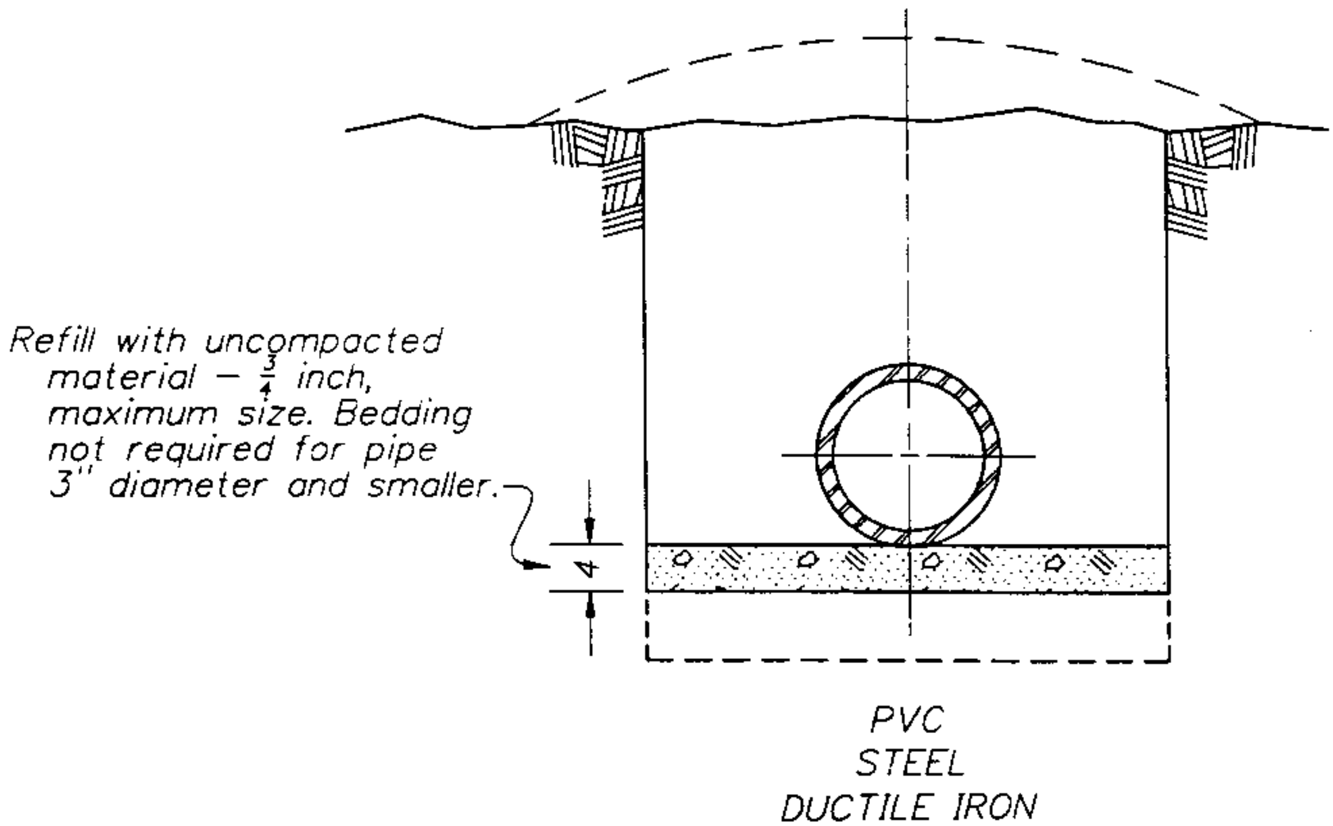


Figure 3-37.—Installation requirements of pipe 250-mm in diameter and smaller.

material must be removed and replaced with compacted material. Removal and replacement of material to 1 to 2 m (3 to 6 ft) depth or more may be necessary to attain a suitable foundation for the pipe.

Unsuitable foundations include potentially expansive material such as shale, mudstone, siltstone, claystone, or dry, dense fat clay (CH). These materials may expand when wetted. Uplift pressures created from expansion of these materials have been known to cause broken backs in rigid pipe.

Another type of unsuitable foundation includes soft, unstable soils such as very wet soils that flow into the excavation, low-density soils, peat or other organic material. A soft, unstable foundation may result in uneven settlement of the pipe. Low-density soils may collapse upon wetting. Very wet, unstable soils must be removed to provide a stable foundation that will maintain grade and provide uniform support for the pipe. Peat or other organic soils are highly compressible; significant settling of the pipe may occur if these soils are left in the foundation. Foundation materials disturbed during construction must be removed and replaced with material that is compacted.

Material is placed and compacted to replace the excavated foundation. Fat clays (CH) must not be used because moisture changes can cause significant volume increase or decrease. Elastic silts (MH), peat, or other organic material must not be used because they are highly compressible; nor should frozen soils be used. Material that allows migration of fines should not be used for a replaced foundation; for example, a crushed rock or gravel material containing significant voids should not be placed next to a fine-grained material. The fine-grained material can migrate into the voids and result in the rock particles floating in a matrix of fine-grained material. This would cause loss of support for

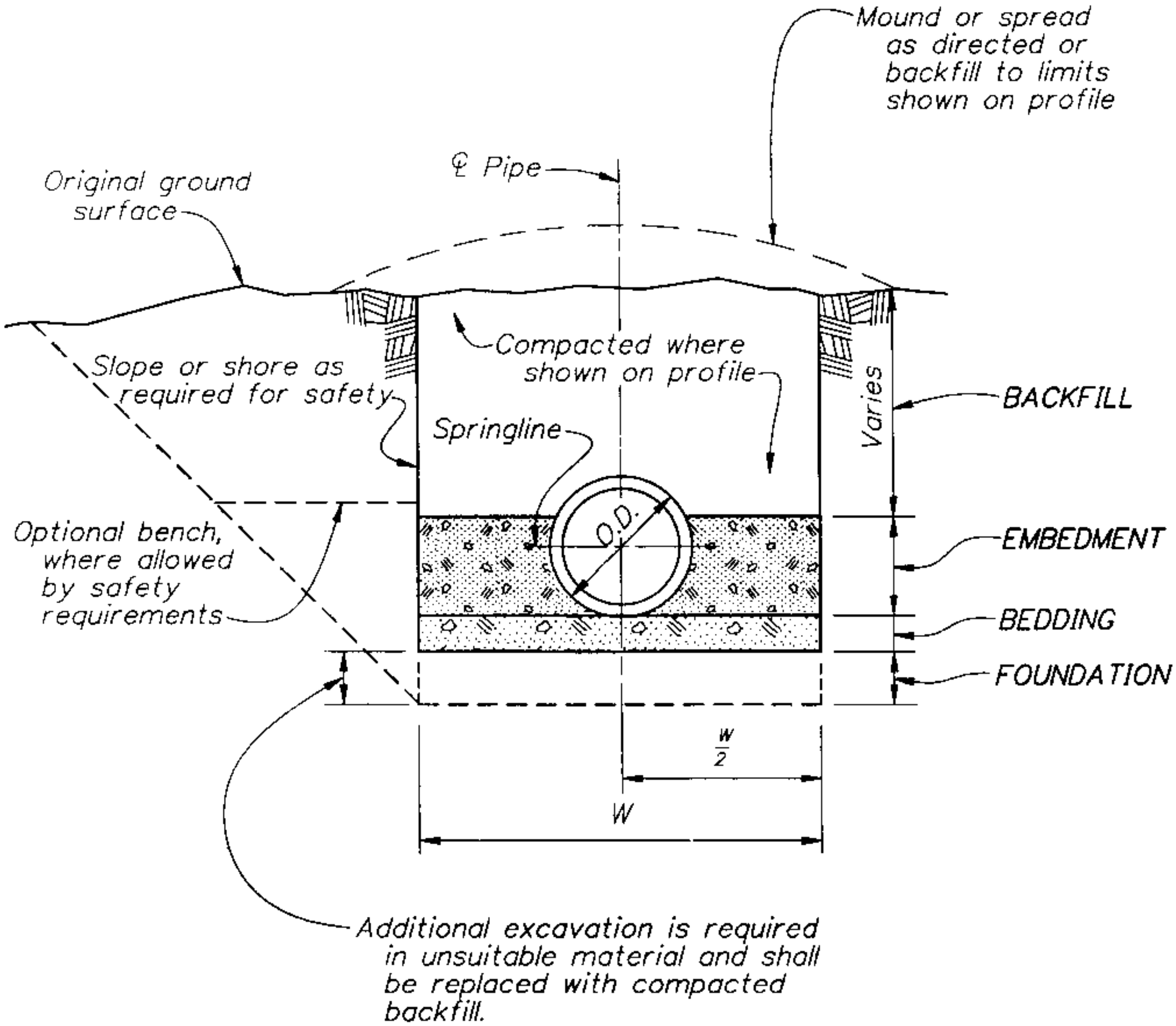


Figure 3-38.—Typical trench details.

the pipe and result in uneven settlement. Any method of compacting the replaced foundation may be used; however, specified density requirements must be satisfied.

For both rigid and flexible pipe, the bedding is a layer of uncompacted select material placed over the foundation (or the replaced foundation). The pipe is laid on the uncompacted bedding. For pipe diameters between 300 and 1,375 mm (12 and 54 in), bedding is 100 mm (4 in) thick. For pipe diameters 1,525 mm (60 in) and over, bedding is 150 mm (6 in) thick.

The surface of the bedding shall be fine graded so that the final grade of the pipe does not exceed specified departure from grade. Settlement of the pipe into the uncompacted bedding soil must be taken into account. For bell-and-spigot jointed pipe, "bell holes" must be excavated in the bedding to provide a space between the bottom of the bell and any soil. This may require that material beneath the bedding also be excavated. This bell hole prevents a point loading condition on the bell end of the pipe.

After laying the pipe on the bedding, embedment soil is compacted underneath and beside the pipe up to the specified height. For rigid pipe, embedment soil is placed and compacted to a height of 0.37 units of the outside diameter of the pipe. Embedment soil must be a select material and must be compacted to a relative density not less than 70 percent. It is important that soil placed in the haunch area of the pipe receive sufficient compactive effort to meet the 70-percent relative density requirement. Compaction by saturation and internal vibration is an effective method of working the select material into the haunch area and of obtaining the required density. For flexible pipe, embedment soil is placed to a height of 0.7 units of the outside diameter of the pipe.

