Quality Control of Soil Compaction

Using ASTM Standards

ASTM INTERNATIONAL
Standards Worldwide

Quality Control of Soil Compaction Using ASTM Standards

Prepared by Committee D18 on Soil and Rock

ASTM Stock Number: MNL70

ASTM International
100 Barr Harbor Drive
PO Box C700
West Conshohocken, PA 19428-2959

Printed in U.S.A.

Library of Congress Cataloging-in-Publication Data

Talbot, James R., 1938-
Quality control of soil compaction using ASTM standards / James R.
Talbot ; prepared by Committee D18 on Soil and Rock.
p. cm.
"ASTM stock number: MNL70."
Includes bibliographical references and index.
ISBN 978-0-8031-7023-0
1. Fills (Earthwork)–Testing. 2. Fills (Earthwork)–Standards. 3. Soil compaction–Quality control.
I. ASTM Committee D-18 on Soil and Rock. II. Title.
TA715.T255 2011
624.15–dc23 2011012808

Printed in Bridgeport, NJ
October, 2011

Foreword

THIS PUBLICATION, ***Quality Control of Soil Compaction Using ASTM Standards***, was sponsored by Committee D18 on Soil and Rock. This is Manual 70 of the ASTM International manual series.

Acknowledgments

This manual is supported by subcommittee D18.08. The following members of that subcommittee submitted material to start the process of reviewing and editing for content in the manual:

James R. Talbot, USDA, Soil Conservation Service
Amster K. Howard, USDI, Bureau of Reclamation
Keith Rademacher, Chem Nuclear Geotech
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Raphael A. Torres, California Dept. of Water Resources
Jeff Farrar, USDI, Bureau of Reclamation
James Talbot and Jeff Farrar collaborated on the initial editing.

In 1999 on July 1–2, committee D18 sponsored a symposium titled "Constructing and Controlling Compaction of Earth Fills". The symposium was held in Seattle, Washington.

The symposium produced STP 1384 which was published in 2000.

Subcommittee D18.08 members Donald Shanklin, Keith Rademacher, and James Talbot were the editors of STP 1384. The final session of the symposium featured a review and discussion of the proposed manual, entitled, "Testing Compaction of Earth Fills Using ASTM Standards".

The final editing of the manual was passed to the Chairman of D18, Terry Hawk and Christopher Hardin. With the sudden death of Terry Hawk in 2004, the uncompleted manual was sent to Donald Shanklin. The bulk of the work remaining was putting together the visual aspects of the manual. This was accomplished with the help of Wendy Pierce, a computer graphical artist for USDA, Natural Resources Conservation Service. Jeff Farrar was also helpful in supplying visual materials from the Bureau of Reclamation.

Beginning with Terry Hawk, then Jim Horton, and finally Ron Ebelhar, all these Committee D18 Chairmen, supported the effort to complete this work. In addition, Bob Morgan, ASTM Staff Manager for D18, has been a continual supporter. Kathy Dernoga, ASTM Managing Editor for Books and Journals has been with the project from the very beginning and finally gets to see a product.

Dedication

This publication, "Quality Control of Soil Compaction Using ASTM Standards," is dedicated to the memory of former Committee D18 Chairman, Terry Hawk. Terry had risen to the leadership of Committee D18 through his 20 years of exemplary hard work and quality performance. He sometimes faltered in pronouncing the names of those receiving awards at Main Committee Meetings, but never faltered in his dedication and performance to the work of an ASTM volunteer. Terry rescued the "Compaction Manual," as it was commonly referred to, and recruited a young engineer, Chris Hardin, from Geo-Environmental Engineering, to work with him and tackle the final editing to keep the project moving forward. They completed the editing and identified most of the visuals needed for the manual. Then, suddenly, on January 24, 2004, Terry Hawk died, unable to complete the project he believed in and had nurtured along. Rest easy, Terry.

Contents

List of Referenced ASTM Standards

ASTM **C127**	Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate
ASTM **D558**	Standard Test Methods for Moisture-Density (Unit Weight) Relations of Soil-Cement Mixtures
ASTM **D559**	Standard Test Methods for Wetting and Drying Compacted Soil-Cement Mixtures
ASTM **D560**	Standard Test Methods for Freezing and Thawing Compacted Soil-Cement Mixtures
ASTM **D698**	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3))
ASTM **D1556**	Standard Test Method for Density and Unit Weight of Soil in Place by the Sand Cone Method
ASTM **D1557**	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))
ASTM **D1558**	Standard Test Method for Moisture Content Penetration Resistance Relationships of Fine-Grained Soils
ASTM **D2166**	Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
ASTM **D2167**	Standard Test Method for Density and Unit Weight of Soil in Place by the Rubber Balloon Method
ASTM **D2216**	Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM **D2435**	Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading
ASTM **D2487**	Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM **D2488**	Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
ASTM **D2850**	Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils
ASTM **D2937**	Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method
ASTM **D3017**	Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods (Shallow Depth) (Withdrawn 2007)
ASTM **D3665**	Standard Practice for Random Sampling of Construction Materials
ASTM **D4253**	Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table
ASTM **D4254**	Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density
ASTM **D4564**	Standard Test Method for Density and Unit Weight of Soil in Place by the Sleeve Method
ASTM **D4643**	Standard Test Method for Determination of Water (Moisture) Content of Soil by Microwave Oven Heating
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ASTM **D4944**	Standard Test Method for Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester
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ASTM **D5080**	Standard Test Method for Rapid Determination of Percent Compaction
ASTM **D5084**	Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter
ASTM **D6938**	Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)

1

Introduction

A. PURPOSE AND SCOPE OF THIS MANUAL

This manual has been prepared to provide guidance to design and construction engineers, technicians, and earthwork construction personnel on the use of ASTM Standard Test Methods for testing compaction of earth fills and other earthwork. The manual includes a discussion of the types of fills in which compaction is used and the theoretical background of compaction. In addition to the theoretical use of ASTM Standard Test Methods, this manual also includes discussion on the overall purpose of compacting fills and the potential negative effects on the engineering properties of compacted materials caused by the improper application of compaction test methods.

It is important for engineers and other technical professionals involved in the placement and testing of fills to understand that ASTM Standard Test Methods have been developed to determine the compaction characteristics of soil and soil/rock materials and establish reference values for density and water content. These reference values are used to determine the density for preparing soil specimens for engineering property tests in the laboratory or to compare with field test results for checking the compaction of earth fills. ASTM Standard Test Methods are also used to establish values for the unit weight or density and the water content of in-place soils, including foundation subgrades, borrow areas, and embankment fills. The test results can be used independently for making judgments on soil properties and characteristics or jointly to make comparisons for the proper control of water content and percent compaction of soil during construction.

This manual was developed to provide a summary of the important items that every experienced earthwork engineer and technician should know and understand to successfully complete an earthfill construction project. It also represents a summary of the skills and terminology that the authors of this manual would want new engineers and technicians to understand and appreciate to obtain quality earthwork construction. Earthfill construction and compaction testing is simple and complex, depending on the nature of the conditions encountered. The most important lesson that can be learned for any professional engineer involved in the placement and compaction of fill material is that soil and rock frequently provide situations that cannot be reliably predicted using solely technical methods. Nonhomogeneous site and soil conditions are frequently the rule instead of the exception when dealing with compacted fills. As frequently described by the early practitioners of soil mechanics, the constantly changing nature of soil and rock continually places engineers and technicians involved with earthwork construction at the “borderline between science and art.”

Into the challenging interface of science and art that is required for successful earthwork construction, ASTM has successfully implemented, for more than 20 years, standard test methods that are used for determining the reference values for the density and water content of soil and rock. These standard test methods were developed as consensus guides by experienced professionals in the geotechnical community and are primarily used to determine in-place density and moisture content of materials for engineering design purposes. By using the technical information provided by the standard test methods, and their previous experience on fill projects, engineers are then able to evaluate whether a fill has been placed to meet the performance requirements of the project. To successfully implement the standard test methods requires the ability to use good engineering judgment for the application of the test methods, the selection of the compaction techniques, and for the evaluation of the skill level of the on-site personnel. This manual was developed to provide a description of the strengths, limitations, and applications of the ASTM Standard Test Methods so that engineers and technicians can make better judgment calls during the completion of earthwork projects.

It is fair to say that the experienced contributors to this manual had at least some difficulty in determining the most important items that needed to be communicated to their fellow professionals involved with fill placement, compaction, and testing. In consideration of these difficulties, it is important to note that this manual is not intended to provide the following:

- This manual is not a “how-to” manual that can be used to solve all problems with earthfill placement and compaction.
- This manual is not a specification guide. The principles and explanation provided in this manual will be useful for specification writing, but the authors have purposely not provided look-up tables and other standard language that could be misleading if applied without qualified engineering judgment.
- Although this manual references the ASTM Standard Test Methods, it does not provide enough detail to tell a technician how to perform an ASTM test.
- This manual is not an introductory textbook on soil mechanics or soil testing, although we anticipate that the information provided will be useful for training and application of the test methods.
- Most important, this manual is not a manual on how to be a geotechnical engineer or how to practice engineering, although one of the goals of the manual is to assist practicing engineers on proper use and application of the ASTM Standard Test Methods used for fill placement and compaction control.

B. GENERAL HISTORY OF SOIL COMPACTION AND METHODS OF CONTROL

One of the most important skills for any engineer or technician involved in the observation and testing of earthfill projects is to develop and maintain a deep appreciation for the history of earthfill construction and the corresponding test methods. Earthfill construction is one of the oldest methods of construction known to humankind and has included innovation and input from many continents and cultures.

The first known earthwork construction was for roads, which dates back to approximately 3000 B.C. in China. The historical information available does not make it clear whether compaction principles were used or understood at this time. Some records exist for highway construction in the mid-1600s and later in 1747 in France when an engineering school was established for educating engineers in highway design and construction. The early instruction dealt mainly with the types of soil that were best suited for construction and provided explanation in descriptive terms common to that time.

There is evidence that some methods of compaction were used in the United States in the late 1800s and early 1900s. It appears that the use of compaction evolved mainly because it improved the performance of roads and other structures. This compaction was accomplished mainly by routing hauling equipment or machinery over the fill. With the invention of tractors for towing, weighted rollers began to be used.

R. R. Proctor conducted the first complete investigation into the theoretical aspects of compacting soil and the development of tests to determine compaction characteristics of soil in the early 1930s. Four famous articles on the compaction of soils used in building earth dams were written by Proctor and published in the *Engineering News Record* during August and September of 1933 [1]. These articles presented a test procedure for determining the compaction characteristics of a soil based on the relationship between soil water content and density for each soil as it is compacted (see Fig. 1). This early work is the basis of the procedure used today with very little change over the years. Proctor's articles further established the basic testing equipment design that is still used today. He further related soil compaction to soil performance (e.g., saturated permeability, swelling pressure, and bearing capacity) and explained how these principles applied to earth dam construction.

Proctor's test used a constant amount of energy (compactive effort) applied to several specimens of the same soil prepared at different water contents. The compacted dry density is plotted against water content for each specimen, resulting typically in a parabolic curve, called "the compaction curve." The peak of the curve defines the "maximum dry unit weight or dry density," which occurs at a certain water content most suitable for compacting that particular soil for the energy applied. The water content at which the maximum dry density occurs is called the "optimum water content." Proctor's test used a hammer of given mass falling a certain distance onto soil placed in a mold of known volume. The soil was placed in the mold in layers, and a certain number of blows were applied to each layer with the hammer. The mass of the hammer, times the height of the fall, times the number of blows per layer, times the number of layers, divided by the volume of the mold, gives the compactive effort in foot-pounds per cubic foot of soil. Proctor also showed that as the compactive effort is increased, the maximum density increases and the optimum water content decreases. In Proctor's articles, a standard test was proposed using a compactive effort or energy of approximately 12,000 foot-pounds per cubic foot. This test has been known over the years as the "Standard Proctor Test" and is now ASTM Standard Test Method D698 that still uses the same basic procedure as proposed by Proctor in 1933 (see Fig. 2 and Chapter 3, Section B).

During World War II, Arthur Casagrande worked with the U.S. military on the design of airfields. His work dealt mainly with the construction of soil subgrades to support the heavy loads of large aircraft landing on paved runways. The main result of his work was a system of classifying soils for engineering uses, which was developed based on the engineering properties of compacted soils [2]. The classification system was mainly related to strength or bearing capacity of compacted soil and the ability to drain or prevent water softening and freezing under airfield pavements. Because the soil subgrade had to support heavy loads, larger equipment was used to achieve higher bearing strength. In 1952, Casagrande's classification system was adopted by most

Three variables determine the density of a compacted soil:
1. **The energy used in compaction**
2. **The water content of the soil**
3. **The properties of the soil**

Then: Using a standard energy, if a series of soil specimens are compacted at increasing water contents, the resultant dry density of the specimens will vary. The density will increase to a peak value, then decrease.

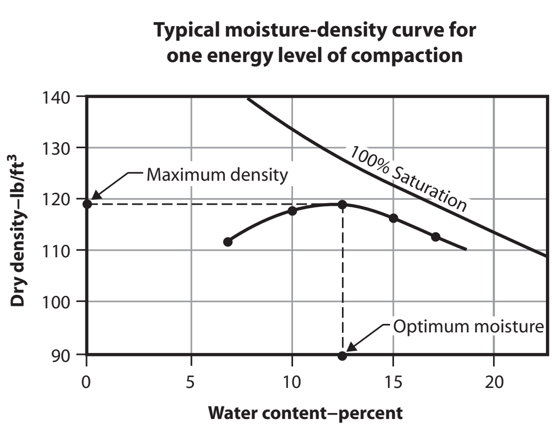
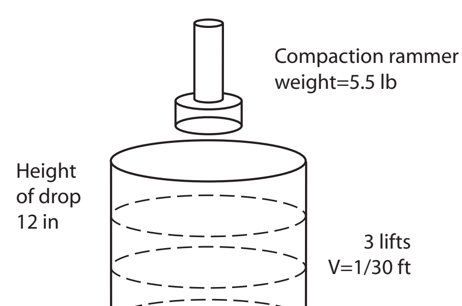


Fig. 1—Proctor's principle.

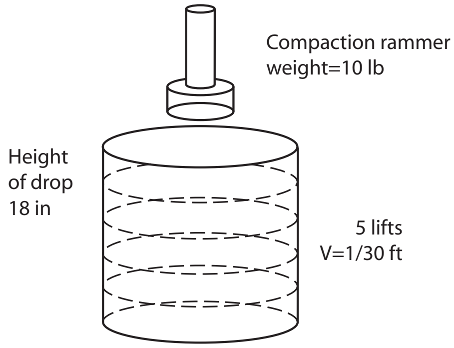


Standard Proctor energy—ASTM D698

$$= \frac{5.5\text{ lb} \times 1\text{ ft} \times 25\text{ blows/lift} \times 3\text{ lifts}}{1/30\text{ ft}^3}$$

$$= 12{,}375\text{ ft–lb/ft}^3$$

Fig. 2—Standard Proctor energy application.



Modified Proctor energy—ASTM D1557

$$= \frac{10\ \text{lb} \times 1.5\ \text{ft} \times 25\ \text{blows/lift} \times 5\ \text{lifts}}{1/30\ \text{ft}^3}$$

$$= 56{,}250\ \text{ft–lb/ft}^3$$

Fig. 3—Modified Proctor energy application.

U.S. Federal Government agencies and called the Unified Soil Classification System (USCS), which is now ASTM Designations D2487 and D2488. (Sample forms for use as a material testing report based on these ASTM Designations can be found in figures 1B and 2B of Appendix B). The compaction Casagrande proposed for airfield use is generally the same as the "Modified Effort" found in ASTM Standard Test Method D1557 (see Fig. 3 and Chapter 3, Section B).

For some comparisons of the standard and modified methods for different soils in the USCS, (see Figs. 4 through 11) [3].

The advancements in soil compaction over the years have been mainly in the equipment used to compact soil. Larger and heavier compactors have been developed for use on large projects. Many of the newer rollers (larger and smaller) are self-propelled, more maneuverable, and faster. Compaction of a fill can be accomplished with fewer passes over the surface at higher speeds so that the desired density is achieved in a shorter time and more efficiently. The heavier compactors apply a larger compactive effort to the soil in a shorter time than the smaller equipment. The modified effort test (ASTM D1557) may more nearly simulate the heavier equipment, whereas the standard effort test (ASTM D698) may more nearly simulate the smaller equipment. It is important to note that the standard or modified test procedure can be used for most projects involving compaction of soils provided the specifications are prepared to reflect the appropriate percentage of maximum density and deviation from optimum water content. The project specifications and guidelines must be properly written to allow for variations in soil conditions that require engineering judgment. Part of a properly written project specification must include a practical application of the test procedures to avoid the development of compaction requirements that cannot be obtained within a reasonable time frame or with materials that are readily available.

C. GENERAL USE OF COMPACTION TESTS, DENSITY TESTS, AND PROJECT SPECIFICATIONS

As mentioned previously, the primary purpose of this manual is to promote the proper use of ASTM standards for tests

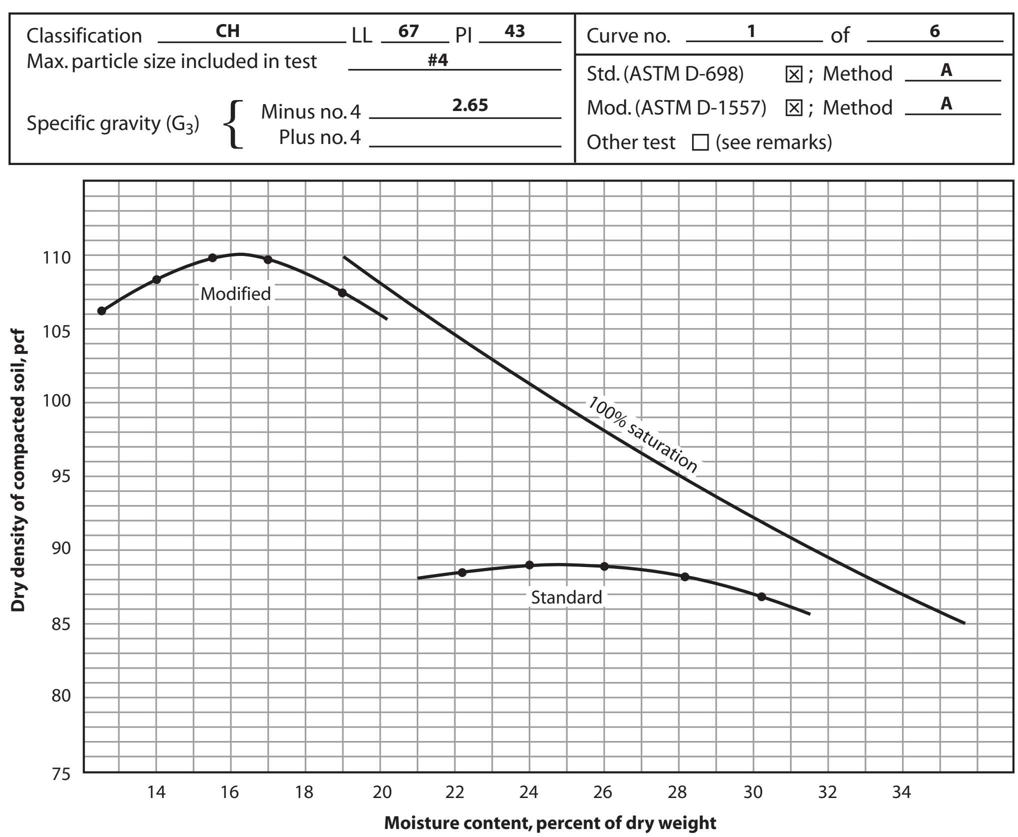


Fig. 4—Typical compaction test results for CH soil.

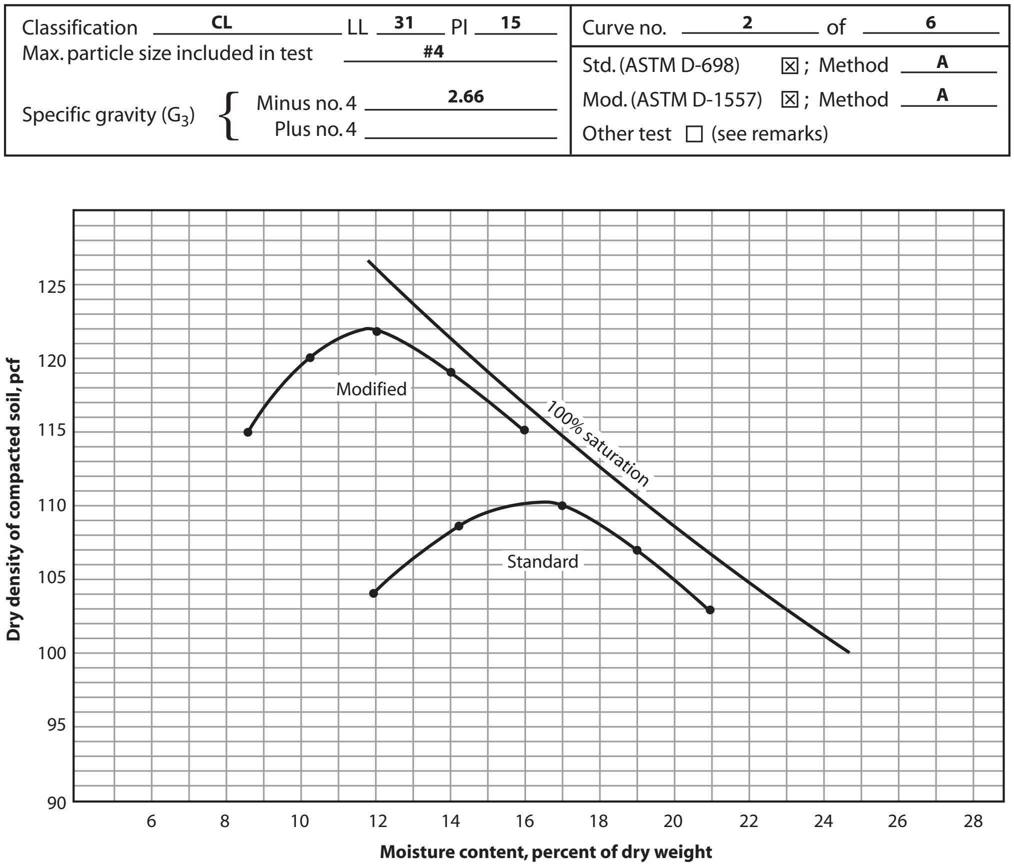


Fig. 5—Typical compaction test results for CL soil.

associated with the compaction of soils. It is the experience of many quality-conscience earthwork engineers when evaluating the quality of the earth or rock fill compaction that problems can occur when the tests are performed correctly but are improperly interpreted. Over the past years there have been numerous case studies completed on earth and rock fill project failures in which the ASTM test methods have been completed correctly, but the test results were misapplied during the evaluation of the data for moisture content, density, and/or percent compaction. Successful completion of earth and rock fill projects, possibly more so than any other type of engineering work, requires a carefully balanced application of the ASTM test methods, geotechnical engineering principles, and old-fashioned engineering common sense. The following is a list of the key items that must be considered when applying the ASTM test methods to a field project and/or writing specifications.

1. Select the standard reference compaction test method that is applicable to the soil and site conditions that are present. The project specifications for the fill should designate the ASTM test method (compactive effort) to be used and the percentage of the maximum density and range of water content (plus or minus of optimum). The specifications should avoid indiscriminate use of test methods that do not meet the needs of the project, can render the project work unnecessarily difficult because the reference standard is not appropriate, or both. It also is important to realize that ASTM test methods for soil testing have a limit to the degree of precision that can be reasonably applied to earthwork projects.
2. To ensure that the presentation of the field in-place density data reflect the degree of observation and testing, it is good engineering practice to describe very clearly whether the testing is provided on a "spot-check" or "full-time observation" basis. This basic engineering practice is essential to ensure that highly accurate or passing test results in a localized area are not used to suggest that a project has been constructed properly over the entire area of work.
3. It is important to remember that time is of the essence in the testing of earth fill, so the test procedures should provide a method for developing preliminary values for moisture content and dry density using direct heating or other rapid moisture content measurements. To ensure consistency and accuracy, the field test values will need to have regular verification using other ASTM methods including oven dry moisture contents using ASTM D2216.
4. Experienced earthwork engineers understand that the reference Proctor curve (standard reference density) may change frequently on a typical earthfill project. To account for this field condition, the project specifications need to provide a method for making proper field decisions on the selection of the standard reference density curves. The project specifications also need to specify the method for determining which curve will be used if the visual classification method is not suitable. Suitable methods for the field selection of the standard reference density include a one-point method, a rapid three-point method, or visual classification, or combinations thereof with a recognized sample of material method. It is important to note that if

Classification ML LL — PI NP	Curve no. 3 of 6
Max. particle size included in test #4	Std. (ASTM D698) ☒; Method A
Specific gravity (G_s) { Minus no. 4 2.68	Mod. (ASTM D1557) ☒; Method A
Plus no. 4	Other test ☐ (see remarks)

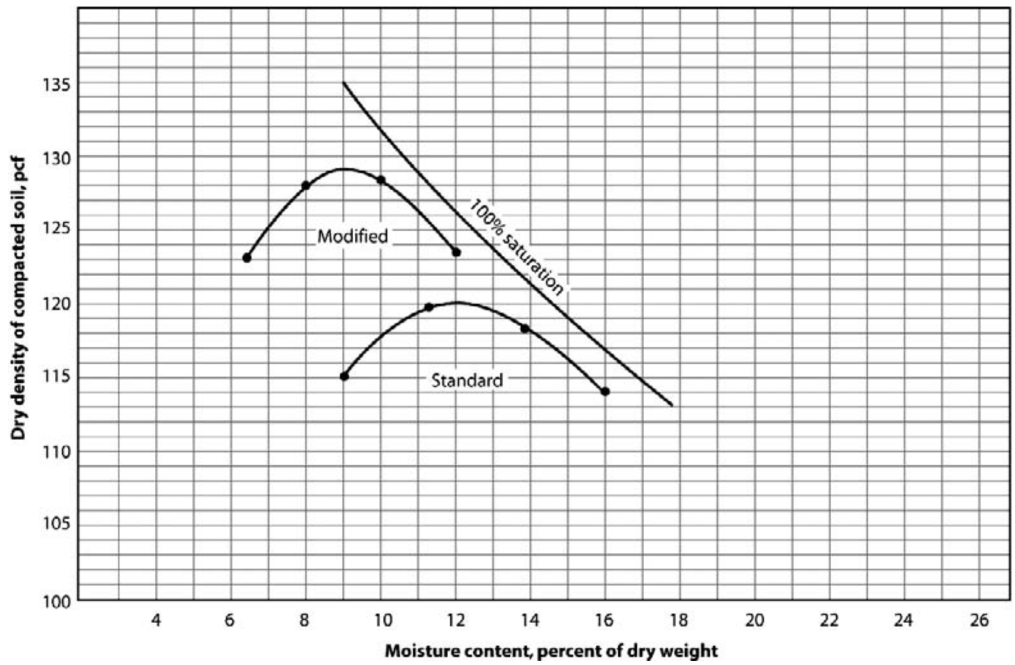


Fig. 6—Typical compaction test results for ML soil.

guidelines on the selection of the standard reference density curve are not provided, then the decision will be made according to the level of experience of the field technician or the contractor at the project site (see Chapter 6 for guidance in this evaluation).

5. A procedure should be set up for timely data review by a qualified geotechnical professional. Allowing technicians alone to select the standard reference density, review their own data, and determine adherence to the selected density and water content criteria typically does not provide adequate checks and balances to ensure a quality earthfill project.
6. It is important to remember that nuclear gauge test results for moisture content and density can be influenced by the presence of carbon, mica, and other materials. The manufacturers of these devices have made provisions to adjust the gauges with moisture and density corrections. Water-content variations will occur on a regular basis whereas density variations may be quite rare. It is important for all earthwork engineers and technicians to realize that the accuracy of the nuclear gauge and other ASTM density methods is only as reliable as the skill level and experience of the equipment operator.

The in-place density/water-content test methods must be compatible with specification requirements regarding placement requirements. These methods should also be implemented at frequencies and test locations that provide good documentation of the earthwork involved. The explanation of the compaction procedures in the project specifications should include lift thickness, compactor type and size, and fill density and water-content requirements. Properly written specifications must provide a balanced emphasis on the importance of compaction techniques, observations by the technician, and adequate testing of the in-place fill materials. During the development of any quality assurance or quality control testing program it is important that the overall purpose of compacting the fill material is not overshadowed by a natural tendency to generate a large volume of testing information. A large amount of very accurate testing without qualified observation of the compaction process will not necessarily ensure proper compaction or proper documentation of the fill material.

Another engineering application of the standard reference value for maximum density and optimum water content includes a direct or indirect comparison to other engineering properties that are used for the design of foundations and embankments. To ensure adequate correlation of the design soil properties to field conditions, it is often necessary to provide other types of testing at the anticipated in-place moisture and density values. These additional tests may include testing for engineering properties such as shear strength, consolidation, and permeability as required for the design and construction of the structure.

Immediately before and during the process of placing the fill, representative samples are obtained of the various soil types that are to be used in the fill. These samples are typically tested for the maximum density, optimum water

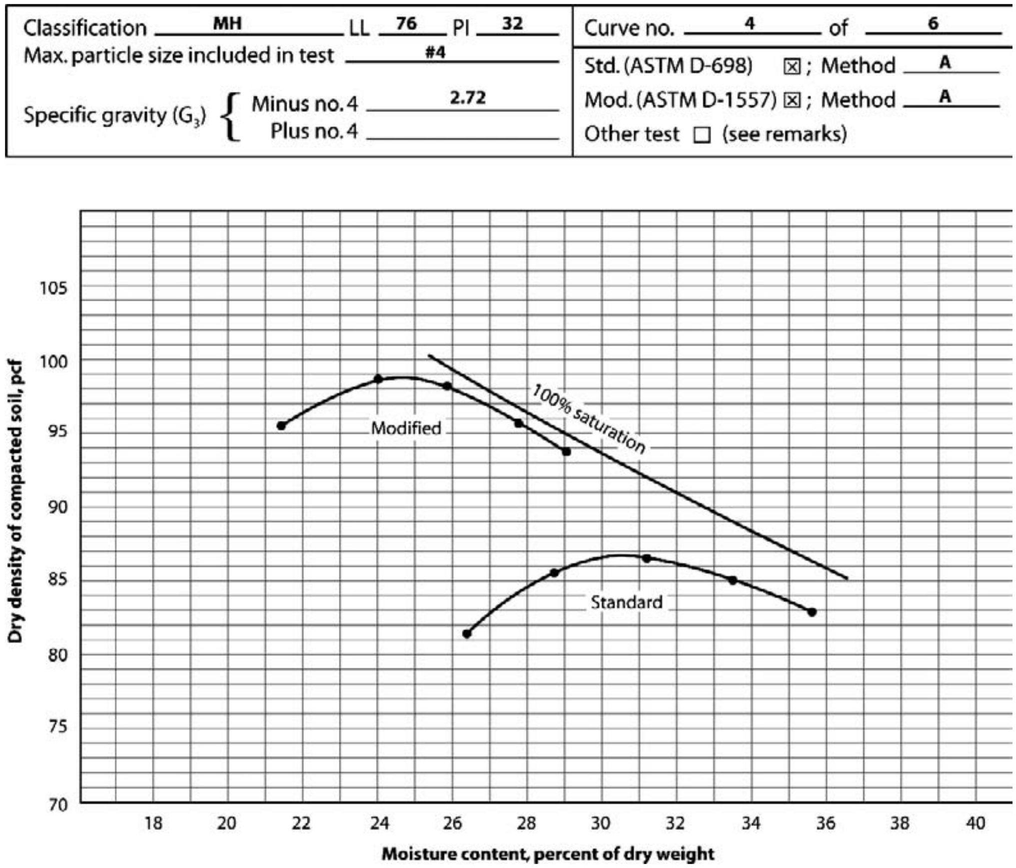


Fig. 7—Typical compaction test results for MH soil.

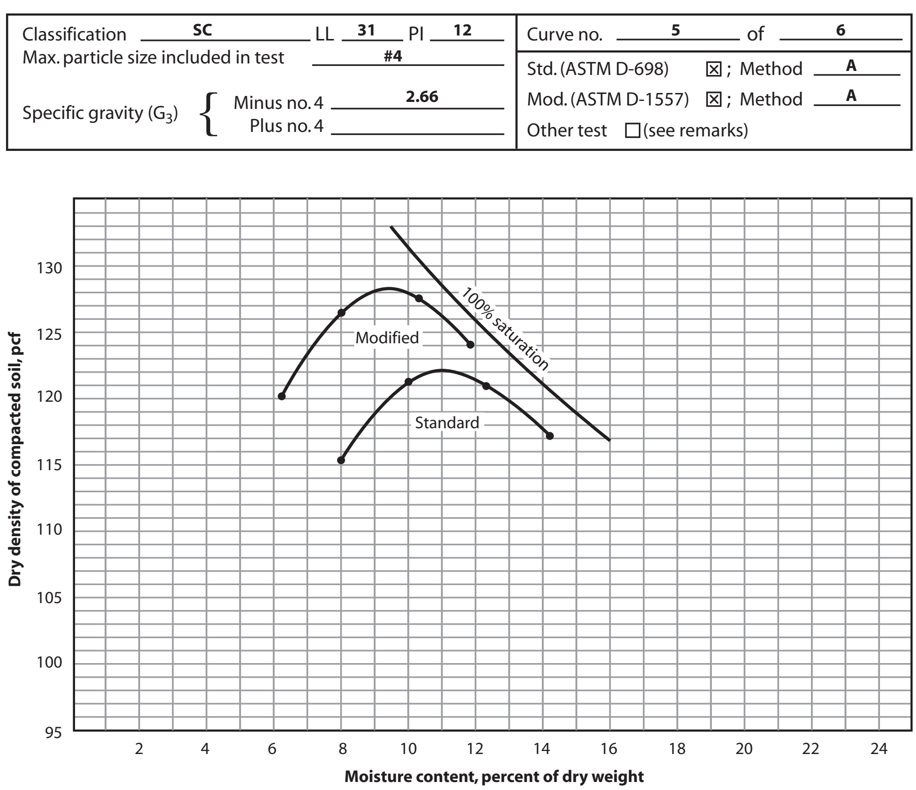


Fig. 8—Typical compaction test results for SC soil.

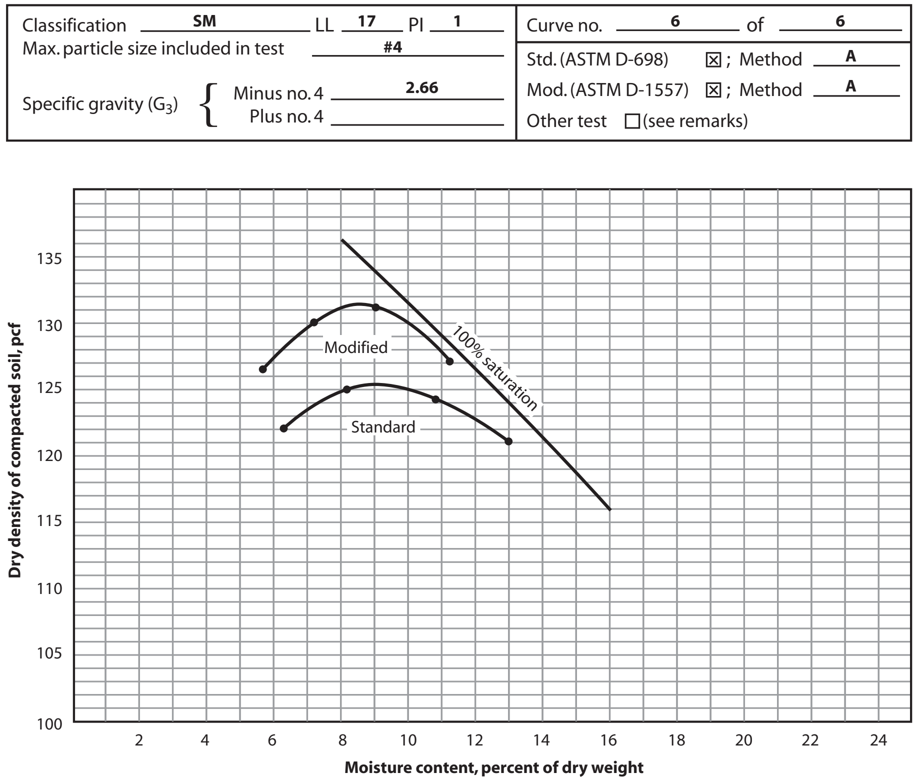


Fig. 9—Typical compaction test results for SM soil.

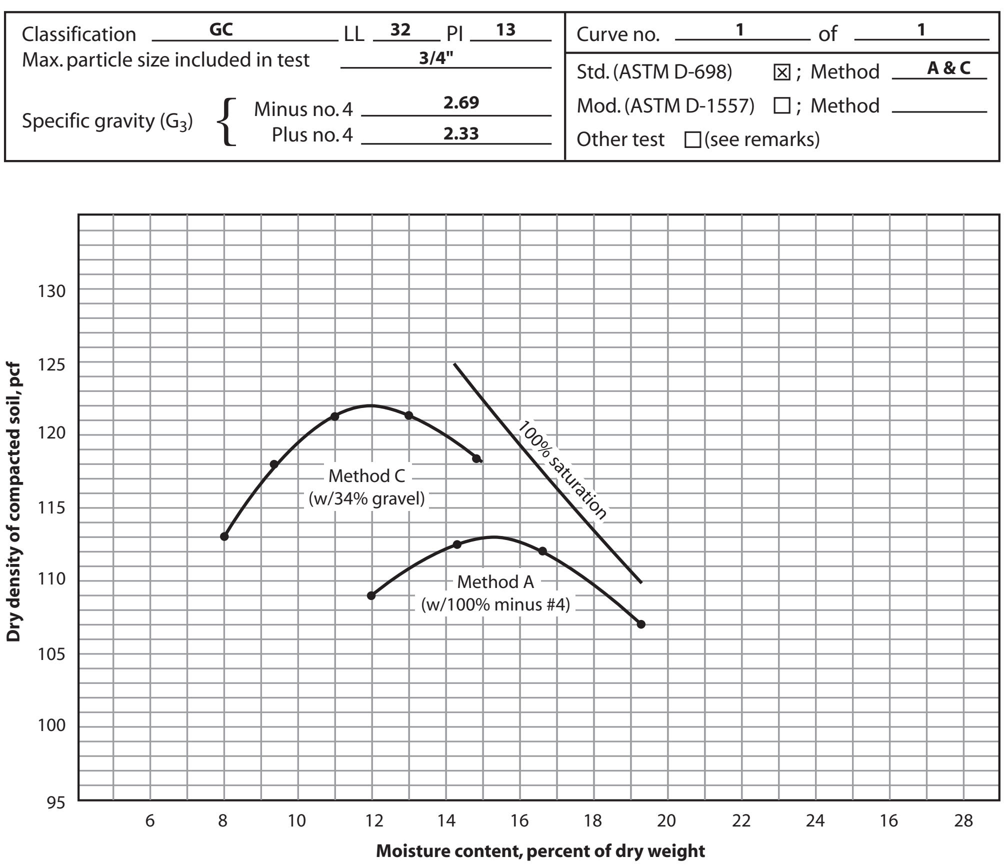


Fig. 10—Typical compaction test results for method A and method C tests.