The select material used for pipe embedment must be cohesionless and free-draining (5 percent fines or less), have a maximum size of 19.0 mm (3/4 in), and not more than 25 percent of the soil shall pass a 300 µm (No. 50) sieve.

Where the pipeline grade exceeds 0.3 units, silty or clayey material may be used instead of the specified select material for bedding and embedment. This change is allowed because of the difficulty encountered in compacting by saturation and vibration on steep slopes. If cohesive soil is used for bedding or embedment, density requirements must still be satisfied; the inspector must be sure that soil placed in the haunch areas is compacted as specified.

Most soil types may be used for backfill over the pipe, except there are maximum particle size restrictions. A limit is placed on the maximum particle size (limitation depends on type of pipe) in a zone 300 mm (12 in) around the pipe. Outside this zone, any rock particle with any dimension greater than 450 mm (18 in) is not allowed in the backfill. Frozen soils shall not be used. Where backfill is to be compacted to the ground surface, such as at road crossings, peat or other organic materials shall not be used. Local requirements for compacted backfill under roads must be satisfied. Backfill material must not be dropped on the pipe.

3-34. Construction Control.—Soil placed against the pipe must be in firm, complete contact with the pipe. Compacting soil in the haunch area of a pipe is the most difficult part of pipeline construction; this compaction must be carefully monitored. A test pit should be dug at regular intervals to inspect the haunch area and to determine density of soil in the haunch area. When using saturation and vibration, the area under the bottom of the pipe must be inspected to ensure the pipe did not "float" during construction. "Bell holes" for bell-and-spigot pipe, sling holes for large diameter pipe, and spaces left for joint treatment—for other than bell-and-spigot pipe—must be filled after the pipe is laid. Embedment soil must be compacted the complete width of the trench. Embedment soil shall be placed to about the same elevation on both sides of the pipe to prevent unequal loading and displacement of the pipe. Where trenches have been left open at pipe-structure junctions, requirements (as previously stated) for pipe embedment must be continued to the structure after it is constructed but before the excavation is backfilled.

For silty or clayey soils, the thickness of each horizontal layer after compaction shall not exceed 150 mm (6 in). For cohesionless, free-draining material such as sands and gravels, the thickness of the horizontal layer—after compaction—shall not exceed:

- 150 mm if compaction is performed by tampers or rollers
- More than 150 mm if compaction is performed by treads of crawler-type tractors, surface vibrators, or similar equipment
- More than the penetrating depth of the vibrator if compaction is performed by internal vibrators

Backfill over the pipe should be placed to a minimum depth of 1 m (3 ft) or one-half pipe diameter (whichever is greater) above the top of the pipe before power-operated hauling or rolling equipment is allowed over the pipe. In addition, limitations may be imposed on weight of equipment traveling over the pipeline. Equipment crossings, detours, or haul roads crossing the pipeline must be approved prior to use.

Pipe 250-mm (10-in) diameter and smaller can withstand the weight of backfill without special soil support. Compacted soil around this pipe is used only on the outside of horizontal curves or where the pipe crosses under another conduit, and under a roadway, canal, ditch, or structure. Previous comments on adequacy of a foundation, however, still pertain to 250-mm pipe.

For pipe 80 to 250 mm (3 to 10 in) in diameter, 100 mm (4 in) of uncompacted material are required as bedding between the pipe and the foundation. Maximum particle size of the bedding material is 19.0 mm (3/4 in). Peat or other organic material shall not be used as bedding. The uncompacted material can be imported or the bottom of the excavation can be loosened by scarifying if the in-place material is suitable. The main requirement for pipe 250 mm and smaller is suitability of the foundation.

3-35. Construction Control Requirements.—Trench dimensions, minimum installation width, slope of the trench walls, trench depth, and side clearance of flexible pipe must always be carefully checked.

A minimum installation width, *W*, is specified to ensure a minimum distance between the pipe and the trench wall. Enough clearance must be allowed to inspect pipe joints and to compact the embedment. This clearance is particularly critical for vertical trench walls. Minimum installation width is measured at the top of the foundation. For trench slopes, the most recent Reclamation or Office of Safety and Health Administration requirements should be consulted. However, State or local regulations may override these requirements. The trench wall slope must begin at the bottom of the excavation, which includes any additional excavation of foundation material.

For flexible pipe, side clearance between the pipe at the springline and the trench wall must be checked for all conditions, including shored trenches, sloping trench walls, and vertical walls. This clearance is required in addition to the minimum installation width and may require, in some cases, a wider trench bottom than the minimum installation width.

Performance of flexible pipe depends on soil at the sides of the pipe for support [24]. This side soil support results from the combination of embedment and trench wall soil. The width of the trench depends on the relative firmness of embedment and trench wall material. If the trench walls are firmer than the embedment, embedment soil is used to fill the space between the pipe and the trench walls. If the trench walls are soft and easily compressible, most of the side soil support must come from the embedment. As shown on figure 3-39, three types of trenches are specified which state the minimum clearance between the pipe and the trench wall measured at springline of the pipe [25].

Trench type 1 is where trench wall material is stronger or firmer than the embedment. Typical trench wall materials would be rock; materials described as:

- Claystone, mudstone, or siltstone
- Strongly cemented soils even though of low density
- Cohesionless soils with in-place relative densities over 70 percent
- Silty or clayey material with in-place densities over 95 percent compaction

For trench type 1 installation, minimum side clearance is 250 mm (10 in) for pipe 450 mm (18 in) and smaller, and 450 mm for pipe larger than 450-mm diameter.

Trench type 2 is where trench wall soil has a strength or firmness equivalent to the embedment. These soils include:

- Silty or clayey material with in-place densities less than 95 percent compaction—but over 85 percent compaction
- Cohesionless soils with in-place relative densities between 40 and 70 percent

Minimum side clearance for trench type 2 installation is equal to the outside diameter of the pipe.

Trench type 3 is where trench walls are much softer than the embedment. Soils in this category include:

- Peat or other organic soils
- Elastic silts (MH)
- Low-density silty or clayey material (below 85 percent compaction)
- Low-density cohesionless soils (below 40 percent relative density)

Minimum side clearance for trench type 3 is equal to two pipe outside diameters—making a total trench width of five pipe diameters measured at pipe springline. Generally, this is impractical so, in trench type 3 areas, rigid pipe may be specified. During construction, if unexpected areas of trench type 3 conditions are encountered, changes to the type of pipe or method of construction must be made to obtain a proper installation.

During investigations where types 2 or 3 trench may be required, areas along the pipeline must be identified. Particular attention should be paid to stream crossings, loessal deposits, talus slopes, and landfills.

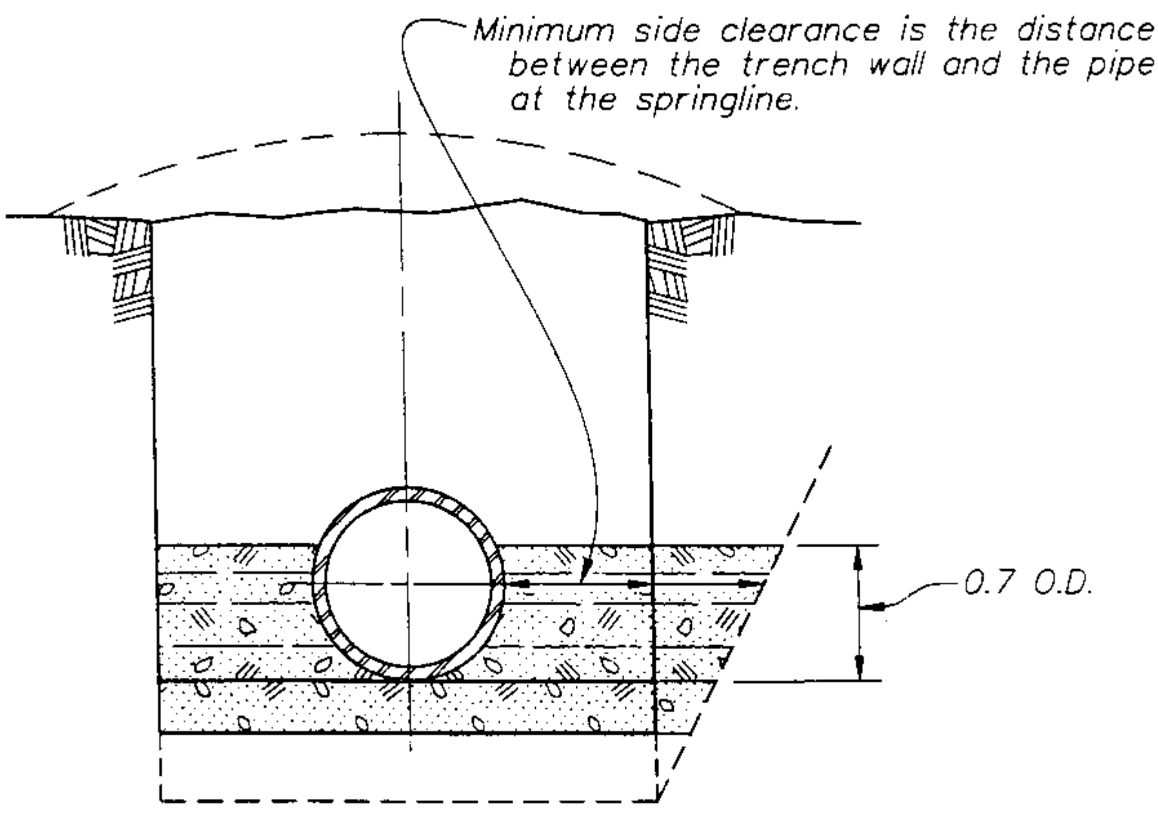
The specifications requirements pertain to one method of pipeline construction acceptable to Reclamation. Other acceptable installation techniques may be used. Sometimes, pipe design may have to be changed to be compatible with the installation method. It is not uncommon to provide a stiffer pipe than specified, so the density requirement for embedment may be lowered.