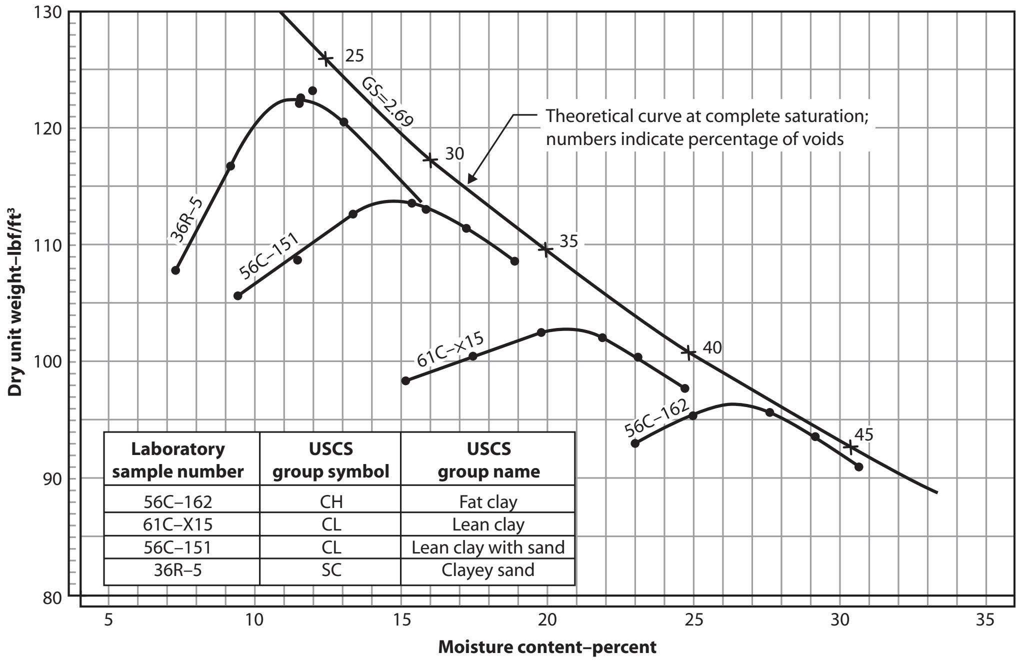


Fig. 11—Family of compaction curves.

content, and classification parameters including Atterberg Limits and grain size distribution. In preparation for placing earth materials in the fill, water is added or removed from the fill material by the contractor to achieve the specified water content. As the materials are placed in the fill, the soil is spread into uniform lifts, and compaction is applied to achieve the specified density. In-place unit weight or density tests and water-content tests are made on the compacted materials to compare with the standard reference values obtained as required by the project specifications.

For practicality in keeping the specimen sizes reasonable, standard reference compaction tests are made using materials in which all particles larger than a specified size are removed by sieving. The maximum density is then determined on the remainder of the soil, eliminating a portion of or all of the gravel-size particles. Tests made to determine the in-place density of the soil must then be based on this same size fraction for proper comparison. Some in-place density tests (e.g., the sand cone or drive cylinder tests) lend themselves to separating these fractions, whereas others (e.g., the nuclear gauge test) do not. In any case, the in-place density and water-content tests must be corrected or done in such a way that the oversize fraction is accounted for. The procedures for making these corrections are explained in Chapter 6 of this manual. A series of flow charts showing the use of applicable test methods on a typical fill project is provided in Figure 1A of Appendix A.

D. UPDATED SCHEDULE FOR ASTM STANDARD TEST METHODS

To ensure that the test methods reflect the most recent state of the practice for the observation and testing of fill materials, ASTM standard test methods are updated as needed or are reapproved at a maximum interval of once every 5 years. This process is continuing so that, on the average, three or four standards used in compaction control are typically reviewed each year. This approach to review or revise (or both) applicable ASTM test methods is an indication of the constantly changing application of test methods and the evolving of new compaction control test procedures. It is important to keep current with the revisions of these standards involved in this compaction control area of earthwork engineering.

The following chapters provide an explanation of the ASTM test methods and industry standard practices that are used for earthfill construction. This manual is intended to provide a basic understanding of test methods that have been developed and to encourage a renewed dedication to quality earthwork engineering. As indicated in the technical information provided, the ASTM test methods provide a general framework and sound starting point for the successful completion of earthwork projects. To successfully complete an earthwork project using these methods requires that engineers continually dedicate themselves to maintain an attitude of constant learning to understand the procedures and difficulties of dealing with soils and compacted materials.

References

[1] Proctor, R.R., "Description of Field and Laboratory Methods," *Eng. News Rec.*, Vol. 111, 1933, pp. 286–289.

[2] Casagrande, A., "Classification and Identification of Soils," *Trans. ASCE*, Vol. 113, 1948, pp. 901–991.

[3] *Soil Mechanics Training Series, Basic Soil Properties, Module #5–Compaction*, National Employee Development Staff, USDA, Natural Resources Conservation Service, Washington, D.C., 1988.

2

Means, Methods, and Mechanics of Compaction

A. DESCRIPTION OF THE TYPES AND TERMS FOR FILL COMPACTION

Earthfill and/or compacted backfill are constructed using earthwork techniques that include excavating, hauling, spreading, soil processing, addition of water, and compacting earth (soil and/or rock). Earthwork has wide application in construction projects. Practically every construction project has a component of earth construction that buildings, structures, or other project components are founded on, supported by, or constructed partially or entirely of soil or rock. Structural stability is usually dependent on proper foundation preparation and the proper placement and control of the earthfill placement and compaction process. Generally, all earth fills must be compacted to a specified density and water content as determined by the design parameters. Controlled compaction is usually required for roads, airfields, highways, building foundations, parking lots and drainage features, pipelines, railways, embankment dams, canals, dikes and levees, clay-lined containment structures and caps, and other related structures.

Fill sections must be compacted to reduce the potential for excessive settlement or differential movements between cut and fill segments. Earth is compacted adjacent to structures such as bridge abutments [1] and in subgrades for roads to achieve uniform compressibility. Subgrade materials that are too soft or weak may be removed and replaced with compacted materials. Several base courses or layers of select soil are compacted immediately below airfield and roadway pavements to improve or control, as much as possible, the compressibility, strength, and drainage characteristics of the subgrade.

Building foundations frequently require several forms of compacted backfill to ensure design performance. Backfill is frequently needed for larger structures where excavation is used to provide a balanced loading of the foundation. Fill placement is required under structures where the site must be elevated for operational requirements or to reduce flooding or both. The perimeter of buildings is often backfilled to prevent undesirable settlements and in some cases to support the walls. Some structures, such as pumping plants, must have compacted, free-draining soils in the foundation to prevent erosion problems caused by leakage. Lightly loaded structures or soils on poor soils may require mat foundations that must have compacted subgrades and backfill to ensure uniform compressibility and maintain differential settlement less than 1 in. for structural integrity and appearance.

Dams, canals, and levees and other earth structures that hold water are another type of structure that requires special earthwork and compaction techniques. Compaction and moisture conditioning of the fill material are used to reduce the permeability of the foundation materials to minimize excessive leakage and erosive piping channels. To accomplish these design objectives, clayey-type soils with low permeability and settlement characteristics are often selected. To meet the design requirements for permeability and density of the soil in the embankments, strict adherence to specified requirements for compaction and water content is required. If these water-retaining structure soils are compacted too dry of the optimum moisture, then the soil particles can remain loose enough to cause collapse (rapid settlement) upon saturation, resulting in tension cracks, shearing, or other problems. To minimize the potential for concentrated leaks due to cracking or internal piping channels, drainage zones are often constructed in embankment dams consisting of sand and/or gravel to serve as filters and drains. These drainage zones also require compaction to reduce the potential for liquefaction, settlement, collapse, or to improve resistance to rotational shear failures, or combinations thereof. For further information and guidance on soil compaction related to these type of structures, see Table 1 [1].

Some earth fills are named or referred to by the process used for construction or predominate characteristic of the fill. An example of this type of project would include earthfill construction projects in which soil or rock materials were dumped in place with carts or carriages with little or no compaction. This type of fill is called “dumped fill” or “uncontrolled fill” and typically exhibits less soil strength than mechanically compacted fills. Another type of earth fill named after its placement method would include hydraulic fills that are placed by mixing soil and water to create a fluid mixture. After the fill material is liquefied, it is then pumped, dredged, or transported through a pipe or flume into a pool where the compaction occurs as the water seeps away or evaporates, allowing a reduction in air voids and closer particle contact. Hydraulic placement of material has been used in the past for dams, dikes, and levees. More recently this method has been used for waste fills or temporary material stockpiles. It is important to note that loosely dumped fills (uncontrolled fills) and hydraulic fills are not compacted by equipment and generally have undesirable characteristics including a relatively high degree of compressibility, increased permeability, decreased slope stability, and an increased potential for liquefaction during earthquakes.

Most engineered earth fills are created using some type of mechanical compaction or rolled compaction of the soil or rock materials. Placement and compaction of earth fill in the modern era is typically accomplished using some type of heavy machinery on steel rollers or rubber tires that impart a compactive energy to the soil and/or rock. Compaction equipment is often referred to as compactors or rollers, and

TABLE 1—Engineering Use of Soil Based on the USCS [Most desirable (1) to least desirable (14)]

	Important properties					Relative desirability for various uses						
						Rolled earth dams			Canal sections		Foundations	
Typical names of soil groups	*Group symbols*	*Permeably when compacted*	*Shearing strength when compacted and saturated*	*Compressibility when compacted saturated*	*Workability as a construction material*	*Homogenious embankment*	*Core*	*Shell*	*Erosion resistance*	*Compacted earth lining*	*Seepage important*	*Seepage not important*
Well-graded gravels, gravel-sand mixtures, little or no fines	GW	pervious	excellent	negligible	excellent	—	—	1	1	—	—	1
Poorly graded gravels, gravel-sand mixtures, little or no fines	GP	very pervious	good	negligible	good	—	—	2	2	—	—	3
Silty gravels, poorly graded gravel-sand-silt mixtures	GM	semi-pervious to impervious	good	negligible	good	2	4	—	4	4	1	4
Clayey gravels, poorly graded gravel-sand-clay mixtures	GC	impervious	good to fair	very low	good	1	1	—	3	1	2	6
Well-graded sands, gravelly sands, little or no fines	SW	pervious	excellent	negligible	excellent	—	—	3 if gravelly	6	—	—	2
Poorly graded sands, gravelly sands, little or no fines	SP	pervious	good	very low	fair	—	—	4 if gravelly	7 if gravelly	—	—	5
Silty sands, poorly graded sand-silt mixtures	SM	semi-pervious to impervious	good	low	fair	4	5	—	8 if gravelly	5 erosion critical	3	7
Clayey sands, poorly graded sand-clay mixtures	SC	impervious	good to fair	low	good	3	2	—	5	2	4	8
Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	semi-pervious to impervious	fair	medium	fair	6	6	—	—	6 erosion critical	6	9
Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	CL	impervious	fair	medium	good to fair	5	3	—	9	3	5	10
Organic silts and organic silt-clays of low plasticity	OL	semi-pervious to impervious	poor	medium	fair	8	8	—	—	7 erosion critical	7	11
Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	semi-pervious to impervious	fair to poor	high	poor	9	9	—	—	—	8	12
Inorganic clays of high plasticity fat clays	CH	impervious	poor	high	poor	7	7	—	10	8 volume change critical	9	13
Organic clays of medium to high plasticity	OH	impervious	poor	high	poor	10	10	—	—	—	10	14
Peat and other highly organic soils	Pt	—	—	—	—	—	—	—	—	—	—	—

— Not applicable

common examples are sheepsfoot or tamping foot rollers, rubber tire rollers, smooth drum rollers, grid rollers, and vibratory rollers. Some rollers incorporate vibration as well as impact and kneading action during the rolling process.

Specialty compaction is typically required in areas with limited access or with materials that may require a combination of methods to meet the specified standard. Specialty compaction may include compaction of fill or backfill close to structures using small, manually directed power (hand) tampers and/or compactors with vibratory action (see Figs. 1(a) and 1(b)). Another type of specialty compaction would be "saturated and vibrated" or "water-induced compaction." This type of compaction refers to a process in which compaction is accomplished by applying water to the fill along with vibratory compaction. The theoretical basis for this compaction method is that the soil/rock particles become saturated, allowing the breakdown of capillary bonds between soil particles and producing compaction of the soil matrix as the water drains from partially saturated material. The "sluiced or water compaction" procedure should only be used on free-draining materials such as sand or gravel materials.

Zoned earth fills are fills that are constructed using different soil or rock materials separated into certain sections or zones of the fill to accomplish water containment, stability, drainage, or other desired engineering characteristics. The zones may include silt and/or clay for low permeability, sand and gravel for filtering and drainage and for strength and stability, and rock for strength and erosion resistance (see Fig. 2). An example of this type of project would be the horizontal zoning of road and airport fills that provide for lateral drainage and uniform pavement support. Another example would be dams and other water-containment embankments that use more vertical zoning for water retention, slope stability, and for intercepting and carrying seepage to an outlet near the toe of the dam embankment.

Soil liners are thin layers of fine-grained soil that range in thickness from 1 to 5 ft and require special moisture conditioning and compaction to meet the required engineering properties. Soil liners are often placed and compacted to provide containment of fluids either inside of a pit, reservoir, waste containment, or as a cover over waste materials to reduce the potential for liquids to enter the contained materials, or combinations thereof. Soil liners are used in clean reservoirs to minimize seepage losses and may be placed over localized subgrade areas with a relatively high permeability or over the entire bottom surface of the reservoir or other liquid containment structures. Soil liners (usually constructed of clay or clay/silt mixtures) are placed under landfills or hazardous waste disposal sites as part of a geosynthetic composite liner system to reduce the amount of leachate that may percolate into the underlying soil or groundwater. Soil liners that are installed as part of a capping system are also used over the landfill waste containment structures to prevent infiltration of precipitation and surface water from entering the waste material and creating more leachate.

To reduce permeability or hydraulic conductivity of soil liners, it is essential to specify moisture contents on the wet side of the optimum moisture content and provide a balanced emphasis on soil permeability and density. Soil liners or capping systems on the soil surface or as placed in the bottom of ponds or reservoirs are often constructed by compacting the natural in situ and on-site soils or by hauling in off-site soils that have soil properties more compatible with the intended engineering use. In general, tighter quality control observation and more frequent compaction testing is required for soil liners used for waste containment purposes because they are subject to more stringent regulatory requirements and the margin for error is much smaller as compared with other types of earthfill projects (see Fig. 3).

Earth fills are usually constructed of soil or rock materials that are placed within a specified water-content range suitable for reworking, compaction, and grading or mixing. For best results, it is typically recommended that water be added in the borrow area in 1–2 % increments and allowed to "cure" (be absorbed and distributed uniformly through the soil) before final compaction or moving to the construction area. Small increments of water (up to a 4 % increase) may be incorporated into the soil on the fill or in a mixing area before compaction of the fill. It must be thoroughly mixed using a disk or other equipment for distributing the water evenly in the soil. Sometimes, the soil is too wet for working or compacting and must be drained and disked to break up clods and promote drying before compaction (see Fig. 4).

Soil materials, rock materials, or both are usually obtained from a borrow area, pit, or quarry that has been


(a)


(b)

Fig. 1—(a) Manual-directed tamping plate (jumping jack) compacting backfill for a pipe; (b) hand-directed vibratory roller.

Fig. 2—Zonefill on an earthfill dam.

Fig. 4—Disking earth fill to break up soil clods and/or incorporate water. The disking may also be used to dry out wet soil.

shown to contain the required engineering properties. The material is hauled to the fill and placed in uniform lifts ranging in thickness from 6 to 18 in. depending on the type of material, the depth of the roller penetration, and the percent compaction that is required. After the fill has been placed and before compaction or any other processing, the contractor removes deleterious materials, such as tree roots, organic-laden soils, and rocks larger than one half of the lift thickness that did not break down during the initial disking and rough grading. Some grading (blading with a power grader or dozer) to achieve uniform lift thickness, the application of additional water, and processing (disking or mixing) of the materials typically occurs before compaction is performed. Compaction is then accomplished by applying a uniform number of roller or compactor passes at a specified frequency and directional pattern. If a fill with uniform engineering properties is required, then the placement, grading, and compaction must be completed in an organized manner so that lift thickness is uniform and most of the fill receives the same conditioning and compactive energy.

One method for organizing the placement and compaction of earth fills that has been used successfully on numerous projects is to place material in two parallel strips or bays that are spaced far enough apart to accommodate

Fig. 3—Linear construction of an agricultural-waste holding pond with a soil amendment.

turning of the hauling and compacting equipment. The reason behind this method of placement and compaction is to leave enough room for working between adjacent strips to avoid damaging previously compacted areas with localized turning and equipment ruts. The strips can be laid out according to a grid pattern, and equipment operators are directed to route hauling equipment so the fill loads are placed and spread end to end for a distance on the fill. During compaction these strips are often bladed or mixed as needed to create a uniform thickness and water content. The compaction equipment is then passed over the strips by traveling one direction on one strip and returning the other direction on the other strip while counting the number of passes as the fill operation progresses. It should be noted that overlapping of spreading and compaction equipment onto adjacent strips is necessary to ensure uniform conditions throughout the fill. The number of compaction passes and water content is noted as the quality control tests are made to verify that the degree of compaction has been achieved. In this way, the equipment operators can develop a practical understanding of the degree of compaction that is required to achieve the required in-place density. The principles of this type of coordinated placement, moisture conditioning, compaction, and quality control testing may vary from project to project, but the intent is to develop a consistent pattern of compaction that can be repeated by the operators and quality control technicians.

Earthfill construction is the process of bringing soil to a suitable moisture condition in the borrow area or at the area of placement followed by spreading, conditioning, and compacting the soil in its final position to create a relatively consistent engineered fill. Quality control/quality assurance testing is conducted to monitor and make adjustments in any or all parts of the process rather than merely to accept or reject the fill. Localized in-place testing is only accurate and representative of the entire fill placement area when a consistent processing and placement operation is in place. High-quality earthfill construction is best achieved when there is close coordination between the testing personnel and other parts of the construction team.

For larger projects and difficult soils and rock fills, a test fill is often constructed at the beginning of the project when more intensive testing is performed as all of the processes, roller size and weight, number of passes, and other factors are adjusted. Once the best combination has been determined, it can be used throughout the rest of the project, usually with less testing and more attention to the processes and coverage being used. Chapter 6 provides a more in-depth discussion of quality control and quality assurance practices as they apply to the successful completion of quality earthwork projects.

B. THEORETICAL BACKGROUND—MECHANICS OF COMPACTION

Compaction is the densification of a soil by means of mechanical manipulation. The compactive effort is applied in a high production manner and includes mechanical energy applied as a kneading, impact, or vibration action that expels air and small amounts of excess moisture. Conversely, consolidation is a process of densification that occurs over a longer period of time after a static or surcharge load is applied. In consolidation, the load is first supported by an increase in pore fluid pressure. During consolidation, water (and air if present) is gradually expelled from the voids and densification occurs over a period of time that is determined by the consolidation characteristics of the soil. The main differences between compaction and consolidation are that with compaction, the load is applied quickly, which expels mostly air, whereas with consolidation, water or water and air are expelled over longer periods of time so that the water content is changed. Some consolidation and/or migration of water can occur after individual lifts are compacted and subsequent lifts are placed over the fill. This type of lower-lift densification is typically not considered part of the compaction process unless heavier rollers are used to induce compaction in lifts deeper than 1 ft below the compaction surface.

The phase diagram in Fig. 5 shows the compaction results of a typical soil. The volume and weight or mass of solids and water remain constant during the compaction process, whereas the volume of air (thus the total volume of the soil unit) decreases, resulting in a higher bulk density or unit weight. Fig. 5 also shows the notation in defining the soil components and in making calculations related to compaction of soil such as unit weight and water content. It should be noted that the water content and unit weight are expressed in terms of the dry mass (the mass of the solid particles). The dry unit density is the mass of the solids divided by the volume of the soil mass including air, water, and solids.

1. Compaction of Silty or Clayey Soils

The following discussion in this chapter applies to a fine-grained soil (silt or clay) or sands and gravels with more than 12 % fines. Clean sands and gravels (containing little or no fines) represent a special case, which is discussed later.

The compaction curve shown in Fig. 6 is a plot of the dry unit weight of the compacted soil versus the water content of the soil. Each point on the curve is obtained by applying the same compactive effort to each of five or six specimens of the soil. The compactive effort is a measure of the energy (m-kg/m^3 or ft-lb/ft^3) applied to each unit volume of soil. It is determined by multiplying the mass of the standard hammer, times the number of blows of the hammer applied to each layer, times the height of the hammer drop, times the number of layers, divided by the volume of the soil specimen. When a roller or other compactor is passed over a lift on the fill, it applies energy to the soil. In the compaction test, the hammer is dropped, impacting the soil to apply energy simulating the

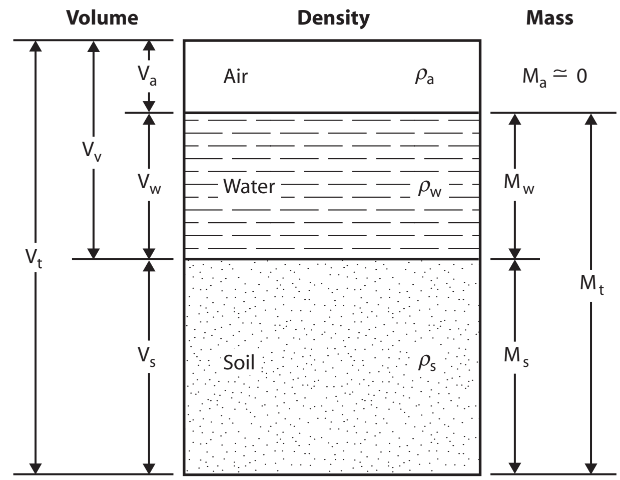


Fig. 5—Relationship between air, water, and solids in a soil mass.

energy applied by the compaction equipment in the field. In the case of a sheepsfoot (tamping) roller, the energy applied is a kneading and an impact action, which may not be fully simulated by the impact hammer; however, it is considered to be a reasonable facsimile of the process. To maintain uniform and predictable conditions, the compactive effort should remain constant for the fill. This process requires uniform water content, a uniform number of passes with the roller over each lift on the fill, and disking and/or blading to produce uniform soil conditions and lift thickness.

For fine-particle soils including silts and clays, it is important to note that soil moisture conditions considerably below the optimum water content increase the frictional resistance and capillary tension between the soil particles. A very dry soil moisture condition makes the fine-particle soil absorb the compactive energy applied and creates elastic rebound in the soil particles. For this condition, the densification process is not as efficient and the soil can contain a higher number of relative air voids after the compactive energy is applied. As the water content of the soil is increased by adding water, there is more lubrication and less capillary tension between soil particles, which allows the impact energy to drive the particles closer together, whereupon the remaining capillary stresses hold them in the tighter arrangement. As more water is added and the same compactive effort applied, the particles are

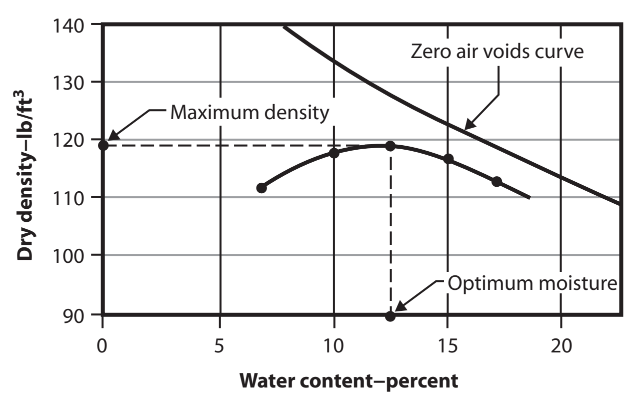


Fig. 6—Typical moisture-density curve for one energy level of compaction.

driven even closer together and the soil/water attractive forces create stronger bonds that increase the plastic rebound of the fine-grained soils. When sufficient water has been added, then the void spaces in the soil matrix begin to be filled or nearly filled with water, and the water, being noncompressible, will not let the particles be pushed together any further. At this point of saturation and at the moment of impact with the hammer, the water absorbs the energy with a momentary increase in pore water pressure because of the soil/water particle attractive forces. At this and at higher water contents, the soil will deform under the hammer impact, but it will undergo a nearly equal rebound adjacent to the hammer impact area. This plastic rebound includes visual evidence of pumping or rutting in the compaction mold and is caused by a localized increase in the pore water pressure and capillary stresses in the soil. The higher the water content, the more pronounced this rebound is and a lower density is typically achieved, which causes a corresponding downward turn in the moisture-density compaction curve. For each soil that is subjected to a given compactive effort, starting at a damp to moist condition and repeating the application of the effort for successive increases in water content, the dry density will increase up to a certain water content, then decrease with successive increases in water content. The water content at which the maximum dry density (γ_{dmax}) can be obtained for a given compactive effort is the optimum water content (w).

$$\gamma_d = \frac{\gamma_{wet}}{1 + \frac{w}{100}} \qquad (2\text{-}1)$$

where:
w = moisture content (percent of dry unit weight)
γ_d = dry unit weight
γ_{wet} = wet unit weight of solid particles plus water

The laboratory compaction test usually involves compacting four to six specimens, starting at a water content approximately 5 or 6 percentage points below the optimum water content and adding sufficient water to each specimen to increase the water content by approximately 2 percentage points between each specimen. A curve plotted through the test points usually takes the general shape of a parabola with the apex at the optimum water content and maximum dry unit weight as shown in Fig. 6. There should be at least two points on each side of the optimum water content to properly plot the parabolic shaped curve. Test methods ASTM D698 and ASTM D1557 provide the details for performing these tests.

The ASTM Standard Test Methods presently cover two compactive efforts, 12,500 ft-lb/ft^3 for standard compaction (Test Method D698) and 56,000 ft-lb/ft^3 for modified compaction (Test Method D1557). Fig. 7 shows a plot of compaction curves for a given soil that was tested using several different compactive efforts. Fig. 7 also demonstrates that as the compactive effort is increased, the maximum dry density increases and the optimum water content decreases. The line drawn through the peaks of the compaction curves is typically defined as the line of optimums.

Figs. 6 and 7 show a typical zero air voids (ZAV) curve that runs adjacent to the reference density curve for a specific soil type. The ZAV curve may also be called the line of 100 % saturation. If the air could be removed from the soil and the voids completely filled with water, then the soil matrix would be 100 % saturated. For some soils this 100 % saturation condition can be achieved by adding water after compaction is complete, but for nearly all soils, it cannot be achieved by expelling all of the air with compaction. Although it is not easily achieved by using conventional compaction equipment, understanding the location of this 100 % saturation line is useful for selecting the compaction curve necessary for evaluating the field moisture/density data.

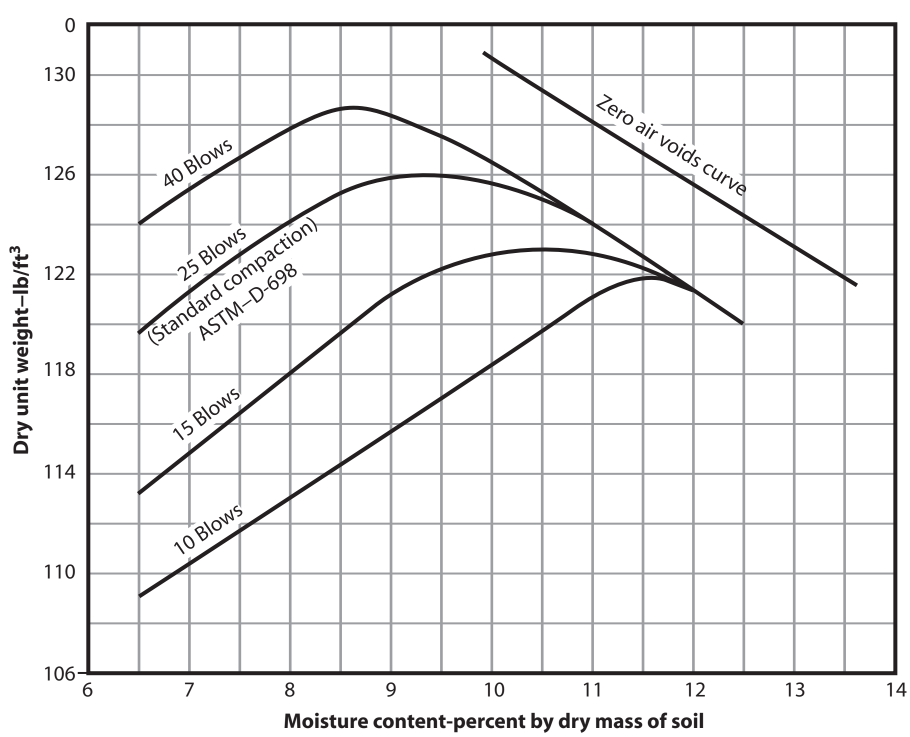


Fig. 7—Effect of compactive effort on the compaction.

Experience has shown that the highest degree of saturation that can be achieved by compaction using conventional compaction equipment is generally between approximately 75 % and 90 % saturation. As shown in Fig. 7, each of the compaction curves drops significantly and becomes approximately parallel to the ZAV curve at the higher water contents on the wet side of optimum water content. In preparing specifications for control of compaction in the field, it is appropriate to study the compaction curve and its relationship to the ZAV curve to make sure the contractor has a suitable range of in-place water contents to work with while attempting to achieve compaction. While performing in-place field density tests, if any points plot to the right of the ZAV curve, there has definitely been an error made because this is an impossible condition. Typically, any points plotting above 95–97 % saturation should be checked for potential errors or the reference density curve selection should be changed to account for a change in soil type.

The equation for calculating the water content at 100 % saturation is as follows:

$$w_{\text{sat}} = \left[\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right] \times 100 \quad (2\text{-}2)$$

where:
w_{sat} = water content at saturation
γ_w = unit weight of water
γ_d = dry unit weight of the compacted soil
G_s = specific gravity of the soil solids

Each soil has unique compaction characteristics. Repeated tests on the same soil sample (using the same compactive effort but different specimens) will generally produce very similar results. A higher compactive effort results in a similar shape of curve, with a higher maximum density and a lower optimum water content. A lower compactive effort results in a similar shaped curve, with a lower maximum density and a higher optimum water content.

2. Compaction of Clean Sands and Gravels

The following discussion in this section applies to clean sands and gravels that have little or no fines (i.e., material passing the #200 sieve).

The compaction curve for clean sands or mixtures of sand and gravel that have little or no fines has a different shape than the curves developed for silts, clays, and mixtures containing silts and clays. Other soil properties for coarser grained soils (i.e., sands and gravels) including soil/water particle interaction and the relationship of the compaction curve to the ZAV line vary significantly from those developed for fine-particle materials (i.e., silts and clays).

Fig. 8 shows a typical compaction curve for clean sands. The compaction curve for clean sands and gravels usually has a shape that has a less defined peak and/or a very irregular progression of maximum dry density with increasing moisture content. Compaction curves for clean fine sands typically have two peaks: one at the dry condition and one in a thoroughly wetted condition. [2] Because water can be easily driven out of clean sand, compaction is not only possible but is also best at the saturated condition. Compaction is best achieved by saturating the sandy material before the compactive effort is applied so that capillary stresses are reduced to zero. With a capillary stress approaching zero, the sand particles are free to move and the water can be more easily expelled during the compaction process. When

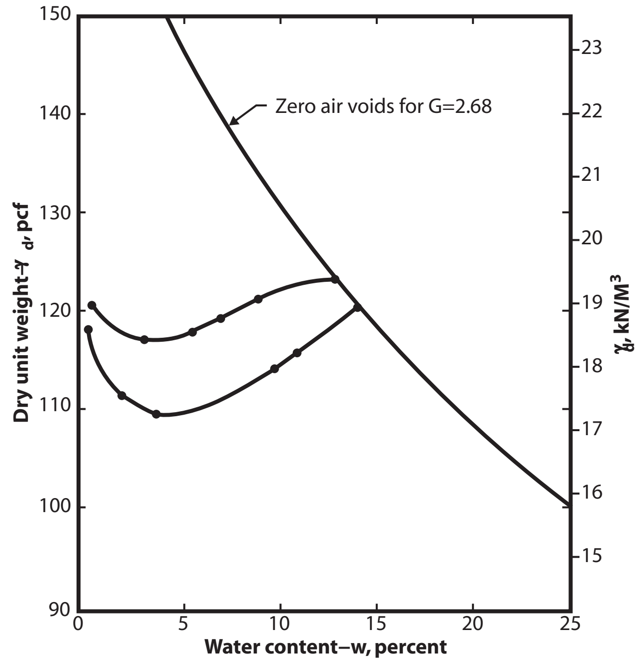


Fig. 8—Compaction curves for sand.

compaction is applied at water contents between the two peaks, the curve is often concave upward, having a low point at the mid-range of water contents. In a moist condition, the sand grains have a film of water around them with capillary stresses that hold the particles in their current loose arrangement, thereby resisting rearrangement by compaction. This water content is referred to as the bulking water content and usually ranges from 3 % to 8 %. These soil properties are most often only associated with clean sands (see Fig. 9).

In consideration of their unique soil properties, the compaction of clean sands and gravels is most frequently tested using relative density criteria. The concept of relative density is used in geotechnical practice to relate engineering

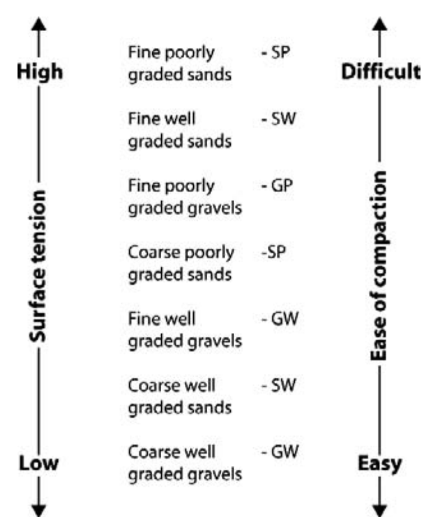


Fig. 9—Surface tension forces and ease of compaction for cohesionless soils.

properties to the range of density conditions possible for cohesionless soils. The relative density test consists of determining the minimum unit weight of a material in its loose state (ASTM D4254) and the maximum unit weight of the material in its most dense state (ASTM D4253). A vibrating table is used to determine the maximum index density because cohesionless soils are more efficiently compacted by vibration than by impact. The in-place density is then expressed as a percentage of the range between the minimum and maximum unit weight. Terzaghi originally defined the relative density of soils, sands, and gravel in 1925. He defined it in terms of the void ratio of the soil, comparing the loosest state to the densest state.

$$D_d = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100 \qquad (2\text{-}3)$$

where:
D_d = relative density
e = in-place void ratio
$e_{\max}$ = maximum void ratio
$e_{\min}$ = minimum void ratio

Typical values for minimum and maximum dry densities for cohesionless soils are shown on Table 2 compiled by Hilf [3]. From these data it can be seen that well-graded cohesionless soils, as defined by the Unified Classification System, can be compacted to very high densities. Well-graded gravels can be compacted to a maximum density approaching 150 lb/ft^3. Poorly graded soils such as uniform fine gravels cannot be compacted this dense.

C. INFLUENCE OF COMPACTION ON ENGINEERING PROPERTIES

In general, the engineering properties of soil including shear strength, compressibility, and permeability (ability to control seepage or leakage) can be improved with compaction at or near the optimum water content as defined by laboratory compaction tests. Shear strength and bearing capacity are increased with additional compaction because the soil particles are compressed into a much tighter arrangement, providing more friction, better interlocking for granular soils, and more soil/water particle attraction for fine-grained soils. Compaction usually produces a lower permeability because the void spaces between soil particles are smaller and the total volume of voids is less than for an equal volume of uncompacted soil. Compaction also usually reduces the compressibility of soil because of the smaller void spaces and the denser soil structure produced by compaction.

Compaction reduces the water holding capacity of the soil. Because the total void space is decreased, the water content of the soil at saturation is less than the void space without compaction. This characteristic of compacted soils may reduce the volume change that takes place when a compacted, saturated soil freezes and thaws as compared with an uncompacted saturated soil. However, compaction will generally not eliminate frost-heave problems in soils that typically exhibit frost-heave problems such as a silty sand (SM) soil.

The effects of compaction on the shrink and swell potential of soil is somewhat less predictable. For certain fine-grained soils (i.e., high plasticity clays), the smaller pore spaces produced by compaction cause higher capillary stresses in the soil during drying. In comparing the shrinkage potential of uncompacted soil with compacted soil, the uncompacted soil has a higher potential for shrinkage because compacted soil is denser and has less void space. However, because the capillary stresses are higher in compacted soil, the shrinkage limit may be lower. Experience has shown that the water content at compaction has an effect on the shrinkage potential of some expansive clay soils. When expansive clay soils are compacted at water contents above

TABLE 2—Compactibility *(F)* of some representative cohesionless soils (where $F = (e_{\max} - e_{\min})/e_{\min}$). A smaller *F* means the soil is more difficult to compact

Classification	$\gamma_{\min}$	$\gamma_{\max}$	$e_{\min}$	$e_{\max}$	Max. size	D_{10}	C_u	C_c	*F*
SP-SM	90	108	0.54	0.84	#16	.058	6.0	2.2	.555
SM	75	97	0.83	1.36	3/4″	.0065	31	5.5	.638
SP-SM	92	113	0.46	0.80	3/4″	.08	3.0	.88	.739
SP	103	124	0.33	0.60	3/8″	.17	5.0	.75	.818
SP	98	122	0.36	0.69	#4	.37	5.1	1.2	.917
SM	99	128	0.31	0.70	3″	.02	240	1.8	1.258
GP-GM	112	129	0.32	0.52	3″	.03	200	.50	.625
GW-GP	111	130	0.27	0.49	3″	.20	105	7.5	.815
GW	111	132	0.25	0.49	3″	2.9	9.7	1.8	.960
GP	114	135	0.22	0.45	3″	2.0	11	.77	1.045
GM	122	141	0.17	0.36	1 1/2″	.025	381	3.0	1.118
GP	114	140	0.18	0.45	3″	1.7	10	.76	1.500
GW	123	146	0.13	0.34	3″	.21	124	1.1	1.615
GW-GM	114	142	0.16	0.45	3″	1.2	15	1.7	1.812

the optimum water content, they generally undergo more shrinkage than clay soils compacted at a water content below the optimum water content. (See Fig. 10)

Clayey soils that contain plastic fines may exhibit swell potential when placed dry of optimum then subsequently wetted. The high swell conditions occur when the field compaction results in dry densities higher than those obtained in the laboratory. Swell potential is reduced near optimum conditions and may be further reduced for conditions wet of optimum and approaching the point of maximum saturation near the ZAV line. These effects are illustrated in Fig. 10 [3].

Compaction at or near the optimum moisture content generally makes the soil stiffer and more rigid when compared with noncompacted soil. In certain cases, this can be detrimental when small movements are expected that may cause cracking of an embankment or other feature constructed of compacted soil. Flexibility can also be improved by compacting soils wet of the optimum water content. In many cases, cracking may be less detrimental than an increase in permeability or low shear strength. The soil embankment structure may be designed with other features to address the cracking problem while achieving the more desirable engineering properties with compaction. An example of this type of balanced engineering design is a compacted dam or dike that is designed to hold water but is also designed with adequate allowance for internal cracking caused by minor differential movements. This type of dam where minor cracking is expected can be designed with a filter-drainage zone in a location where it will intercept additional drainage and localized concentrated leaks. This type of engineered earthwork design internally limits unacceptable cracks and prevents the development of additional cracks by providing a seal at the filter face.

The effects on engineering properties previously discussed in this chapter are achieved when compaction is accomplished at or near the optimum water content. If optimum conditions are not reached, there could be important performance consequences. Conversely, there may be some instances in which

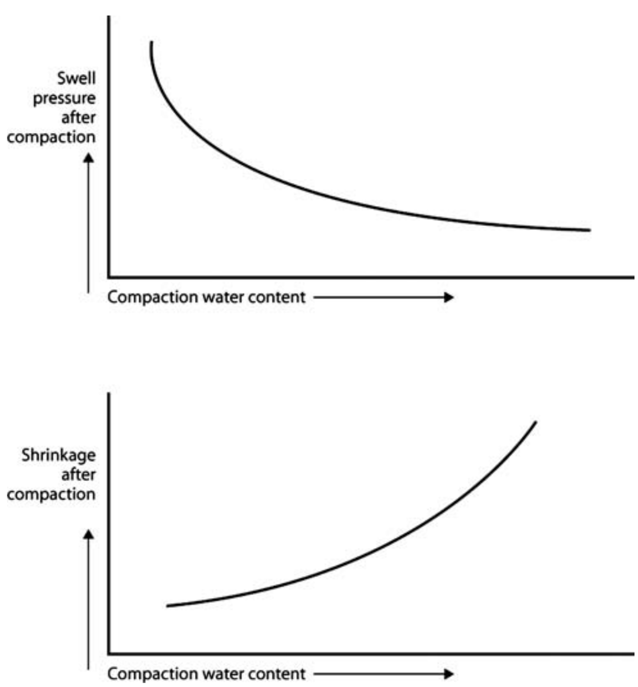


Fig. 10—Effect of compaction water content on shrink–swell of soil.

compaction conditions wet or dry of optimum are desired. There are many definitive texts discussing engineering properties of compacted soils [3–5]. A brief review of how soil moisture conditions at the time of compaction influence some engineering properties follows in this section.

In general, silty and clayey soils have more defined soil structures associated with compaction at conditions wet and dry of optimum. At conditions dry of optimum, soil particles have a flocculated structure dominated by aggregates of soil particles that are held together by capillary forces. Wet of optimum, particles become more aligned. The difference in soil structure affects the engineering properties and the overall soil performance during and after construction.

One acceptable method for measuring the bearing capacity of soil compacted to laboratory conditions dry of optimum is provided by the penetrometer needle test (ASTM D1558). Needle tests can be performed on laboratory specimens to develop a penetration resistance curve, shown on Fig. 2A in Appendix A. The penetration resistance is very high for conditions dry of optimum. As long as the compacted fill remains unsaturated, unconfined compression strength and modulus are very high. However, for soil that is compacted considerably dry of the optimum water content, collapse of the soil structure may occur when it is saturated under loading.

At water contents wet of optimum, needle resistance is very low, indicative of low bearing capacity. The laboratory observation can also be observed during construction when soil is compacted 3–5 % wet of the optimum water content. A low needle resistance and measured bearing capacity may be apparent in the field by observations of "rutting" or "pumping" of the fill under the action of heavy equipment. Compaction of soil wet of the optimum water content will result in higher compressibility and lower shear strength as compared with soils compacted near the optimum water content. In larger embankments or deeper fills, excess pore pressures may build up in lower lifts of the fill placed wet of the optimum water content. These higher pore pressures will cause lower shearing resistance and near-surface rutting and pumping in the embankment or structural fill until the pore pressures dissipate by the removal or displacement of excess moisture.

Needle penetration resistance and bearing capacity is a function of water content and compactive effort. Some individuals have attempted to use the needle penetration tests or probe rods for checking water content and the degree of compaction in the field, but this is recommended only for obtaining a preliminary indication and never to replace testing for compliance with the specifications. This application is not an accurate way to make determinations of water content or unit weight because penetration resistance can also be influenced by soil particle size, angularity, cohesion, and other factors that are not directly related to soil moisture content and density. Because needle resistance converges at optimum conditions for different compactive efforts it may be somewhat useful as a check for moisture; however, it should not be considered accurate or consistent enough to determine compliance with specifications. Proctor (1955) gives guidance on use of the penetrometer needle for field construction control [6].

For conditions dry of optimum, silty or clayey compacted fill appears dense and hard during construction, giving those unfamiliar with soil behavior a false impression that the compaction has been effective in achieving the required density. In-place density tests of soil compacted dry of optimum may plot considerably below the maximum density on the

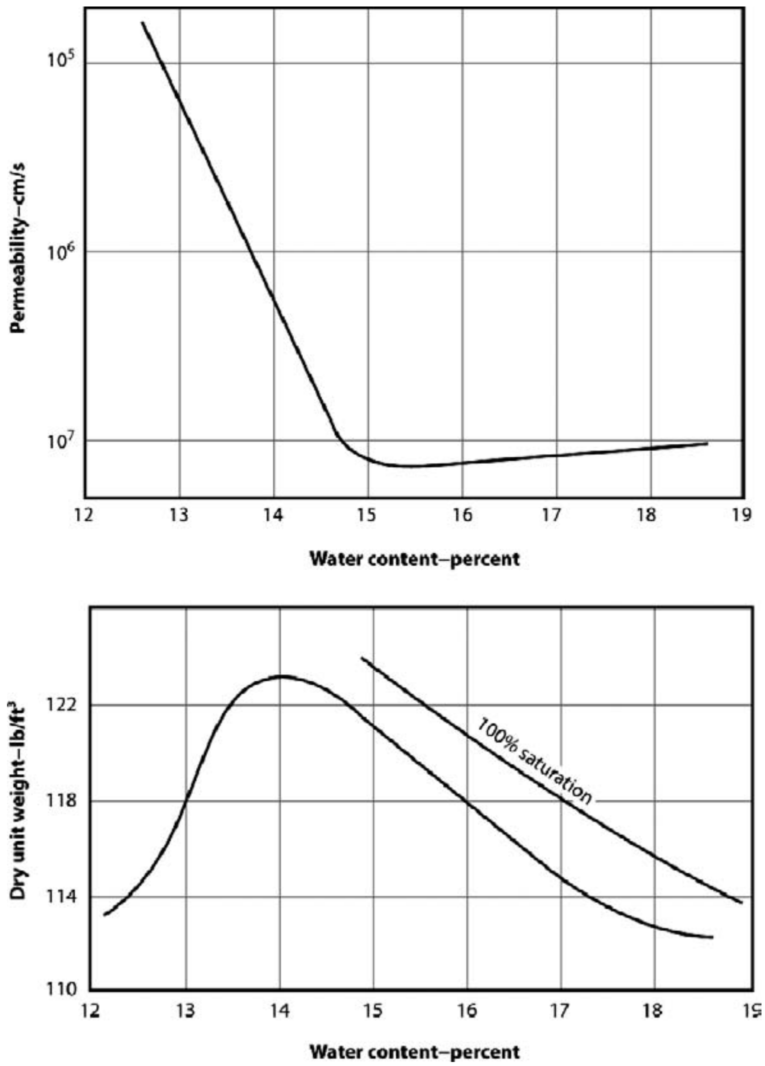


Figure 11—Change in permeability and density with molding water content.

compaction curve for the soil. Consolidation tests on clays compacted at water contents greater than 3 % or 4 % dry of optimum usually exhibit sudden collapse of soil structure when saturated under load [7]. For most construction applications, water content is limited to within 2 % dry of the optimum water content to avoid these problems. Some specifications allow water contents 2–4 % dry of optimum if the settlement characteristics of the soil are understood and in-place density approaching 90–95 % saturation is achieved [11].

Permeability of silty or clayey soils is generally significantly lower when they are compacted wet of the optimum moisture content. Fig. 11 illustrates the results of permeability tests performed on compaction specimens through the range of water contents. These effects are especially important for clayey soils. When clays are excavated and placed at conditions dry of optimum, they tend to form in aggregations sometimes called "clods" or "peds." During compaction the aggregations may remain in the fill, forming a more permeable macroscopic structure than if the peds are completely broken down and remolded. To alleviate this problem, clay soils should be wetted, thoroughly disked, and cured in the borrow area if possible. Sufficient time must be allowed for moisture to penetrate the clods. The penetration can be assisted with repeated disking to break up the clods. The best construction practice, when using dry clays from a borrow area, is to irrigate these soils before the planned use and allow time for water penetration and curing to obtain suitable moisture conditions [8].

D. MECHANICS OF COMPACTION FOR SOILS IN ADVERSE WEATHER CONDITIONS

The practical side of earthwork involves working with available materials and site conditions to develop a high-quality compacted earthwork that meets the design requirements of the project. Most specifications contain some prohibitions regarding the use of frozen materials. As practitioners of soil mechanics, engineers and technicians frequently encounter conditions that are borderline or not within the specified range and unacceptable. These conditions, if handled properly, can provide a quality earthwork project. When borderline conditions are handled improperly, adverse soil and site conditions can be disastrous and lead to future failures. This section provides general guidance on engineering principles and the application of soil mechanics as materials are placed and compacted in the adverse conditions of freezing, too wet, or too dry.

The only materials that can be effectively compacted at temperatures below freezing are rocks and dry granular soils. Other materials exist as hard frozen chunks that cannot be effectively densified. When thawing occurs, the compacted material will be weak, highly compressible, and unsuitable for carrying loads.

Unfrozen soils may be effectively densified at air temperatures below freezing. The key is to complete the compaction process before the water in the soil can freeze. Various techniques can be used to reduce the risk of freezing the water in the soil. These include the use of inclined lifts, round-the-clock compaction, and working in smaller areas. Most specifications prohibit the compaction of soils when they are frozen and/or when the ambient air temperature is less than 32°F. In practice, the crust of frozen soil can be removed and unfrozen borrow material placed and compacted as long as surface freezing does not take place between lifts.

Working extremely wet soils is governed by principles similar to those for frozen soils. Removal of excess water from soils is influenced by several factors, including the evapotranspiration ability of the soil, the soil particle size, the ambient air temperature, the equipment available for mixing and drying of the soil, and the skill of the operators involved in the drying operation. The skill of the on-site technician is very important for determining when "rutting" or "pumping" of the soils is an indication that the in-place moisture content is outside of the range where compaction can be achieved. There are also times when minor discrepancies in the test results can provide nonrepresentative high or low moisture content readings that must be confirmed by skilled observation of the placement and compaction operation To determine whether it is feasible to dry soils or postpone earthwork operation to a drier time, the earthwork contractor typically must develop an understanding of the rate of drying per hour on the basis of the number of equipment passes of a disc or rotary mixer machine. Methods for increasing the rate of drying include the addition of quicklime, repeated reworking, and mixing in drier borrow materials.

Extremely dry soil conditions can also provide unique challenges for obtaining suitable compaction, especially for high-plasticity, fine-grained soils. As mentioned previously, dry silt and clay soils can form clods and provide a bridging effect that can be a detriment to consistent, homogeneous compaction. This soil condition can cause significant problems with long-term soil performance because, when saturated, these soils can become highly compressible months or years after the initial compaction. The problem of soil bridging and lack of compaction can be further increased if the field technician selects the wrong compaction curve so that the percent compaction is falsely evaluated. This condition is very common on sites with unskilled technicians because dry compacted soil may look very hard and highly compacted to an untrained eye.

To minimize the difficulties of dealing with the compaction of dry soils, it is best to focus on three main areas. First, the proper reference density curve must be selected and be checked on a regular basis using the one-point (ASTM D698) or rapid three-point (ASTM D5080) methods. Second, the borrow area needs to be controlled to select areas that are closest to the optimum water content, have been moisture conditioned before placement and compaction, or both. Attempting to adjust the water content more than 4 percentage points in the fill placement area results in excessive processing of a lift to properly incorporate the water. Third, and possibly most important, is to emphasize to contractors and owners that moisture conditioning and the degree of saturation are equally important as density in compaction of soil.

One of the most useful skills that can be obtained by an earthwork engineer is the ability to approximate the in-place water content of soils using visual observation of soil properties and of the interaction of heavy equipment with the soils. For example, if a technician obtains a water-content reading of 1 % dry of the optimum moisture content and field observations indicate that the construction equipment is leaving 4- to 6-in. ruts typical of much wetter soil conditions, then the conflict between the field observation and field data should indicate that additional assessment is required. Field observations are an important part of the information that is used for assessment of the earthwork operation along with the field-testing data. If the field-testing data do not provide adequate correlation to field observations by a qualified inspector or engineer, then additional testing information and the use of possible comparative testing should be considered. It is good engineering practice to verify field-testing methods (e.g., the sand cone or drive cylinder) with an occasional nuclear gauge test. Likewise the nuclear gauge test data may be verified with an occasional sand cone or drive cylinder test.

E. COMPACTION EQUIPMENT

A wide variety of compaction equipment is available for compaction of soils. When selecting the compaction equipment for a project, the type of compactor selected should be dictated by soil type, moisture conditions, and the intended function of the compacted fill. Table 2A in Appendix A provides a summary of the compaction equipment and typical application requirements [9]. In addition to this table, there are several good texts that report on compaction equipment and the research that has been performed [1,3,10]. Table 3A in Appendix A summarizes these same compaction characteristics as they relate to the soils of the Unified Soil Classification.

The effectiveness of compaction equipment depends on equipment details such as weight and contact pressures, coverage or number of passes, lift thickness, water content, soil type, and compactor drum configurations. In addition to the soil and equipment variables, lift thickness is an important variable during compaction. As shown in Table 3A in Appendix A, lift thicknesses for heavier compaction equipment ranges from 6 in. to 1 ft. If lighter equipment is used, it is often not sufficient to compensate by increasing the number of passes because the contact pressure zone of influence is not deep enough. For most work with light equipment in tight areas, in addition to equipment passes, the lift thickness will need to be reduced to obtain satisfactory compaction.

The sheepsfoot roller was developed for compacting fine-grained soils in earth dams. Today, these rollers may be called "tamping foot compactors" and may be somewhat different than the original sheepsfoot rollers. Whether they are called sheepsfoot or tamping foot rollers, the principles are the same and compaction is achieved by the protruding feet that start compacting the soil near the bottom of the loose lift and compacts higher in the lift with succeeding passes. The ability of the sheepsfoot roller's protruding feet to impart kneading and mixing action to the fill helps to produce a more homogeneous fill.

Sheepsfoot rollers are typically recommended for compacting cohesive soils in dams, dikes, landfill liners, or other fills that contain or divert water, or combinations thereof.

Typical sheepsfoot rollers have drums ranging from 48 to 72 in. in diameter and foot lengths ranging from 7 to 10 in. The feet have cross-sectional areas ranging from 5 to 12 in.^{2} and there is a large variation of foot shapes available. Sheepsfoot or tamping roller drums are usually hollow so that they can be loaded with liquid or sand to control the weight of the roller and the pressure applied to the soil by the feet. Loaded sheepsfoot roller weights range from 6,000 lb for older equipment pulled by a tractor up to 80,000 lb for modern, self-propelled rollers. As shown in Table 3A, the foot spacing is such that contact pressures can range from 200 to 1,000 lb/in.^{2}. Compacted lift thickness is normally 6 in. The roller begins compaction at the bottom of the lift and progressively works its way up the lift until it "walks out" (the lift becomes sufficiently dense to support the tips of the roller feet near the surface of the lift). If the roller fails to walk out after multiple passes, the soil is most likely too wet for the foot contact pressure. This usually occurs when the water content of the soil is more than several percentage points wetter than the optimum water content. It may be impossible to obtain a specified density near the maximum density when the soil is this wet.

Depending on the foot spacing and the in-place water content of the soil, it usually takes two to four passes to have the feet contact the entire horizontal plane of the surface being compacted. It usually takes 6–12 passes for the roller to walk out. For light rollers, this may be equivalent to the standard compactive effort (ASTM D698), but for the heavy, modern rollers, the compactive effort will be more nearly at the modified compactive effort (ASTM D1557) or greater.

Vibrating tamping foot rollers are produced to compact a wide variety of soils. They can effectively compact silty, low-plastic soil materials. The addition of vibration is not as effective for clayey soils. It is sometimes used for cohesive soils wet of optimum when a high degree of compaction is not required but one desires remolding. The use of vibration in close proximity to a water table is not advisable

Smooth wheel rollers are often used for highway construction for rolling of subgrade or base course materials. These types of rollers are designed to obtain compaction by static pressure. The compaction surface left by these rollers is also smooth, which is good for proof rolling and paving operations (see Fig. 12).

Vibratory smooth drum rollers are most often used for compaction of cohesionless soils in large areas. Single and double drum rollers are available in static weights up to 15 tons. These are equipped with eccentric vibrators that operate at frequencies from 1,000 to 2,000 cycles per minute. Lift thickness depends on the size of equipment. Smaller rollers and vibrating plate compactors are used with lift thicknesses of 6–12 in. Large double drum rollers are used for compacting rock fills with lift thickness up to 3 ft. Moisture content, as previously discussed, is only critical to cohesionless soils containing appreciable sand content.

Some densification of cohesionless soils may be obtained by vibration under the tracks of crawler tractor equipment. This method of compaction is sometimes used for minor fills not requiring a high degree of compaction. Because the soil pressure under crawler tractor tracks is low, the compaction is achieved mainly from vibration and many passes are needed to make sure the tracks cover the entire surface a sufficient number of times to achieve the desired compaction. This method is not recommended for earth dams, highway fills, or airfield runways.

Rubber-tired rollers can be used on a wide variety of soils. The compaction is achieved by a combination of static weight and near-surface kneading action that occurs near the tire interface. Information on recommended weights, tire pressures, passes, and lift thicknesses is provided in Table 3A. Soil moisture should be such that excessive rutting is avoided. These rollers compact the soil near the surface of the lift to a higher degree than soil near the bottom of the lift. They also may leave a relatively smooth surface in silty and clayey soils, which are typically scarified to improve bonding with the next lift and minimize seepage planes in earth dams or other water-retaining structures. See Figs. 1C to 4C in Appendix C, "Compaction and Testing Equipment," for additional photos.

With the wide range of equipment available for compaction, it is often difficult to specify a single piece of equipment for a particular need. In many cases there will be various pieces of equipment that may be used to achieve the desired product. Many agencies and firms will use combined performance and method specifications for compaction. Previously successful methods may be specified, such as a standard range of dimensions and weights for sheepsfoot rollers. The goal of most compaction standards is that performance or end result is the most important product. If the contractor proposes alternative equipment, test fills are often used to evaluate if the appropriate degree of compaction and the desired fill characteristics (homogeneity and lift bonding) is achieved. Tests can also be made on the test fill to verify that the desired engineering properties have been obtained. More intensive testing is recommended at the beginning of the project to ensure that the proposed equipment is suitable for the job. The ballast (weight of sand or water filling the roller) and number of passes of the equipment can be adjusted to achieve the desired product. Once procedures have been verified and agreed to by the contractor, testing can usually be performed less frequently. It is important to remember that use of the compaction equipment is only one part of the job. Proper excavation in the borrow area, processing, mixing, placing, moisture conditioning, and compacting of the earth fill in an organized manner all contribute to and are equally important in obtaining a quality earth fill.

Fig. 12—Smooth wheel by static pressure and/or vibration as needed.

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3

Laboratory Compaction Tests

A. PURPOSE AND USE OF LABORATORY COMPACTION TESTS

The ASTM test methods for compaction are used to develop a laboratory-derived standard reference density that is used to determine the percentage of compaction and deviation from the optimum water content that is measured by the field in-place density tests. The ASTM Standard Test Methods used for determining compaction characteristics of soil are D698, D1557, D558, D4253, and D4254. These test methods are performed in the laboratory on samples obtained from the borrow area and are used to establish specimen compaction requirements for engineering property testing. The compaction test methods are also periodically performed in the field as construction proceeds to account for variations in soil types and to verify the previously run laboratory standard reference densities.

In general, the impact methods of compaction (ASTM D698 and D1557) are useful for evaluating soils with a percent fines ranging from 5 % to 95 %. The impact methods of compaction testing are typically used as an evaluation standard on fine-grained soils including silty sand, sandy silt, clay silt, silty clay, and clay soils. Because it is difficult to obtain a well-defined moisture density curve for most cohesionless, free-draining soils, an alternative method of density evaluation using minimum (ASTM D4254) and maximum (ASTM D4253) relative density methods was developed using a vibratory table. The relative density test methods are applicable to cohesionless soils with a percent fines ranging from 5% to 15% and a maximum particle size passing the 3-in. sieve. The relative density method of compaction is typically used as an evaluation standard for select sand, gravel, and other free-draining materials.

As indicated by the range of the percent fines provided above, there is some overlap in the use of compaction test methods for soils used in construction. The selection of the most applicable method for the design and field quality control is best determined by the design engineer on the basis of a careful consideration of the required soil performance properties and the practical use of the test methods in the field. The following sections provide an explanation of how the results of compaction tests are used in the design and construction of earth fills.

1. Laboratory Testing for the Design and Determination of Field Control Values

The engineering properties and performance of soil are dependent on many factors, including the in-place density and water content. Several of the engineering properties including shear strength, compressibility, and permeability can be controlled to some degree by altering the amount of compaction applied to the soil. It is a typical design practice to perform compaction and other index tests in the laboratory on soil borrow samples before the tests on shear strength and permeability tests are performed. This two-step process allows the selection of a specified percentage of density and water content for the preparation of specimens that are used for determining other engineering properties. After the maximum dry density and optimum water content have been determined, then the test specimens are prepared at the selected density and water content for performing the shear strength, consolidation, or permeability tests, or combinations thereof to determine if the design engineering properties are achieved. The remolded samples are often tested for triaxial shear (ASTM D2850), unconfined compressive strength (ASTM D2166), consolidation characteristics (ASTM D2435), or permeability (ASTM D5084), or combinations thereof. If the desired engineering properties are not obtained, the remolded density, water content, or both can be adjusted to see if more desirable engineering properties can be achieved. The engineering properties used in the design are selected based on laboratory-measured engineering properties of the samples prepared at the density and water content that the engineer decides can be reliably and economically obtained in the field. Generally, the design analyses are based on the test results of engineering properties for the soils that give the worst-case conditions that are likely to develop in the field.

After the target compaction values used in the design analyses are established in the laboratory, these values are specified for controlling the field compaction process of the earth fill or back fill. The basic assumption is that the engineering properties of the larger mass of soil being placed in the fill can be controlled within the range of density and moisture content values obtained in the laboratory tests. To be valid, a sufficient number of tests must be made to represent all of the soils that will be used. The number of these tests is often determined by engineering judgment or regulatory guidelines.

In special cases for critical structures or if variation in the material occurs, undisturbed samples can be obtained from the actual fill as it is constructed or from a test fill that has been constructed using the specified compaction control. The field verification of the shear strength, permeability, and consolidation characteristic can be based on test values obtained from specimens prepared from the undisturbed samples to make sure the compaction has achieved the desired engineering properties. The number and frequency of the verification are most often determined by the risk associated with the proposed construction and the expense and time delays associated with the test procedures.

2. Field Quality Control and Compaction Tests

Laboratory-controlled compaction tests for standard reference density and optimum water content are made on all soils expected to be used in the construction of the fill or back fill. Some standard reference compaction tests should be performed during the process of construction on materials obtained from the fill, especially when considerable variation is found in the borrow area. Another reason for standard reference compaction testing of the material during placement is to account for mixing of different materials during the earthwork construction process and to verify that the laboratory compaction test curves are representative of the borrow material properties. Standard reference compaction curves are used as the comparison of field in-place conditions to determine compliance with the construction specifications. Measured in-place dry density and water content must be within the range of values specified. The specified range, when compacted, is usually expressed in terms of a minimum percentage of the maximum dry density at a range of water content above and below the optimum water content. For example, the specified range of compaction might be 90–95 % of the maximum dry unit weight at a water content ranging from the optimum moisture content to 4 percentage points above the optimum water content.

It is worth emphasizing that laboratory and field compaction tests provide the maximum dry density and optimum moisture content that is used as the standard reference for each specific and unique soil type. Field in-place density tests measure the actual density that is developed in the field by the compaction of the soil. As noted before, there may be regular variations in the maximum dry density of a soil because of blending or variations in the borrow area that require additional laboratory or field compaction testing or the use of methods described in Chapter 6. The accuracy of the in-place evaluation of density is only as accurate as the standard reference compaction test that is used for comparison. The laboratory or field compaction test is analogous to a frame of reference and the field in-place density is analogous to a moving point that must be quantified. The difficulty occurs when the on-site soil conditions create the old-fashioned engineering problem of a "moving point in a moving frame of reference." To account for these soil and site conditions without resorting to a large amount of redundant testing requires that the geotechnical engineer obtain a practical understanding of the soil and practical application of the ASTM compaction test methods on a wide variety of soil conditions. Chapter 6 provides some guidance to assist in this evaluation of testing.

B. DESCRIPTION OF STANDARD AND MODIFIED COMPACTION TESTS

1. ASTM D698, Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort [12,400 ft-lbf/ft^3(600 kN-m/m^3)] and ASTM D1557, Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort [56,000 ft-lbf/ft^3(2,700 kN-m/m^3)]

These two test methods are identical except for the compactive effort that is applied to the soil. The only differences are the mass of the rammer, the height of rammer drop, the number of rammer blows, and the number of layers of soil to which compaction is applied in the mold. Both test methods are used to determine the compaction characteristics of a soil with a specified compactive effort. Fig. 1 shows the number of layers, the mass of the rammer, the height of the rammer drop, and a sample calculation of the compactive effort for each of the two test methods.

The compaction characteristics are demonstrated by the shape of the compaction curve, its relationship to the 100 % saturation curve, and the maximum dry density and optimum water-content values that are determined from the compaction curve. The compaction curve for each method is plotted from the calculated results obtained by following the test procedures for a series of compaction specimens.

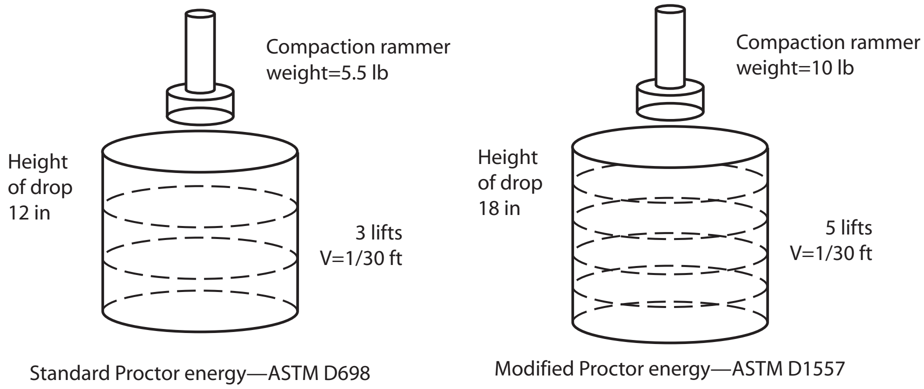
These test methods can be used for soils that have 30 % or less by weight of particles retained on the 3/4-in. (19.0-mm) sieve. Three alternative procedures are included that require different mold sizes and maximum particle sizes in the test specimens. For each procedure, the particles larger than a size specified by the test procedure are removed from the soil by screening before the test is made.

Procedure A uses a 4-in. (101.6-mm) diameter mold that is 4.584 in. (116.4 mm) in height and has a volume of approximately 1/30 ft^3 (with some variance for dimension tolerances). The material is screened to pass the No. 4 sieve so that the largest particle in the test is 4.75 mm in diameter. This procedure may be used for all soils in which 20 % or less of the mass of the material is retained on the No. 4 sieve. If more than 20 % is retained on the No. 4 sieve, other procedures must be used. For the standard compactive effort (D698), the material is placed in the mold in three layers, and five layers are used for the modified effort (D1557). Each layer is to have approximately equal thickness after compacting. After each layer is placed in the mold in a loose condition, compaction of the layer is achieved using a 5.5-lbf (24.4-N) rammer dropped from a height of 12 in. (305 mm) for the standard effort (D698) and using a 10-lbf (4.54 kg) rammer dropped from a height of 18 in. (457.2 mm) for the modified effort (D1557). The rammer is dropped 25 times on each layer for both test methods. The rammer is moved around to uniformly cover the surface of the layer while making the drops.

Procedure B also uses a 4-in. (101.6-mm) diameter mold that is 4.584 in. (116.4 mm) in height and has a volume of approximately 1/30 ft^3 (with some variance for dimension tolerances). The material is screened to pass the 3/8-in. (9.5-mm) sieve so that the largest particle in the test is 9.5 mm in diameter. For the standard effort (D698), the material is placed in the mold in three layers, and five layers are used for the modified effort (D1557). After each layer is placed in the mold in a loose condition, compaction of that layer is achieved using the same rammers and procedures as specified for Procedure A.

Procedure C uses a 6-in. (152.4-mm) diameter mold that is 4.584 in. (116.4 mm) in height and has a volume of approximately 0.075 ft^3 (with some variance for dimension tolerances). The material is screened to pass the 3/4-in. sieve so that the largest particle in the test is 19.0 mm in diameter. This procedure may be used for all soils in which 20 % or more of the mass of the material is retained on the 3/8-in. sieve and less than 30 % by weight of the material is retained on the 3/4-in. (19.0-mm) sieve. If more than 30 % is retained on the 3/4-in. sieve, other procedures must be used (see Section C in Chapter 4 of this manual). For the standard effort (D698), the material is placed in the mold in three layers, and five layers are used for the modified effort (D1557). After each layer is placed in the mold in a loose condition, compaction of that layer is achieved using the same rammers as specified for Procedure A of each test method. For Procedure C, the rammer is dropped 56 times while moving it around to uniformly cover the surface of the layer.

The test method procedures for ASTM D698 and D1557 contain all of the details for performing each test. The



$$= \frac{5.5\ \text{lb} \times 1\ \text{ft} \times 25\ \text{blows/lift} \times 3\ \text{lifts}}{1/30\ \text{ft}^3} = 12{,}375\ \text{ft–lb/ft}^3$$

$$= \frac{10\ \text{lb} \times 1.5\ \text{ft} \times 25\ \text{blows/lift} \times 5\ \text{lifts}}{1/30\ \text{ft}^3} = 56{,}250\ \text{ft–lb/ft}^3$$

Fig. 1—Standard Proctor energy application and modified Proctor energy application.

following items are provided to point out some precautions or provide suggestions for making the tests and checking for errors or both. The basic steps and commentary on both test methods (D698 and D1557) are as follows:

STEP 1

The material is screened over a certain size sieve as specified by the method. It is helpful for controlling the water content to have the entire sample of material at a uniform water content before dividing it into five or six specimens before testing. The material may be dried, but this is not always necessary if the oversize can be removed without having a lot of the finer portion sticking to the oversize. There is an advantage in dividing the soil into specimens of equal mass; 5 lbm (2.3 kg) is suggested for Procedures A and B and 13 lbm (5.9 kg) is suggested for Procedure C.

STEP 2

The five or six specimens of the material are prepared at different water contents, usually approximately 2 percentage points apart with one specimen near the optimum water content and two specimens wet and two specimens dry of the optimum water content. It is suggested the specimen at optimum water content be prepared first while keeping track of the amount of water added to the specimen. The trial water content for optimum is based on judgment, and some guidance is given in the test method for making these judgments (add water until it clumps into a firm ball when squeezed in the hand). The spread of 2–3 percentage points in water content and can be controlled by adding the amount (mass) of water to bring the soil to the desired water content relative to the first trial water content for optimum. Each specimen is placed in an airtight container and allowed to "cure" (stand) for a period of time as specified in the ASTM procedure to allow the water content to become uniform throughout the specimen.

STEP 3

The compaction mold is then assembled and the mass of the empty mold is determined. Each mold must be periodically measured using precise calipers and an inside micrometer or by using the water filling method to see that its volume is within tolerance. The measured volume is recorded to the nearest 0.0001 ft^3 (1 cm^3). This measured volume (not the nominal volume) must be used in calculating the unit weight values for the test results. Failure to use the actual volume of the mold in favor of the nominal volume for calculating the unit weight makes this one of the more common sources of error.

STEP 4

The upper collar is assembled on the mold and the soil is compacted in the mold in equal layers. The number of layers, the rammer size, fall distance, and the number of blows are specified by the method and procedure. Because the soil is placed in the mold in a loose state then compacted, the layer thickness before compaction is determined by judgment and usually requires experience to obtain uniform compacted layer thicknesses. The procedure in the standard gives guidance on control of the rammer guide sleeve, on keeping the rammer vertical, and on the rate and coverage of the rammer blows to the surface of the soil layer. In addition, when using the manual rammer, care should be exercised to prevent the rammer from bouncing or impacting more than once for each blow by catching the rammer handle after the first impact. All of these precautions are important in obtaining accurate results.

STEP 5

After all of the layers are compacted, the top collar is removed without disturbing the compacted soil in the mold. This sometimes requires using a spatula or knife to separate the compacted soil from around the inside edge of the collar before twisting it to remove it. After the collar is removed, the top layer of soil is trimmed to be level with the top of the mold. As the soil is trimmed from the top of the mold, some of the larger particles that protrude across the plane of the top of the mold may be removed, leaving a small void. These voids should be filled in with finer soil particles using finger pressure to make it firm. The mass of the mold filled with compacted moist soil is then determined on a calibrated scale and the soil is removed from the mold.

STEP 6

The mass of the empty mold is subtracted from the total mass and the unit weight of the moist, compacted specimen is calculated by dividing the mass of the moist soil by the measured volume of the mold. Steps 3–6 above are repeated for all of the five or six specimens and the results of wet unit weight are compared to make sure two points will be located on each side of the optimum water content. A comparison of wet unit weight values can be made to determine when two points are on each side of the optimum water content.

STEP 7

Immediately after removing the soil from the mold, a sample of the compacted soil is obtained from each of the compaction test specimens for a water-content test or the entire specimen is used. The procedures outlined in the ASTM Standard Test Methods give guidance on obtaining specimens for the water-content tests. ASTM Standard Test Method D2216 is usually used to determine the water content. Other methods of water content may be specified for a specific project, such as when tests must be made in field conditions where more rapid results are desired or where the specified oven is not available. These water-content test methods may be D4959 (by direct heating), D4643 (by microwave oven), or D4944 (by carbide gas pressure tester).

STEP 8

After the water-content samples have dried, the specified length of time in the temperature-controlled oven or by using other methods as may be specified and the water content are calculated for each specimen, the dry density is calculated as outlined in the procedure. Using an assumed or measured value for specific gravity, the 100 % saturation line or zero-air voids line is plotted. The dry density compaction curve is then plotted. Sometimes a wet density curve may also be plotted. It is a good idea to use the same graph scales each time for plotting compaction curves so that the shape of the curves can be compared for different materials. Good curves can usually be obtained where a square grid is used with the unit weight in lbf/ft^3 twice as large between grid points as the water content in percent; for example, 4 lbf/ft^3 between grid points for unit weight on the ordinate and 2 % water content between grid points on the abscissa. For example forms, see 3B, 4B, and 5B in Appendix B.

STEP 9

Checks for errors should be made after the curves are plotted. For soils having more than 10 % fines (material passing the No. 200 sieve), the dry density curve should have a well-defined peak and a general concave downward parabolic shape. The wet side slope of the compaction curve should become nearly parallel to the 100 % saturation curve so that the soil is between approximately 88 % and 95 % saturated. Any compaction point wetter than 95 % saturation is likely in error. The percent saturation can be easily determined by dividing the water content at any point on the curve by the corresponding water content for 100 % saturation at the same unit weight (see equation 2 from chapter 2). If a relatively smooth curve cannot be plotted or if errors are indicated from percent saturation checks, the test should be done over again using another portion of the sample (not previously compacted) if possible. If errors are indicated from percent saturation checks, then the compaction curve or specific gravity tests should be retested. Fig. 2 shows an example of the determination of the curve and a check for percent saturation.

For sand or gravelly sand soils having less than approximately 5 % fines, the dry density curve may have a concave upward shape with a peak very near 100 % saturation and another peak near a completely dry condition. Using clean sands or gravels, it may be possible to have points plot very close to the 100 % saturation line provided the soil can retain sufficient water during the compaction process.

For soils having between approximately 5 % and 10 % fines, the shape of the curve can be concave upward or downward depending on the nature of the fines and their ability to overcome the bulking tendency of the sand particles. No point should plot on the wet side of the 100 % saturation curve for any soil.