3-36. Soil-Cement Slurry Pipe Bedding.—When conditions warrant, the contractor may choose to install pipe using soil-cement slurry as an alternate to the standard compacted soil embedment. The trench is trimmed to a semicircular cross section (or other approved shape) slightly larger than the pipe (as shown on figure 3-40). A soil-cement slurry is used to fill the gap between the pipe and the in place soil. Since soil-cement transfers the load from

SIDE CLEARANCE TABLE

TRENCH TYPE	MINIMUM SIDE CLEARANCE (INCHES)
1	10 INCHES FOR 12" THRU 18" I.D. 18 INCHES FOR OVER 18" I.D.
2	ONE O.D.
3	TWO O.D.

For location of Trench Types, see Specifications.



FLEXIBLE PIPE

Figure 3-39.—Side clearance requirements for flexible pipe.

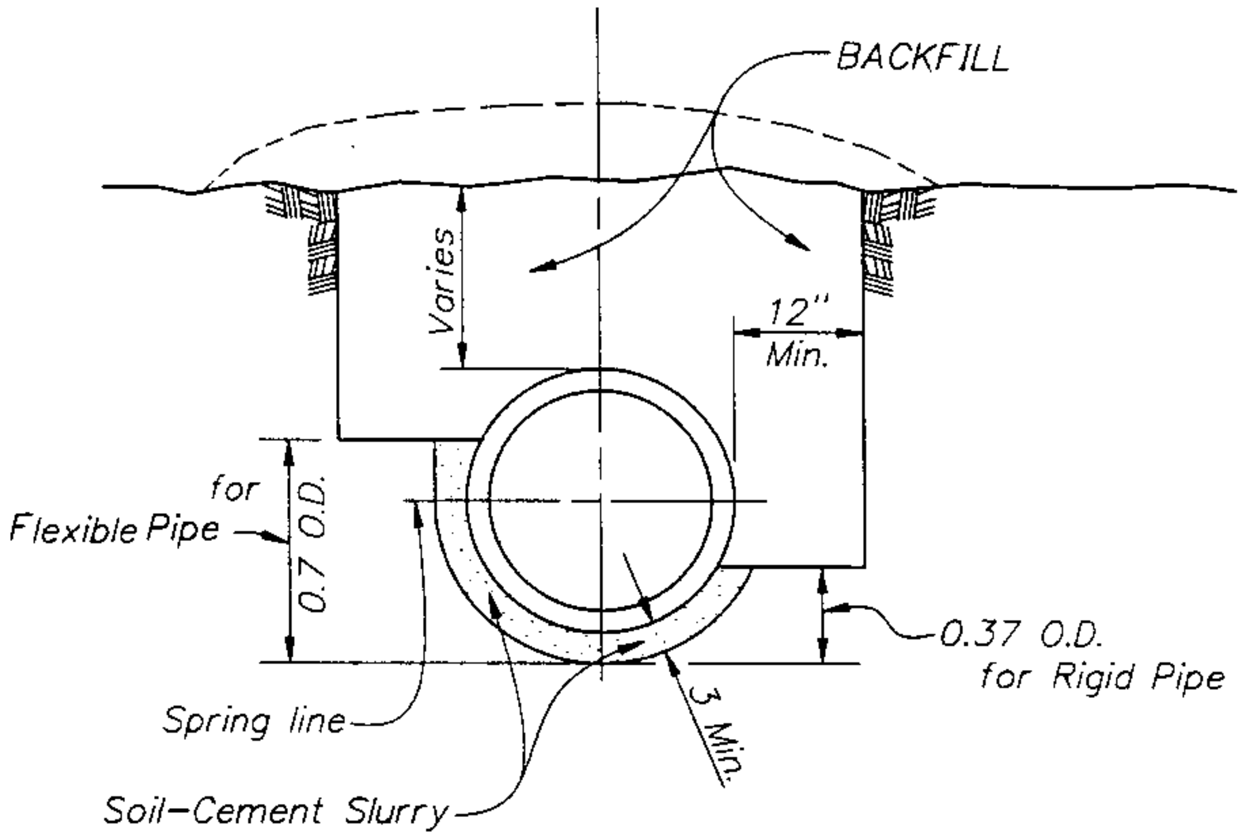


Figure 3-40.—Soil-cement slurry installation.

the pipe to the trench material, the native soil must be able to provide the necessary support for the pipe. Typically, in-place materials that categorize as trench type 1 would qualify.

The trimmed shape is advantageous because it reduces excavation quantities and handling of excavated soil. Placing soil-cement slurry is quicker than replacing and compacting soil beneath and beside the pipe. This faster installation is a distinct advantage where construction work space is limited or where time of installation is critical.

Soil-cement slurry is a combination of soil, portland cement, and enough water so the mixture has the consistency of a thick liquid. In this form, the slurry flows readily into openings and provides a hardened material with strength greater than the in place soil.

Allowable ingredients and specific placement requirements are given in the construction specifications. Typically, soil-cement slurry contains about 6 to 10 percent portland cement by dry mass of soil. Some cementitious fly ashes have been successfully used in place of cement.

One distinct advantage in using soil-cement slurry is that soil-cement may be produced using local soils. The soil can contain up to 30 percent nonplastic or slightly plastic fines. Although clean, concrete sands have been used, the presence of fines helps keep the sand-size particles in suspension. This allows the mixture to flow easier and helps prevent segregation. The allowable maximum particle size, limit of organic material, and other requirements are usually specified.

Soil-cement slurry typically has a water-cement ratio of about 2.5 to 3.5 to attain required flow properties. Generally, internal vibrators are used to ensure that the slurry completely fills the annular space around the pipe.

Central batch plants—with the slurry delivered in ready-mix trucks—or portable batch plants have been used to mix soil-cement with the latter normally used when soil comes from trench excavation. Materials to be used and method of mixing depend on availability of local materials and equipment.

Construction control of soil-cement slurry involves only two factors: (1) proper proportioning of the mixture and

(2) ensuring that the slurry completely fills the annulus between pipe and trench walls. Proper mixture is checked by determining the unconfined compressive strength at 7 days for minimum and maximum strength requirements of 700 and 1,400 kPa (100 and 200 lbf/in^2). Higher strength is not desirable because structural characteristics of the pipe may be altered.

G. Miscellaneous Construction Features

3-37. Highways and Railroads.—During construction, public utilities, communication systems, or transportation systems (including highways and railroads) may need to be relocated. Normally, a relocation agreement is entered into with the owner of the facility. Provisions of the relocation agreement will require either the owner or Reclamation to design and construct the relocation of facilities according to design and construction standard agreed upon.

a. Design Features.—The design of a highway or railroad consists of selecting and laying out the roadway or roadbed alignment, designing and detailing required structures, and preparing specifications for constructing the facility. As a basis for agreement on the design and construction of relocated highways, Reclamation uses American Association of State Highway and Transportation Officials standards and manuals, the standards of the state highway departments, and general highway construction practice. For the design and construction of relocated and access railroads, Reclamation uses American Railway Engineering Association manuals, the railroad's own standards, and general railroad construction practice.

Access roads for use by the contractor and the Government are designed and constructed by Reclamation according to Reclamation standards. These standards are based on local proven methods of road construction which permit using local materials and services of local contractors experienced with construction equipment designed for such work.

b. Earthwork Specifications Provisions.—Specifications for constructing a railroad or highway will contain a paragraph on roadway excavation. This paragraph will require the contractor to construct the roadway to dimensions shown on drawings or as staked and to prescribed lines and grades. Drawings include profiles of gradelines either to finished grade or to subgrade. The term "subgrade" refers to the top of embankments and to the bottom of excavations ready to receive the roadway surface or the railroad ballast. In rock excavation, the bottom of the cut is excavated to 150 mm (6 in) below subgrade; and, in common excavation, all loose rock fragments, cobbles, and boulders are removed or excavated to a depth of not less than 150 mm (6 inches) below subgrade.

Excavation below subgrade is refilled to subgrade with material obtained from excavation for the roadway or from borrow pits; the refilled material is compacted equivalent to the compaction required for embankments. Provisions are made in the paragraph on roadway excavation for dealing with slopes shattered or loosened by blasting and slides extending beyond the established slope lines or below subgrade. Provisions for constructing side drains and for excavating around trees, poles, or other objects that are to remain within the right-of-way for the roadway are also given in the paragraph on roadway excavation.

Provisions are included in paragraphs on preparation of surfaces under roadway embankments for surface treatment and for removal of unsuitable materials. Surfaces of sloping ground underneath embankments must provide bond with the embankments and to prevent slipping. Where embankments placed on smooth, firm surfaces and where low embankments are placed, the original ground surface is thoroughly plowed or stepped to ensure proper bonding of new and existing material. When embankments are placed on steep rock slopes, trenches are blasted into the rock surface for "keying" the embankment to the rock surface. The contractor must strip the area under an embankment of all unsuitable material to depths as directed.

Roadway embankments are constructed to established lines and grades and increased by such heights and widths as necessary to allow for settlement. Brush, roots, stumps, sod, or other unsuitable materials are not permitted in embankments. Hard material or hard lumps of earth more than 300 mm (12 in) in size must be broken down, or removed, and boulders and cobbles more than 150 mm (6 in) in diameter are not permitted in the upper 600 mm (24 inches) of embankment. Given a choice of materials, the best are used in the upper 300 mm of the embankment. Material is not permitted in embankments when either the material or the surface on which it is to be placed is frozen. Good distribution of cobbles and gravel within the embankment is required when such material is being used. Rockfill embankments are constructed so large spaces are not unfilled. When directed, rock fragments are deposited in the outer portions of embankments to protect the slopes against erosion.

The contractor's combined excavation and placing operations should ensure that material in the embankment will be blended sufficiently to secure the best possible degree of compaction and stability. The required thickness of layers is 300 mm (12 in) after compaction for earthfill and not more than 600 mm (24 in) after compaction for rockfill. Layers are constructed across the entire width of embankments and must be built to the required slopes rather than widened with loose material dumped from the top. On sidehill fills, where the width is too narrow to accommodate hauling equipment, end dumping is used until the width of the embankment becomes great enough to permit using hauling equipment. The contractor is required to route hauling equipment over the layers already in place and to direct travel evenly over the entire width of the embankment so as to obtain the maximum amount of compaction possible.

Moistening and compacting roadway embankments may be necessary where normal construction techniques will not result in proper densification. Where sufficient moisture is not available in the material, additional water must be provided by sprinkling as the layers of material are placed on the embankments. Moistening must be uniform; harrowing, disking, or other working may be necessary to produce a uniform moisture content. Material containing excess moisture must be permitted to dry to the extent required before being compacted.

The amount of embankment rolling depends on the nature of the material compacted and the extent of compaction required; rolling will be specified for each embankment or portion of an embankment. Reclamation usually follows State highway specifications when embankments are built for a State highway department. Generally, highway department specifications are of the performance type where only the end result is specified. For this type of specification, the contractor usually selects the equipment to obtain the end result. If highway and railroad embankments are not built according to State specifications, Reclamation will build them using an end-result type of specification requiring the fill to be compacted to 95 percent laboratory maximum dry density.

Additional material for construction of embankments and for excavation refills in rock cuts is obtained by widening the cuts, flattening the slopes on either or both sides of the centerline, or from borrow pits. If additional material is taken from borrow pits adjacent to the roadway, a berm of original unbroken ground, not less than 3 m (10 ft) wide, is required between the outside toe of the embankment and the edge of the borrow pits, with provisions for a side slope of 1.5h:1v to the bottom of the borrow pit. Where directed for drainage or other requirements, the berm between the toe of the embankment and the borrow-pit slope should be no less than 12 m (40 ft) wide. Provisions are made for connecting pit to pit with waterways, unless otherwise directed, and to provide ample drainage and leave no stagnant pools. Unless otherwise approved, the bottoms of pits near bridge and culvert openings are not permitted to be excavated below the culvert or bridge. The borrow pits should be left in a reasonably smooth and even condition.

c. Control Techniques.—Many of the control techniques discussed in preceding paragraphs are applicable to highway and railroad embankments. Moisture content-density relationship of soils and related field control methods are fundamental for any earthwork construction where compaction of soil is required. However, the same degree of control may not be practical nor required for highway and railroad embankments.

Control requirements vary depending on standards adopted by the various agencies involved with Reclamation in the relocation agreement. The inspector should first become familiar with the appropriate requirements and standards and adapt the techniques discussed for embankment dam embankments to the requirement. The principal differences in control are:

1. Moisture, if required, for the most part will be added to material on the fill. Usually, moisture content should be as near as possible to the optimum required for maximum compaction.
2. Density requirements are usually specified as percentages of laboratory maximum dry density and vary according to the maximum dry density of soil.
3. The frequency of testing required to maintain control on these embankments is not standardized. For each fill location, the quantity of water and the amount of rolling required must be determined rapidly so the information may be used when constructing the fill. Since construction of a fill may be a relatively short operation, moisture and rolling requirements must be established quickly. The rapid method of construction control (USBR 7240) can be used for compacted highway and railroad embankments.[6] Where compaction is required by the specifications, one in-place density test should be made for each 2,000 m^3 (2,000 yd^3) of compacted fill.

[6] 7240: Performing Rapid Method of Construction Control

4. Conformance with certain specifications provisions can be determined by visual inspection, such as preparation of foundation soils, and deposition of material. Selection of borrow materials required in embankment dam construction may not be required for highway or railroad work.
5. Thickness of layers, whether they are to be compacted by roller or equipment, is normally specified; control of layer thickness is the inspector's responsibility. Where embankment is to be compacted by equipment travel, distribution of travel over the embankment should be directed to obtain the maximum amount of compaction.

3-38. Miscellaneous Structures.—

a. General.—In addition to large earth structures such as dams and embankments, many earthwork features relate to smaller structures built by Reclamation. These structures include:

- Pumping plants
- Powerplants
- Bridges
- Substations
- Warehouses
- Residences

various types of canal structures such as:

- Checks
- Drops
- Turnouts
- Siphons
- Pipelines
- Overchutes.

Although foundation loadings for most of these structures are normally light, moderately heavy loadings on soils are not uncommon for larger units. Even for small structures soft, highly compressible, or expansive soils—at natural moisture content or in a future wetted state—must be examined carefully. Care must be exercised when compacting embankment and compacting backfill about these smaller structures.

b. Structure Foundations on Soil or Rock.—The contractor is required to use methods of foundation soil preparation, including moistening and tamping if necessary, which will provide appropriate foundation soil conditions. Excavation is made to lines and grades shown on drawings or as directed when subsurface conditions become known. Additional excavation is refilled and compacted, as specified. Loosened or disturbed natural material must be either compacted or removed and replaced with suitably compacted material. When excavation for concrete structures is in rock, rock should be excavated so there will be more than 150 mm (6 in) between rock points and the bottom of the concrete slab. This space is filled with compacted soil. For other structures, concrete is often placed directly on rock which has been previously cleaned and dampened. Large unit loadings may be used in this case.

Construction of small miscellaneous structures usually involves a variety of earthwork problems because of the numerous structures involved, the extent of the work, and the many different soils encountered in foundation soils and backfill materials. For larger structures (i.e., powerplants and pumping plants), where loadings are moderately high and settlements must be small, explorations are normally conducted before designs are prepared to determine competency of foundation soils. Measures to improve the competency of foundation soils, if needed, are usually known ahead of time and are included in plans and specifications. Often, it is impractical to perform adequate predesign explorations, so provisions for drilling exploratory holes or performing tests—after the excavation has been completed—are contained in the specifications. Usually, it is impractical to perform detailed investigations of foundation soils for the numerous small structures built by Reclamation. Regardless of the amount of prior exploration, the inspector's responsibility is to make sure that soil conditions, which can be observed upon completion of excavation to grade, are compatible with design assumptions. If any doubt exists concerning adequacy of foundation soils, this fact should be brought to the attention of the supervisor.

Several methods can be used to check properties of foundation soils. When sands are involved, the field penetration test, with split-tube sampler (USBR 7015), provides a rapid method for determining the relative density of soil.[7] The relationship of blow counts to relative density was shown on figure 1-15. Unless otherwise specified, 70 percent relative density of sands can normally be considered adequate for all but very heavy loadings. For very fine sands, 80 percent relative density is often used. Gravelly soils containing at least 50 percent gravel and graded up to coarse sizes can be considered to have adequate shear strength and satisfactory consolidation properties for most Reclamation structures if the in-place relative density is 60 percent or greater. In-place density tests (USBR 7205,

[7] 7015: Performing Penetration Resistance Testing and Sampling of Soil.

7206, 7215, 7220, 7221, or 7230)[8] and the index unit weight tests (USBR 5525 and 5530) can be used with USBR 7250 to determine the relative density of sand, sand-gravel, and gravel soils.[9]

As presented on figure 1-16, the relative consistency of saturated fine-grained soils can be estimated using data from the standard penetration test (USBR 7015).[10]

Cone penetration tests (USBR 7020 or 7021)[11] and pressuremeter tests (ASTM D 4719) [26] may be used to evaluate the adequacy of foundation soils. Both tests may be performed in cohesive or cohesionless soil deposits.

Hard, saturated fine-grained soils normally will support moderately heavy to heavy loads. Very firm soils usually are adequate for light loadings. When fine-grained cohesive soils are not saturated, data from most *in situ* test methods are misleading and should not be used for evaluating foundation soils for hydraulic structures because properties of the soils in a later saturated condition cannot be determined. In-place density tests provide a good index as to the supporting capacity of saturated or unsaturated fine-grained soils.

Fat clays (CH), lean clays with liquid limits above 40 percent, or high plasticity clay shales should be checked to determine whether they are sufficiently expansive to cause undesirable uplift of a structure. This can be accomplished by determining the gradation, plasticity index, and shrinkage limit of the soil. Any such soils falling into the "low expansion" grouping of table 1-2 are satisfactory.

When qualitative tests indicate doubtful support more detailed testing is necessary. Usually, this is accomplished by securing undisturbed soil samples and testing them in the laboratory.

When bottom grade of a structure is known to be below the water table, provisions are usually contained in the specifications for dewatering foundation soils and unwatering the excavation. When water is encountered, the inspector must ensure that foundation soils are not disturbed by removal of the water. This may be accomplished by beginning the dewatering operations as soon as ground water is encountered or at an elevation at least 1.5 m (5 ft) above bottom grade elevation. During dewatering operations, the water level should be maintained continuously at least 300 mm (12 in) below base grade until 48 hours after sufficient concrete has been placed to overcome uplift pressure. Dewatering may be performed by well points, deep wells, trenches, or sumps, or by other methods which will prevent upward movement of water through soils within the area of the structure foundation.

After excavation has proceeded to grade, care should be taken to ensure that foundation soils are not disturbed by equipment working in the area. Excessive drying or wetting may damage certain types of soils. Shales may require a cover of pneumatically applied mortar, soil, or a thin cover of concrete to prevent slaking; however, placement immediately after final cleanup is usually sufficient.

When a structure is designed for placement on impervious soils, the inspector should ascertain that the material is adequately impervious and that strata or lenses of permeable soil do not exist which might cause subsequent piping. Cutoff walls should be constructed to the depth shown on the drawings and into impermeable soils as required. Construction of cutoff walls into soil must be such that piping cannot occur around the walls. Normally, specifications require either that concrete for transitions and cutoff walls be placed against the sides of the excavation without use of intervening forms or that compacted backfill of silty or clayey soils be used between the concrete and natural ground after the forms are removed.

When the foundation area consists of more than one type of soil, care should be taken to prevent appreciable differential settlements. Often, pockets of soft or loose materials must be removed and be replaced with suitable compacted soil to provide uniform bearing over the entire foundation area. When a foundation area is partially on soil and partially on rock, sometimes it is necessary to remove the soil to rock and refill with lean concrete. At times, excavating additional rock and replacing it with compacted soil is desirable. The

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[8] 7205: Determining Unit Weight of Soils In-Place by the Sand-Cone Method.

7206: Determining Unit Weight of Soils In-Place by the Rubber Balloon Method.

7215: Determining the Unit Weight of Soils In-Place by the Sleeve Method.

7220: Determining Unit Weight of Soils In-Place by the Sand Replacement Method in a Test Pit.

7221: Determining Unit Weight of Soils In-Place by the Water Replacement Method in a Test Pit.