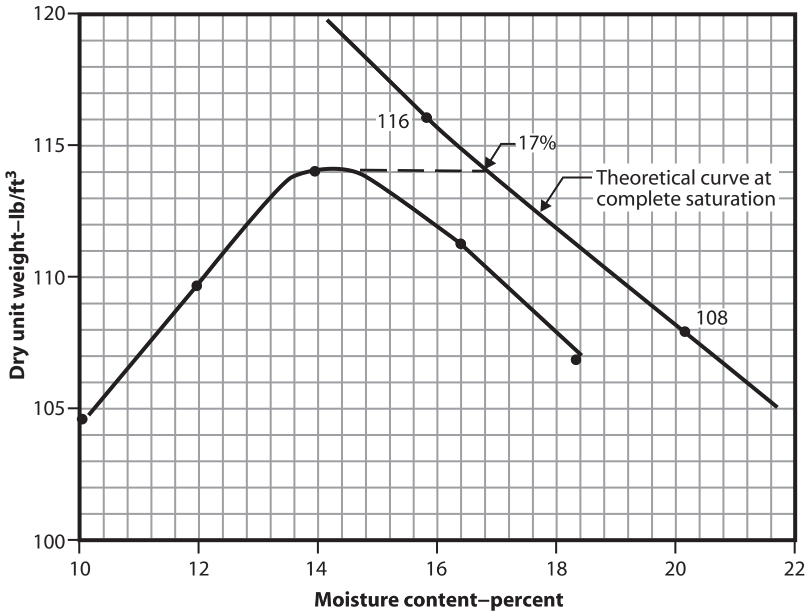
The standard and modified compaction test methods (D698 and D1557) are most often used to represent compaction of soils having more than 10 % by weight of fines (passing the No. 200 sieve). These soils are best compacted by impact or kneading compaction. This includes use of sheepsfoot rollers, vibrating sheepsfoot rollers, pneumatic rollers, and impact manually directed compactors.

It should be noted that the compaction characteristics determined by these methods can be related to the soil index properties of gradation, specific gravity, and plasticity. Soils having different gradations, specific gravity, and plasticity will have different compaction characteristics. Sometimes small differences in these values can cause substantial differences in the compaction test results. The soil tested also has a certain maximum particle size. If oversize particles were retained on the sieve used in preparing the sample for testing and discarded, the compaction characteristics may not represent the characteristics of the total soil. Corrections can be made to the results to account for the oversize particles that were discarded. These corrections are explained in Chapter 4.

2. ASTM D558, Test Methods for Moisture-Density Relationship of Soil-Cement Mixtures

These test methods are used to determine the compaction characteristics of a soil-cement mixture for the compactive effort of the test that is [12,400 ft-lbf/ft^3 (600 kN-m/m^3)], the same as the standard compaction test (D698). The compaction characteristics are demonstrated by the shape of the compaction curve and its relationship to the 100 % saturation curve, and by the maximum dry unit weight and optimum water-content values that are determined from the shape of the compaction curve. The compaction curve is plotted from the calculated results obtained by following the test procedures for a series of compaction specimens and constitutes the final results of the test.

Two alternative procedures (methods) for the preparation of soil cement specimens are included that allow different maximum particle sizes in the test specimens. For each method, the particles larger than a specific size are removed from the soil by screening before the test is made. Both methods require a 4-in. (101.6-mm) diameter mold that is 4.584 in. (116.4 mm) in height and has a volume of approximately 1/30 ft^3 (with some variance for dimension tolerances). Also, both methods use a 5.5 lbf (2.5 kg) rammer dropped from a height of 12 in. (305 mm). The rammer drops are 25 times on each of three layers for both methods. The rammer is moved around to uniformly cover the surface of the layer while making the drops. If a manual rammer is used, it must be equipped with a guide sleeve that controls the drop height and has certain requirements for clearance between the rammer head and the sleeve. These requirements are detailed in the test method.



Classification *SM/ML*
Gravel *13* %
Sand *40* %
Fines *47* %
sat

Atterberg limits
Liquid limit *28* %
Plasticity index *4* %
Shrinkage limit ____%

Specific gravity
Minus no. 4 *2.66*
Bulk ____
Fines ____
Apparent ____
Absorption ____%

Remarks____

Compaction
Method____
Percentage larger than tested *13*
Maximum dry unit weight *114.0* lbf/ft^3
Optimum moisture content *14.0* %
Degree of saturation @ opt ____%
Penetration resistance @ opt *1000* lbf/ft^2

Note: Theorectical curve at complete saturation and the zero air voids curve from Chapter 2 are one and same.

- **Determine the 100% saturation curve using selected dry density values with the following formula (equation 2-2):**

$$w_{sat}(\%) = 100 \times \left[\frac{\gamma_{water}}{\gamma_d} - \frac{1}{G_s} \right]$$

1. For γ_d of 116 lb/cf

$$w_{sat}(\%) = 100 \times \left[\frac{62.4}{116} - \frac{1}{2.66} \right]$$
$$= 15.8\%$$

2. For γ_d of 108 lb/cf

$$w_{sat}(\%) = 100 \times \left[\frac{62.4}{108} - \frac{1}{2.66} \right]$$
$$= 20.2\%$$

- **Check the percent saturation at optimum moisture and maximum dry density.**

At 114 lb/cf the water content at 100%=17.0%

Percent saturation at optimum moisture
=14.0/17.0×100=82.3%

Fig. 2—Theoretical curve at complete saturation and a check of percent saturation.

For Method A, the material is screened to pass the No. 4 sieve so that the largest particle in the test is 4.75 mm in diameter. For Method B, the material is screened to pass the 3/4-in. sieve. Special requirements for treatment of the particles between the 3/4-in. sieve and the No. 4 sieve are required when using Method B.

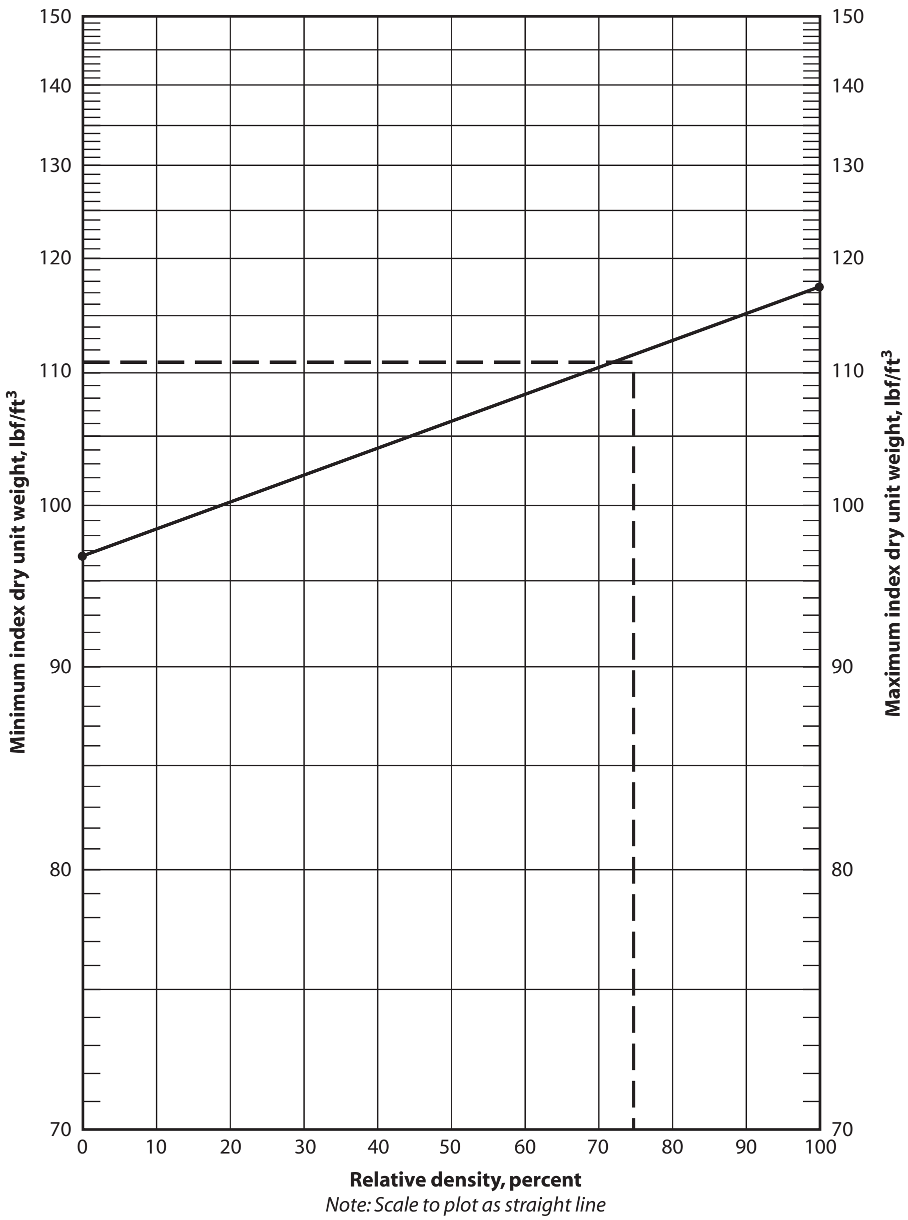
The general procedures and precautions discussed for test methods D698 and D1557 apply to this method; however, some details differ because this test is used in conjunction with methods D559 and D560 and because some special techniques apply specifically to soil cement. The procedures of the method should be studied and strictly followed in making the test.

C. DESCRIPTION OF THE INDEX DENSITY AND UNIT WEIGHT TEST METHODS

As mentioned previously, for cohesionless, free-draining soils, it is often more representative to use a relative density value as the standard reference for determining if an earth or rock fill has adequate density. The index density and unit weight test methods are applicable for soils containing 15 % or less fines and a maximum particle size of 100 % passing the 3-in. sieve, provided the soils have cohesionless, free-draining characteristics. This section provides a description of the test methods that ASTM has developed for evaluating the density of cohesionless, free-draining soil and rock using a vertically vibrating table.

1. ASTM D4253, Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table, and ASTM D4254, Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density

These test methods determine the minimum and maximum index dry density/unit weight that are then used to compute the relative density/unit weight for design and field quality



Minimum index dry unit weight ___96.6___ lbf/ft³ In-place dry unit weight ___111.0___ lbf/ft³

Minimum index dry unit weight ___117.3___ lbf/ft³ Relative density ___74___ %

Sample no. ___5___ Hole no. ___3___ Depth ___10___ Ft.

Test no. ___4___ Station ___12+40___ Elevation ___6685___ Offset ___28R___

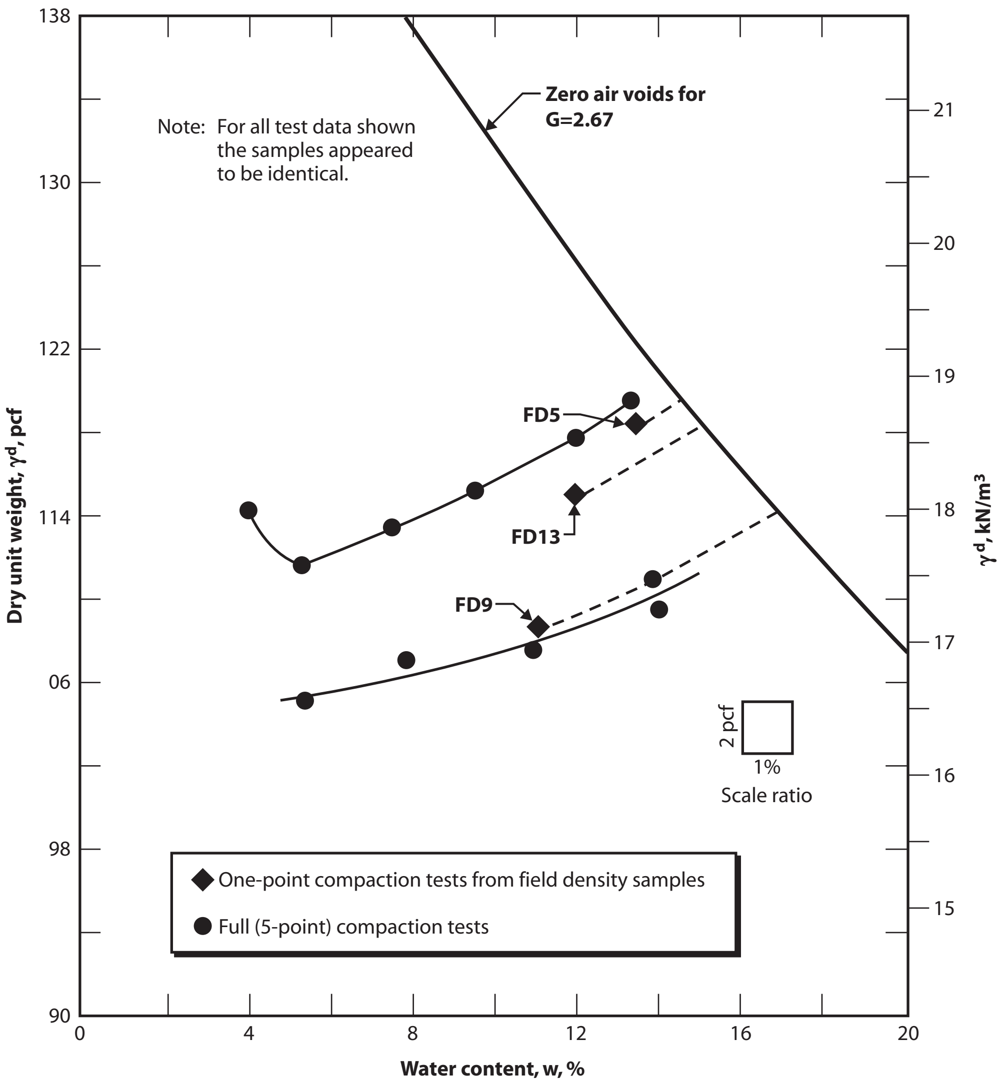
Prepared by ______ Checked by ______ Figure ______

Fig. 3—USBR 7250 relative density plot *(example).*

control purposes. The relative density/unit weight expresses the degree of compactness of a cohesionless soil with respect to the loosest or least dense and most dense condition as defined by standard laboratory procedures. Only when viewed against the possible range of variation, in terms of relative density/unit weight, can the density/unit weight be related to the compaction effort used to place the soil in a compacted fill. In a similar manner, this relative density/unit weight also indicates a volume change and stress-strain tendencies of the soils when subjected to external loading.

The maximum index density/unit weight of a free-draining soil is determined by placing an oven-dried soil (Test Methods 1A and 1B) or a wet soil (Test Method 2A and 2B) in an appropriately sized mold on the basis of the maximum particle size of the soil. After the soil is placed in the mold, a 2-lb/in.^{2} surcharge weight is applied to the surface of the soil and a vertical vibration is applied to the mold, the soil, and the surcharge weight. The vertical vibration may be applied using an electromagnetic, eccentric, or cam-driven vibrating table providing a sinusoid-like, time-vertical displacement relationship in accordance with the requirements of ASTM D4253. Following the vibration, the height of the specimen is obtained for use in conjunction with the cross-sectional area of the mold to calculate the volume of the specimen. The soil is then dried to obtain the moisture content of the sample, and the maximum density/unit weight is computed by dividing the oven-dried mass of the soil specimen by the calculated volume.

As with any test methods, some potential sources of error may occur during the testing of samples for maximum density/unit weight. One potential source of error can occur if the water level during vibration is above the soil surface. This testing condition can influence the impact of the vibration on the surcharge weight, thereby providing a maximum density/unit weight that would be lower than the actual value. Other potential sources of error include innapropriate use of the test method for a given soil, segregation of the soil during



Soil tested: Clean, gravelly sand
Compaction procedure: ASTM D1557, Method D
Year tested: 1982

Fig. 4—Compaction curves and field unit weight tests for clean sand.

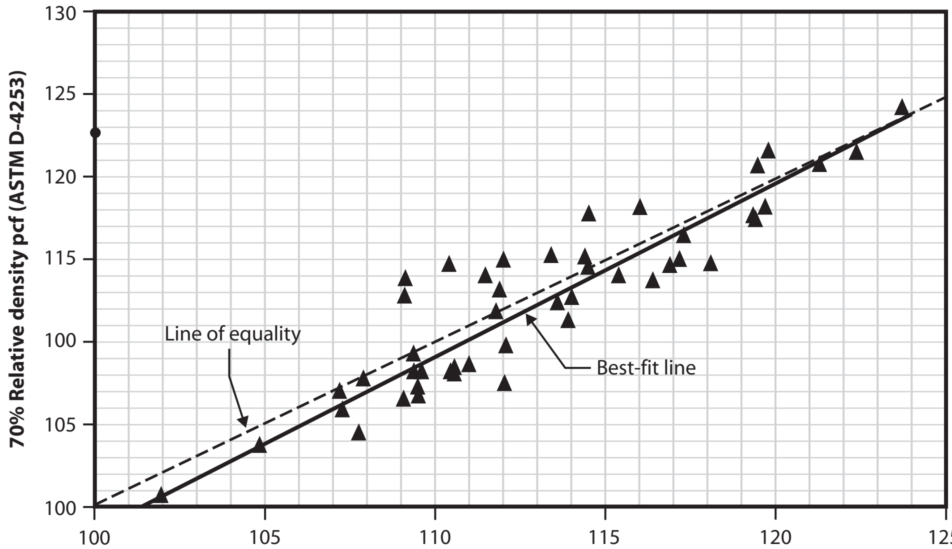


Fig. 5—Seventy percent relative density vs. 1 pt. compaction.

processing, increase in soil moisture due to condensation or absorption of moisture in the air, or misalignment of the guide sleeve in the mold, or combinations thereof. These sources of potential error are typically minimized or eliminated by careful attention to detail or review of the test results by a qualified geotechnical engineer or both.

The test method for minimum density/unit weight is ASTM D4254 and it represents the theoretical loosest condition that can be attained by a cohesionless, free-draining soil using a standard laboratory procedure. The test method consists of loosely pouring an oven-dried cohesionless soil into a mold in a manner that prevents bulking, particle segregation, and compaction of the soil. Potential sources of error include inadvertent jarring of the mold, impact compaction of the soil's particles if the soil is placed too fast, and segregation of the soil when filling the mold. Adhering to the requirements of the test method minimizes these potential sources of error.

Experience has shown that for a clean sand, maximum dry density and minimum dry density are not independent variables, thereby indicating that there is a relationship between these values. Because this is the case, it may not be necessary to measure both values. If the in-place sands and gravels are consistent, then the compaction can also be controlled or checked by comparing the in-place measured values to the maximum unit weight. The ASTM Standard Test Method for Determination of Relative Density includes the measurement of maximum and minimum unit weight of the soil and the calculation of relative density by using two standard test methods. ASTM Designation D4253 is the ASTM Standard Test Method for Maximum Index Density and Unit Weight of Soils Using a Vibrating Table. ASTM Designation D4254 is the ASTM Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density. The value of relative density is then obtained by using a formula such as equation 3-1, which is the Terzaghi formula from equation 2-3 expressed in terms of unit weight.

$$D_d = \frac{\gamma_{d\,\max}}{\gamma_d}\left[\frac{\gamma_d - \gamma_{d\,\min}}{\gamma_{d\,\max} - \gamma_{d\,\min}}\right] \times 100 \qquad (3\text{-}1)$$

where:
D_d = relative density
γ_d = in-place dry unit weight
$\gamma_{d\max}$ = maximum dry unit weight
$\gamma_{d\min}$ = minimum dry unit weight

In practice, a relative density of 70 % or greater has been found to be satisfactory for most conditions. For field control, the value specified can be determined by the use of the chart in Fig. 3. Individuals who want to use a comparison of the maximum unit weight to control an earth fill can use one test (i.e., D4253), and those who want to use the relative density procedure can use both test procedures (i.e., D4253 and D4254).

Experience has also demonstrated [1] that impact compaction tests (ASTM D698 or ASTM D1557, or both) can also be applied to clean sands and gravels to get similar results for the maximum dry unit weight as those provided by the vibrating table method. Fig. 4 shows this relationship. Fig. 5 [2] shows a correlation between a one-point ASTM D698 test and relative density values for 29 clean filter sands. When using these methods it is important to note that particle breakdown may also cause problems with obtaining consistent test results; therefore, regular observations and confirmatory gradation tests may have to be conducted to provide representative testing information.

References

[1] Poulos, S.J., "Compaction Control and the Index Unit Weight," Geotech. Test. J., Vol. 11, 1988, pp. 100–108.

[2] McCook, D., "Correlations between a Simple Field Test and Relative Density Values," Technical Note, J. Geotech. Eng., Vol. 122, 1996, pp. 860–862.

4

Standard Test Procedures for Determining Density or Unit Weight of Soil in Place

A. PURPOSE AND USE OF IN-PLACE DENSITY OR UNIT WEIGHT TESTS

In-place density or unit weight tests are performed on the soil to determine the undisturbed or in-place soil properties for field quality-control purposes to determine whether an earth or rock fill has been compacted to the desired or specified density or unit weight. The field in-place density tests are also conducted for making comparisons and calculations related to volume and weight relationships such as void ratio or degree of saturation. For design purposes, the tests are often conducted to provide information for calculating soil loads or stresses within a soil mass or exerted by a soil mass. Another important reason for these test procedures is to assist in the control and management of the borrow area by providing information for calculating volume differences between excavated volume and the volume of the compacted fill areas.

B. STANDARD TESTS FOR DETERMINING IN-PLACE DENSITY OR UNIT WEIGHT

This section provides a description of the methods that are most often used for the determination of in-place density or unit weight. To assist the practitioner in earthwork application, the description of the test methods also provides a listing of common applications, advantages, disadvantages, and potential sources of error. The most common ASTM test methods that are used to measure the in-place density or unit weight are the following:

- D1556–Density and Unit Weight of Soil in Place by the Sand cone Method (sand cone method)
- D2167–Density and Unit Weight of Soil in Place by the Rubber Balloon Method (rubber balloon method)
- D6938–In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (nuclear method)
- D2937–Density of Soil in Place by the Drive Cylinder Method (drive cylinder method)
- D4564–Density of Soil in Place by the Sleeve Method (sleeve method)
- D4914–Density of Soil in Place by the Sand Replacement Method in a Test Pit (sand replacement method)
- D5030–Density of Soil in Place by the Water Replacement Method in a Test Pit (water replacement method)
- D5080–Rapid Determination of Percent Compaction (rapid method)

1. Sand cone Method (D1556)

The purpose of the sand cone test is to determine the volume and mass of soil material from a hole excavated into the compacted fill. The test is made by excavating a hole in the compacted fill and saving all of the material that is removed in a covered container to prevent moisture loss. The volume of the hole is measured by filling it with sand having a known unit weight. The apparatus is shown in Fig. 1. See Photo 5C in Appendix C.

To ensure consistency and accuracy of the test results, the surface of the fill where the hole is to be made must be smooth. A template or base plate is placed over the spot where the hole is to be made before beginning the excavation. The base plate has a machined receptacle ring or guide to receive a metal cone attached to a container that holds a known amount of sand. The volume of sand used in the hole is usually calculated from the known unit weight of sand and determinations of the mass of sand in the container before and after filling the hole. An apparatus having a valved orifice, funnel, and cone is used to place the sand in the hole. There is a mass of sand required to fill the apparatus cone and the base plate, which must also be determined before the test is made so it can be subtracted from the total mass of the sand used.

The material excavated from the hole is placed in a plastic bag or other waterproof container and transported to a field laboratory where the mass and the water content of the material removed from the hole are determined. The wet and dry unit weight and the water content of the in-place soil can be calculated from the determinations of mass of material taken from the hole, the volume of the hole, and the water content of the material from the hole. The water content of the soil is determined by using one of the standard test methods explained in Chapter 5 of this manual.

APPLICATIONS

This test is used on cohesive soils including all fine-grained soils (CL, ML, CH, MH) and silty or clayey sands or gravelly sands (SM, SC) or silty or clayey sandy gravels (GM, GC) with particles up to approximately 1.5 in. in diameter (see ASTM D4287). Most users limit the test to these materials; however, it can be used on materials containing larger particles by using a 12-in.-diameter cone to provide a test-hole volume large enough to be representative of the soil. This test is generally not used on noncohesive soils or soft, saturated soils because it is difficult to maintain a stable hole while performing the test. The test procedure in the ASTM D1556 test method provides a more detailed list of explanations and precautions.

ADVANTAGES

Because a hole is excavated in the in-place soil, the tester can visually observe the nature of the soil being tested and make judgments relating to whether it is representative of the fill. If a large enough volume of soil is removed, then a one-point compaction test can be performed on the soil removed from the excavating hole, thereby making sure the maximum density and optimum water content used for comparison are the correct ones. When the sand cone test is used as part of the

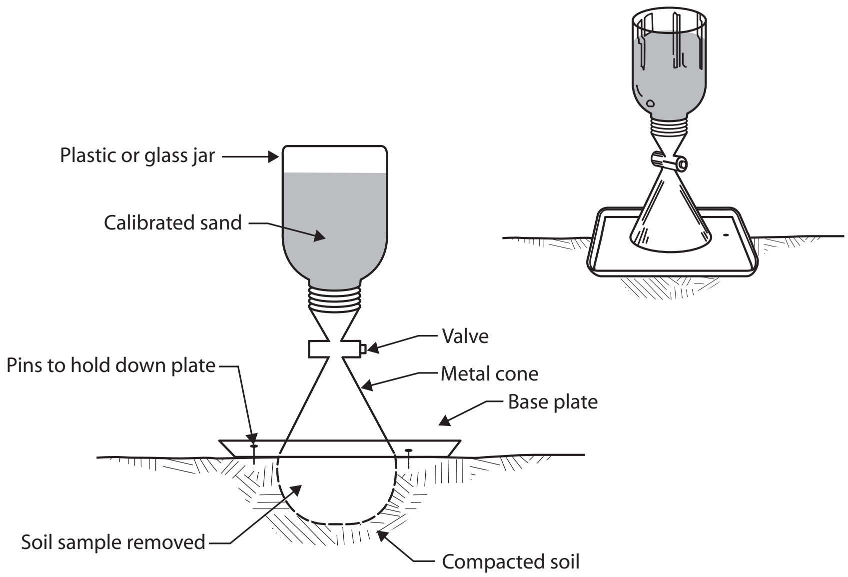


Fig. 1—Sand cone method (ASTM D1556).

rapid method (D5080), the in-place density is measured and a rapid three-point compaction test is performed on the material excavated from the hole. When used together, the rapid method (D5080) and sand cone method (ASTM D1556) provide a good way to achieve accurate results within a few hours. With the sand cone test, the oversize particles can be removed from the material excavated from the hole by screening to determine the exact percentage of oversize or determine the properties of the oversize, such as their mass and volume. It is important to note that the accuracy of the test is very much operator dependent, and careful attention should be given to all of the measurements and calculations involved in the test procedures.

DISADVANTAGES

The test can be rather labor intensive and takes considerable time to complete one test at one location. The construction process must be disrupted or at least curtailed for a period of time near the location of the test to provide safe access for those performing the test and to reduce the potential for vibration while the sand is flowing into the hole. The results of the test may not be available for up to an hour if moisture determination by direct heating is required. The final moisture content may take up to one day if the oven-dry method is used. If the sand cone test is used as part of a coordinated testing program with the rapid method (D5080), testing for water content is not necessary and results can be obtained within approximately 2 h.

PRECAUTIONS AND POTENTIAL SOURCES OF ERROR

As with most field determinations of density or unit weight, there are precautions that should be noted and potential sources of error that need to be understood to ensure the accuracy of the information provided by the sand cone test. The most significant precautions or potential sources of error for the sand cone test are as follows.

Selection and Storage of Sand

The sand must be uniform in size (narrowly graded), dry, free flowing, and devoid of fines. Rounded or surrounded particles are best because angular sand is more susceptible to bridging across irregularities. Sand that is too coarse will also be subject to bridging across irregularities. Sand that is too fine will be subject to large changes in unit weight from changes in atmospheric humidity. The standard test method procedure spells out the limits for the type and gradation of the sand that must be adhered to for accurate testing.

Sand should never be reused without screening out all of the fines, drying it, and allowing the temperature to stabilize. Sand should be stored in a dry location where the temperature is approximately the same as the air temperature at the location where the tests are to be made. The sand should be kept in containers that will protect it from contamination with soil, dust, moisture, or any foreign matter.

Determination of Unit Weight of Sand

The unit weight of the standard sand is determined by placing the sand into a container of known volume and determining the mass of the sand to fill the container. The mass of sand to fill the container of known volume can be determined by filling the container from the sand cone apparatus and striking off the sand to be even with the top of the container or by using the sand cone and template method, which requires no striking-off, as explained in U.S. Bureau of Reclamation (USBR) procedure 1435-89 found on page 123 of the *Earth Manual* [1]. The sand should be placed in the container through the same apparatus as will be used in the field to place the sand in the hole. It has been determined through previous study that if the sand falls further, it will have a higher unit weight. The size and shape of the orifice on the sand cone can also affect the unit weight of the sand after free falling. Some orifices have a fine stream and others have a coarse stream; this difference in sand flow stream can make a substantial difference in the unit weight of the sand in the hole. Failing to fully open the valve also changes the sand stream size and can have a similar effect on the unit weight of the sand. The container of known volume that is used to determine the unit weight of the sand should have a shape such that the sand will fall about the same distance into it as it will in the excavated hole in the field. During the filling of the known volume container and after the container is full, great care must be exercised to avoid vibrating the sand in any way, such as while striking off the sand to the top of the container, because this will increase the unit weight to a higher value than the value that will be measured in the field. To minimize this potential source of error once the strike-off is completed, the container can be vibrated to densify the sand below the rim so spilling can be avoided.

One potential source of error is an increase in the moisture content of the sand because of changes in atmospheric humidity. Many users of the sand cone method simply do not determine the unit weight of the sand often enough. Most sand, particularly finer sand, is highly susceptible to bulking and volume changes with small changes in water content. The water content can change significantly with a change in atmospheric humidity as a water film is formed over the surface of sand particles. In some humid climates, the unit weight of the sand may need to be determined as often as several times each day, whereas in dry climates once each week may be enough. The needed frequency can be determined by comparing the results with previous tests. If the results are the same each time, the frequency of testing can likely be reduced.

An associated procedure necessary for this test is the determination of the volume of the apparatus cone and the plate above a level surface. This determination should be made regularly (usually at the same time as the sand calibration). Care should be taken to align the cone in the same location on the plate each time because slight warping or manufactured irregularities cause a different volume of sand to be used with differing alignments. See Fig. 6B in Appendix B for the sample form, "Bulk Sand Density Determination and Calibration of the Cone and Base Plate."

Preparing the Soil Surface at the Test Location

Calibration of the volume of the cone and base plate is made with the base plate on a smooth, level surface. To be accurate, the field condition should have a smooth, level surface before starting the hole excavation. It is sometimes difficult to obtain such a surface on the earth fill, particularly if the soil contains some gravel-size particles that stick up or cause holes in the surface. When a level surface cannot be obtained, the test procedure allows for an initial measurement of the test cone volume and base plate on the prepared non-smooth surface using sand from a second container. If this method is used, all of the sand should be cleaned up before the hole is excavated. The cleanup of the sand is best achieved when a thin membrane is placed on the surface before the initial sand volume measurement is made.

Excavating the Hole and Retaining the Soil

To minimize the potential errors, the hole should be as large as is practical. After the test area has been prepared with a smooth surface, the base plate is set and not moved until the hole is excavated and the sand is placed in the hole. Some base plates have provisions for driving a pin through the plate in each corner to secure it to the soil surface and reduce the potential for movement during the testing. If the base plate does not have provisions for securing it to the soil surface, marks on the soil surface or a large spike can be driven next to each side to secure the base plate.

In excavating the hole, care should be exercised to ensure that the hole is conical shaped with smooth sides and no overhangs. If a gravel-size particle is removed from the side, the hole must be reshaped above it to eliminate any overhang that would cause voids as the sand flows into the hole. Overhangs at the soil contact surface with the plate must also be avoided. Care should be exercised not to deform the material surrounding the sides of the hole during excavation. This can cause inconsistencies in the volume measurement with the test sand. Working on boards or taking other measures to avoid excessive pressures on the sides of the excavation may be required.

Care should be exercised to prevent loss of any soil excavated from the hole or loss of any moisture from the soil during the testing process. All of the excavated soil should be carefully and quickly placed in a container with a lid or sealed in a plastic bag to prevent moisture loss until the wet density is determined and a water content specimen is obtained. The container of material from the hole should not be stored where direct sunlight on the container will cause condensation and drying of the soil. Minor changes in the temperature of the soil after it is excavated will drive water out and onto the container sides, making the water content measurement inaccurate.

Placement of Sand in the Hole

The main precautions and potential sources of error for the placement of the sand are as follows: (1) place the cone in the plate in a vertical position, (2) carefully align the cone in the plate at the marked location used previously for determining the volume of the cone and plate, (3) quickly open and close the valve to a fully open and fully closed position at the beginning and end of the sand placement to minimize vibration and provide consistent flow of the sand, and (4) stop all equipment or other sources of vibration within at least 100 ft of the test location while the valve to the sand cone is open.

Obtaining Water-Content Specimens

The water-content test can be made using any one of five procedures outlined in Chapter 5 of this manual. The specifications should indicate which test is to be used to avoid confusion or dispute. To provide a representative moisture-content sample, the sample should be divided and processed enough to break up clods and other nonhomogeneous soil conditions. The water content can be determined by using a representative portion of the in-place test material with a minimum sample size as required by the ASTM test methods. On many projects it is standard practice to run a field moisture-content test with a confirmation oven-dry moisture content in accordance with ASTM D2216. If this process is used, then the sample should be combined and then divided into samples of appropriate size for the field and oven-dry moisture content.

If the Standard Practice D4718 for Correction of Unit Weight and Water Content of Soils Containing Oversize Particles is used, the water-content specimen should contain a representative portion of the oversize material. The dried specimen should be saved and screened to determine the percentage of oversize material as a percentage of the total sample.

Equipment Care

The equipment used in the sand cone test is generally not a large source of error. The most common errors caused by equipment are balance scales out of adjustment or ovens with the wrong temperature. The sand cone apparatus should be handled carefully to prevent damage, such as distortion and poor fit with the base plate. Straight edges used to strike off calibration containers must be kept straight.

Records of Testing

Sample forms for recording test data and the final test record can be found in Figs. 7B and 8B of Appendix B.

Quality-Control Checks

Some quality-control checks can be made to determine if the final test results using the sand cone method are reasonable. See Chapter 6, Section E for details on these checks.

2. Rubber Balloon Method (D2167)

The purpose of the rubber balloon test is to determine the volume and mass of soil material from a hole excavated into the compacted fill. After the volume and mass of the hole for the rubber balloon method have been determined, the values are converted to a wet density/unit weight that is used to compute the dry density from the measurement of in-place moisture content. This test is made by excavating a hole in the compacted fill and carefully saving all of the material that is removed in a covered container to prevent moisture loss. The volume of the hole is measured using an apparatus that uses fluid (typically water) under pressure to inflate a rubber balloon in the hole. The volume of the hole is determined by measuring the volume of liquid required to fill the balloon. The apparatus for this test is shown in Fig. 2 and 6C in Appendix C.

As with the sand cone test, the surface where the hole will be excavated is made smooth and a template or base plate is placed over the spot where the hole is to be made before beginning the excavation. The base plate is machined to fit the base of the balloon apparatus. The balloon apparatus is equipped so that an externally controlled pressure and a small vacuum can be applied to a fluid chamber, which is connected to a rubber balloon. The rubber balloon apparatus is designed so that the balloon can be inflated with fluid in a downward direction into the base plate and hole. The amount of fluid to fill the hole is measured by reading a volume indicator on the fluid reservoir before and after inflating the balloon. An initial volume of the space between the apparatus and the prepared surface (including the space created by the base plate) is made before the hole is excavated

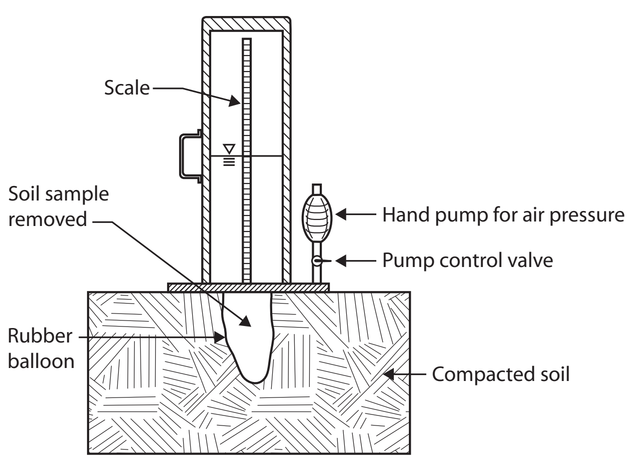


Fig. 2—Rubber balloon method (ASTM D2167).

and is subtracted from the final reading when the volume of the hole is measured. This allows for some small irregularities in the prepared surface before the hole excavation is started, and these will need to be accounted for in the initial reading. The equipment must be calibrated using a known volume container before the test is made. Pressure adjustments are used to achieve the proper volume measurements. The equipment must allow for weights to be added to the apparatus to keep it from being pushed up by the pressure in the balloon. The operator must ensure sufficient weights have been added to prevent any movement of the apparatus. Small movements can cause large errors in the results.

As with sand cone test, once the material has been excavated from the hole, the container of excavated material is transported to a field laboratory where the mass and water content of the material removed from the hole are determined. The wet and dry unit weight and the water content of the in-place soil can be calculated from the determinations of mass of the material taken from the hole, the volume of the hole, and the water content of the material from the hole. The water content of the soil is determined by using one of the standard test methods explained in Chapter 5 of this manual.

APPLICATIONS

This test is typically used on cohesive soils, including all fine-grained soils (CL, ML, CH, MH) and silty or clayey sands or gravelly sands (SM, SC) or silty or clayey sandy gravels (GM, GC) with less than 5 % oversize fraction. It is generally not practical to use this method on soils having particles larger than 3/4 in. because the larger particles create voids and irregularities that the balloon cannot accurately measure. This test is generally not used on noncohesive soils or soft, wet, or saturated soils because it is difficult to maintain a stable hole while performing the test. See the test procedure in ASTM D2167 for additional explanation and precautions.

ADVANTAGES

Because a hole is excavated in the compacted soil, the tester can visually observe the nature of the soil being tested and make judgments relating to whether it is representative of the fill. A compaction test can be performed on the soil removed by excavating the hole; thereby making sure the maximum density and optimum water content used for comparison are the correct ones. When the rubber balloon test is used as part of the rapid method (D5080), a three-point compaction test is performed on material excavated near the test site and representative of the material removed from the test hole. Testing for the water content is not necessary, making this a very good test for accurate results within approximately 2 h. With the rubber balloon test, the oversize particles can be removed with a screen to determine the exact percentage of oversize or to determine the properties of the oversize, such as their mass and volume. The accuracy of the test is very much operator dependent, and careful attention should be given to all of the measurements and calculations involved in the test procedures.