7230: Determining Unit Weight and Moisture Content of Soil In-Place — Nuclear Moisture-Density Gauge.

[9] 5525: Determining the Minimum Index Unit Weight of Cohesionless Soils.

5530: Determining the Maximum Index Unit Weight of Cohesionless Soils.

7250: Determination of Percent Relative Density.

[10] 7015: Performing Penetration Resistance Testing and Sampling of Soil.

[11] 7020: Performing Cone Penetration Testing of Soils—Mechanical Method.

7221: Determining Unit Weight of Soils In-Place by the Water Replacement Method in a Test Pit.

type of treatment depends upon the type of structure, structural loadings, and strength and compressibility of the soil.

c. Pile and Drilled Shaft Foundations.—When piles are required to provide desired bearing capacity or to minimize settlement, specifications contain requirements as to the type of piles, the cross-sectional area, and, in some cases, the length of the pile. Frequently, minimum depth of penetration is specified. A pile must never have a minimum penetration less than that shown on the drawings. Piles are of such length that the top of the pile will be at the specified elevation even after any damaged or battered portion of the top of the pile is cut off. Piles are required to be driven accurately to required lines and depths with either drop (gravity) hammers—single acting or double acting diesel hammers—or vibratory or hydraulic drivers. To reach the minimum penetration specified on the drawings, preboring or jetting prior to driving may be required. A usual requirement is that the number of jets and the volume of water, with adequate pressure, be available to freely erode material adjacent to the pile. When the minimum penetration called for is reached, the jets shall be removed and the pile finally set to grade using normal driving techniques. When holes are prebored, the diameter of the prebore holes shall be slightly smaller than the diameter of the pile, and the depth of the holes shall be such that the piles will reach the specified minimum penetration.

The pile driving specifications contain detailed provisions for:

- Shoes
- Cutoff
- Preboring or jetting
- Cutting
- Splicing
- Pile materials
- Preservative materials
- Preservative treatment
- Miscellaneous accessories

The inspector is required to ensure that all piles are located properly and are driven to specified lines, grades, and tolerances. The inspector must ensure that piles are not damaged during driving and that required penetration is obtained. Damaged piles or piles not driven to correct line and grade are ordered removed. When piles are in place in freezing weather, the top elevation should be checked before placing of concrete. If heaving has occurred, the piles should be redriven to grade.

Sometimes, static pile load tests are specified for determining bearing capacity or tensile capacity of single piles or pile groups. When pile load tests are required and, unless otherwise specified, they are in accordance with:

- ASTM D 1143: Piles Under Static Axial Compressive Load [27] (or USBR 7400: Testing Foundation Members Under Static Tensile Load)[12]
- ASTM D 3689: Individual Piles Under Static Compressive Load [28]
- ASTM D 3966: Piles Under Lateral Loads [29]

In special cases, other tests may be required; special instruction will be provided for those tests.

When pile load tests are performed, the inspector is responsible to verify that:

- Test piles are placed to specified line, grade, and length
- Driving records are secured
- Test loadings are applied in proper manner, amount, sequence, and period of time
- Data are obtained and recorded as required

Static load capacity of individual piles also may be determined using dynamic testing methods. When using dynamic test methods, transducers are mounted to the top of a pile to obtain force-time or velocity-time data from individual hammer blows. The data can be analyzed using wave equation theory to determine:

- Pile integrity
- Static load capacity (bearing capacity)
- Hammer performance
- Stresses in the pile during driving

Dynamic testing should be performed in accordance with ASTM D 4945: High-Strain Dynamic Testing of Piles [30].

Drilled shaft foundations (also called caisson, drilled pier, drilled caisson, or bored pile foundations) sometimes are used in lieu of pile foundations when it is necessary to transfer heavy foundation loads to deep, competent bearing strata. Drilled shaft foundations may be of constant diameter, tapered, or belled-out at the bottom. Belled-out shafts (piles) have been used in expansive clay as anchors to prevent structure uplift. Usually, shafts are drilled with large auger boring equipment, belled if needed, and cleaned;

[12] 7400: Testing Foundation Members Under Static Axial Tensile Load.

then, reinforcing steel and concrete are placed as specified. The inspector is responsible to ensure that:

- All loose soil is removed from the bottom of the shaft
- Soil at the bottom is competent for design loadings
- The bells, if required, are properly excavated in undisturbed material

Shafts may be cased or uncased—depending on soil and ground-water conditions. If water is encountered, the hole must be dewatered by approved methods until some specified time after the concrete is placed (usually 6 hours), unless tremie concrete is allowed in the specifications.

For more information on pile and drilled shaft foundations, see references [31, 32, and 33].

d. Backfill.—Backfill is specified in specifications variously as:

- Backfill about structures may be specified simply as backfill for which compaction is not required, or as compacted backfill for which the compacted dry density is specified.
- Backfill may be further classified on the basis of material required, for example, "cohesionless free-draining materials of high permeability, commonly called pervious material," or "clayey and silty materials of low permeability, commonly called impervious material."
- Compacted backfill is normally specified if backfill under structures is required.
- Compacted backfill may be specified as compacted clayey or silty materials for which compaction is done by tamping rollers, power tampers, or other similar suitable equipment.
- Compacted backfill may be specified as compacted cohesionless free-draining materials for which compaction is accomplished by thorough wetting accompanied by operation of surface vibrator, internal vibrator, tractor, or other similar suitable equipment.

Backfill, or compacted backfill is placed as shown on the drawings or where prescribed by the contracting officer. Type of material used, amount, and manner of depositing backfill are subject to approval of the contracting officer. Insofar as possible, backfill materials are normally secured from required excavation. When suitable materials required for a specific structure are not available from required excavation, the materials are secured from approved borrow sources. Distribution of material must be uniform and such that compacted backfill is free from lenses, pockets, streaks, or other imperfections.

When silty and clayey soils are placed as compacted backfill, required dry density of soil is 95 percent of laboratory maximum dry density (USBR 5500) unless otherwise specified.[13] Prior to compaction, these soils normally are required to have a uniform moisture content. When cohesionless, free-draining soils are placed as compacted backfill, required dry density of soil is that equivalent to 70-percent relative density (USBR 7250) unless otherwise specified.[14] Higher density requirements may be specified for fine sands for large structure loadings, when structure vibrations are severe, or when only very small settlements can be tolerated. After compaction, the depth of layers, unless otherwise specified, is 150 mm (6 in) when rollers are used, or not more than 300 mm (12 in) when vibrators or surface tractors are used. A normal requirement is that these fine sands be thoroughly wetted during compaction.

To provide adequate protection for compacted backfill about structures where backfill adjoins embankment, specifications often provide that the contracting officer can direct the contractor to place a sufficient amount of embankment material over the compacted backfill within a specified length of time (often 72 hours) after compaction of backfill has been completed. This provision is included so the backfill will not dry and shrink and cause cracking and pulling away from the structure.

Sand and gravel backfill material is often specified for placement:

- Under and about structures
- Under canal linings
- For filters
- At bridge approaches
- At weep holes, or against retaining walls
- For other purposes

Various gradations of material may be specified for different structural purposes. Laboratory tests should be made at frequent intervals to ensure that materials meet specified gradation limits. Unless otherwise specified, maximum particle size for backfill materials is 75 mm (3 in); and, the amount of material finer than the 75 µm (No. 200) sieve size for sand-gravel backfill is limited to 5 percent or less.

Generally, control techniques given for compacted earth embankments and compacted linings apply to backfill. Control of compacted backfill consists of:

---

[13] 5500: Performing Laboratory Compaction of Soils—5.5-lbm Rammer and 18-in Drop.

[14] 7250: Determination of Percent Relative Density.

- Inspecting materials
- Checking amount and uniformity of soil moisture
- Maintaining thickness of layers being placed
- Determining the percentage of laboratory maximum dry density or relative density obtained by compacting operations

Also, determination should confirm that the soils are fully compacted to specified lines and grades.

e. Filters.—Protective filters for canals and miscellaneous structures consist of one or more layers of free-draining soil placed on less pervious soil. The soil to be protected by the filter is commonly referred to as the base soil or the base material. The purposes of a protective filter are to:

- Safely carry off seepage from the base soil
- Prevent erosion or piping of the base soil
- Prevent damage to overlying structures from uplift pressure

The base soil can be protected by filter materials that have a certain range of gradation. Gradation of the filter material bears a definite relationship to the gradation of the base material. Filters may consist of a single layer or several layers, each with a different gradation. Multiple layer filters are known as zoned filters.

Material selected for the protective filter must satisfy four main requirements:

1. Filter material should be much more pervious than base material. This requirement prevents excess hydraulic pressures from building up in either the filter material or the base material.
2. Voiding the compacted filter material must be small enough to prevent base material from penetrating the filter. Penetration of the filter can cause the filter to clog with base material or piping of the base soil through the filter. Either condition can result in failure of the filter system.
3. The filter must be thick enough to provide good distribution of particle sizes throughout the filter to provide adequate hydraulic capacity for the volume of water flowing out of the base soil.
4. Filter material must be prevented from moving into drainage pipes by providing sufficiently small slot openings or perforations, or by additional coarser filter zones if necessary.

Filter design based on gradation criteria is generally credited to G.E. Bertram, with the assistance of Karl Terzaghi and Arthur Casagrande [34]. Over the years, studies by the U.S. Army Corps of Engineers [35], Reclamation [36, 37], and the Natural Resources Conservation Service [38] have produced various, and sometimes conflicting, filter criteria. Stability criteria are the oldest criteria and are the only criteria having successfully been demonstrated in laboratory studies; they are the only criteria that seem to be universally accepted. Stability criteria (also referred to as the stability ratio) state that the ratio of the D_{15} size of the filter soil to the D_{85} of the base soil must be equal to or less than 5. The stability ratio has been shown to be conservative but effective in preventing penetration of the base soil into or through the filter.

Permeability of filter material should be at least 25 times larger than permeability of the base material. Generally, permeability criteria are satisfied if the D_{15} size of the filter material is equal to or greater than 5 times the D_{15} size of the base material.

In addition to stability and permeability criteria, filters should satisfy the following requirements: ([14], ch. 5; and [35, 38]).

1. Filter material should be clean, cohesionless sands or gravels with C_u (coefficient of uniformity) between 1.5 and 8. Filter material should pass the 37.5-mm (1-1/2 in) sieve to minimize particle segregation and bridging during placement. Smaller maximum particle sizes may be specified. Filters must have less than 5 percent minus 75 µm (No. 200) sieve size material to prevent clogging caused from excessive movement of fines in the filter and drainage pipes and to maximize permeability of the filter.
2. Filter material adjacent to drainage pipes should have sufficient coarse sizes to prevent movement of filter material into the drainage pipes. The maximum size of perforations or joint openings of drainage pipe is selected as one-half of the D_{85} grain size of the filter material.

The specifications will normally describe the following requirements for filter construction:

1. Before filter placement, the base soil should be firm and, if necessary, be lightly tamped or rolled.
2. Clean filter material should be protected from contamination during and after placement; the placement method should minimize segregation in the filter.
3. Filters are compacted to 70-percent relative density unless otherwise specified, in a manner similar to free-draining sand-gravel backfill to prevent settlement.

4. Filter layers are often specified at a 150-mm (6-in) minimum thickness. However, for extreme conditions such as high heads, variations in base material, or filter gradations that are near the extreme coarse limit, a minimum thickness of 200 mm (8 in) may be specified. For zoned or graded filters these minimum thicknesses may be specified and are maintained for each layer.
5. Where drainage pipe is used in a filter system, hydraulic capacity of the pipe should be sufficient to collect the expected volume of seepage water and to convey it to a place for discharge.
6. While drainage pipe is being laid, the openings are often protected from inflow of fines from the filter material.

Figure 3-41 shows an example using filter criteria. For additional information on filter design, see references [38, 39]. Often ASTM C-33 concrete sand can serve as a protective filter material for many materials.

Recently, geosynthetic materials (geotextiles, geonets, and geocomposites) have gained wide acceptance for use as filters and drains in civil engineering works. These specialized materials have been used successfully in:

- Landfill liner systems
- Highway edge drains
- Retaining walls and basement walls drains
- Highway subgrade drains
- Strip or wick drains

Design methods for these materials, while similar to the design of soil filters, require specialized testing to determine required properties of the geosynthetics. Design methods and applications for these special materials is discussed in reference [40].

H. Stabilized Soils

3-39. General.—Soil stabilization is the chemical or mechanical treatment of soil to improve its engineering properties. Chemically stabilized soils consist of soil and a small amount of additive such as cement, fly-ash, or lime. The additive is mixed with the soil, and the mixture is used in compacted fills, linings, or blankets. Quality and uniformity of the admixture and the uniformity of moisture are closely controlled to produce a high quality end product. Therefore, processing equipment and procedures are specified to ensure that the relatively small amount of additive is uniformly distributed throughout the soil mixture before placement and compaction. Uniformity of soil in the mixture is a major factor in controlling desired uniformity of the final product; and soil gradation, plasticity, and moisture content should be controlled prior to mixing with the additive—or prior to adding the stabilizer for in-place mixing.

A mixing plant must be calibrated over the range of soil gradation stated in the specifications. Adjustments to the mixing plant should then be made during construction to accommodate variation in soil gradation or for other variable conditions. These adjustments are based on mixing plant calibrations obtained before and during construction.

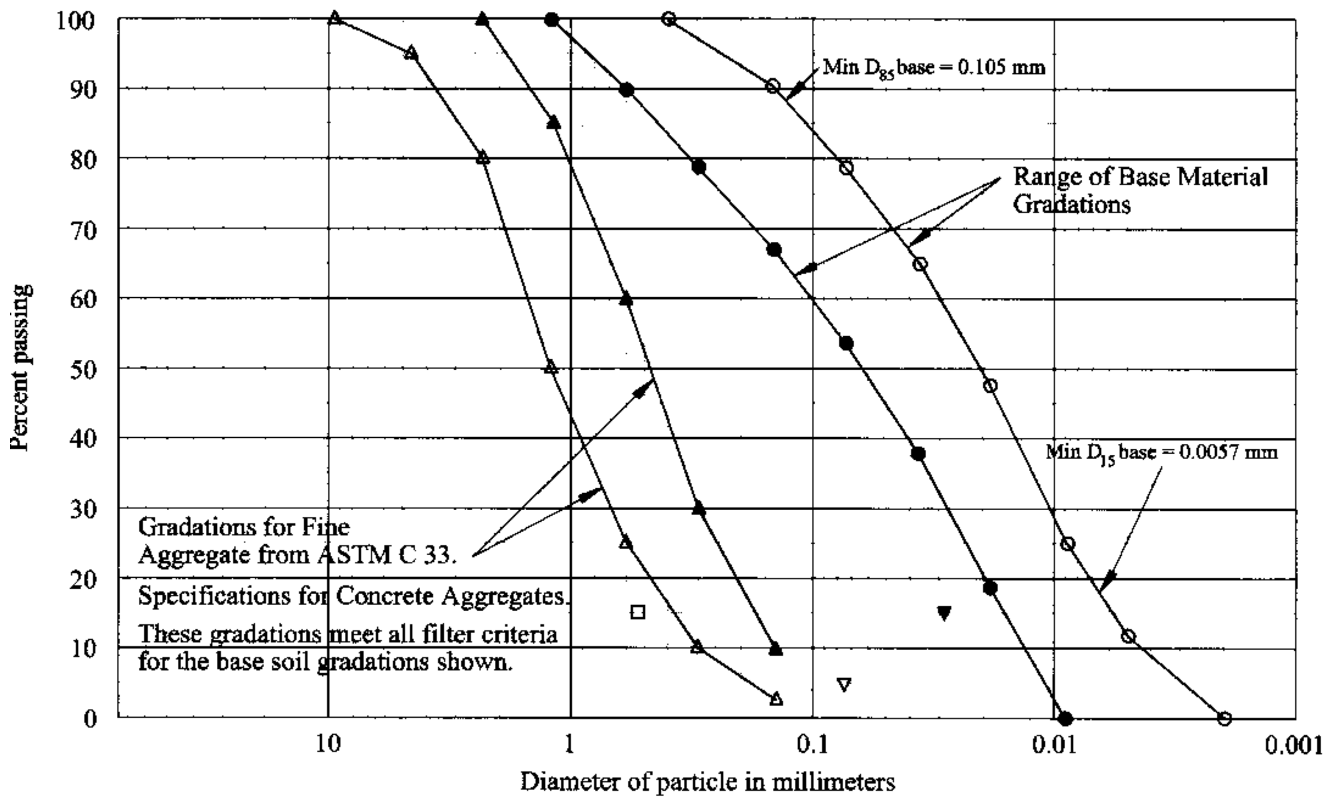
Sometimes, soils are stabilized to deeper depths by grouting or by injection methods to solve particular foundation problems (see sec. 3-19). These methods for stabilization require specialized knowledge and understanding of the materials being used, and quality control procedures are specifically developed for the particular situation.

3-40. Compacted Soil-Cement.—

a. Design Considerations.—Compacted soil-cement has been used as upstream slope protection for embankment dams and as blankets, linings, or other applications. Soil-cement used as upstream slope protection is placed in successive horizontal layers ranging from 150 to 300 mm (6 to 12 in) in compacted thickness to protect the slope from wave action. The layers are placed successively up the slope, and the outer edges form a stairstep pattern. When soil-cement is placed as a blanket or lining it is usually placed in layers up to 600 mm (2 ft) thick with the layers parallel to the slope. Although roads are a minor part of Reclamation work, compacted soil-cement has been used extensively for construction of road bases by others. Figures 3-42 and 3-43 show soil-cement facings constructed on the upstream slope of two Reclamation structures.

To satisfy design requirements for slope protection, a layer must be:

1. Formed into a homogeneous, dense, permanently cemented mass that fulfills the requirements for compressive strength
2. In intimate contact with earth slopes, abutments, or concrete structures



Plot symbol	Gradation ranges
○	Base material - finest
●	Base material - coarsest
△	Filter material - coarse limit of ASTM C 33 Fine Aggregate
▲	Filter material - fine limit of ASTM C 33 Fine Aggregate

Plot symbol	Filter design requirements
▽	Limit of fines in filter = 5% passing 0.075 mm
■	Maximum particle in filter = 100% passing 75 mm
▼	Permeability criteria : D_{15} filter > or = 5* D_{15} base D_{15} filter > of = 0.029 mm
□	Stability criteria : D_{15} filter < or = 5 * D_{85} base D_{15} filter < of = 0.525 mm

Figure 3-41.—Filter gradations.

3. Durable and resistant to "wetting and drying" and "freezing and thawing" action of water
4. Stable with respect to the structure and of sufficient thickness (mass) to resist displacement and uplift

Performance of soil-cement facings on Reclamation dams has generally been excellent. However, inspections of these facings have revealed that the bond between lifts is a weak point in the facing. Test results on cores taken from these faces show that the bond is much weaker than the remainder of the soil-cement [41, 42]. Since the layers are not well bonded, they perform as a series of nearly horizontal slabs on the slope of a dam. Each slab is offset from the previous one by a distance equal to the layer thickness multiplied by the slope of the dam. If the layers were well bonded, the entire facing would act as a massive unit instead of as individual layers, and damage by wave action would be greatly minimized.

Several studies have been performed to identify methods for enhancing bond between layers. Currently, the most promising method investigated is to apply a water-cement slurry to a layer just before placing the overlying layer. This technique was used at Davis Creek Dam in Nebraska to

Figure 3-42.—Soil-cement facing at Lubbock Regulating Reservoir in Texas.

improve bonding between layers and to improve overall durability of the facing for an elevation of about 6 m (20 ft) at normal reservoir operating level.

b. Construction Provisions.—The specifications describe the type and amount of cement, the quality and amount of water, and the borrow area for soil or aggregate. The permissible range of soil and aggregate gradation also is specified. If investigations show the deposit is variable, selective excavation and processing may be required to produce uniformity. Oversize particles and other objectionable materials must be removed.

A stationary mixing plant is usually required. Either a batch type or a continuous-feed pugmill type plant is acceptable. Control over mixing time; positive interlocking of cement and soil flow; and controls for accurately proportioning soil, cement, and water—all from appropriate storage—should be incorporated into the plant design. A mixing plant used at Lubbock Regulating Reservoir in west Texas is shown on figure 3-44.