DISADVANTAGES

The test is rather labor intensive and takes considerable time to complete one test at one location. The construction process must be disrupted or at least curtailed for a period of time near the location of the test to provide safe access for those performing the test and to reduce the potential for vibration, which may cause inaccuracies in the test results. The results of the test may not be available for up to 1 h and up to 1 day if the oven-dry method is used to determine the water content. The graduated cylinder may be made of glass and subject to breakage from field use. The whole operation is dependent on the proper functioning of a rubber bulb assembly to increase or decrease air pressure. As the bulb deteriorates, leakage may be a problem and the test results become inaccurate.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

The rubber balloon method has precautions and potential sources of error that should be understood to minimize the influence of the errors on the test results. The most significant precautions and potential sources of error for this method are as follows.

Calibration of the Apparatus

The calibration of the apparatus is explained in the Annex of the ASTM test method for D2167. The procedure should be followed closely to minimize the potential sources of error. For the calibration of the apparatus, the container of known volume should be approximately the same size as the hole that will be excavated in the field test to minimize inherent dimensional influence and edge effects. If the compaction molds specified in the compaction standards (D698 or D1557) are used, their measured volume must be determined and used in the calibration comparison rather than using the nominal volume of the mold (e.g., 1/30 ft^3). If water is used to determine the volume of the compaction mold, it may require a gasket at the base plate connection, which should be left in place during the volume measurement using the rubber balloon apparatus. Calibration is recommended periodically (i.e., annually as a minimum) even if the equipment has not been repaired or subject to any observable damage.

Expelling Air from the Fluid and the Hole

Ways of expelling air from the known volume containers during calibration are explained in the calibration procedure in the Annex of the test method. Expelling air from the excavated holes in the field is usually not a problem, but placing a string down the side of the hole as explained in the calibration procedure may be needed when the material in the hole is smooth, such as for a compacted clay soil.

Determining Weights to be Applied for Various Pressures

Very slight movements of the apparatus caused by an increase in the fluid pressure can cause significant errors in the test results. Sufficient weight must be applied to the apparatus so that the fluid pressure does not lift the apparatus. Fluid pressure of 5 psi can cause up to 100 lb of uplift for large holes. If the weight to hold the apparatus down is supplied by the operator standing on the apparatus, another person may have to take the readings so a constant load can be applied to the apparatus. To provide consistent results, the same amount of weight should be used on the apparatus during testing as during calibration.

The rubber balloon apparatus is not recommended for testing where the soils contain sharp, angular particles because they may puncture the balloon. Using a double balloon may help solve this problem, but if a double balloon thickness is used, the apparatus must be calibrated using a double balloon. The operator should always observe closely for leaks along the surface of the balloon, at the connection at the top of the balloon, at the pump, and at the valves.

Excavating the Hole and Retaining the Soil

As indicated above during the previous description of the sand cone method, a larger hole is desirable to reduce the influence of errors. The base plate is set and an initial reading is taken using the rubber balloon apparatus. The base plate is not moved until the hole is excavated and the final reading is made using the apparatus to inflate the balloon in the hole, which was excavated within the opening in the plate. Larger hole volumes typically require a larger rubber balloon apparatus. Smaller apparatus (using small balloons) will not work in large holes because the balloon will not stretch far enough to provide accurate readings. Using a hole smaller than those specified in the test method affects the precision and invalidates the test.

In excavating the hole, care should be exercised to ensure that the hole is conical shaped with generally smooth sidewalls, no overhangs, or sharp intrusions or extrusions from the surface of the hole. If a gravel-size particle is removed from the side, the hole must be reshaped above it to eliminate any overhangs and narrow cavities in the near vicinity of where the particle was removed. Overhangs at the contact with the plate must also be avoided. Care should be exercised to not deform the material surrounding the sides of the hole during excavation of the hole.

Care should be exercised to prevent loss of the soil or moisture, or both from the material excavated from the hole before obtaining samples for the moisture/density computations. The material excavated from the hole should be placed in a container with a lid or sealed in a plastic bag to prevent any moisture loss until the mass is determined and a water-content specimen is obtained. The container of material from the hole should not be stored where direct sunlight on the container will cause heating of the soil and drive moisture out of the soil and onto the container sides, making the water-content measurement inaccurate. The material from the hole should be protected until all necessary tests are made on it. When appropriate, ASTM D4718 may have to be used to correct for oversize particles.

Obtaining Water-Content Specimens

The water-content test can be made using any one of five procedures outlined in Chapter 5 of this manual. The specifications should indicate which test is to be used to avoid confusion and provide consistency as part of a structured testing program. The water content is typically determined using a thoroughly mixed portion of the in-place test material. If ASTM Standard Practice D4718 for Correction of Unit Weight and Water Content of Soils Containing Oversize Particles is used, then the water-content specimen should contain a representative portion of the oversize material. For samples requiring oversize correction, the dried moisture content specimen should be saved and mixed back with the total sample for screening and determining the percentage of oversize in the total sample.

Equipment Care

The equipment manufacturer's recommendations for care of the equipment should be followed to minimize damage that could cause erroneous test results. Care should be exercised in using and transporting the equipment so that it is not dropped or jarred such that any part of the apparatus is bent or broken. Damaged equipment can adversely impact the results and should never be used in testing.

Records of Testing

Sample forms for recording test data and final test records can be found in Figs. 9B and 10B of Appendix B.

Quality-Control Checks

Some quality-control checks can be made to determine if the final test results are reasonable. See Chapter 6, Section E for details on these checks.

3. Drive Cylinder Method (D2937)

This test is made by driving a thin-walled metal cylinder or tube of known volume into the soil using a drop hammer. The cylinder is sharpened at the bottom and has a blunt end that slides into a drive head on the top. The drop hammer slides on a rod attached to the drive head. The cylinder is driven completely into the soil so that the soil protrudes out the top of the cylinder a small distance. The cylinder full of soil is then excavated out of the soil with a shovel or other tool by carefully removing the soil from around and under the cylinder. During the excavation, a small amount of soil is left protruding out the bottom end of the cylinder. The soil that protrudes beyond the ends of the cylinder is then carefully trimmed off flush with the ends using a knife or straight edge. The mass of the full cylinder is then determined, the mass of the empty cylinder is subtracted from it, and the unit weight is calculated using the volume of the cylinder. The water content is determined by testing a portion or all of the material removed from the drive cylinder and the dry unit weight is calculated. The apparatus can be found in Fig. 3 and 7C in Appendix C.

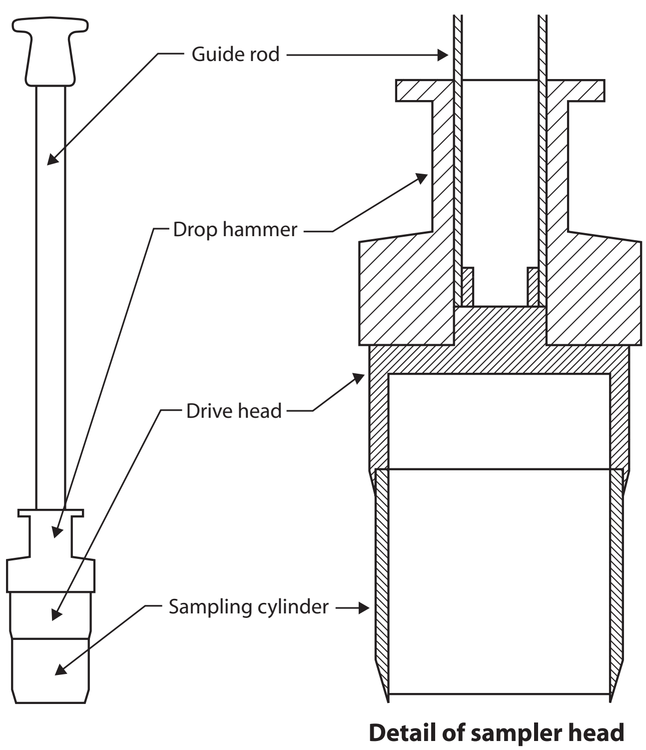


Fig. 3—Drive-cylinder method (ASTM D2937).

APPLICATIONS

This test is used on cohesive soils, including all fine-grained soils (CL, ML, CH, MH) and silty or clayey sands (SM, SC). Generally, it is not applicable to soils that have appreciable quantities of coarse sand and gravel-size particles; however, it may provide satisfactory results when the gravel is limited to a few isolated particles in a matrix of mostly silts and clays. This method is not recommended for use in organic soils, noncohesive or friable soils such as sand, soft or saturated soils that are easily deformed, or soils that may not be retained in the drive cylinder. See the test procedure for ASTM D2937 for explanations and precautions. Soils that are very hard, brittle, and dry are very difficult to test using this method and the results are subject to more error because of disturbance of the soil or possible distortion of the cylinder sidewall.

ADVANTAGES

Because this method includes obtaining a partially disturbed sample of the in-place soil, it is useful for providing a visual indication of the placement and compaction of the layers and other features of the fill. The tester can visually observe the nature of the soil being tested and make a technical evaluation on the overall quality of the fill. The oversize particles can be removed with a screen to determine the exact percentage of oversize or determine the properties of the oversize such as their mass and volume. The test generally takes less time for testing than the sand cone or rubber balloon tests and can have fewer sources of error for an operator with basic understanding and technical skill. Unlike the sand cone test, the earthwork equipment can continue to work nearby the test location because vibration typically does not affect the test results.

DISADVANTAGES

The test is somewhat labor intensive and can take considerable time to complete because of the excavation required to get an intact sample out of the compacted soil. The construction process may be disrupted for the period it takes to drive the cylinder into the soil and excavate around it. The results of the test may not available for up to 1 day if the oven-dry method is used to determine the water content. However, other methods can be used to determine the water content, or the test can be used with the rapid method procedure (D5080) where water content testing is not needed. Gravelly and stony soils cannot be tested using this method because of the influence of the gravel/stone on the volume of the sample and on the sidewall of the drive cylinder. Depending on the soil moisture content, soils with low plasticity that are loose may not stay in the cylinder.

There are several sources of error associated with this method, and care must be exercised in making the test. Driving the cylinder into the soil may change the unit weight of the soil because of disturbance. Please refer to the following section on precautions or sources of error.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

There are several precautions that should be followed to minimize the sources of error, which can substantially affect the test results when using the drive cylinder method. The most significant ones are as follows.

Cylinder Drives Crooked

If adequate precaution is not taken, the drive cylinder will become crooked during the driving process if one side penetrates easier than the other. When this occurs, the soil is cut in an elliptical shape so that there are voids on one side and sidewall disturbance on the other. When a drive cylinder is driven crooked, the test should be stopped and moved to a new location because the results will be in error.

Equipment Size
Some commercially available drive cylinder equipment is too small to meet the sample size requirements for accurate testing. A minimum diameter of 4 in. is recommended.

Coarse Fragments or Gravel Size Particles
During driving of the cylinder rock, fragments can be encountered by the cylinder cutting edge. If the rock is pushed ahead of the cylinder, the material is disturbed and a groove is made in the soil that is retained in the cylinder. Void caused by rock or gravel fragments in the soil can be detected when the ends are trimmed or when the soil is removed from the cylinder, or both. The results of tests with voids due to gravel are questionable and should not be used.

Overdriving the Cylinder into the Soil
Overdriving may result in deformation or compression of the sample such that the unit weight measurement is incorrect.

Inaccurate Trimming of the Soil at the Ends of the Cylinder
A straight edge and a knife are typically used to trim the soil so it is even with the ends of the cylinder. A sharp, straight knife or straight edge should be used to trim the soil from the ends of the cylinder with a sawing motion and the ends should be checked to determine the degree of progress toward a level surface. If sufficient care is not exercised, the soil may break off below the surface of the cylinder, causing a void in the compacted soil. When coarse sand or gravel-size particles are encountered during the trimming process, some technical judgment is required to determine if the particles should be removed and replaced with the excess soil from the ends of the cylinder. Soil that is placed in small voids should be compacted with a straight edge or finger pressure to limit the amount of disturbance to the sample.

Errors When Obtaining Water Content Specimens
The water content test that is used for the drive cylinder test can be any one of five procedures outlined in Chapter 5 of this manual. The specifications or recorded test results should indicate which test is to be used to avoid confusion or dispute. The water content can be determined by using a portion of the in-place test material or all of it. If a portion of the drive cylinder sample is used, the material should be thoroughly mixed before the specimen is obtained. If Standard Practice D4718 for Correction of Unit Weight and Water Content of Soils Containing Oversize Particles is used, the water content specimen should contain a representative portion of the oversize material. The dried specimen should be saved and mixed back in with the total sample for screening the oversize material out and determining its percentage of the total sample.

Equipment Calibration and Care
The most common equipment problem is that the drive cylinder becomes distorted or the cutting edge becomes nicked and dull from driving it into hard soil or rock fragments. To minimize potential errors because of a damaged cylinder, the cutting edge can be sharpened; however, a careful determination of the volume is necessary after the sharpening procedure is completed. The volume and mass of the cylinder should be checked often. Distortion of the cylinder edge or sidewall can substantially change the volume. A change in volume will cause a corresponding change in the sample mass when the cylinder is used in wet or dry conditions. Care should be exercised in following the steps outlined in the ASTM test procedure to determine the volume because it is critical to accurate test results. As required for all density tests, the balance scales and ovens should be calibrated at regular intervals. Straight edges used to strike off calibration containers must be kept straight.

Records of Testing
Sample forms for recording test data and final test records can be found in Figs. 11B and 12B of Appendix B.

Quality-Control Checks
To minimize the number and degree of the potential sources of error, quality-control checks should be made on a regular interval to determine if the final test results are reasonable and consistent. See Chapter 6, Section E for a summary and details on the procedures of the quality-control checks.

4. Nuclear Density and Moisture-Content Methods (D6938)
The nuclear test method for determining field density and moisture content is very popular for density and unit weight measurements because of the ease and relative speed of the method as compared with other field density methods. This section includes a discussion of methods for determining density and water content using the nuclear gauge because both tests use one instrument or apparatus that requires explanation regarding the equipment, its calibration, and some precautions regarding its use. The apparatus and testing modes are shown in Figs. 4a–4d. Photos 8C and 9C are in Appendix C.

The nuclear gauge can be used to determine the in-place density and water content of most soil/aggregate mixtures commonly used in earth fills and structural backfill (except rock fills or coarse crushed rock). The wet density or mass density is determined indirectly by gamma rays in backscatter or by direct transmission through the materials immediately under gauge. Neutrons in backscatter through the material under test indirectly determine the water content. The dry density is then computed by the gauge from these two basic measurements.

The backscatter mode of testing results in a bias toward near-surface soils, with the top 4 in. of material having a large influence on the measurement. The direct transmission mode, available with most gauges, measures an average density to the depth at which the gamma source rod is positioned and is preferred by most users.

The water-content measurement by the nuclear gauge is measured by the backscatter mode with all gauges, which causes it to be biased toward conditions near the surface. The measurement depth is a function of the water content and decreases with increased water content. The depth of measurement averages 6 in. (15 cm) at a water content of 15 pcf (0.240 g/cm^3). If the soil profile moisture is uniform with depth, this will not present a problem. If the soil moisture profile is variable and the test depth greater than 6 in. (15 cm), then the gauge measurement may not be representative of the soil being measured in the direct transmission mode for soil

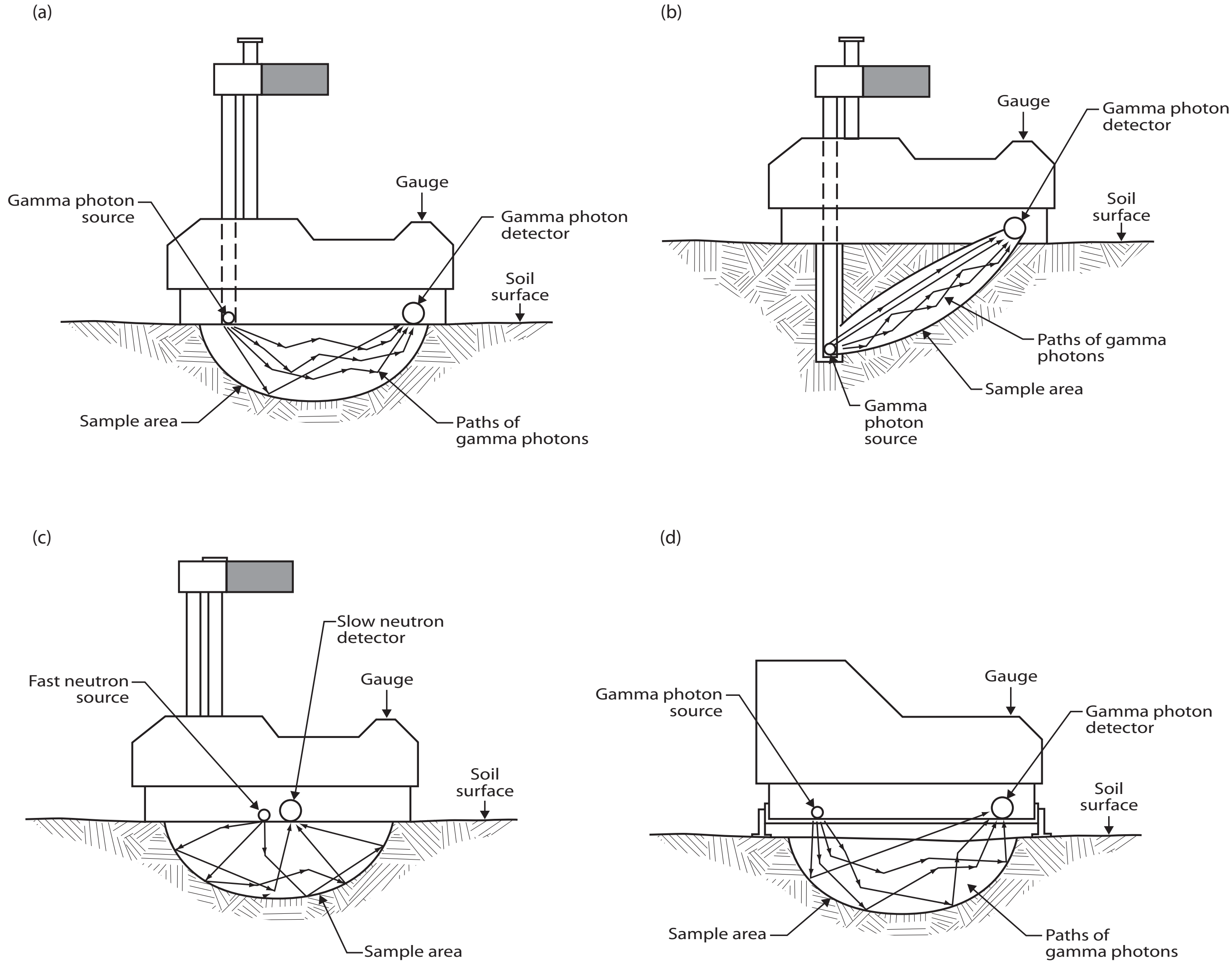


Fig. 4—(a) Backscatter density measurement; (b) Direct transmission density measurement; (c) Backscatter moisture measurement; and (d) Air-gap density measurement.

density. If the latter situation is the case, then an appropriate sample needs to be taken from the soil under the gauge and an alternative water content method should be utilized.

The density measurement and the water-content measurement are covered by ASTM D6938, Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth).

ADVANTAGES

The advantages of the nuclear gauge for soil compaction control testing are as follows. The tests can be performed in a short time period, making it possible to perform more tests for more complete coverage of an earth fill during construction. There is typically less disruption to the construction operations because of the speed and nature of the testing and the ability to provide more rapid information to help ensure timely corrective action of defective work. The gauge has the ability to take multiple tests with consistency and good repeatability of test results. This method can also provide nondestructive testing of compacted soil covers for synthetic membrane liners by using the backscatter mode. The nuclear method, if properly used, can provide a rapid testing for density at multiple depths using the direct transmission mode. This method can also provide nondestructive testing of compacted soil covers for synthetic membrane liners by using the backscatter mode. The nuclear method also provides more convincing results because of a perceived high technology and fewer perceived operator errors.

DISADVANTAGES

Some of the disadvantages of the nuclear gauge for soil compaction control testing are as follows. Depending on the applicable nuclear regulatory guidelines, there can be cumbersome and costly licensing requirements, fees, inspections, and work associated with the owning, operating, special storage, and transporting requirements of the gauges. Operator training and certification are required to use the gauge, and operators also must be part of a radiation exposure detection program when operating the nuclear gauge.

It is important to note that the nuclear gauge can be subject to several potential errors due to field conditions and soil chemistry. The backscatter mode for density measurement is biased toward the surface layers of the soil, is very sensitive to the surface condition of the test site, and can provide inconsistent density measurements. The water-content measurements are all made by the backscatter mode, and this measurement method is biased to the moisture content present in the near-surface layers of soil. The moisture-content measurement tends to penetrate deeper in drier soil, but rarely exceeds approximately 6 in. (15 cm) in depth. The water content measurements are also affected by the soil chemistry. **A correction must be applied to the gauge reading for the effects of bound hydrogen in the soil.** The proper adjustment of the nuclear gauge for errors caused by soil chemistry requires some awareness and skill by the field operators.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

The nuclear density and moisture-content method(s) have precautions and potential sources of error that should be understood to minimize the influence on the test results. The most significant precautions and potential sources of error for this method are as follows.

Test Location Selection, Preparation, and Site-Specific Gauge Calibration

To ensure representative test results, a good visual or physical inspection of the soil surface should be made for anomalies because visual observation of the soil tested is typically not inherent in the test method. Nuclear gauges cannot be used close to trench walls, vertical banks or walls, or certain materials such as steel pipes without some special corrections or test procedures to ensure accurate test results. Special calibration may be needed for certain "nonstandard" soils containing compounds that affect nuclear gauge operation.

For nuclear gauges that are capable of direct transmission measurements of soil density using a source probe, it is recommended to have a source probe length that allows testing up to a 12-in. depth. The gauge should be capable of accurately measuring wet density and water content with a simple correction procedure for the water-content measurements.

The nuclear gauge is calibrated for a standard material by the manufacturer before being delivered to the purchaser. This calibration is accomplished using a set of standard blocks and historical curve data to cover the range of densities normally found in mineral soils. Checking instrument calibration and any adjustments of calibration should be done in accordance with the manufacturer's recommendations and in accordance with the procedures and intervals recommended in ASTM D6938. Once the calibration is verified, the daily standardization count required at the start of each day's use is a reasonable indicator that the instrument is in satisfactory condition and may be used to give reliable results. It is important to note that this daily gauge calibration does not preclude the need for operator skill and knowledge of the soil materials and the influence of their properties on the use of the gauge. Field adjustments using other ASTM density and moisture-content methods to provide verification may be required to develop moisture and at times density offsets that take into consideration the unique properties of the soil. Regardless of the verification test method used, prior knowledge of the soil materials is essential for the proper use of the nuclear density gauge.

Water-Content Correction

A water-content correction is generally needed in the field for ordinary soil compaction control work when using the nuclear density gauge. The nuclear gauge uses neutrons to detect hydrogen ions in the soil material. Most of these ions in soil are in the form of water (H_2O); however, it is common for hydrogen other than water to be present in the soil. Possible sources of hydrogen in the soil include hydrocarbons, hydrous minerals (e.g., gypsum), clay minerals, and organic matter in the soil. This type of soil chemistry often results in a correction factor that may run as low as 1 % to as high as 10 percentage points in highly micaceous soils.

Hydrogen in forms other than water typically results in water-content measurements in excess of the true value. The correction factor is nearly always negative, thereby reducing the gauge water content reading and increasing the dry density computed by the gauge. Some chemical elements such as boron, chlorine, and minute quantities of cadmium will cause measurements lower than the true value that will require a positive correction factor. Before the nuclear gauge is used for soil/aggregate density and water-content control, it must be calibrated to determine the site/location-specific water content correction for each material type that is tested. This is done by taking nuclear gauge readings in the appropriate materials, sampling these materials from directly under the gauge, and determining the water content by acceptable alternate methods. The most common and consistent method is the oven-dry method (ASTM D2216), but this method typically takes a 12-h period to obtain results. Other methods that provide more rapid moisture content measurements are the ASTM D4643 microwave method, the ASTM D4959 direct heating method, and the ASTM D4944 gas pressure tester method. The latter method should be the last choice because of inaccuracies caused by the small sample size, particularly with soils of higher plasticity where the sample size is further reduced to accommodate the higher water-content range.

The water-content correction method for the nuclear gauge is reliable if the difference in the measured water content between the nuclear gauge and other water-content methods is fairly consistent (i.e., less than a 1–2 % difference between the gauge and the check methods) for the soil type. If the difference is quite variable within the same soil type, then the nuclear water-content measurement should be abandoned and one of the alternate soil water content methods substituted. The gauge can still be relied on for the wet density, and with the entry of an alternate water content, the gauge will compute an accurate dry density for the soil tested.

A practical application of the procedure for water-content correction is as follows:

1. Take water-content readings at a minimum of three different locations in the same soil or soil-rock material. These readings can be made in the borrow area or in fill materials as they are brought to the fill area and processed for compaction. The gauge will be in the 1-min setting for most field work. One 4-min reading may be taken if a higher degree of precision is determined to be significant.
2. Obtain a sample of representative material from the top 6 in. directly under the gauge at each location tested and determine the water content by one of the previously discussed ASTM Standard Test Methods. The minimum size for each sample should be as specified in the appropriate ASTM Standard Test Method.
3. Using the average of the three water-content values, determine the bias or correction value for that type of soil material. Be alert for single values that are significantly different from the other values. These should be discarded and a new test performed to replace the suspect value.

4. Repeat the procedure for each soil type to be used in the fill operations and for any new materials added as the work progresses. The fill is probably the preferred location for testing and sampling soils for water correction because the borrow source often has a stratification of materials and natural moisture variability. Sampling of blended materials is acceptable as long as the blending process can be reasonably controlled and is also reflected in the preparation of the standard reference density being used for compaction control.
5. For larger projects with a wide variety of fill materials and a range of moisture-content correction, it is good engineering practice to develop a database of moisture-content corrections on the basis of material classifications and the most recent moisture-content corrections. This method will allow the nuclear gauge operator and engineer to track trends in the moisture offset used and make corrections depending on the location in the borrow area.

When determining the water-content correction, it may be useful to plot the nuclear gauge water content versus the oven-dry (or other alternate method) water content to get a visual picture of the relationship. This procedure will allow detection of a change that is increasing or decreasing in magnitude. This procedure may be useful in the case in which the soil contains a significant portion of micaceous material and the water-content correction is not uniform.

Field Density and Water Content Testing

Select the test site to be representative of the location, depth, and fill lift that has been compacted and is ready for testing. As discussed in other parts of this guide, this may be a bias selection of a test site or it may be intended as a random selection. Refer to ASTM D3665 Standard Practice for Random Sampling of Construction Materials. It may be necessary to excavate to the desired elevation in the fill by using a bulldozer, grader, or scraper blade as available. After the test location has been selected, prepare the test surface in accordance with the procedures in ASTM D6938. Select the desired depth for the gauge source probe on the basis of the thickness and location with respect to depth of the lift (layer of earth fill placed at a specific elevation on the fill). Set this depth on the gauge and verify that the gauge has recognized the testing depth. Take the measurement and record the wet density, water content, and the resulting computed dry density. A sample form for recording test data and the final test record can be found in Figs. 13B and 14B of Appendix B. Photos are in Appendix C.

When testing gravelly or stony soils, there may be considerable variations in the distribution of the coarse fraction of material. For material with nonhomogeneous soil and gravel mixtures, take at least three tests by rotating the gauge around the source probe to a new location. After the test is complete, it is also advisable to dig out part of the test area to inspect the in-place materials and visually verify the rock distribution and to sample the test area for measurement of the oversized material fraction. Digging out the test area is advisable any time there is a question about the uniformity or nature of the material being tested.

Calibration

Calibration is very important for nuclear gauges. The calibration is explained in the Annex of ASTM D6938. The recommended calibration interval should be followed to ensure that the gauge is operating with the desired accuracy at all times. Daily standardization and checking for each instrument do not necessarily relate to proper calibration.

Recognizing Erroneous Readings

One common error is associated with gauges that have a manual set for the testing depth of the probe in direct transmission. The manual set and the probe depth must be in agreement for a good reading. Because most new gauges have an automatic feature for this, the problem has been eliminated. As in any test, an understanding of the materials being tested is required to be able to recognize unrealistic testing values and then find the reason for those values.

Correction for Oversize Particles

The nuclear gauge measures the mass density of the soil or a soil-rock mixture. A correction will have to be made if the quantity of oversize is greater than 5 % more than what was used in the standard reference density test. See Section B of Chapter 6 of this manual for the procedure for making oversize rock corrections. Another consideration is to rotate the gauge and take multiple readings to determine if the oversize is consistent or just one large rock.

Selection of the Standard Reference Density

Because the nuclear gauge is a nondestructive test, sampling the material under the gauge by digging it up is a good way to look at the nature of all of the materials involved in the test. This may be enough to make a visual and textural identification of the material and match it with a density standard. Other procedures for selecting the appropriate density standard are covered in Section E of Chapter 6 of this manual.

Sleeve Method (D4564)

This test method is relatively new and was developed by employees of the U.S. Bureau of Reclamation for use in testing uniform, granular soils used for pipeline bedding. The density measurement is made by preparing a smooth, level working area and placing a base plate on the area. A thin metal sleeve with a beveled cutting edge is rotated into the soil with the aid of a driver plate that fits on the top of the metal sleeve. The material inside of the sleeve is removed and placed in a container with a lid to protect against moisture loss. Material can be removed in several stages during insertion of the sleeve into the soil until the sleeve is level with the base plate. The bottom of the hole is made level and measured by placing a measurement plate on the surface of the bottom and measuring the depth to the surface of the plate at four locations. The volume of the excavation is calculated using the inside diameter of the sleeve and the average depth of the excavation. The mass and the water content of the material removed from the sleeve are measured and the in-place dry unit weight and water content are calculated and reported.

Because of the recent development of the test method, there is not much information available on which to base further guidance and potential sources of error. The procedure provided in the test method should be followed and care given to performing each step of the procedure and the calibration. The nature of the test is such that a high degree of accuracy should not be expected and technical judgment will be required to determine the applicability and accuracy of the test method for the acceptance or rejection of construction work.

APPLICATIONS

The application of the sleeve method is typically limited to determining density on cohesionless, granular soils for which the sand cone, rubber balloon, drive cylinder, nuclear gauge, and test pit methods are not practical. It has mainly been used for testing the in-place unit weight for materials used as a bedding and backfill for pipeline trenches. The sleeve method is typically applicable for soils that are cohesionless with a maximum of 5 % fines and a maximum particle size of 3/4 in.

Sand Replacement in a Test Pit Method (D4914)

This test method is based on the same principles and procedures as the sand cone test (ASTM D1556). The sand replacement test is performed by positioning a metal ring or square frame on a relatively flat surface. The ring can be round or rectangular and is usually 2–4 ft or more in diameter or side dimension and is sufficiently rigid so as not to bend or deform when performing the test. Calibrated sand is placed in the ring on the surface of the test location using a pouring device that is calibrated before excavating the hole. The sand is leveled to the top of the ring surface and the amount of sand to fill the ring is measured on a mass per unit basis. The filling of the space between the top of the soil and ring may require several containers of sand. The test hole is excavated while saving the excavated material in containers that can be closed to protect against moisture loss. The volume of the hole is determined by calculating the amount of sand used to fill the excavated hole and by using sand with a known unit weight. A calibrated pouring device is used to help in achieving a uniform density of sand in the large hole. Fig. 5 shows test pit configurations for this test.

Two methods are indicated in the procedure (Method A and Method B). Method A is for determining the in-place mass unit weight of the fill for checking against some specified unit weight value. Method B is used when the in-place unit weight will be compared with a standard reference density test (laboratory compaction test). When large particles are removed in making the laboratory compaction test, a correction for oversize must be made to the in-place test or the compaction test so that the comparison is made for soils with the same particle sizes.

APPLICATIONS

This test is specifically designed for determining the in-place unit weight of soil material containing a considerable amount of gravel, cobble, or rock-size particles, or combinations thereof. These materials are often used in the outer shells of zoned earthfill or rockfill dams or in some road fills. Other

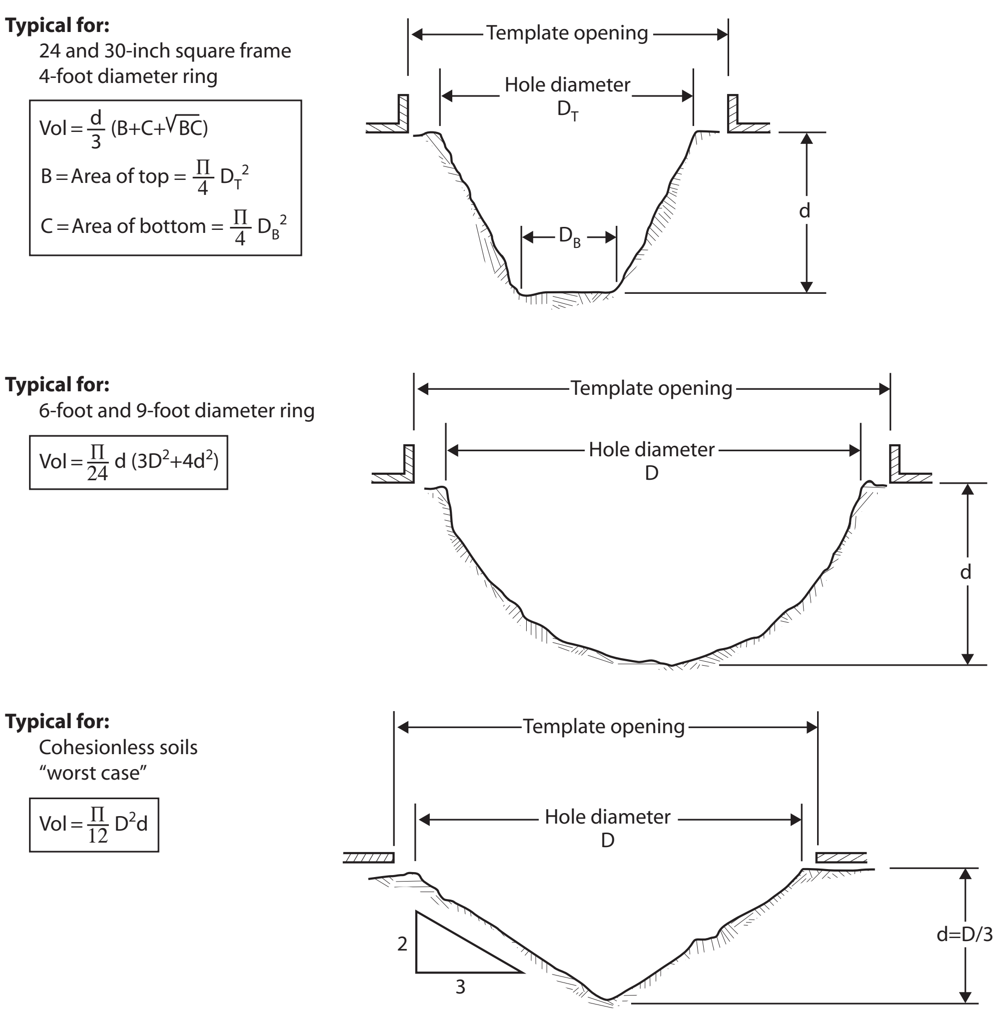


Fig. 5—Test pit configurations USBR 7720.

in-place density test methods cannot be used for large-particle-size soils because the required limitations in the specimen and hole size make the required test hole size impractical. This method uses the same principles as the sand cone test (ASTM D1556) but provides a large test hole volume to obtain a representative sample of the soil/gravel in fill constructed with large particle material. The test results can be used to compare the in-place unit weight against a specified unit weight or a percentage of a maximum unit weight as determined by a standardized laboratory compaction test, or both. When the standardized compaction tests are conducted on materials with larger particles, a correction factor is typically used. It is important to note that the same correction factor/method must be used for the in-place test results to allow comparison with the standardized compaction test results as explained in Chapter 6.

ADVANTAGES

Large test hole volumes are often required so that the test can be representative of materials containing large particles. The soil excavated from the hole can be observed, judgments can be made about whether it properly represents the fill, and the oversize particles can be screened out and measured to determine their mass and volume for developing correction factors.

DISADVANTAGES

This test often takes a considerable amount of time and work effort, usually taking two or more individuals to perform it. The samples and sand containers are numerous, bulky, and heavy, making it very difficult to perform many tests without a large technician work force. The sand and sample containers must be labeled and carefully recorded to prevent confusion and error. Special excavating and transporting equipment may be required, as well as small-scale construction equipment for handling, drying, and screening the material. The construction process is typically disrupted for a considerable length of time in the area of the test. If the test method is not properly implemented, there can be many sources of error associated with this method and the technician staff needs to use substantial care to achieve a representative test result.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

There are precautions and good technical practices that should be followed to minimize the potential sources of error that would affect the test results for this test method. The published ASTM test method provides a description of these precautions and a list of detailed procedures that should be carefully studied before running the test and followed while the test is being performed. A list of the primary areas of concern is as follows.

Selection and Storage of Sand

The sand must be uniform in size (narrowly graded), dry, free flowing, and devoid of fines. Rounded or surrounded particles are typically best because angular sand is more susceptible to bridging across irregularities. Sand that is too coarse will also be subject to bridging across irregularities. Finer sand may be more susceptible to changes in unit weight from changes in atmospheric humidity.

The sand should never be reused without screening out all of the fines, drying it, and allowing the temperature to stabilize. Sand should be stored in a dry location where the temperature is approximately the same as the air temperature at the location where the tests are to be made. The sand should be kept in containers that will protect it from contamination with soil, dust, moisture, or any foreign mater.