Trucks for transporting the soil-cement mixture should have tight, clean, smooth beds and protective covers. The soil-cement spreader used for laying the soil-cement must produce a smooth uniform loose layer of required width and thickness. Usually, layers are placed horizontally; however, a slope toward the outer edge as steep as 8h:1v is sometimes permitted to increase working width. The maximum time for hauling and spreading the soil-cement after mixing is usually specified as 30 minutes. The tractor and spreader box used for placing soil-cement on Merritt Dam and Reservoir in Nebraska is shown on figure 3-45.

Generally, compaction must be completed within 60 minutes after spreading with no more than 30 minutes between operations. Compaction is accomplished by several passes of a sheepsfoot tamping roller, followed by several passes of a pneumatic-tire roller, as shown on figure 3-46. The minimum number of passes by each roller should be determined by constructing a test section. The rollers should have provisions for ballast loading so the masses can be adjusted to provide optimum compaction. Vibratory, smooth

Figure 3-43.—Soil-cement facing at Merritt Dam in Nebraska.

steel drum rollers have been used to compact coarse grained soil-cement. A combination of vibratory and static mode may be used. The roller mass and the frequency and amplitude of vibration should be set for optimum compaction without damage to the soil-cement and to minimize localized shear failure in the upper part of a layer. A test section should always be constructed to determine: optimum equipment usage, roller mass, and number of passes—and vibratory characteristics if a vibratory roller is used. Rollers may be towed or self-propelled. After compaction, the compacted layer is cured by keeping the exposed surfaces continually moist using a fog spray until the overlying or adjacent layer is placed, or for a minimum of 7 days. A blanket of moist earth may be used for permanently exposed surfaces. The surface of a completed layer may require brushing to remove soil or other debris just before placing an overlying layer, as shown on figure 3-47.

c. Control Techniques.—Compacted soil-cement is similar to compacted earthwork in that careful observations and additional control tests are required during the early stages to check planned control against results obtained under field conditions. These observations and tests are used to establish placing conditions and to develop procedures for use in the remaining construction. Control should begin during excavation and stockpiling of the soil to ensure that the material is within gradation requirements and has a uniform moisture content. Gradation and moisture content are controlled by:

- Directing the excavation
- Mixing the stockpile by spreading and cross-dozing
- Sloping the stockpile surfaces to provide runoff without water catchments
- Sampling and testing during stockpiling

After the mixing plant is set up, the cement, soil, and water feeds must be calibrated individually to establish curves (or tables) of equipment settings versus quantity produced. The plant must be calibrated over the full range of anticipated production rates. A cement vane feeder and soil feed belt to a pugmill are shown on figure 3-48. Calibration is accomplished by timing and weighing quantities of moist soil, cement, and water to check feed and meter settings. To facilitate computation of dry soil production, moisture content of the wet soil should be determined at the plant by

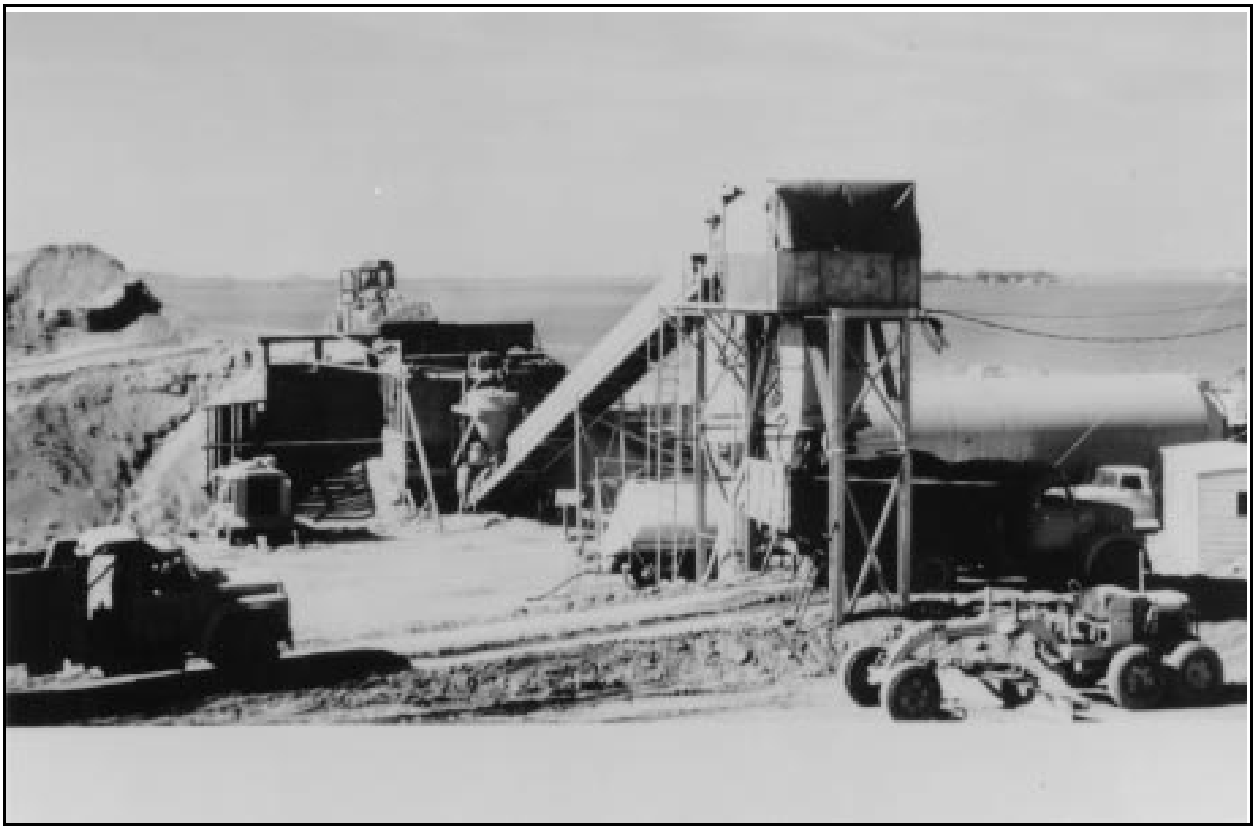


Figure 3-44.—Soil stockpile and pugmill for mixing soil-cement at Lubbock Regulating Reservoir in Texas.

a quick method, such as the calcium carbide reaction device (USBR 5310), a microwave oven (USBR 5315), or the Moisture Teller device (USBR 5305).[1] The cement feed is perhaps the most crucial calibration because of variation in the amount of cement can critically affect the properties of soil-cement. The cement feed is calibrated over the range of required cement content by varying the feeder speed. The cement feed is usually quite consistent because cement is a fairly uniform product and is fed under reasonably uniform conditions in a well-designed plant. Cement feed calibration allows accurate adjustment if soil feed rates change during progress of the job. Plant performance and production is checked under full load by:

- Timing and weighing truckloads of the soil-cement mixture
- Checking mixing time
- Inspecting the mixture for:

  - Uniformity in texture
  - Moisture
  - Distribution of cement

The soil and soil-cement mixture is sampled to determine:

- Cement content
- Moisture content by both quick and ovendry methods
- Occurrence of "clay balls" (rounded balls of clayey fines and sand which do not break down during ordinary processing)

During construction, the inspector should check plant operation periodically. Inspection frequency will depend on performance of the plant and uniformity of soil gradation and moisture content. The soil feed rate should be checked at the beginning of each shift by timing and weighing a truckload of moist soil. By using a quick method to determine moisture content, the feed rate of dry soil can be computed, and the proper cement and water feed rates can be set. At regular intervals, the moisture content of the soil and soil-cement mixture should be determined by a quick method to

[1] 5310: Determining Moisture Content of Soils Using the Calcium Carbide Reaction Device.
5315: Determining Moisture Content of Soils by the Microwave Oven Method.
5305: Determining Moisture Content of Soils Using the Moisture Teller Device.

Figure 3-45.—Soil-cement placement at Merritt Dam in Nebraska.

provide a basis for making adjustments in the water feed if necessary. Also, the inspector does a check calibration each time a record control test is performed by timing the production and weighing a truckload of soil-cement. Based on water content and cement feed rate, the soil feed rate can be determined. The soil and soil-cement are sampled for moisture tests at the plant and for record tests in the laboratory.

The placement inspector should observe placing and compaction procedures and should verify that loose lift thickness, texture, and surface are uniform and that the lift is to specified dimensions.

Compaction begins with the roller starting at the outer edge. The number of passes and adequacy of overlap should be checked. When using a sheepsfoot roller, each pass over the entire width of the lift should be completed before the next pass begins, as the tamping feet tend to follow previous tracks if successive passes are made without overlap. Because of the sandy material, the sheepsfoot roller usually does not "walk out" completely but should begin to "walk out" on the last passes. Each layer is completed by compaction with a pneumatic-tire roller. This is in contrast to normal earthwork where successive layers are compacted together with a sheepsfoot roller. With the time limits used in soil-cement construction, it is usually not feasible to place successive layers quickly enough to compact them together; hence, each lift must be compacted individually.

The pneumatic-tire roller also starts compaction at the outer edge of the lift to minimize the amount of lateral spreading of the soil-cement. Smooth, even surfaces after compaction are desired. Rutting usually indicates a high placement water content. Thickness and width of the completed compacted lift should be checked. The placement inspector should be sure that the surface of the compacted layer is kept continuously moist until the overlying layer is placed, or for a minimum of 7 days. If required, the surface should be thoroughly cleaned by brooming (brushing) just before placing an overlying layer (fig. 3-47).

Access ramps for trucks hauling soil-cement are built of soil on the soil-cement facing; these should also be checked to verify that they satisfy specified thickness requirements [usually 600 mm (2 ft)]. Ramps are constructed to adequately protect the edge of the top layers on which soil-cement is being placed. Inadequate ramps will result in

Figure 3-46.—Compaction of soil-cement at Lubbock Regulating Reservoir in Texas.

damage to the outside edges of soil-cement layers and decrease durability of the finished product.

d. Control Testing.—During construction, control testing is required for every 500 m^3 (500 yd^3) of soil-cement placed, or a minimum of one test per shift. The control test consists of timing a sample through the process to ensure that specified time constraints are met. Times recorded are (1) when the truck leaves the plant, (2) when the load is spread, (3) when compaction is completed, and (4) when the control test is taken in the compacted material. Procedures for performing construction control of soil-cement are outlined in USBR 5830.

Control testing begins by timing the production of a truckload of soil-cement and obtaining a representative sample of soil from the soil feed. The mass of the timed truckload is determined, and a representative sample of soil-cement is obtained for laboratory testing. When the load is spread, the approximate center of the load is marked by the placement inspector. Moisture contents of the soil and soil-cement are determined by a quick method at the time of sampling, and the remainder of the samples are used for laboratory tests.

For the soil, the percentage of fines is determined, and the moisture content is obtained by the standard ovendrying method. The cement content of the soil can be determined by chemical titration. If the untreated soil contains a significant amount of calcium, a chemical analysis of the soil is performed. The presence of calcium should have been determined when the soil was being stockpiled. A complete gradation and specific gravity should be determined on specimens from every fourth control test.

For the soil-cement, the percentage of "clay balls" is determined, and the percentage of cement is determined by either the heat of neutralization method or the chemical titration method. A three-point compaction test is performed using the rapid method of construction control. The compaction test should be performed at the same time that material (from the sampled truck load) is being compacted. This is necessary to allow for time effects on compaction properties of soil-cement caused by hydration of cement.

Figure 3-47.—Power brooming of surface prior to placement of next lift at Lubbock Regulating Reservoir in Texas.

The remainder of the soil-cement sample is used to prepare three or four compression test specimens to be tested at ages 7, 28, and 90 days. One 7-day and two 28-day specimens should be formed for each control test with an additional specimen for a 90-day test for every fourth control test. Specimens tested at 90-days are for design verification only and not for construction control. Specimens are placed at the density determined from the field density test of the compacted soil-cement layer. These specimens should be formed as soon as possible after the field density has been determined. Procedures for preparing specimens for compressive strength testing are discussed in USBR 5806 and testing of soil-cement cylinders is described in USBR 5810.

A field density test is performed at the point marked by the placement inspector when the timed load was spread. This test should be performed as soon as compaction is complete. Care should be taken so the test is not performed where roller overlap has occurred. When the test hole is complete, but before determining the volume, the depth of the lift is measured and recorded.

Control testing includes obtaining record cores from the compacted soil-cement at least 28, 90, and 360 days after completion. One hole should be drilled for each 5,000 m^3 (5,000 yd^3) placed. Locations of core holes should be spaced to be representative of the area covered, with some cores near abutments or structures. In drilling, care should be exercised to obtain a continuous core; comments on bond strength should be included with other information accompanying the cores. The core holes should be carefully backfilled with cement grout and a reinforcing bar placed flush with the surface and located for future reference. Record cross sections of the compacted soil-cement are obtained at locations where 28-day core samples are taken. The compressive strength of a section of core from each of the holes should be determined in the field laboratory; this strength is compared to those of the record construction control cylinders representative of the immediate vicinity. The remainder of the cores should be sent to the laboratory for durability, direct shear, and compression tests. A summary of compressive strength and age of record core, and compressive strength of specimens of material in the immediate vicinity, should accompany the cores. Procedures for performing record coring and cross sectioning of compacted soil-cement are detailed in USBR 5835.

3-41. Compacted Soil-Lime.—

*a. General.—*Use of lime as a soil additive is the oldest known method of chemical stabilization; it was used by the Romans to construct the Appian Way. Soil-lime is a mixture of soil (usually clay), lime, and water which is compacted to form a dense mass. Experience has shown that mixtures of most clay soils, either quick or hydrated lime, and water will

Figure 3-48.—Cement vane feeder and soil feed belt to pugmill at Cheney Dam in Kansas.

form cementitious products in a fairly short period of time. Reclamation applications for water resources work have been limited to use of lime to stabilize expansive clay soils and dispersive clay soils.

b. Adding Lime to Highly Plastic Clay Soils.—Adding lime to highly plastic clay soils produces several effects on physical properties:

1. The liquid limit decreases, and the plastic limit increases, radically decreasing the plasticity index (sometimes, by a factor of 4 or more)
2. The finer clay-size particles agglomerate to form larger particles, which makes the soil more friable and easier to work. By absorbing water, lime also assists in breaking up clay clods during mixing.
3. Lime dries the soil by absorbing water to hydrate the lime and makes wet soils easier to handle and compact.
4. Unconfined compressive strength increases many fold.

Reclamation's Friant-Kern Canal, California, was constructed during 1945-51; about 87 km (54 mi) of the canal traverse an area of expansive clay. Of these 87 km, 37 km (23 mi) are earth lined, and the remaining 50 km (31 mi) are concrete lined. After 3 years of operation, portions of the canal traversing expansive clay soils began cracking, sloughing, and sliding with failures occurring in both the concrete-lined and earth-lined sections. Because these conditions caused continuing, expensive maintenance problems, in 1970, rehabilitation began for the worst failed areas. Lime stabilization was selected as the most effective method of treatment [43].

Riprap that had been dumped into slide areas was removed. Then, all material to be stabilized with lime and recompacted was removed by a benching operation. A series of long sloping benches or ramps were cut from the top of the bank down to the canal bottom, with the cut extending far enough into the slope to remove the entire depth of required excavation material. Two-percent quicklime was spread over the bench surface, and 0.3 m (1 ft) of material from the bench was mixed with the quicklime, and the lime-clay mixture was pushed into the canal bottom. The material was spread on the canal bottom and an additional 2 percent lime was added. Water was added to at least 2 percentage points above optimum moisture, and about 0.3 m depth of material was mixed with dozers and graders. After about 2 m (6.6 ft) of material had been mixed and cured for 24 hours, dozers began spreading the material on the slopes, which were then compacted with a sheepsfoot roller moving up and down the slope. The side slopes were constructed in three compacted lifts for a 1.1 m (3.6 ft) compacted depth normal to the slope.

In subsequent rehabilitation work, the lime-treated soil was placed and compacted in successive horizontal layers stepped up the slope in the same manner in which soil-cement is placed on the face of an earth embankment. This placement method was found much more desirable than placing material parallel to the slope. Placing and compaction were much more efficient, and the finished product was of higher quality.

The amount of lime used was controlled by periodically placing a canvas on the ground where lime was to be spread, and after spreading, mass of lime was determined for a given area.

Four-percent lime by dry mass of soil was used during rehabilitation of Friant-Kern Canal; compressive strength increased to 20 times that of untreated soil. The rehabilitation has proved durable after about 20 years of additional service.

c. Adding Lime to Dispersive Clay Soils.—Dispersive clay soils are those that erode in slow-moving or even quiet water by individual colloidal clay particles going into suspension and then being carried away by the flowing water. A concentrated leakage channel (crack) must be present for erosion to initiate in dispersive clay. This mechanism is totally different than that for piping where erosion begins at the discharge end of a leak and progresses upstream through the structure until it reaches the water source.

The design lime content for controlling dispersive clay soils is generally defined as the minimum lime content required to make the soil nondispersive. In addition, it may be desirable to increase the shrinkage limit to near optimum water content to prevent cracking from drying when using lime-treated soil in surface layers. In all known cases investigated to date, dispersive clay soils were made nondispersive by addition of 1 to 4 percent lime by dry mass of soil. However, in construction specifications, the design lime content is often increased 0.5 to 1.0 percent to account for losses, uneven distribution, incomplete mixing, etc.

Dispersive clay soils were identified throughout the borrow and foundation areas during investigations for McGee Creek Dam, Oklahoma; it was determined to be practical to stabilize these soils with lime during dam construction. Since dispersive clay soils were not concentrated in specific areas, lime treatment was considered more economical than attempting to identify the randomly occurring dispersive clays and selectively wasting them. Treating these clay soils with lime rendered them nondispersive and allowed their use in constructing the embankment-foundation contacts as erosion resistant material on the downstream slope of the embankments, and for placement as specially compacted backfill in areas of high piping potential such as along conduits through the embankment. Material was used directly from the borrow areas to construct the remainder of the dam and dike embankments. Specially designed filters were used to guard against any possible erosion of dispersive clay soils in untreated areas. Another benefit of using lime-treated soil was the improved workability of some of the highly plastic clay soil encountered.

d. Construction Procedures for Lime.—Treated Dispersive Clay Soils.—The following general procedures have proved satisfactory for handling, mixing, and placing lime-treated soil.

Soil to be lime treated is pulverized in a high speed rotary mixer or with a disk harrow prior to applying lime, and the moisture content is brought to within 2 percent of optimum. Lime is uniformly spread on the pulverized soil to the specified percent lime by dry mass of soil. Lime is mixed with the soil using a rotary mixer, and additional water is added as necessary to again bring the mixture to within 2 percent of optimum (or other specified value). When mixing is completed, the soil-lime mixture is cured for at least 96 hours before placing and compacting. Exposed surfaces of the mixture are either lightly rolled to prevent moisture loss or the mixed material is stockpiled and the surface sealed.

Each section of the foundation is carefully prepared coincident with final mixing and pulverization of the lime-treated material. The soil-lime is mixed until 100 percent passes the 25 mm sieve and 60 percent passes the 4.75-mm sieve (1 in and No. 4). Immediately after final mixing, the lime-treated earthfill is placed and compacted in horizontal lifts of no more than 150 mm (6 in) after compaction. The material is compacted to no less than 95-percent laboratory maximum dry density, using a tamping roller followed by a pneumatic-tire roller. The top of each compacted lift is scarified or disked before the next lift is placed [44].

The following items should be monitored to ensure high quality earthwork construction control:

1. Soil pulverization (gradation)
2. Lime content
3. Soil dispersivity (before and after lime treatment)
4. Compacted density
5. Moisture content of both soil and soil-lime mixture

If embankment materials are placed on the compacted lime-treated earthfill within 36 hours, special curing provisions are not required. Otherwise, the exposed surface of the lime-treated earthfill is compacted with a pneumatic-tire roller to seal the surface, and it is sprinkled with water for 7 days or until embankment material is placed.

Construction control testing is the same as that for other earthwork.

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