Determining Unit Weight of Sand

The unit weight of the standard sand is determined by placing the sand into a known volume container and determining the mass of the sand to fill the container. The mass of sand to fill the known volume container can be determined by filling the container from the sand-pouring apparatus and striking off the sand to be even with the top of the container. The sand should be placed in the container through the same apparatus as will be used in the field to place the sand in the hole. It has been found that if the sand falls further, it will have a higher unit weight. Some orifices have a fine stream and others have a coarse stream; this can make a substantial difference in the unit weight of the sand in the hole. The known volume container used to determine the unit weight of the sand should have a shape such that the sand will fall about the same distance as it will in the excavated hole in the field. During the filling of the known volume container and after the container is full, care must be exercised to avoid vibrating the sand in any way, such as while striking off the sand to the top of the container (if this method is used) because this will increase the unit weight to a higher value than in the field. Once the striking-off is completed, the container can be vibrated to densify the sand below the rim so that spilling can be avoided.

One potential source of error is not determining the unit weight of the sand often enough. Most sand, particularly the finer sand, is susceptible to bulking and volume changes with small changes in water content. The water content can change significantly with a change in atmospheric humidity as a water film is formed over the surface of sand particles. In some climates, the unit weight of the sand may need to be determined as often as several times each day, whereas in dry climates once each week may be enough. The required frequency for checking the unit weight of the density sand can be determined by comparing the results with previous tests. If the results are consistent and based on experience appear to be representative of the soil/gravel that is being tested, the frequency of testing can likely be reduced.

Preparation of the Soil Surface at the Test Location

The ring or frame should be placed on a relatively level surface and care should be taken to make sure it is in contact with the surface at all locations. Gaps or overhangs under the ring will cause errors in the measurement of the volume of the ring before excavation or the volume of the hole, or both. The test procedure requires an initial measurement of the ring on the prepared surface using sand from containers other than those to be used for measuring the volume of the excavated hole. After this step is completed, all of the sand must be cleaned up before the hole is excavated. A thin plastic membrane 1–3 mil thick is recommended in the procedure to help with the cleanup of the sand used for measuring the ring volume.

Excavating the Hole and Retaining the Soil

To measure the volume of soil/gravel removed, the ring is set and not moved until the hole is excavated and the sand is placed in the hole. The ring should be secured with pins or weights to keep it from moving during the placement of sand or excavation of the hole. During the excavation of the hole, care should be exercised to make sure the hole is conical shaped with smooth sides and no overhangs. If gravel, cobble, or rock-size particles are removed from the sidewalls, the hole must be reshaped above where the particles are removed to eliminate any overhang where the particles were removed. Overhangs at the contact with the ring must also be avoided. Care should be exercised not to deform the material surrounding the sides of the hole during excavation of the hole to minimize potential voids as the sand is poured.

As the soil is removed from the hole, care should be exercised to prevent loss of any soil material before weighing and determining the volume of the hole. The excavated soil should be placed in containers with lids to prevent any moisture loss until the mass is determined and a water-content specimen is obtained. Containers of material from the hole should not be stored in direct sunlight before the subsequent water-content tests are made. Allowing the soil to be heated will drive moisture out and onto the container sides, possibly making the water-content measurement inaccurate.

The material from the hole should be protected until all of the tests are made on it. The tests typically include a mass determination, water-content test, and percent retained on the appropriate sieve size to determine the amount of oversize particles. The number and grain size of the oversize particle are required for making corrections and for comparing with the standardized compaction test results. Other required test procedures may include determining the specific gravity of the oversize or its volume to satisfy other correction procedures.

Placing Sand in the Hole

All equipment or other sources of vibration around the test location should be stopped while the sand is poured into the ring. To minimize localized vibration, care should also be exercised to prevent vibrating the ring while striking off the surface of the sand in the ring.

Obtaining Water-Content Specimens

The procedure in the test method must be followed according to the method (Method A or Method B) being used. In some cases the water content of the oversize material and the finer fraction are needed separately. For Method A, the water content of a representative specimen of the entire soil sample is made. When the soil contains large particles, larger water-content specimens are required (see tables in Test Method D2216). Because of the size of the material, some water-content methods such as D4643, D4944, and D3017 may not be applicable. The specifications and the technician's records should indicate which test is to be used to avoid confusion or dispute. For these large samples, the water content will be determined by using a portion of the in-place test material. The material should be thoroughly mixed before the specimen is obtained.

7. Water Replacement in a Test Pit Method (D5030)

This test method is similar to the Sand Replacement in a Test Pit Method, except that water is used to determine the volume of the ring and the hole that is excavated. This ASTM method uses a watertight membrane to isolate the water so it is also similar to a large-scale rubber balloon method in its practical application.

APPLICATIONS

This method is mainly used with 4- to 6-ft-diameter rings or frames as a method for determining the volume and density of rockfill construction. The method is almost identical to the previous method discussed–Sand Replacement in a Test Pit Method (D4914). The main difference is that water is used instead of sand to determine the volume of the ring on the surface before the hole is excavated and the volume of the hole and ring after the hole is excavated. A watertight membrane (3–6 mil, plastic or rubber) is used in each volume measurement to contain the water.

The test procedures and applications for Test Method D5030 are the same as for D4914 except for the use of water instead of sand. The advantages and disadvantages are also very similar, except for the disadvantages associated with the use of water instead of sand as a volume determination method. It is the experience of many earthwork professionals that the use of water in Test Method D5030 may provide fewer disadvantages and difficulties than using sand in Test Method D4914. The difficulties associated with the use of water as a volume determination method are discussed in the sections below. Sample forms for recording test data are in Appendix B.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

The precautions for Method D5030 are very similar to those previously presented for Method D4914, except for those indicated for sand or the calibration and use of sand.

Errors Associated with Using Water to Measure Volumes

If the mass of water is used to determine the ring and test hole volume, then the temperature of the water must be measured at the time it is used in the hole. After the water is placed in the hole, the volume of the hole is calculated based on the unit weight of water at the temperature measured. Care must be exercised to avoid losing any of the water into the underlying soil where the test hole is excavated. A leak in the membrane will cause two errors: (1) the water content of the in-place soil of the test will not be representative of the fill, and (2) the volume measurements will be incorrect. A typical method involves pumping the water from a tank on a truck. In this method of providing the water, a gauge on the tank or a flow meter gauge can be used to determine the volume of water used for each measurement. The measurement gauges should be accurate to 1 % and volume corrections should be made for temperature differences in the water between the time of calibration and testing.

C. STANDARD TESTS FOR DENSITY TESTING AND QUALITY CONTROL OF VERY-COARSE-GRAINED SOILS (ROCK FILL)

Rock fills are very coarse soils containing mostly gravel and cobble-size particles. These materials are used in many forms of embankment construction. Rockfill material generally has less than 5 % fines, but in some cases this type of fill could be classified as dirty with higher sand and fines contents. Rock fills are often compacted with vibratory rollers with a lift thickness of 1–3 ft, depending on the particle size and the compactive effort of the equipment. Maximum particle size for most rockfill materials range from 12 to 18 in. Water may or may not be used during compaction, depending on the amount of sand and fines. Rock fill is an inherently strong and low-compressibility material. As long as good compaction is applied, it may not be necessary to perform routine check testing. Inspection of rockfill projects typically involves observation to ensure good coverage of the compactor and to identify problem areas in the fill.

Density testing of coarse rockfill material is most often expensive and labor intensive. Extensive testing is frequently limited to projects with critical design parameters such as dams or power plants. Often, rockfill compaction is tested at the beginning of a project using test fills to develop placement parameters that can be used for the remainder of the project. During the test fill construction, the rockfill material is placed at various lift thickness and number of passes of the equipment to determine the optimum parameters for construction.

A sample form for recording test fill data is in Fig. 16B of Appendix B.

As part of the test fill procedure, the rock fill can be monitored for settlement after incremental compaction of lifts using simple survey elevation measurement equipment. Once the optimum compaction method is established from the test fill information, the ongoing rockfill quality control is based on visual observation and records to ensure that that the compactor operator is providing a sufficient number of passes. On many projects, visual observation of the compaction process, the number of compactor passes, and lift thickness are often the best indication that adequate compaction is being attained.

The field in-place density of some rockfill materials can be determined using D5030 (water replacement in test pit). This test can handle maximum particle sizes of up to 12 in. in 6- to 7-ft-diameter rings. Fig. 5 shows some typical pit excavations. For most rockfill projects the particle is such that the moisture content in not a significant parameter. For finer particle rock fill with a significant portion of sand and gravel, the Oven-Dry Method (D2216) or Direct Heating Method (D4959) can be used to determine the moisture content necessary for computing the in-place dry density.

The method for the determination of the laboratory maximum dry density for coarse rock fill has not been standardized. To provide some indication of the maximum attainable density, special equipment has been built to determine a maximum density in a controlled condition. Additional information on the laboratory-determined values for the maximum density of rock fills can be found in the ASTM Standard Test Procedure 523 on Relative Density testing [2] and in other publications [3]. The projects described in these publications describe density determination using large containers up to 36 in. in diameter that are equipped with concrete form vibrators. The soil and rock fill in these test procedures is placed in layers and vibrated with a surcharge mass. Depending on the amount of oversize, control techniques are similar to previous sections where oversize corrections are made. It is important to note that many of the test methods for the maximum dry density of coarse rock fill are in the process of being evaluated by the ASTM at this time.

Finer rock fill, including gravel fill, with less than 30 % larger than 3 in. in dimension and less than 10–15 % fines can be controlled successfully with the Relative Density test (D4253 and D4254) as the standard for comparison of field-derived values. The procedures for these methods are similar to those for relative density provided in Chapter 3. Quality control of finer rock fills can be based on percentage of relative density or by percentage of the vibrated maximum density. Once the percentage of particles larger than 3 in. increases past 30–40 % the smaller material begins to "float" in the matrix of coarse particles, and compaction of the control fraction is reduced. In the case in which a larger percentage of coarse particles exists, the quality-control program will require that larger dimension compaction tests be developed.

References

[1] *Earth Manual*, 3rd Ed., Part II, U.S. Department of the Interior, Bureau of Reclamation, U.S. Government Printing Office, Washington, DC, 1990.

[2] *Construction Testing of Embankments Containing Large Particles*, U.S. Committee on Large Dams, Denver, CO, 1988.

[3] ASTM STP 523, Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils, ASTM International, West Conshohocken, PA, 1973.

5

Standard Test Procedures for Determining the Water Content of Soils

A. PURPOSE AND USE OF WATER CONTENT TESTS

The water content of soil is determined as a means to calculate the dry density of the soil or to determine if the water content is within the range that has been specified for the compaction of the soil. Water content testing is also used to provide preliminary quality-control information that can be used as a guide during construction on whether water needs to be added or removed from the soil. The water content is always an important factor in controlling earthfill compaction. The values of water content are often needed quickly and sometimes in remote borrow areas.

The terms "water content" and "moisture content" have historically been used interchangeably. Both terms may be found in the ASTM standards. There is no significance to the use of one term over the other.

The definition of water content is "the ratio of the mass of water contained in the pore spaces of soil or rock material, to the solid mass of particles in that material, expressed as a percentage." The equation is

$$w = \frac{M_w}{M_s} \times 100 \tag{5-1}$$

where:

w = water content in percent
M_w = mass of water
M_s = dry mass of soil

It is important that only the mass of water contained in the pore spaces is included in the calculation. For most inorganic soil, this is the mass of fluids driven off by heating the soil to a constant mass at a specified temperature of 110 ± 5°C or 212°F. It is important to note that when the soil contains some types of organic matter, gypsum, or other hydrated compounds that dehydrate at temperatures under 110°C, a drying temperature of 60°C is used. The solid mass of particles is the constant mass at the end of the drying period at the specified temperature of 110 ± 5°C or 212°F. It is also important to note that the water content is at the required drying temperatures provided in the test methods and that more moisture may be removed from finer-grained soils if the temperature is taken above the specified temperature of 110 ± 5°C or 212°F. The water content is calculated as the mass of the water (difference in mass before and after drying) divided by the mass of soil remaining after drying, multiplied by 100.

B. STANDARD TESTS FOR DETERMINING WATER CONTENT OF SOIL

There are several different ASTM standards for measuring the water content of soils used for construction purposes. The principal ASTM standard for the moisture content of soil and rock is D2216, Test Method for Water (moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures. This is the "reference" method, or the method that the other tests for water content are compared to. It is also the method used when the most accurate results are required. It has the disadvantage of taking up to 24 h (or longer when lower temperatures are required) to obtain results, so other faster methods have been developed that can provide results that are accurate enough for most field work if adequate care is provided by implementing the precautions described in the test methods.

The other methods that can be used to determine the water content of soil are described in the following ASTM standards:

- D6938 In-Place Density and Water Content of Soil and Soil Aggregate in Place by Nuclear Methods (Shallow Depth)
- D4643 Determination of Water (Moisture) Content of Soil by Microwave-Oven Heating
- D4944 Field Determination of Water (Moisture) Content of Soil by the Calcium Carbide Gas Pressure Tester Method
- D4959 Determination of Water (Moisture) Content of Soil by Direct Heating Method

Although these four methods are used primarily to give more rapid, and at times more approximate, results, they most often result in values comparable to the "oven-dry" method (D2216). To ensure that the more rapid methods provide representative results, the tests need to be carefully performed on soils according to ASTM procedures. The proper performance of the method includes performing the required equipment calibrations and implementation of the required quality-control checks/correlations.

In some instances it may be necessary or desirable to test the material using two water-content methods for comparison. Representative testing may require that the sample be split into two or more separate size fractions. For easy handling or during sample preparation for other tests, separating the material into fractions may be desirable if only a few random, significantly larger particles are present. Because only 100 g are typically needed for a specimen with most, minus No. 4 sieve material, separating on the No. 4 sieve may be advantageous to conserve the minus No. 4 material.

To calculate the water content of a material that has a wide range of sizes, the percentage (by dry mass) and the water content of each size fraction must be determined. This is typically accomplished by separating the material into two or more fractions using the appropriate sieves. The minimum mass for each fraction must conform to the requirements listed in the ASTM method. The percent of each size fraction and the water content must be determined.

$$w = [(w_1 \times P_1) + (w_1 \times P_2) + (w_n \times P_n)] \times 100 \qquad (5\text{-}2)$$

where:
w = water content of total material (percent)
P_1, P_2, P_n = percent (by mass) of each size fraction in total material (percent)
w_1, w_2, w_n = water content of each size fraction (percent)
100 = constant representing total material (percent)

A data sheet for the summary of ASTM methods used can be found in Fig. 16B of Appendix B.

1. Oven-Dry Method (D2216)

APPLICATIONS

This method includes all of the measurements needed to calculate water content according to its definition. Some other ASTM test methods for water content of soil/rock refer to D2216 as the reference method and the standard that the other methods are correlated or compared to. ASTM D2216 is the method most often specified and the one preferred when sufficient time and facilities are available (equipment, power source, etc.) to perform the test.

The water-content test specimen is dried in a thermostatically controlled oven at 110 ± 5°C or 212°F to a constant mass. The loss in mass due to drying is considered to be water content of the soil. Sands with small amounts of fines may often be dried to constant mass in approximately 4 h while most soils with some percentage of fine particles can take 12–16 h. Typically, the soils are dried overnight.

SPECIAL APPLICATIONS FOR SOILS CONTAINING HYDRATED WATER

The drying temperature of 110 ± 5°C or 212°F may not be appropriate for determining water content of soils containing cement, organdies, gypsum, or other materials containing significant hydrated water. The user is referred to Paragraph 1.3 in the Scope Section of Method D2216 for further description of the materials requiring special consideration. If a material with a significant amount of hydrated water is suspected to be in the soil, a companion specimen should be dried at 60°C or at room temperature to a constant mass and the results compared with the results of the specimen dried at 110 ± 5°C or 212°F. The 60°C or room-temperature drying temperature will likely require a much longer drying time to reach a constant mass. If a difference greater than 5°C is found, the lower temperature should be used on all subsequent tests on this soil. For these tests, the test report should always indicate the temperature used for the drying process.

Materials containing water with substantial amounts of soluble solids (such as salt in the case of marine sediments) when tested by this method will give a mass of solids that includes the previously soluble solids. These materials require special treatment, including moisture offsets and controlled rates of heating to account for the presence of precipitated solids in the dry mass of the specimen or for a qualified definition of water content.

ADVANTAGES

The advantages of the oven dry methods are

- Less safety hazards than other procedures for soil containing nonhazardous materials.
- Test results are widely accepted.
- Most soils can be left drying in an oven over the weekend.
- Equipment is durable.
- There are good commercial ovens available for drying soils that do not require any modification.

DISADVANTAGES

The disadvantages of the oven-dry methods are

- Most often 12–16 h are required to dry soil to a constant mass. This turnaround time is typically unacceptable for field quality control in which moisture content and density results are needed in less than 1 h.
- The method typically requires a laboratory or location protected from the weather with an electric power source.
- Some commercial ovens do not have sufficient power to consistently dry soils or do not meet the ASTM requirements for consistent temperature control, or both. Care must be exercised in selecting the proper oven.
- Ovens with more than 1 m^3 (10 ft^3) of space may need a 220-V power source.

CALIBRATION AND PRECAUTIONS

There is not an ASTM standard for calibrating ovens, nor does test method D2216 give a procedure for calibrating ovens.

The following is a step-by-step procedure that is used by most soils laboratories for calibrating ovens used for moisture-content testing. These procedures are modeled after a Bureau of Reclamation procedure and are provided in “USBR 1020, Procedure for Calibrating Ovens” [1]. This calibration procedure should be performed upon receipt of the oven and typically on an annual basis thereafter.

The only equipment needed is an etched-stem glass thermometer with an eyehook at the end for easy hanging, a 0 to 300°C temperature range, and 2°C graduations that is filled with mercury. The procedure is as follows:

1. Check to see that the thermometer to be used for this procedure has a certificate of inspection or calibration verification from the manufacturer. If there is any doubt as to the accuracy of the thermometer, it should not be used for this procedure. A thermometer with a verifiable accuracy should be obtained and used.
2. During the calibration procedure, the room temperature should not vary more than ±20°C.
3. Locate and record the serial number and any other identifying markings of the oven to be calibrated.
4. Remove any material (sample containers, pans, etc.) from the oven.
5. Carefully hang the thermometer by its eyehook with a piece of wire as close to the center of the oven chamber as possible. Make sure that the thermometer hangs freely.
6. Determine the applicable temperature at which the oven will be calibrated and set the thermostat of the oven to that temperature. For most geotechnical purposes, the oven will be calibrated at 110°C or 212°F. If calibration of the oven is desired at another temperature, the procedure is similar. If a range of calibrated temperatures is desired, the calibration should be performed in intervals of 5°C over the desired range of temperatures.
7. Allow sufficient time for the oven temperature to stabilize and record the thermostat setting.
8. Determine the thermometer reading and record the value. The oven temperature will drop drastically when the oven door is opened. The thermometer reading should be taken as quickly as possible once the door is opened.
9. Calculate the difference between the thermostat setting and the thermometer reading.

10. If the thermometer reading is within ±5°C of the thermostat setting, the calibration procedure is completed. If the thermostat reading differs from the thermostat setting by more than 5°C, adjust the thermostat by the amount of the difference. Retest for the temperature to verify that the new setting will result in the desired temperature.
11. If the thermostat setting is different from the required temperature, a notice should be placed next to the thermostat.
12. To check if the temperature is uniform throughout the drying chamber, the thermometer can be located in various positions in the oven.

PRECAUTION OR POTENTIAL SOURCES OF ERROR

As indicated in ASTM D2216, there are several precautions and potential sources of errors that need to be understood to minimize the potential for errors in the test results. The following sections list the most important sources of error and recommendations for decreasing their influence on the test results.

Sample Selection and Preparation

The selection and preparation of the sample are the most important procedures that can increase the degree of error in the water-content test results. The important parts of sample selection include selecting a sample of sufficient size and a large enough cross section of a nonhomogeneous sample to be representative of the soil that is being tested and following the procedures outlined in D2216 for including or not including oversize materials or deleterious materials in the sample. In addition to the sample selection, another important part of minimizing errors is to prepare the sample with proper technique. Some of the key items for preparing samples are (1) protecting the sample from sunlight or heat before preparation to minimize the amount of moisture loss from the soil, (2) sieving and placing the sample in the drying container rapidly to minimize the moisture loss to the ambient air before obtaining the predrying weight, and (3) including a representative amount of oversize and deleterious materials to be representative of the soil that is tested.

Equipment Condition and Calibration

In general, the water-content test is a simple and easily performed test as long as the basic procedures for maintaining equipment condition and calibration are followed. The oven is obviously the most important piece of equipment and the procedures listed above provide a practical process to ensure calibration. In addition to the oven calibration, the accuracy of the scale and condition of the oven-dry pan should also be checked to ensure that they are accurate before and after water mass is obtained. Typical sources of error are oven-dry pans with inaccurate tare weights and scales that have not been accurately calibrated to an independent standard of weights and measures.

A detailed step-by-step procedure for performing the water content test is also provided in Test Method D2216. It is important to note that the water content is significantly affected by the selection of a representative sample. Although not included in the other water-content methods, the principles in this section apply to all of the other methods. A sample form and an example computation for this method can be found in Fig. 17B of Appendix B.

2. Microwave-Oven Method (D4643)

APPLICATIONS

This method uses a microwave oven to dry the soil instead of the conventional oven and is used when rapid results are needed. Typically, the water content can be determined in less than 15 min. Microwave heating is a process by which heat is induced within a material from the interaction between molecules of the material and an alternating, high-frequency electric field. Microwaves are electromagnetic waves with 1-mm to 1-m wavelengths.

Test Method D4643 uses an incremental drying procedure to avoid overheating the soil. A moist soil specimen is placed in a suitable container and its mass is determined by weighing on a calibrated scale. The sample is then placed in a microwave oven, subjected to an interval of drying, and removed from the oven and its new mass is determined. The interval of drying must be kept short to avoid overheating the soil or causing damage to the oven, or both. This procedure is repeated and the result is recorded until the mass becomes nearly constant within a percent difference specified in Test Method D4643. The difference between the mass of the moist specimen and the dried specimen at its steady state or constant mass condition is used as the mass of water originally contained in the specimen. The water content is determined by dividing the mass of water by the mass of dry soil then multiplying by 100. For a given soil and sample size, the time to achieve a constant dry mass can be noted and used as a minimum drying time for subsequent tests using the same size specimen of the same soil.

Although the microwave-oven method is typically used to get rapid, approximate results, more accurate results can be obtained when the method is correlated to the oven method for specific soil types. All parties interested in the test results should be aware of and agree that the results from the microwave-oven method can be used.

The incremental drying method can be tedious and time consuming. A system using a computer-controlled balance to continually monitor the soil specimen mass has been developed and is available [2,3].

ADVANTAGES

The advantages of the microwave-oven method are

- Results are typically available in less than 15 min.
- This method can be used in place of conventional oven method D2216 when proper correlation studies and periodic comparison to the results of D2216 are provided. Typically a set of 5–10 tests for correlation is adequate for smaller projects and a set of 20–30 tests and weighted average for the entire project is used for larger projects.
- Microwave ovens are typically compact and portable within the laboratory.
- Equipment is usually available locally.
- Data sheets for oven-drying methods can be used.
- Microwave ovens only need a 110-V power source.

DISADVANTAGES

The disadvantages of the microwave oven method are

- Specimen size can be limited because of the small volume of the microwave heating chamber.
- Because of potential particle explosion and loss of the sample, the maximum particle size in soil should not exceed 25 mm (1 in.). This method is best suited for minus No. 4 size material.

- This method is not reliable for soils containing a significant amount of halloysite, mica, montmorillonite, gypsum, organic matter, or dissolved salts in pore water.
- Metal drying pans or containers cannot be used in the oven.
- Use of a microwave oven to dry soils has unique hazards not associated with food preparation (see *Precautions* section of the standard). Operators may neglect required safety measures, which can cause injury or property damage.
- Generally, a laboratory or location protected from the weather with an electric power source is required, which may not be available at all field locations.
- Lower-cost microwave ovens can have an inconsistent source that may not be comparable to the temperature generated by the more consistent temperature required by D2216. The inconsistent heat source can cause overdrying of fine-particle materials in a manner that provides steady-state microwave moisture content readings that do not correlate to the referee Test Method D2216. The consistency of the microwave source may also change over time as the microwave is used on multiple sample and soil types.
- The use of a microwave oven for the drying of soils may be considered abusive by the manufacturers and constitute voiding of warranties. It is common practice for the ovens used for the drying of soils to have a significantly shorter life than standard microwaves because the goal of D4643 is to regularly dry the sample completely with no water source remaining to protect the microwave oven from damage.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

The use of the microwave method for determining the water content of soils requires an understanding of the precautions and potential sources of errors listed above in D2216. In addition to the technical concerns for D2216 and D4643, the drying of soils with a microwave oven has certain hazards unique to its use as listed below.

- Water-saturated particles may explode during and after drying. Steam is generated more rapidly in the interior of a particle than it can escape through particle pores. Eye protection should be worn during the heating, mixing, and weighing of the test specimen. A covering over the sample container may be appropriate to prevent operator injury or oven damage. A cover of heavy paper toweling has been found satisfactory for this purpose. This also prevents the scattering of the test sample in the oven during the drying cycle.
- The oven door must seal properly to prevent microwave radiation leakage. The door of the oven is designed with interlocks (reed switches and door-strike switches) to prevent open-door use. No attempt should ever be made to defeat these interlocks. The microwave system should never be operated if the door has been damaged or warped. Additionally, the system should be checked periodically for leakage with a microwave leakage detector. An allowable emission of microwave radiation is 5 mW/cm^2 measured at 5 cm from the surface of the oven. If leakage intensity is higher than this level, a qualified technician should service the door.
- Highly organic soils and soils containing oil or other contaminates may ignite into flames during microwave drying. Means for smothering flames to prevent operator injury or oven damage should be available during testing. Fumes from contaminated soils or wastes may be toxic, and the oven should be vented accordingly.
- A "heat sink" (container of liquid or other material containing moisture) should be placed in the oven while drying soil. The heat sink is to provide some moisture in the oven after the soil is completely dry and will help to reduce the potential for damage to the oven when the soil approaches a dry condition. When using a heat sink on multiple samples, the inside of the microwave can become saturated with excess water or condensation in a manner that can add moisture to the sample or make consistent drying difficult. If this condition occurs, it is best to dry out the inside of the microwave, place cooler water in the oven, and restart the drying process.
- The containers used to hold the soil placed in the oven for drying must not contain any metal. The soil or other material being dried must not contain any metal or metal-like material. There are other safety precautions mentioned in Test Method D4643.

A sample form and example computation for this method can be found in Fig. 18B of Appendix B.

To minimize the potential sources of errors, the step-by-step procedure described in Test Method D4643 should be followed closely without exception. To provide consistent results, it is the experience of most senior technicians that the data sheets used for the conventional oven-drying method can be used for this method as long as the multiple results are recorded to document when the sample weight is within the maximum percent change described in Test Method D4643.

The moisture content of soils containing gravel particles can be successfully determined by separating out the gravel particles and using the microwave oven to determine the water content of the minus No. 4 fraction. A water content can be assumed for the gravel particles, typically approximately 3 %, and the combined water content can be calculated using equation 5-2.

3. Direct Heating Method (D4959)

APPLICATIONS

This method includes applying direct heat to a metal pan that contains a sample of moist soil for the purpose of drying the soil. The direct heat is typically applied using a hotplate, gas stove, a blowtorch with a control-setting valve, or a similar device that can give quick and useful results. The test method is typically used by field technicians when the requirements of the project will allow a water-content value that is not as precise as the Oven-Dry Method D2216. The accuracy of the method can be increased by the judgment of skilled technicians, the use of methods that provide a low, steady heat to the sample, and the comparison of test results to a database of split test samples using the Direct Heat Method D4959 and the Oven Dry Method D2216.

Using heating devices that do not need electric power (gas stove, camp stove, etc.) makes direct heating a very useful method for remote locations. Because some soils can be altered or organic matter burned from overheating, or both, the user is cautioned to refer to the Significance and Use Section of Test Method D4959. The direct heating water-content test is sometimes used to provide a quick indication of the approximate value or to determine when other tests should be performed, but it may be used for acceptance testing on those soils that are not adversely affected by overheating, such as clean sands and gravels.

The proper application of Test Method D4959 includes the preparation and selection of the samples using techniques similar to those used for D2216. After weighing, the moist soil specimen is put into a suitable container and the container is heated by the hotplate, gas stove, etc. It is important to note that direct flame is never applied directly to a soil that is being properly tested using Test Method D4959.

ADVANTAGES

The advantages of the direct-heat drying method are

- Test results are obtained rapidly, which is especially advantageous for construction testing and quality control of large earth- and rockfill projects.
- The equipment for direct-heat moisture-content testing is typically simple, rugged, portable, and does not need to be protected or stored in a laboratory-controlled environment. It can be used easily in unprotected field locations without a readily available electric power source.
- Bottled gas for field ovens and stoves can be used in remote areas without electric power.

DISADVANTAGES

The disadvantages of the direct-heat drying method are:

- Test results may not be accurate enough for soils in which overheating alters the soil. Test results may be inconsistent when the direct heat method is used on organic soils, some types of micaeous soils, moisture-sensitive silts, and soils in which the chemically bound water will be dehydrated at high temperatures.
- Coarse sand-size particles and larger may explode during heating with the direct heating method.
- The soil is not dried uniformly and may become too hot, which will change its characteristics. It is limited to soils that do not contain any combustible materials or compounds that will dehydrate or be altered by overheating.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

The use of the direct heating method for determining the water content of soils has similar precautions and potential sources of errors as the previous methods listed above. In addition to the technical concerns for D2216 and D4643, the direct heating of soils requires that the technician understand the following potential sources of errors.

Overheating or Burning of Soils

To provide water content results that are similar to the results for Test Method D2216, it is essential that the temperature of the sample be controlled to avoid overheating of the sample. To accomplish a controlled yet rapid heating of the sample, experienced technicians use various methods to determine when most of the moisture has left the sample, but the sample is not burned. These techniques include the following: (1) placing a glass lid in the pan with the sample that will condense excess moisture and remain dry when the sample is mostly dry, (2) heating the sample near completion and removing the sample and weighing it several times until a steady-state weight is established, (3) observing the sample color and stirring the sample regularly to ensure that the soil is heated uniformly and not burned, and (4) sieving the sample carefully to pulverize sample particles that would hold moisture and allow nonuniform heating of the sample.

Unintentional Moisture Loss from the Sample

This potential source of error typically occurs during sample preparation or during the heating process. Typical sources of error that can be minimized by the skill of the field technician include allowing heat or sunlight to condense moisture out of the sample and onto the sample bag or container. Storing the sample carefully and weighing the sample within 20–30 min after it is obtained from the field most often minimizes this source of error. Another source of error that can be minimized is loss of the sample during the heating process caused by popping of large particles or dropping of soil particles with the mixing spoon, or both. Carefully mixing and processing of the sample minimize this source of error. A sample form and example computation for this method can be found in Fig. 19B of Appendix B.

4. Calcium Carbide Gas Pressure Tester Method (D4944)

APPLICATIONS

This test is primarily known as the "Speedy Moisture Meter Test." The soil moisture is related to the gas pressure produced when the wet soil is mixed with a reagent, causing the reagent to react with the water in the soil. The amount of gas produced is determined by the amount of water in the soil.

A measured volume of calcium carbide is placed in the testing apparatus along with two steel balls and a small specimen of soil having all particles smaller than the No. 4 sieve size and having a mass equal to that specified by the manufacturer of the instrument or equipment. The apparatus is shaken vigorously in a rotating motion so that the calcium carbide reagent can contact all of the available water in the soil. Acetylene gas is produced proportionally to the amount of available water present. The apparent water content is read from a pressure gauge on the apparatus calibrated to read in percent water content for the mass of soil specified.

A calibration curve is developed for each instrument and each soil type by plotting the pressure gauge reading and the water content determined from Test Method D2216 using representative specimens of the soil. The calibration curve is used to determine a corrected water content value for subsequent tests on the same type of soil.

This test method is used when results are needed within a short time period and in locations where it is not practical to install an oven or to transport samples to an oven. See photos 10C and 11C in Appendix C.

ADVANTAGES

The advantages of the Calcium Carbide Gas Pressure Tester Method are

- The test results are usually available in less than 5 min.
- The equipment is portable and can be transported and operated by one person. No laboratory or protection from the weather is needed except that the calcium carbide must be kept dry.
- Electric power is not needed for the test device.
- The equipment is rugged and will usually not be damaged under normal transportation in field vehicles.

- Some soils containing compounds or minerals that dehydrate with heat (such as gypsum) that are to have special temperature control with Test Method D2216 may not be affected (dehydrated) in this test method.

DISADVANTAGES

The disadvantages of the Calcium Carbide Gas Pressure Tester Method are

- The specimen size is very small (26 g), which can cause the test results to have a low degree of accuracy if the specimen is so small that it is not representative of the soil intended. When the sample has to be reduced because of high water content of the material to be tested, the size is even smaller at 13 g.
- Because the reagent must react with all of the water in the soil, the test is limited to friable soil that will pass a No. 4 sieve. Highly plastic soils or any soil containing clods or clumps that will not break down cannot be tested.
- There may be some soils containing certain compounds or chemicals that will react unpredictably with the reagent and give erroneous results.
- The safety hazards associated with the equipment require conscientious use by the operator to avoid injury to people or damage to property (see *Precautions*).
- Calcium carbide quality will deteriorate with time after it becomes exposed to the atmosphere or any source of moisture. Careful monitoring of the quality of the calcium carbide being used and periodic purchase of a new supply is recommended. Calcium carbide that has deteriorated will usually turn color from a dark gray to light brown.
- Equipment limitations require the use of samples significantly smaller than what is typically recommended to properly represent the soil that is being tested. Extra care must be exercised to select samples that are truly representative of the soil.
- Federal U.S. Department of Transportation (DOT) hazardous materials regulations require that calcium carbide reagents be shipped only by certified shippers and can go by ground or air.
- The soil/carbide mixture left at the end of the test can be hazardous unless it is properly disposed of. Precautions are necessary to prevent igniting the explosive gas that is produced.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

The use of the Calcium Carbide Gas Pressure Test Method for determining the water content of soils requires an understanding of the precautions and potential sources of errors listed in D2216 and other methods listed above. In addition to the technical concerns for D2216, the water-content determinations using a calcium carbide gas test device have certain hazards unique to their use.

- When combined with water, the calcium carbide reagent produces a highly flammable or explosive acetylene gas. Testing should not be carried out in confined spaces or in the vicinity of an open flame, embers, or other sources of heat that can cause combustion. Care should be exercised when releasing the gas from the apparatus to direct it away from the body. Lighted cigarettes, hot objects, or open flames are extremely dangerous in the area of testing. **If a fire results, water must not be used to put out the fire**. To provide adequate protection from fire, a blanket or an ABC class dry chemical fire extinguisher should be used.
- The operator should take precautions to avoid the gas fumes and use clothing with long sleeves, gloves, and goggles to keep the reagent from irritating the eyes, respiratory system, or hands and arms.
- Attempts to test excessively wet soils or improper use of the equipment (e.g., adding water to the testing chamber) could cause pressures to exceed the safety level for the apparatus. This may cause damage to the equipment and an unsafe condition for the operator.
- Clean out the soil/reagent mixture from each test in a well-ventilated area, preferably outdoors when possible.
- Care should be taken to keep the calcium carbide reagent stored in a dry place and avoid contact with water because it will produce an explosive gas.

A sample form and example calculation for this method can be found in Figs. 20B and 21B of Appendix B. Photos are in Appendix C.

CALIBRATION OF EQUIPMENT

Calibration kits are available from manufacturers for testing gasket leakage and for calibrating the gauge. Periodic checks for gasket leakage are recommended. The gasket should be changed when leakage is suspected. Gauge calibration problems can usually be detected when the instrument calibration curves are made. When the gauge needs adjusting, a manufacturer-approved calibration method should be used.

CALIBRATION FOR TESTING

The following steps are the calibration procedure stated in Test Method D4944. They are given here verbatim to emphasize that calibration curves must be developed for the soils to be tested. Too often, this test is misused by running the test and using the number on the gauge as the moisture content without correcting the reading using calibration curves. The tester normally has a calibration curve or table furnished; however, to make this procedure more accurate, a calibration table should be prepared for each individual device and for the range of soil types to be tested. A different soil type means a different classification of soil by the Unified Soil Classification System (ASTM D2487).

1. The manufacturer-supplied equipment set, including the testing chamber with attached gauge and the balance scales, are calibrated as a unit and paired together for the testing procedure.
2. Calibration curves must be developed for each set of equipment using the actual soil types to be tested and the expected water content range of the soil. As new materials are introduced, further calibration is needed to extend the curve data for the specific instrument. If tests are made over a long period of time on the same soil, a new calibration curve should be made periodically, not exceeding 12 months.
3. Calibration curves are produced by selecting several samples representing the range of soil materials to be tested and having a relatively wide range of water content. Each sample is carefully divided into two specimens by quartering procedures or by use of a sample splitter. Taking care to not lose any moisture, one specimen is tested in accordance with the procedure of this test method without using a calibration curve, and the other specimen is tested in accordance with test method D2216.

4. The results of the oven-dry water content determined by Test Method D2216 from all of the selected samples are plotted versus the gauge reading from the calcium carbide tester for the corresponding test specimen pair. A best-fit curve is plotted through the points to form a calibration curve for each soil type. Comparisons should be relatively consistent. A wide scatter in data indicates that this test method or Test Method D2216 is not applicable to the soil or conditions. Fig. 21B in Appendix B shows a typical calibration curve.
5. A comparison of this test method with Test Method D2216 for a given soil can be made using the calibration curve. Points that plot off the curve indicate deviations. Standard and maximum deviations can be determined if desired.

Each calibration curve should be compared to the previous calibration curve on the same soil. If the gauge readings are more than 2 percentage points different, the device itself needs to be recalibrated, repaired, or discarded. A change in the reading of more than 2 percentage points may be an indication of an equipment problem, such as a pressure leak.

DESCRIPTION OF TEST PROCEDURE

The step-by-step procedure for the Calcium Carbide Gas Pressure Test Method is described in Test Method D4944. Detailed procedures are also published by the Bureau of Public Roads [4] and by the manufacturer [5]. In addition to the procedures provided in Test Method D4944, careful evaluation of the data should be made when the soil water content is below 5 % or above 30 %. To ensure consistent results, it is best to purchase calcium carbide in small containers with airtight replaceable lids, to store it in a dry place, to keep the lid on the container at all times except when measuring out a portion for use in a test, and to use a complete container before opening a new one.

5. Nuclear Methods (ASTM D6938)

This test is covered in Chapter 4, Section B.3. A major precaution in the use of the nuclear gauge is the awareness that for most soils used in earthwork, a moisture correction is necessary to obtain accurate values for water content and dry density. This has already been covered in Chapter 4 and is mentioned again in Chapter 6.

References

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[4] Blystone, J.R., Pelzner, A., Steffens, G.P., "Moisture Content Determination by the Calcium Carbide Gas Pressure Method," Bureau of Public Roads, Vol. 31, pp. 177–181, 1961.
[5] "Speedy Moisture Testing Procedure," Operation Manual, Alpha-Lux Company, Philadelphia, PA.

6

Quality Control and the Coordinated Use of Laboratory and In-Place Tests for Compaction Testing

DURING THE INSTALLATION OF A TYPICAL EARTH-fill project, the density, water content, and degree of compaction of the earth fill is confirmed and documented by testing the in-place compacted soil. The compacted soil is measured to determine its dry density and water content. These values are then compared with the maximum dry density (standard reference density) and optimum water content of the soil being evaluated. The project specifications usually require the in-place dry density to be a minimum percentage of the maximum dry density. The specifications will usually require the placement water content of the fill to be within a range of water content relative to the optimum water content of the soil being tested.

The in-place density and water content of compacted soils are measured in the field using various methods. Several methods are also used to determine the laboratory reference values for maximum dry density and optimum water content to which the field measurements are compared. Specifications should state which methods are acceptable for measuring the field density and water content as well as which method(s) is (are) acceptable for determining the reference values for the soil.

The ratio of the in-place field density test to the standard reference density test for that soil is called "the percent compaction." Compute percent compaction by dividing the in-place dry density by the maximum dry density of the standard reference density test, then multiply by 100 and express as a percentage. The maximum dry density determined by the standard reference density test must be determined for the same soil or a previously tested sample of the same soil type that represents the soil where the in-place test was made. Refer to Section F of this chapter for guidelines on selecting the proper standard reference density used as a comparison for in-place tests. The computed degree of compaction and in-place water content should be reported to the nearest whole number because the tests are not accurate enough to state results to any higher precision. For example, the specifications may indicate that compaction shall be to 95 % (not 95.0 %) of the maximum dry density determined by ASTM D698. The specifications may also indicate that the water content shall be from 2 % (not 2.0 %) below to 2 % above the optimum water content determined by ASTM D698. An example of reporting the measured degree of compaction is to state the measurement as 96 % compaction, not 95.6 %. In reporting measured water content, the result would be stated as 1 % wet of optimum, not 1.2 %.

To accurately determine the percent compaction, the maximum dry density (standard reference density) must be determined accurately. This can be done by performing compaction tests on all of the different types of soils that will be used in the earthfill construction. The borrow soils and all practical combinations (or blends) of soils should be represented by the standard reference densities. This may be relatively simple for uniform materials in which the test results are nearly the same from test to test. In many alluvial deposits or where the borrow area contains layers or pockets of differing material, hauling and spreading operations often mix the various materials in different proportions. These differences must be recognized and provisions must be made for establishing appropriate standard reference densities or utilizing methods to verify the appropriate reference densities. Refer to Section F of this chapter for procedures that establish which standard reference density test values are appropriate for a given field density test. Procedures such as the Rapid Determination of Percent Compaction (D5080) and the Family of Curves–One-Point Method (AASHTO T 272) can be used.

The steps required to compare field in-place measurements to standard reference density test values are explained in the following section for various soil types. Different procedures are used depending on the properties of the soils being tested. The gravel content of the soil being tested can significantly affect these procedures. These steps are also illustrated in the flow chart in Fig. 1A (see Appendix A). Table 1A (Appendix A) can also be helpful in relating to the compaction properties of different soil types.

A. SILTS AND CLAYS OR SANDY FINE GRAINED SOIL WITH LITTLE OR NO GRAVEL

For soils with less than 5 % oversized particles (gravel retained on the No. 4 sieve), the comparisons between the in-place compacted soil and its reference test values can be made directly without corrections. The standard laboratory compaction tests that are used to determine the compaction index properties (maximum dry density and optimum water content) are D698 or D1557 or D558 for soil cement. The standard tests for measuring the in-place density of the compacted soil are D1556 (sand cone), D2167 (rubber balloon), D6938 (nuclear gauge), and D2937 (drive cylinder). The applicable water-content test procedures are D2216 (oven dry), D4643 (microwave oven), D4959 (Direct Heating), D4944 (Calcium Carbide Tester), and D6938 (Nuclear Method).

1. Procedures

The procedures for checking an earth fill for compliance with density and water-content requirements when the soil contains less than 5 % oversize particles (particles larger than the No. 4 sieve) are as follows:

1. Determine the maximum dry density and optimum water content of the soil being evaluated using Procedure A of D698 or D1557. The amount of material retained on the No. 4 sieve must be less than 5 % of the dry weight for this procedure to apply. The soil is passed through the No. 4 sieve regardless of whether it has oversize material or not. This ensures that the soil is properly broken up to run the test. The test procedures must be followed as outlined in the appropriate standard and as discussed in Chapter 3 of this manual.
2. Measure the in-place field density at a randomly selected location of the compacted fill or in areas that are suspected to have localized substandard compaction, or both. The specifications should indicate one or more acceptable methods for measuring the in-place dry density and water content. Follow the test procedures as outlined in the appropriate standard and as discussed in Chapter IV of this manual.
3. Determine the water content of the in-place test by using a representative portion of the in-place test material or use the test procedure for the nuclear gauge (ASTM D6938). The specifications should indicate which of the five water-content test procedures indicated above are acceptable, with periodic correlation to the Oven-Dry Method (ASTM D2216). Follow the test procedures as outlined in the appropriate standard and as discussed in Chapter V of this manual. If the Rapid Method (D5080) is used, the water content need not be measured because the test results provide the variation from optimum water content without drying a specimen of soil.
4. Calculate the in-place dry density as directed in the in-place test procedure. Dividing the in-place dry density by the maximum dry density of the appropriate standard reference density test that represents the soil of the in-place test and multiplying by 100 calculates the percent compaction. No corrections for oversize materials are needed because the soil contains less than 5 % oversize particles.
5. Check compliance with the specifications by comparing the percent compaction computed from the test results with that specified and the water content with the optimum water content of the standard reference density test selected.

B. SILTY OR CLAYEY SOIL WITH GRAVEL (5 % OR MORE RETAINED ON THE NO. 4 SIEVE AND UP TO 30 % RETAINED ON THE 3/4-IN. SIEVE)

For soils with more than 5 % retained on the No. 4 sieve, a correction must be made to values of measured density and water content to account for the presence of the oversize particles before comparing the results of in-place tests to the standard reference density test results. These corrections can be made in several ways. The standard reference density test methods that are used for this soil are ASTM D698 or D1557 or D558 for soil cement. The standard in-place field density tests that apply are D1556 (sand cone), D2167 (rubber balloon), D6938 (nuclear gauge), D4914 (sand replacement in a test pit), and D5030 (water replacement in a test pit). The applicable water content test procedures for this soil are D2216 (oven dry), D4643 (microwave oven), D4959 (direct heating), D4944 (calcium carbide tester), and D6938 (nuclear method).

Procedures

The procedures for checking an earth fill for compliance with density and water content requirements for soils with more than 5 % oversize content are as follows:

1. Determine the maximum dry density and optimum water content using Procedures A, B, or C of ASTM D698 or D1557. The procedure used is based on the percentage of oversize particles and the method used is selected as described in the applicable ASTM Standard Test Method, ASTM D698 or D1557. The percentage of oversize particles should be determined and recorded as the soil is prepared for the compaction test. When the amount of material retained on the sieve specified for the procedure being used is more than 5 %, a correction will be necessary. The oversize material is not used in performing the standard reference density test. The maximum dry density and the optimum water content are determined for that portion of the soil that passes the specified sieve size required for the appropriate procedure (A, B, or C). This portion of the soil becomes the control fraction for any future evaluations.
2. An in-place field density test is made at the location selected to represent the compacted fill or in localized areas that are suspected to have substandard compaction, or both. The specifications should indicate which of the four in-place field test procedures indicated above are allowed. Follow the test procedures as outlined in the appropriate standard and as discussed in Chapter 4 of this manual. During the performance of the in-place field test, the method for making oversize corrections must be selected. These are as follows:
   - Method A—The volume and mass of the oversize material can be determined and subtracted from the volume of the hole and the mass of the material taken from the hole. This method only applies to the sand cone, rubber balloon, drive cylinder, and the test pit methods. The mass and volume of the oversize is determined by screening the oversize out of the total sample using the sieve size corresponding to the appropriate method (A, B, or C) of the compaction test and measuring it on balance scales weighed in air and in water (in water by using a net hanging from the balance scales with the oversize completely submerged). The volume of the oversize can be calculated using the difference in mass in and out of water and dividing by the density of water.

   The procedure is as follows:

   1. Determine the volume of the hole (using sand cone, rubber balloon, drive cylinder, or test pit methods) = V_T.
   2. Determine the mass of the wet material removed from the hole = M_1.
   3. Screen the material from the hole through the appropriate sieve for the method used in the standard reference density test.
   4. Determine the water content of the material passing the screen (control fraction) using a representative specimen of this material and the specified or chosen method (e.g., microwave-oven method) = w_1.
   5. Wash the oversize using a minimum amount of water and blot with a towel or cloth to a surface-dry condition and determine its wet mass = M_2.

6. Determine the volume and mass of the oversize using procedures in ASTM C 127 for determining the bulk specific gravity, except the 24-h soaking period is eliminated. This is accomplished by determining the mass of the oversize in water (suspended beneath the balance scale in a submerged net) = M_3. The difference between the two masses ($M_2 - M_3$) divided by the density of water is the volume of the oversize = V_{os}.
7. Determine the volume of the control fraction, which is the volume of the hole minus the volume of the oversize = $V_c = V_T - V_{os}$.
8. Determine the wet mass of the control fraction (M_c) by subtracting the mass of the oversize from the mass of the wet material removed from the hole. $M_c = (M_1 - M_2)$.
9. Calculate the wet density of the control fraction (γ_{wet}) by assuming 1 lbm = 1 lbf and dividing the mass of the control fraction by the volume of the control fraction. $\gamma_{wet} = M_c/V_c$.
10. Calculate the dry density of the control fraction (γ_d) by dividing the wet density by 1 plus the decimal equivalent of the water content of the control fraction. Percent compaction = $(\gamma_d/\gamma_{d\max})$ 100.
    - Method B–The correction can be made for the oversize by using the procedure of Standard Practice D4718, "Correction of Density and Water Content for Soils Containing Oversize Particles."

This method can be used with any of the in-place density test methods including the nuclear gauge method. If this method is used, the total mass and percentage of oversize material of the in-place test must be determined using the same sieve size corresponding to the method (A, B, or C) used for the standard reference density test. The procedures in standard practice D4718 can be used to correct dry density and water content determined in the in-place test for the total material to the corresponding dry density and water content with the oversize removed. The procedures of standard practice D4718 can also be used to correct the maximum dry density and optimum water content determined in the standard reference density test with the oversize removed to the corresponding maximum dry density and optimum water content for the total soil (with the oversize included).

For example, an alternative method is to correct the standard laboratory reference compaction standard to a value corresponding to the soil without oversize removed. A new line of maximum density and optimum water content can be developed by calculating values of density and water content at various percentages of oversize material, say 10 %, 20 %, and 30% (using equations in D4718). The field test values for percent oversize are entered into this new curve to obtain the adjusted values for maximum density and optimum moisture. The percent compaction and the deviation from optimum water content can be determined using the corrected values

When using the D4718 procedure, the soil sample that is prepared for the standard reference density compaction test (D698, D1557) or the sample representing a field in-place test (or from under the nuclear gauge) is screened to remove the oversize fraction. After the oversize fraction has been removed, the mass of the moist fine fraction of the

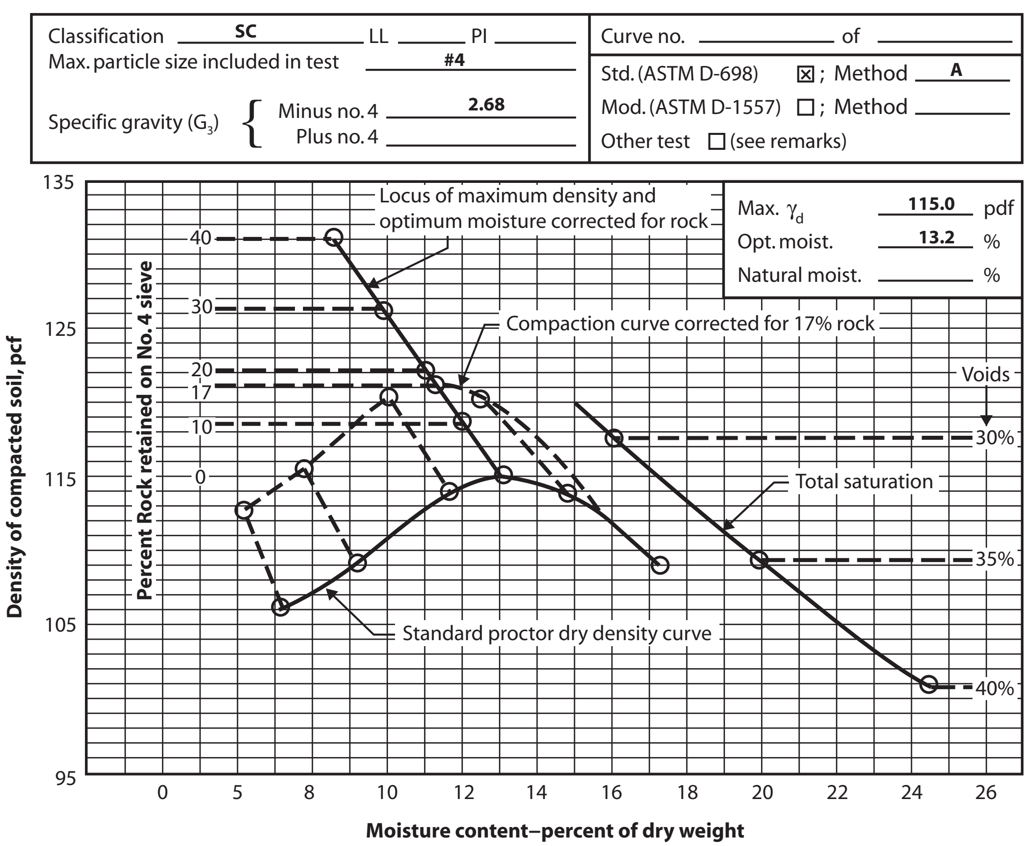


Fig. 1—Rock correction using ASTM D4718 procedures.

sample and the mass of the moist oversize fraction (plus No. 4, plus 3/8-in., or plus 3/4-in.) is determined. The water content of each fraction is then determined. The practice standard indicates that D2216 (oven-dry method) should be used to determine the water content; however, if faster results are desired, other methods could be used to determine the water content of the fine (control) fraction and the water content of the oversize fraction estimated between 1 % and 3 %. The water content of the oversize fraction usually does not vary much and can be estimated based on previous oven-dry tests in the same material. The dry mass of the control fraction and the oversize fraction are calculated by dividing the wet mass by 1 plus the water content (in decimal form). With this information, the percentage of the oversize can be calculated according to procedures and equations presented in practice standard D4718. The corrections of the in-place test results are then made by developing values corresponding to the oversize removed.

The nuclear method measures the density and water content of the total soil under the gauge, which includes the oversize particles. When an oversize correction is needed, Standard Practice D4718 must be used.

3. For determining the water content of the in-place test, a representative specimen is usually selected from the soil control fraction, but all of the in-place control fraction could be used. The entire oversize fraction is typically used for the water content test on this material. Test Method D6938 contains the procedures to be used for making water content tests when using the nuclear gauge. The project specifications should indicate one of the five water-content test procedures indicated at the beginning of this section. Test Method D4944 (calcium carbide tester) is not recommended for testing the water content of soils with coarse fragments because the specimen is too small to be representative. Chapter 4 of this manual provides further guidance for water-content testing along with each of the standard in-place field test methods.

C. SILTY OR CLAYEY SAND OR GRAVEL MIXTURES, OR BOTH WITH MORE THAN 12 % FINES AND MORE THAN 30 % RETAINED ON THE 3/4-IN. SIEVE

If a soil contains more than 30 % larger than the 3/4-in. sieve, standard reference density tests do not apply. Specifications for compacting fills constructed with these soils may require a method control for compaction. This approach specifies a certain number of passes of a prescribed roller, a definite lift thickness, and a given water content. The number of passes is usually determined by constructing a test fill in which the roller is passed over the required lift until more passes result in little or no additional densification of the fill. Measuring the elevation of the lift surface after each roller pass or measuring the in-place density of the fill during the compaction process are methods used to evaluate the desired number of passes and the overall methodology to be used. The in-place dry density of the soil in the test fill can be determined using appropriate field in-place density tests. The dry density in the actual fill can be checked periodically using these same test methods if desired.

Test fills may be used for any earth fill to demonstrate that the desired characteristics of the fill can be obtained and which methods will best achieve these characteristics. Test fills are particularly useful for evaluating the methods for constructing fills from materials having a large percentage of oversize. The test fill can be used to develop the techniques that will provide the most efficient method for placing and compacting the fill.

The specifications should spell out the material properties and the method of compaction to be used and indicate whether field in-place density tests are to be made as part of the field quality control. Most material containing a large percentage of oversize is placed in a lift thickness, which can accommodate the largest particle size. The material may be dumped from trucks or scrapers, leveled with dozers, and compacted with the appropriate equipment. When in-place field density testing will not be used to control the density of the fill, a method compaction control will be used. In a method compaction, the following items need to be specified: (1) the type and size of roller, (2) the number of passes for each lift, and (3) the thickness of each lift. (see Table 3 from Chapter 2).

D. RELATIVELY CLEAN SAND OR GRAVEL MIXTURES, OR BOTH WITH LESS THAN 12 % FINES (MATERIAL PASSING THE NO. 200 SIEVE) AND UP TO 30 % RETAINED ON THE 3/4-IN. SIEVE

Clean sands and gravels are used for drain fill, filter-drainage zones, or shell zones of embankment dams or in many locations of road fills and other fills, or combinations thereof. Compaction of these materials usually requires vibrating rollers or vibrating hand compactors. The laboratory test methods used as the reference standard for this soil type have historically been D4253 (Maximum Index Density) and D4254 (Minimum Index Density and Calculation of Relative Density). The measured dry density of the compacted fill is compared to the values determined by these methods to compute the relative density of the fill by standard equations (see Chapter 2, equation 2). Recently, a more popular method is to compare the in-place dry density of the fill to a maximum dry density as is done with other soils.

The maximum dry density can be determined by using Test Method D4253 or Test Methods D698 or D1557. Test Method D4253 includes procedures for including particles up to 3 in. in diameter and may be more useful for soils with appreciable gravel content. Test Methods D698 and D1557 are limited to 3/4-in. particle size. Test Method D4253 requires a vibrating table that is expensive and may be difficult to maintain at a field location.

The standard in-place field tests that apply are D1556 (sand cone), D2167 (rubber balloon), D6938 (nuclear gauge), D4914 (sand replacement in a test pit), and D5030 (water replacement in a test pit). The applicable water content tests for this soil are D2216 (oven dry), D4643 (microwave oven), D4959 (direct heating), D6938 (nuclear method), and D4944 (calcium carbide tester–for sand only). The sand cone and rubber balloon methods are generally limited to sands that do not contain a significant content of large gravels, and that will maintain an excavated hole for running the test. The calcium carbide tester and microwave oven are also limited to soils without appreciable amounts of gravel. The nuclear method may have problems where a relatively smooth surface cannot be obtained and where large rock particles under the gauge may prevent making a suitable hole for the insertion of the probe. The best in-place tests for soils with larger rock fragments are the test pit tests D4914 and D5030, where large specimens can be taken and sand or water replacement is used to determine the hole volume.

The procedures for checking an earth fill for compliance with density and water-content requirements are the same as listed for Chapter 6, Section B.

E. QUALITY-CONTROL CHECKS

For most test methods, checks can be made after the final calculations have been completed to determine if the results are reasonable. Important checks are as follows:

1. First, be sure the standard reference density test used to compute degree of compaction is for the same material that the field measurement of dry density and water content was made (see Section F of this chapter).
2. The results of the in-place test should be plotted on a standard plot with dry density as the *y*-axis plotted against water content as the *x*-axis. The plot should include a zero-air voids (100% saturation) plot. If the moisture content and dry density value of the field measurement plots to the right of the zero-air voids line, there is an error. The error may be in the measurement of the density or water content of the field test or in the value of specific gravity used to determine the zero-air voids line. If the specific gravity values and the resulting zero-air voids line are suspect, the soil should be retested to determine an accurate value.
3. If the computed degree of compaction is more than approximately 110 % or less than approximately 85 %, for standard reference density effort D698, an error should be suspected.
4. If the computed degree of compaction is more than approximately 105 % or less than approximately 75 % for modified reference density effort D1557, an error should be suspected.

The results can also be checked against Table 3A (see Appendix A), which gives a range of average values for various materials. Some deviation from the range of average values can be expected in some cases; however, differences should not be excessive and should be explainable.

F. SELECTION OF THE STANDARD REFERENCE DENSITY

The two most important aspects of quality-control testing for soil compaction are (1) accurately measuring the density and water content of the compacted fill in place and (2) selecting the appropriate value of the standard reference density for the soil. If borrow materials are quite variable and are being mixed together during the normal processing of the fill materials, accurately determining an appropriate value for the standard reference density test is difficult. The most accurate method is to actually perform a standard reference density test on the soils at the same location where the field in-place test was made. This solution is not very practical and does not provide a timely response to a production operation.

Two additional methods (other than performing a full standard reference density test) are recommended in this manual for selecting the standard reference density that is used in comparing field in-place test results to determine percent compaction. The methods are the Standard Test Method for Rapid Determination of Percent Compaction (D5080) and the Family of Curves and One-Point Compaction Test Method. The One-Point Proctor Test is not an ASTM Method but can be found in American Association of State Highway and Transportation Officials (AASHTO) T 272, Standard Test Method for Family of Curves–One-Point Method. The standard reference density to be used for computing the degree of compaction is often a haphazard selection, usually based largely on the judgment of the person making the comparison. This judgment is based primarily on a visual comparison of previously tested samples to the soil in question. Without a standardized approach, disagreements and uncertainty over the conclusions from the testing are common.

1. Rapid Determination of Percent Compaction (Test Method D5080)

The Standard Test Method for Rapid Determination of Percent Compaction (D5080) is an accurate method for establishing a value for standard reference density to use in computing the degree of compaction of the soil on which field density and water contents were measured. If the borrow materials are variable on the project, or if there is any question about which standard reference density values should be used, the rapid method provides a practical means to obtain accurate values for reference density and water content.

The rapid method is frequently used for larger earthwork projects, including earth dams, landfill soil liners, and airfields. This method is used infrequently on smaller projects because it requires an on-site quality-control laboratory and technicians who are experienced with performing the rapid test method. The rapid method gets its name because the water content of the in-place test is not required to determine the percent compaction and the deviation from optimum water content. The test requires less effort and can be performed more quickly than a full compaction test made on soil from the same location in the fill, but it provides similar accuracy results. One of the assumptions that simplify this procedure is that the shape of the compaction curve is a true parabola. Because all compaction curves are parabolic in shape, this is a valid assumption.

The test is performed by determining the in-place wet density of the fill material using any one of the regular in-place tests as discussed in Chapter 4 of this manual. The soil from the in-place test is used to make three reference density tests. If the in-place field test is not large enough to provide enough material for making three test specimens, additional soil must be obtained immediately around the location of the in-place field test. When the nuclear method (D6938) is used with the rapid method, the soil for performing the compaction test part should be obtained from the area on the fill immediately under the gauge at the test location. The compaction test procedure (mold size, rammer size, number of drops, and number of layers) is in accordance with the test method and procedure specified for the project. Any standard reference density test procedure can be used.

The standard test method procedure (D5080) includes all of the steps, spelled out in detail. See Figs. 2 and 3 for graphs used in the application of this method. This method can be used on any soil in which the standard reference density test methods apply and is particularly appropriate for earthfill projects in which fairly rapid results are needed because it eliminates any question about what value for standard reference density is valid. Corrections for oversize rock can be made as previously discussed in Section B of

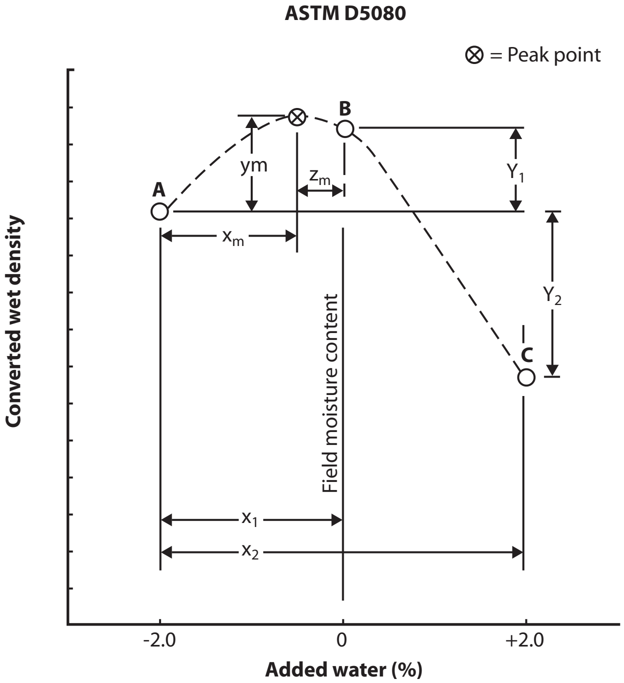


Fig. 2—Determining peak point of compaction curve.

this chapter. The procedure for making these corrections is also provided in the published test method

With this method, it is possible to determine the percent compaction and the deviation from optimum water content without determining the actual water content of any samples. The test procedure suggests that the actual water content be made for the record, but the fill compaction is usually accepted or rejected based on the initial same-day results. An accurate maximum density and optimum water content may be obtained by drying a field sample by Test Method D2216. The most significant thing about the procedure is that the water content and maximum dry density values obtained are for the exact same soil as the in-place field density test.

ADVANTAGES/DISADVANTAGES

Because this method uses the in-place field test procedures and the standard reference density procedures as specified, it has the same advantages and disadvantages as explained for these tests in Chapters 3 and 4. The advantages unique to this method have been explained.

The disadvantages unique to this method are that the test is a little more work and takes more time to complete than a one-point test. The rapid method typically requires an on-site field quality-control laboratory and qualified technicians to provide proper test results in a timely manner. Gaining

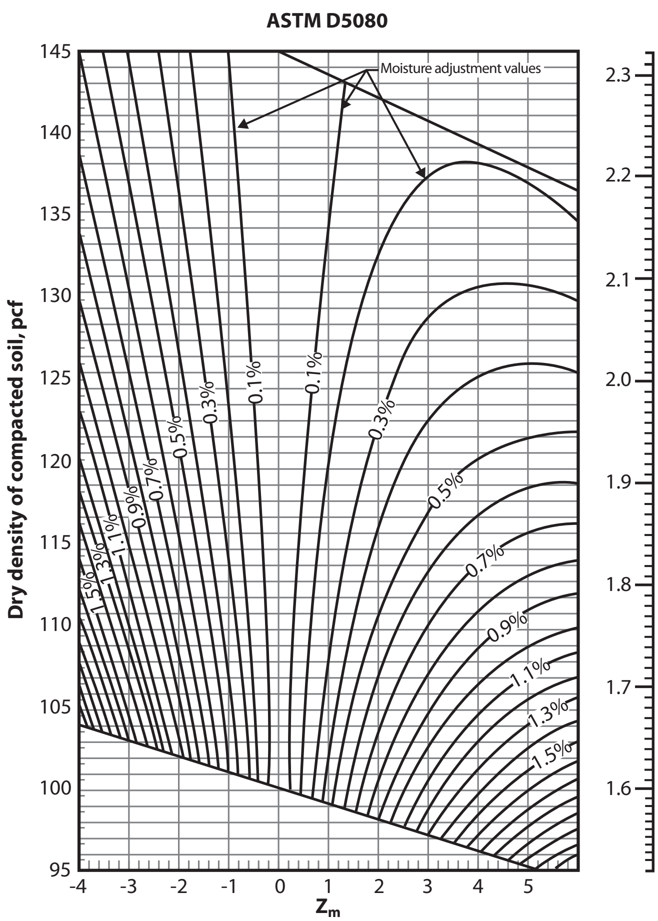


Fig. 3—Moisture adjustment values.

proficiency in performing the test requires considerable experience and training of technicians. The computations involving the parabolic adjustment of the data are complicated and computer programs are helpful in that regard.

PRECAUTIONS OR POTENTIAL SOURCES OF ERROR

The precautions of the standard test procedures used in conjunction with this method also apply to this procedure. Things such as properly calibrated molds and hammer weights and drop heights are important. The main precaution unique to this method is that when the fill water content is at or above the optimum water content, at least one specimen will have to be dried to a lower water content. If heat is added to dry the soil, care must be exercised to dry only halfway and then let the soil cool while it is drying the remainder. The soil must be at room temperature or the temperature of the mold and rammer when the compaction is accomplished. If the soil is at a temperature higher than the surrounding air and the equipment, moisture will escape from the hot soil while the test is being performed and condensation will collect on the equipment. The soil should be remixed after drying and the soil broken down to make sure the clods are not just dry on the surface and still wet on the inside.

2. The Family of Curves and One-Point Compaction Test Method for Determining Percent Compaction

The Family of Curves and One-Point Compaction Test Method for Determining Percent Compaction is not an ASTM Method but can be found in AASHTO T 272 Standard Method of Test for Family of Curves–One-Point Method. Many state highway departments have developed this method for their use in quality control and selection of a standard reference density to be used for computing degree of compaction and water-content deviations. (See Figs. 3A and 4A in Appendix A.) This method utilizes typical relationships between optimum water content and maximum dry density for soils from a specific borrow source being used for a compacted earth fill. The relationship exists most often when residual soil materials have been formed in place from the weathering of parent rock materials. Alluvial materials in flood plains can also exhibit these same relationships if they are derived from materials that have a common geological origin. When a valid family of curves exists, a smooth curve can be drawn through the values of maximum dry density and optimum water content of a series of compaction curves developed from soil materials of varying weight, particle size, and plasticity within the borrow area. Developing a series of these curves from field data and interpolation between the known points produces the family of curves for that specific borrow source.

When a family of curves has been developed, it can be used in conjunction with a one-point compaction value from the appropriate method in D698 or D1557 to determine the optimum water content and maximum dry density for the soil used in the one-point compaction test. The soil for the one-point compaction test is taken from alongside the in-place field test on the fill. The in-place field test may be made with any of the standard test methods appropriate for the material being tested.

Procedures for Developing a Family of Curves

The compaction test curve data used to develop a family of curves should be from tests made on soil materials taken from a specific construction site. They may be supplemented with tests made on soils of similar geologic origin from adjacent areas.

The first step is to plot the optimum water content and maximum dry density values from the available compaction test curves on a regular compaction test report form with the scale values set to accommodate the full range of values. A best-fit curve is then drawn through these points, which will represent the true maximum points for all of the materials tested. This smooth curve forms a reference line and is the baseline from which the family of curves is developed. Soils represented by plotted values that fall more than 2 water content percentage points from the reference line should not be included in the family of curves and should be identified for special testing and the use of other methods. See Fig. 4 for an example of the initial plot of a family of curves.

All compaction test curves to be used in the family of curves are then plotted to scale along the reference line. The plotting is accomplished by shifting the water-content values to the right or left as necessary to locate the maximum point on the reference line. The curves plotted in this shifted position on the reference line will serve to guide the shapes of the entire family of curves. If large gaps exist between plotted curves along the reference line, then additional standard reference density test curves may be needed to fill in the gaps and provide the appropriate shape of the curve in all ranges. See Fig. 4 for a final and corrected plot of the family of curves.

It is suggested, when possible, that the family of curves be developed with one curve at each 5-bl maximum density interval beginning at selected points on the reference line. Each new curve of the family of curves is then drawn as guided by the standard reference density test curves previously plotted. Care should be taken to evenly space each curve in relation to adjacent curves of the family to minimize overlapping. The family of curves consists of these newly drawn curves and the reference line. It can be lifted from the worksheet used in developing it to a new sheet of cross-section paper for field use.

Making the One-Point Compaction Test Specimen

The most significant precaution in using the family of curves has to do with the water content of the one-point compaction test (D698 or D1557). This one-point test must be run at a water content at least 1–2 % dry of the true optimum water content of the soil material.

The curves of the family of curves begin to overlap on the wet side of the optimum moisture content. Therefore, if your one-point sample is not dry enough, it may be impossible to determine what existing curve is the correct one to match in shape with a new curve.

Plotting the One-Point and Drawing the New Curve

The one-point compaction test identifies one point on a compaction curve. When the one point is plotted on the family of curves, it will identify the location of a new curve representing the material of the in-place test. The new curve is then drawn through this point and shaped to conform to the adjacent curves of the family.

Locating and Recording the Optimum Water Content and Maximum Dry Density

The optimum water content and maximum dry density values for the specific soil sample are determined by noting these values defined by the peak of the new curve that

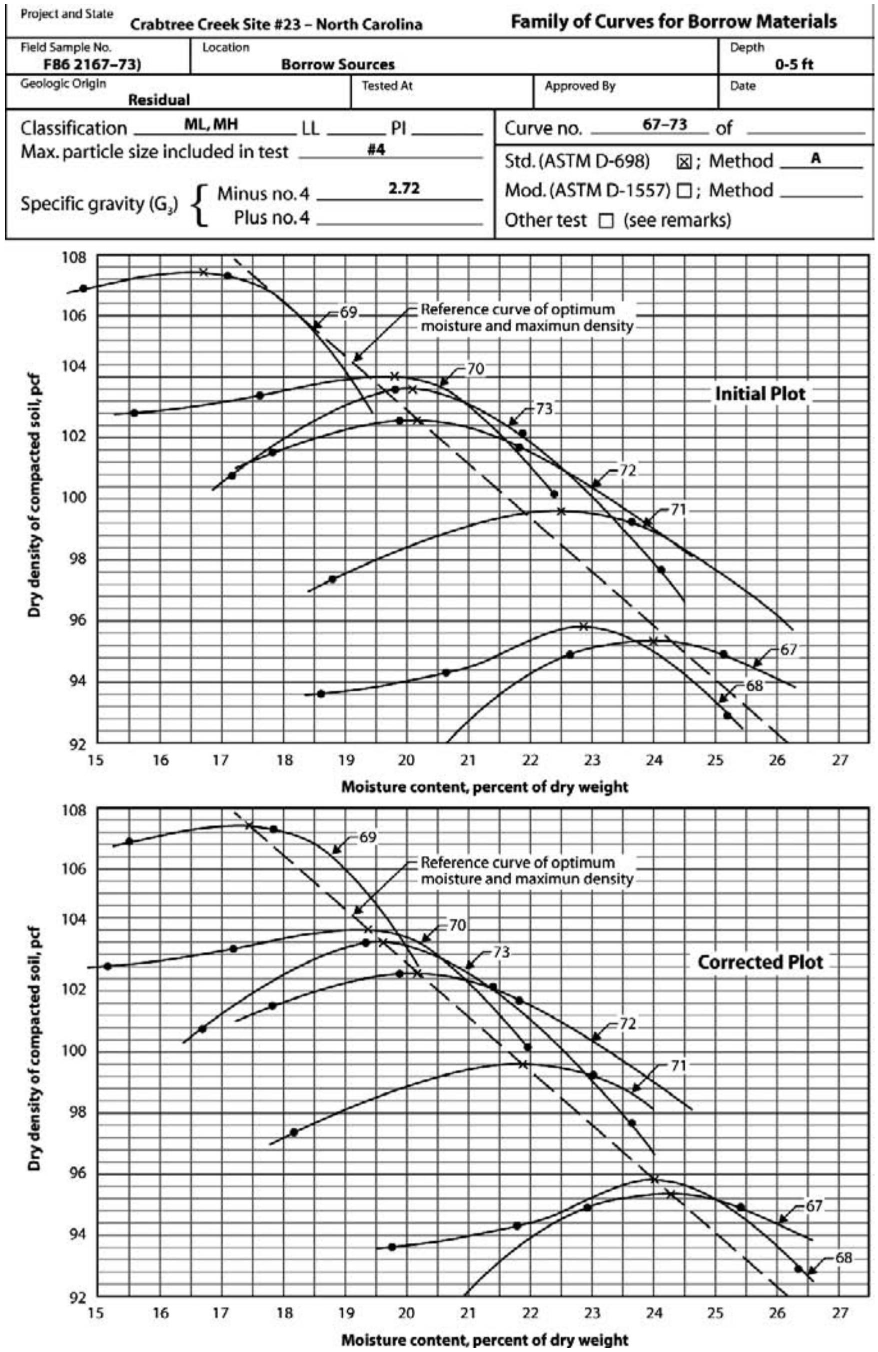


Fig. 4—Family of curves for borrow materials.

correspond with the intersection of the curve and the reference line on the family of curves. They can be recorded for reference on the in-place test report.

G. SUMMARY

The greatest source of error in the evaluation and control of earthfill compaction is usually in selecting the proper standard reference density test to compare with the in-place field density test. Poulos (1988) [2] has shown that basing this comparison on soils that appear visually to have the same gradation leads to significant error. The Rapid Determination of Percent Compaction (Test Method D5080) prevents this error because the standard reference density is determined on the same soil as the in-place field density test. The Family of Curves–One-Point Method (AASHTO T 272) helps prevent this error by identifying the proper curve or establishing values for a new curve.

Some checks can be made after the results have been calculated to determine if the results are reasonable. The results of the in-place field density tests should be plotted on the appropriate standard reference density curve. If the degree of saturation is greater than 95 %, there is likely an error. Compacting soils to the point where they are more than 95 % saturated is highly unlikely because water is incompressible. In no case should any dry density value plot to the right of the zero-air voids (100 % saturation) curve. These guidelines are dependent on having accurate specific gravity values from which to plot the zero-air voids curve.

While comparing the in-place field density test against the standard reference density test (D698), there may be an error if the dry density in-place is more than approximately 108 % or less than approximately 75 % of the maximum dry density. While checking against the modified reference density test (D1557), there may be an error if the dry density in place is more than approximately 104 % or less than approximately 65 % of the maximum dry density. The results can also be checked against Table 3A (see Appendix A), which gives a range of average values for various materials. Some deviation from the range of average values can be expected in some cases; however, differences should not be excessive and should be explainable.

References

[1] *Earth Manual*, 3rd Ed., Part II, U.S. Department of the Interior, Bureau of Reclamation, U.S. Government Printing Office, Washington, DC, 1990.

[2] Poulos, S.J., "A Compaction Control and the Index Density," Geotech. Test. J., Vol. 11, 1988, pp. 100–108.

[3] Standard Test Method for Family of Curves–One-Point Method, AASHTO Designation: T 272.

[4] *Plotted Ohio Typical Density Curves, Manual of Procedures for Earthwork Construction*, Vol. II, Ohio Department of Transportation, Columbus, OH, 1998, Chapter 4.

[5] *Family of Curves and the One-Point Proctor Procedures, Certified Technician Program Training Manual for Construction Earthworks*, Indiana Department of Transportation, Indianapolis, IN, 2003, Chapter 15.

Appendix A

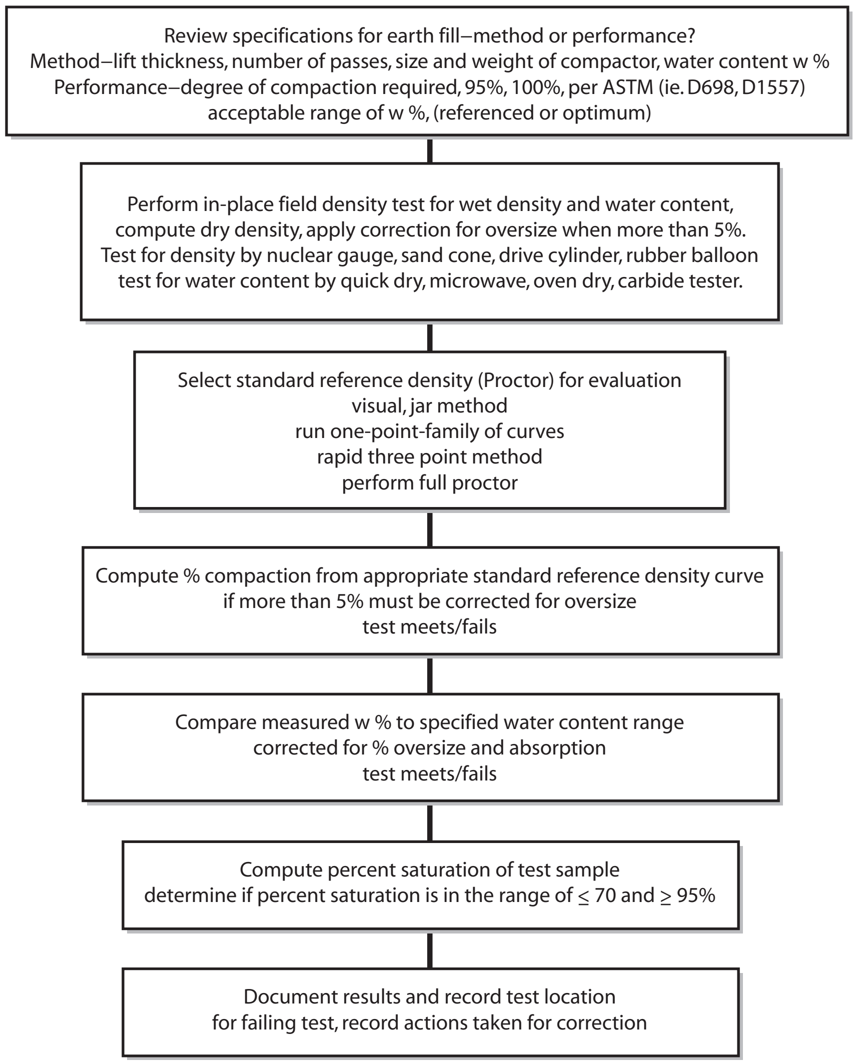


Fig. 1A—Flow chart for field compaction quality control of earth fill

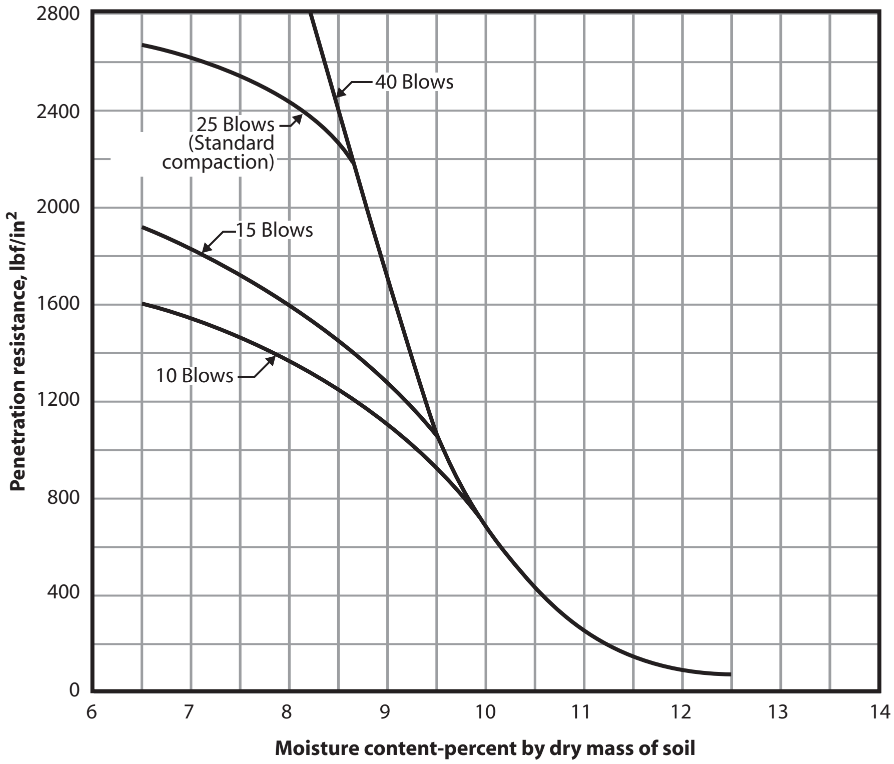


Fig. 2A—Effect of compactive effort on penetration—resistance curves. Earth Manual, USBR.

Ohio DOT—Family of Curves

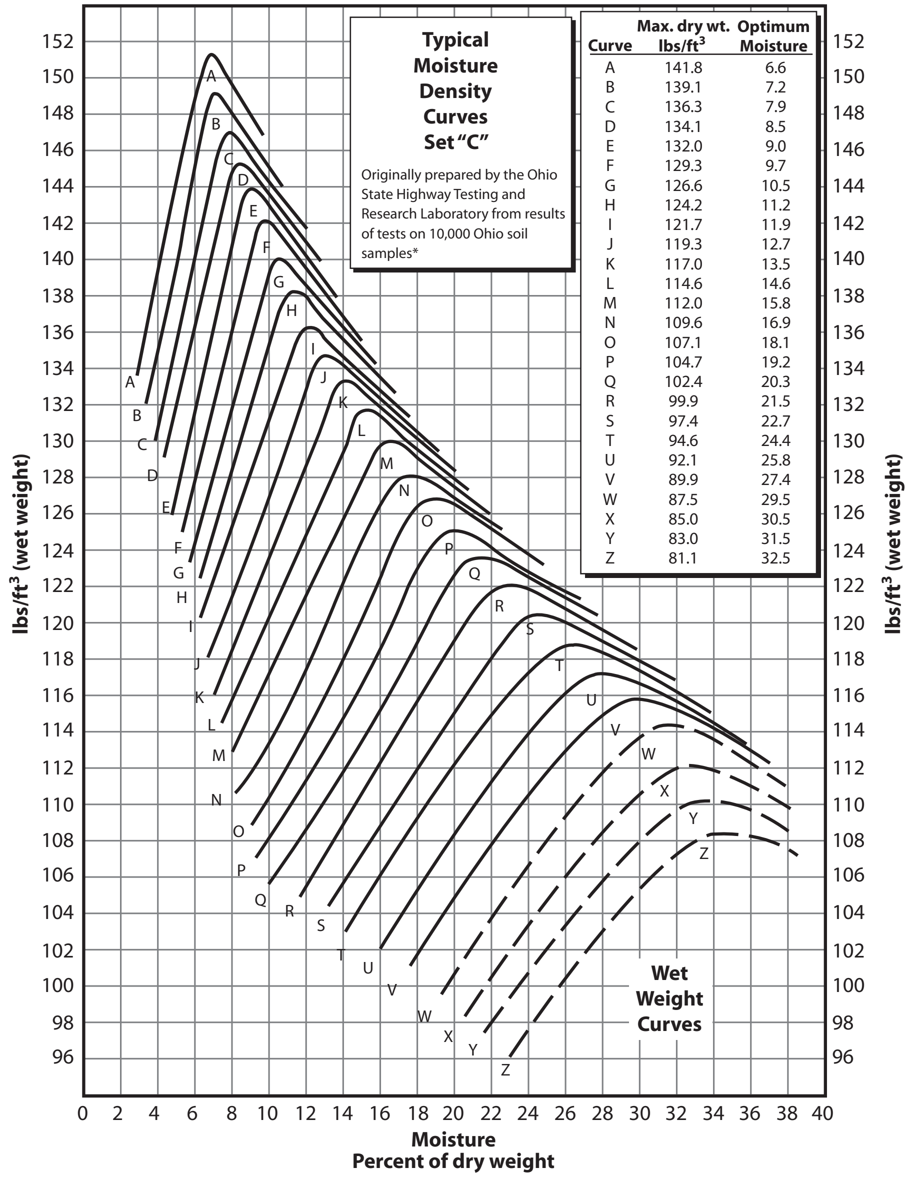
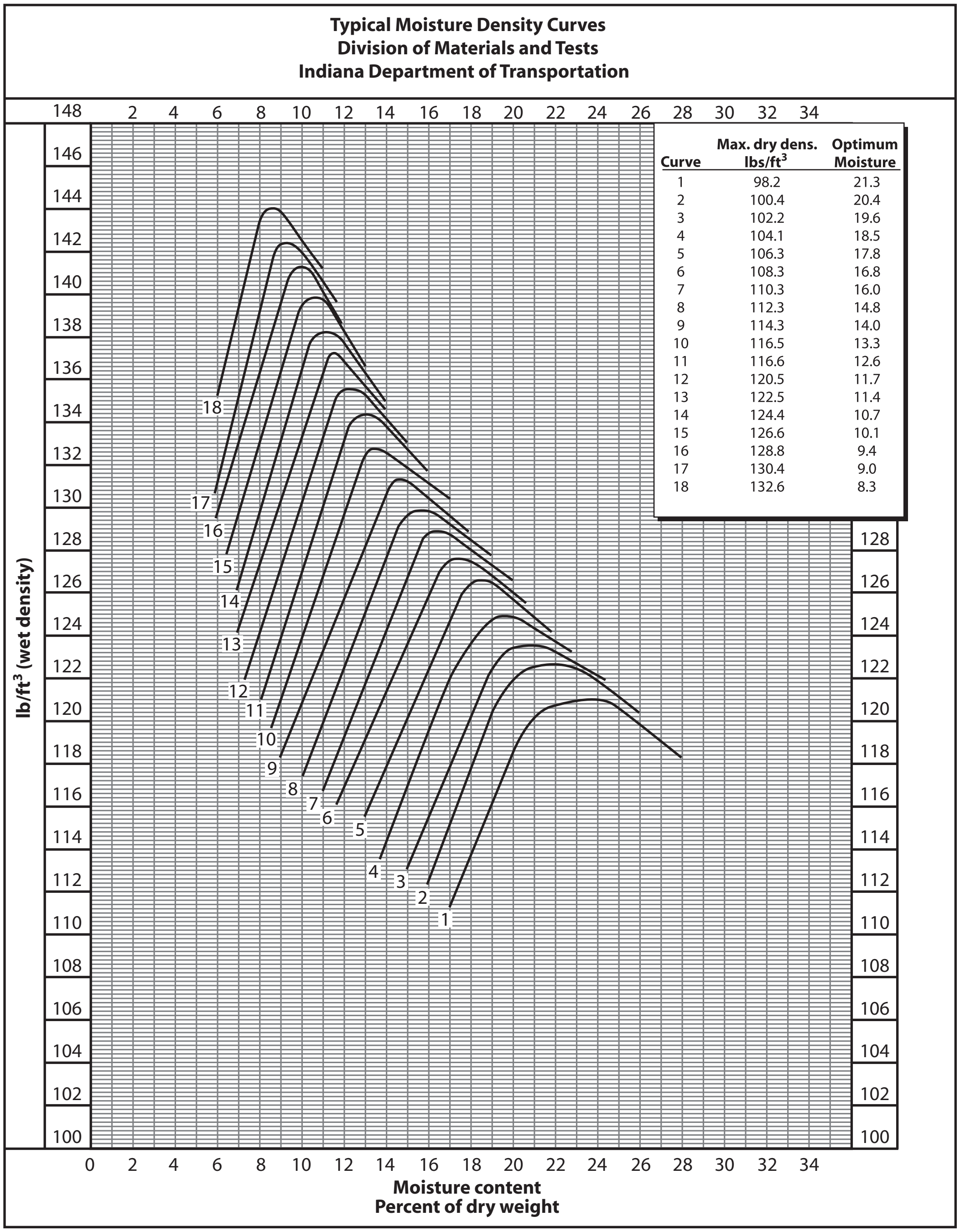


Fig. 3A—Ohio DOT family of curves.



Curve	Max. dry dens. lbs/ft³	Optimum Moisture
1	98.2	21.3
2	100.4	20.4
3	102.2	19.6
4	104.1	18.5
5	106.3	17.8
6	108.3	16.8
7	110.3	16.0
8	112.3	14.8
9	114.3	14.0
10	116.5	13.3
11	116.6	12.6
12	120.5	11.7
13	122.5	11.4
14	124.4	10.7
15	126.6	10.1
16	128.8	9.4
17	130.4	9.0
18	132.6	8.3

Fig. 4A—Indiana DOT family of curves.

TABLE IA—Typical properties of compacted soils

Group symbol	Soil type	Range of maximum dry unit weight pcf	Range of optimum moisture percent	Typical value of compression		Typical strength characteristics				Typical coef-ficient of permeability f t/min	Range of CBR values	Range of subgrade module k lb/in^3
				At 1.4	At 3.6 tsf tsr (20 psi) (50 psi) Percent of original height	Cohesion (compacted) psf	Cohesion (saturated) psf	(Effective stress enve-lope degrees)	Twoϕ			
GW	Well graded clean grav-els, gravel-sand mixtures.	125–135	11–8	0.3	0.6	0	0	>38	>0.79	5×10^{2}	40–80	300–500
GP	Poorly graded clean grav-els, gravel-sand mix.	115–125	14–11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30–60	250–400
GM	Silty gravels, poorly graded gravel-sand-clay.	120–135	12–8	0.5	1.1	. . ...	. . ...	>34	>0.67	$>10^{-6}$	20–60	100–400
GC	Clayey grav-els, poorly graded gravel-sand-clay.	115–130	14–9	0.7	1.6	. . ...	. . ...	>31	>0.60	$>10^{-7}$	20–40	100–300
SW	Well-graded clean sands, sand-gravel mix.	110–130	16–9	0.6	1.2	0	0	38	0.19	$>10^{-3}$	20–40	200–300
SP	Poorly graded clean sands, sand-gravel mix.	100–120	21–12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10–40	200–300
SM	Silty sands, poorly graded sand-silt mix.	110–125	16–11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10–40	100–300

TABLE IA—Typical properties of compacted soils (Continued)

Group symbol	Soil type	Range of maximum dry unit weight pcf	Range of optimum moisture percent	Typical value of compression		Typical strength characteristics				Typical coef-ficient of permeability f t/min	Range of CBR values	Range of subgrade module k lb/in^3
				At 1.4	At 3.6 tsf tsr (20 psi) (50 psi) Percent of original height	Cohesion (compacted) psf	Cohesion (saturated) psf	(Effective stress enve-lope degrees)	Twoϕ			
SM-SC	Sand-silt clay mix with slightly plas-tic fines.	110–130	15–11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5–30	100–300
SC	Clayey sands, poorly graded sand-clay-mix.	105–125	19–11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5–20	100–300
ML	Inorganic silts and clayey silts.	95–120	24–12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100–200
ML-CL	Mixture of inorganic silt and clay.	100–120	22–12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	. . ...	
CL	Inorganic clays of low to medium plasticity.	95–120	24–12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50–200
OL	Organic silts and silt-clays, low plasticity.	80–100	33–21	. . ...	. . ...	. . ...	. . ...	. . ...	. . ...	. . ...	5 or less	50–100
MH	Inorganic clayey silts, elastic silts.	70–95	40–24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-2}$	10 or less	50–100
CK	Inorganic clays of high plasticity.	75–105	36–19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50–150
OH	Organic clays and silty clays.	65–100	43–21		. . ...	. . ...	. . ...	. . ...	. . ...	. . ...	5 or less	25–100

Notes:
1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR, which are for "Modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR date.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown. (. . ...) indicates insufficient data available for an estimate.

TABLE 2A—Compaction equipment and methods

		Requirements for compaction of 95 to 100 percent standard proctor maximum density			
Equipment type	**Applicability**	**Compacted lift thickness, inches**	**Passes or coverages**	**Dimensions and weight of equipment**	**Possible variations in equipment**
Sheepsfoot rollers	For fine-grained soils or dirty coarse-grained soils with more than 20 percent passing No. 200 sieve. Not suitable for clean, coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important.	6	4 to 6 passess for fine-grained soil 6 to 8 passes for coarse-grained soil		For earth dam, highway and airfield work, articulated self propelled rollers are commonly used. For smaller projects, towed 40- to 60-inch drums are used. Foot contact pressure should be regulated so as to avoid shearing the soil on the third or fourth pass.
Rubber tire roller	For clean, coarse-grained soils with 4 to 8 percent passing the No.200 sieve.	10	3 to 5 coverages	Tire inflation pressures of 35 to 130 lb/in^2 for clean granular material or base course and subgrade compaction. Wheel load 18,000 to 25,000 lb.	Wide variety of rubber tire compaction equipment is available. For cohesive soils, light-wheel loads, such as provided by wobble-wheel equipment, may be substituted for heavy-wheel load if lift thickness is decreased. For granular soils, large-size tires are desirable to avoid shear and rutting.
	For fine-grained soils or well-graded, dirty coarse-grained soils with more than 8 percent passing the No.200 sieve.	6 to 8	4 to 6 coverages	Tire inflation pressures in excess of 65 lb/in^2, for fine-grained soils of high plasticity. For uniform clean sands or silty fine sands, use large size tires with pressures of 40 to 50 lb/in^2.	
Smooth wheel roller	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures.	8 to 12	4 coverages	Tandem type rollers for base course or subgrade compaction 10- to 15-ton weight, 300 to 500 lb per lineal inch of width of rear roller.	3-wheel rollers obtainable in wide range of sizes. 2-wheel tandem rollers are available in the range of 1- to 20-ton weight. 3-axle tandem rollers are generally used in the range of 10- to 20-ton weight. Very heavy rollers are used for proof rolling of subgrade or base course.
	May be used for fine-grained soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands.	6 to 8	6 coverages	3-wheel roller compaction of fine-grained soil; weights from 5 to 6 tons for materials of low plasticity to 10 tons for materials of high plasticity.	

TABLE 2A—Compaction equipment and methods (Continued)

		Requirements for compaction of 95 to 100 percent standard proctor maximum density			
Equipment type	**Applicability**	**Compacted lift thickness, inches**	**Passes or coverages**	**Dimensions and weight of equipment**	**Possible variations in equipment**
Vibrating sheetsfoot rollers	For coarse-grained soils, sand-gravel mixtures	8 to 12	3 to 5	1 to 20 tons ballasted weight. Dynamic force up to 20 tons.	May have either fixed or variables cyclic frequency.
Vibrating smooth drum rollers	For coarse-grained soils, sand-gravel mixtures, rock fills.	6 to 12 (soil) to 36 (rock)	3 to 5 4 to 6	1 to 20 tons ballasted weight. Dynamic force up to 20 tons.	May have either fixed or variables cyclic frequency.
Vibrating baseplate compactors	For coarse-grained soils with less than about 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	8 to 10	3 coverages	Single pads or plates should weigh no less than 200 lb. May be used in tandem where working space is available. For clean coarse-grained soil, vibration frequency should be no less than 1,600 cycles per minute.	Vibrating pads or plates are available, hand-propelled, single or in gangs, with width of coverage from 1-1/2 to 15 ft. Various types of vibrating drum equipment should be considered for compaction in large areas.
Crawler tractor	Best suited for coarse-grained soils with less than 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	6 to 10	3 to 4 coverages	Vehicle with "Standard" tracks having contact pressure not less than 10 lb/in^2.	Tractor weight up to 85 tons.
Power tamper or rammer	For difficult access, trench backfill. Suitable for all inorganic soils.	4 to 6 in. for silt or clay, 6 in. for coarse-grained soils	2 coverages	30-lb minimum weight. Considerable range is tolerable, depending on materials and conditions.	Weight up to 250 lb., foot diameter 4 to 10 in.

TABLE 3A—Summary of compaction characteristics of unified soil classes							
Unified class	**Relative ease of compaction**	**Compacted lift thickness (inches)**	**Importance of water content**	**Preferred type of equipment**	**Number of passes**	**Typical dry unit weights (PFC)**	**Typical water content (percent)**
GW	Very easy	10–12	Either dry or saturated	Crawler tractor vibratory roller	3–4	125–135	9–12 [2]
GP	Good to excellent	10–12	Either dry or saturated	Crawler tractor vibratory roller	3–4	115–125	12–16 [2]
GM	Good with close control	6–8	Fairly important	Tamping roller	4–6	120–135	8–13
GC	Good	6	Very important	Tamping roller	4–6	115–130	9–14
SW	Excellent	10–12	Either dry or saturated	Crawler tractor vibratory roller	3–4	110–130	10–18[2]
SP	Fair	10–12	Either dry or saturated	Crawler tractor vibratory roller	3–4	100–120	13–22[2]
SM	Fair	6–8	Important	Rubber-tired or tamping roller	4–6	110–125	10–16
SC	Good	6	Very important	Tamping roller	4–6	105–125	10–18
ML	Fair	6	Important	Rubber tired or tamping roller	4–6	95–120	12–22
CL	Good to fair	6	Very important	Tamping roller	4–6	95–120	12–24
MH	Poor	6	Very important	Tamping roller	4–6	70–95	22–40
CH	Very poor	6	Critical	Tamping roller	4–6	75–105	20–40
OL	Fair	6	Important	Tamping roller	4–6	80–100	20–32
OH	Very poor	6	Important	Tamping roller	4–6	65–100	20–45
Pt	Not suitable	Not suitable for most fills - usually placed with draglines and little compaction					

[1] All conditions are for compaction of 95 of 100 percent standard proctor maximum density at optimum water content, except for soils covered by footnote
[2] Saturated water content

Appendix B

MATERIALS TESTING REPORT:
UNIFIED CLASSIFICATION SYSTEM
ASTM D2487

Project Name: ______________________ Location: ______________________

Contract No. ____________ Contractor: ______________________

Inspector: ______________________ Date: ____________ Time: ____________

Sample location: ______________________ Field sample No.: ____________

Depth: ________ Geologic origin: ____________ Type of sample: ________ Tested at: ____________

Approved by: ________ Date: ________ Symbol: ________ Description: ____________

Identification				Coarse fraction					Fine fraction				Total soil			Classification	
Testing section sample No.	Test hole No.	Field sample No.	Depth (ft)	Maximum size (mm)	Particle shape	Particle condition	Gravel (3 in. to No. 4)	Sand (No. 4 to 200)	Fines (- No. 200)	Plasticity	Dry strength	Dilatance	Organic odor (wet)	Reaction to HCL	Color (wet)	Description (classification, grading, structure, consistancy, moisture condition, inclusions, etc.)	Group symbol

Remarks: ______________________ Signature: ______________________

______________________ Title: ____________ Date: ________

Fig. 1B—Materials testing report: unified classification system (ASTM D2487).

MATERIALS TESTING REPORT:
UNIFIED SOIL CLASSIFICATION SYSTEM VISUAL-MANUAL PROCEDURE
ASTM D2488

Test	Description	Symbol	Identification
	Angular	A	Irregular shape: sharp edges.
	Subangular	SA	Irregular shape; fairly sharp edges.
	Subrounded	SR	Irregular shape; rounded edges.
	Rounded	R	Fairly regular shape; rounded edges.
Particle condition			
	Soft	S	Rubber pestle will break particles.
	Vesicular	V	Individual grains contain air voids.
	Dense	D	Massive: grains contain no air voids.
Plasticity	High	H	Tough thread, will remold before plastic limit.
	Medium	M	Medium tough thread, crumbles below plastic limit.
	Low	L	Weak thread, will not remold at plastic limit.
	None	N	Will not form thread.
Dry strength	High	H	Difficult to break by finger pressure.
	Medium	M	Considerable finger pressure to crumble
	Low	L	Will crumble at light finger pressure.
	None	N	Will not form soil pat.
Dilatance	Rapid	R	Water surfaces immediately.
	Slow	S	Water surfaces slowly.
	None	N	Water will not surface.
HCL	Positive	+	Effervescence
	Negative	–	No reaction
Organic odor	Strong	S	Strong odor when moist and hot.
	Weak	W	Weak Odor when moist and hot.
	None	N	No organic odor.

Group	Organic odor	Visual examination			Character of fines (- No. 40)		
		Grading	Percent fines	Dominant fraction	Dilatance	Dry strength	Plasticity
ML	Weak	Not a criterion for classification	Over 50	Fines	Rapid	None-slow	None-Low
CL	“		“	“	None-slow	Medium-High	Medium
CH	“		“	“	None	High	High
MH	“		“	“	None - slow	Low- medium	Low-medium
OL,OH	Strong		“	“	None	Low- medium	Medium (spongy)
SM	Weak		12 - 50	Sand	Fines classify as ML or MH		
GM	“		“	Gravel			
SC	“		“	Sand	Fines classify as CL or CH		
GC	“		“	Gravel			
SP	“	Poor	Under 5	Sand	Not a criterion for classification		
GP	“	“	“	Gravel			
SW	“	Well	“	Sand			
GW	“			Gravel			
Pt	Strong	Identify by high fibrous organic content					

Fig. 2B—Materials testing report: unified soil classification system visual-manual procedure (ASTM D2488).

Material Testing Report
Reference Density Compaction Curve

Project ________________________________ Laboratory No. __________

Field sample No. __________ Location ________________ Depth __________

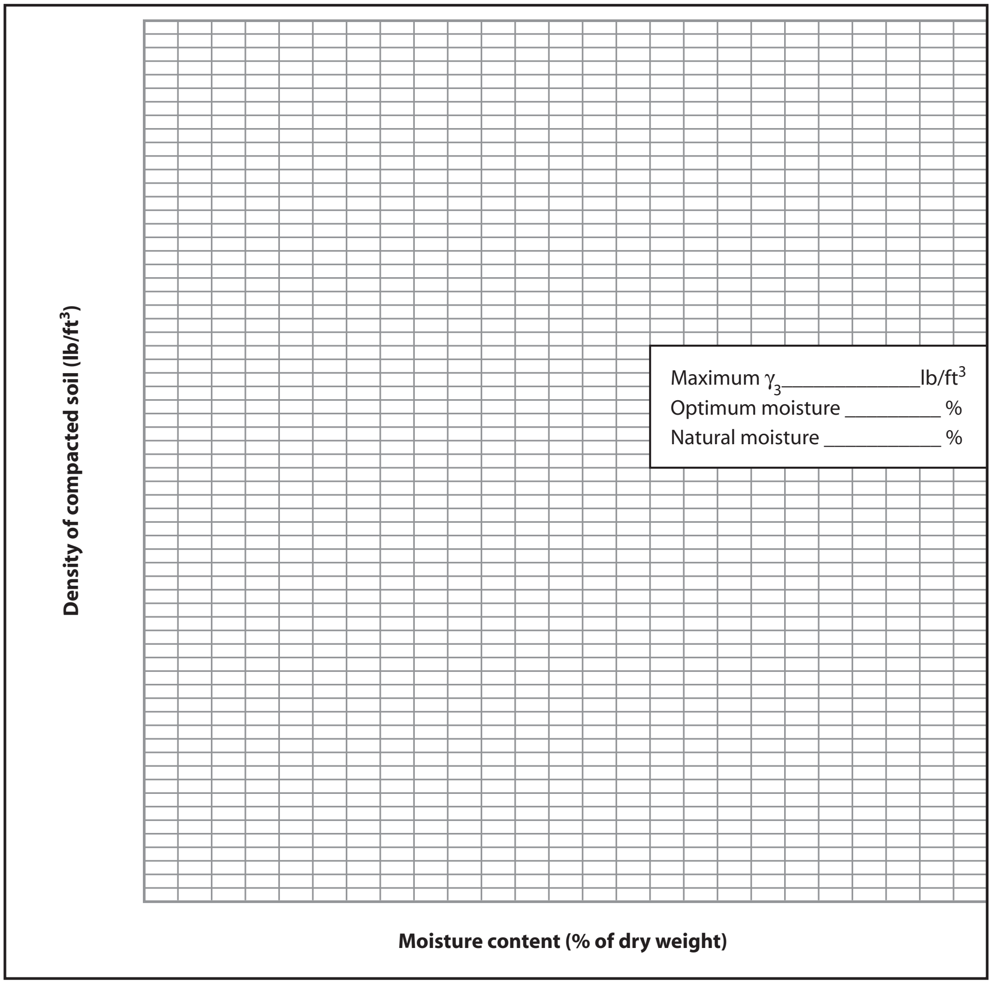
Geologic origin __________ Tested at __________ Approved by __________ Date __________

Classification __________ LL __________ PI __________ Curve No. __________ of __________

Maximum particle size in test __________ Standard (ASTM D-698), method __________

Specific gravity (Gs): -No. 4 __________ Modified (ASTM D-1557), method __________

+No. 4 __________



Remarks ____

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Fig. 3B—Material testing report, reference density compaction curve.

Worksheet for Reference Density Compaction Data

Project ____________________ **Site** ____________ **Sample No.** ______

Compaction Data						
1. Weight of cylinder plus moist soil __________(lb)						
2. Weight of cylinder __________(lb)						
3. Weight of moist soil = [1] - [2] __________(lb)						
4. Wet density = [3] ÷ volume of cylinder __________(lb/ft^3)						
5. Dry density = ([4] × 100) ÷ 100 + [6]) __________(lb/ft^3)						

Moisture Determination Data						
6. Moisture content[1] = ([10] ÷ [12]) × 100__________(%)						
7. Container No. __________						
8. Weight of container plus moist soil __________(g)						
9. Weight of container plus dry soil __________(g)						
10. Weight of moisture = [8] - [9] __________(g)						
11. Weight of container __________(g)						
12. Weight of dry soil = [9] - [11] __________(g)						

Volume of cylinder __________ ft^3 using: ASTM Standard D 698/D 1557 __________, method __________

Procedure data: weight of hammer: __________ lb, drop __________ in., number of lifts __________

Completed by __________ Date __________ Computed by __________ Date __________

Checked by __________ Date __________ Recorded by __________ Date __________

Fig. 4B—Worksheet for reference density compaction data.

BULK SAND DENSITY DETERMINATION AND
CALIBRATION OF CONE AND BASE PLATE FOR ASTM D1556

Project Name: ______________________ Location: ______________________

Contractor: ______________________ Contract No. ____________ Test No. ____________

Material source: ______________________ Tested by: ____________ Date: ____________

Bulk Density of Sand

		Trial 1	Trial 2	Trial 3	Avg.
(1) Volume of Mold, ft^3 (predetermined)					
(2) Initial Weight of Jar + Sand	(lbs)				
(3) Final Weight of Jar + Sand	(lbs)				
(4) Weight of Sand in Cone & Plate	(lbs)				
(5) Weight of Sand in Mold, lbs	(2) – (3) – (4)				
(6) Bulk Density of Sand, lbs/ft^3	(5) / (1)				

	Trial 1	Trial 2	Trial 3
Percent Difference From Average			

% Difference from Avg. = [(Avg. of 3 trials – Trial #___) / Avg. of 3 Trials] x 100

(Trials should not exceed 1% difference from the average.)

Weight of Sand in Cone & Plate

		Trial 1	Trial 2	Trial 3	Avg.
(7) Initial Weight of Jar + Sand	(lbs)				
(8) Final Weight of Jar + Sand	(lbs)				
(9) Weight of Sand in Cone and Plate (8) – (7)	(lbs)				

	Trial 1	Trial 2	Trial 3
Percent Difference From Average			

% Difference from Avg. = [(Avg. of 3 trials – Trial #___) / Avg. of 3 Trials] x 100

(Trials should not exceed 1% difference from the average.)

Fig. 5B—Bulk sand density determination and calibration of cone and base plate for ASTM D1556.

IN-PLACE MOISTURE-DENSITY DETERMINATION:
TEST RECORD FOR SAND CONE METHOD
ASTM D1556

Fined grained soils—less than 5% + oversize[1]

Location: ______________________ Site No. ______________

Watershed: ______________________ Subwatershed: ______________

Contract No. ______________ Contractor: ______________________

Tested by: ______________ Computed by: ______________ Checked by: ______________

Test No.	Date	Location of test			Moisture (%)	Material classification
		Station	Centerline offset	Elevation		

Size of sand cone: ______________________

Test No.	Date	Spec. requirements		Test results	
		Moisture range (%)	Mass dry density (lb/ft^3)	Moisture (%)	Mass dry density (lb/ft^3)

Remarks: ____

____

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[1]Oversize correction required based on method selected in ASTM D698 or D1557.
Indicate weight and volume units used in test.

Fig. 6B—In-place moisture-density determination: test record for sand cone method (ASTM D1556), fine-grained soils–less than 5% + oversize[1].

IN-PLACE MOISTURE-DENSITY DETERMINATION: TEST DATA FOR SAND CONE METHOD ASTM D1556

Fined grained soils—less than 5% + oversize[1]

Volume Determination	Test No. 1	Test No. 2	Test No. 3	Test No. 4
1. Bulk density of sand (predetermined): ______				
2. Initial weight of sand, cone, and container: ______				
3. Final weight of sand, cone, and container: ______				
4. Weight of sand in hole, plate, and cone = [2] – [3]: ______				
5. Weight of sand in plate plus cone (predetermined): ______				
6. Weight of sand in hole = [4] – [5]: ______				
7. Volume of hole = [6] ÷ [1]: ______				

Moisture Determination	Container No. 1	Container No. 2	Container No. 3	Container No. 4
Sample tested using: direct heat ____ oven ___ microwave ___				
8. Weight of moist sample and container: ______				
9. Weight of dry sample and container: ______				
10. Weight of moisture = [10] – [11]: ______				
11. Weight of container: ______				
12. Weight of dry sample = [9] – [11]: ______				
13. Moisture content = ([10] ÷ [12]) 100: ______				
14. Correction for ignition: ______				
15. Corrected moisture content = [13] – [14]: ______				

Density Determination	Container No. 1	Container No. 2	Container No. 3	Container No. 4
16. Weight of moist sample plus container: ______				
17. Weight of container: ______				
18. Weight of moist sample = ______				
19. Wet density = [18] ÷ [7]: ______				
20. Dry density = [18] ÷ [1 + [15]/100]: ______				
21. Required density = ______				
22. Ratio[1] = ([20] ÷ [21]) 100: ______				

[1]Oversize correction required based on method selected in ASTM D698 or D1557.
Indicate weight and volume units used in test.

Fig. 7B—In-place moisture-density determination: test data for sand cone method (ASTM D1556) fine-grained soil–less than 5% + oversize[1].

IN-PLACE MOISTURE-DENSITY DETERMINATION:
TEST RECORDS FOR THE RUBBER BALLOON METHOD
ASTM D2167
Fine-grained soils—less than 5% + no. 4 sieve

Location: ______________________ Site No. ______________

Project Name: ______________________

Contract No. ______________ Contractor: ______________________

Tested by: ____________ Computed by: ____________ Checked by: ____________

Test No.	Date	Location of test			Borrow source, location, and depth	Material classification
		Station	Centerline offset	Elevation		

Test No.	Date	Spec. requirements (%)		Test results (%)		Curve No.	Wet density check	
		Moisture range	Compaction	Moisture	Compaction		1-Point	Curve

Remarks: ______________________

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Fig. 8B—In-place moisture-density determination: test records for the rubber balloon method (ASTM D2167) fine-grained soils–less than 5% + no. 4 sieve.

IN-PLACE MOISTURE-DENSITY DETERMINATION: TEST DATA FOR THE RUBBER BALLOON METHOD ASTM D2167

Fine-grained soils—less than 5% + no. 4 sieve

Volume Determination	Test No. 1	2	3	4
1. Final base reading:________				
2. Initial case reading:________				
3. Volume of hole = [1] – [2]: ________				

Moisture Determination	Container No. 1	2	3	4
Sample tested using: direct heat ___ oven ___ microwave ___				
4. Weight of moist sample and container: ________				
5. Weight of dry sample and container: ________				
6. Weight of moisture = [4] – [5]: ________				
7. Weight of container: ________				
8. Weight of dry sample = [5] – [7]: ________				
9. Moisture content = ([6] ÷ [8]) 100: ________				
10. Correction for ignition: ________				
11. Corrected moisture content = [9] – [10]: ________				

Density Determination	Container No. 1	2	3	4
12. Weight of moist sample plus container: ________				
13. Weight of container: ________				
14. Weight of moist sample = ________				
15. Wet density = [14] ÷[3]				
16. Dry density = [15] ÷ [1 + [11]/100]:________				
17. Required density: ________				
18. Ratio[1] = ([16] ÷[17]) 100: ________				

[1]Oversize correction required based on method selected in ASTM D698 or D1557.
Indicate weight and volume units used in test.

Fig. 9B—In-place moisture-density determination: test data for the rubber balloon method (ASTM D2167), fine-grained soils–less than 5% + no. 4 sieve.

IN-PLACE MOISTURE-DENSITY DETERMINATION:
CALIBRATED CYLINDER METHOD TEST RECORD
ASTM D2937
Fine-grained soils—less than 5% + no. 4 sieve

Location: ______________________________ Site No. ______________

Project Name: ____

Contract No. ______________ Contractor: ______________________

Tested by: ____________ Computed by: ____________ Checked by: ____________

Test No.	Date	Location of test			Borrow source, location, and depth	Material classification
		Station	Centerline offset	Elevation		

Test No.	Date	Spec. requirements (%)		Test results (%)		Curve No.	Wet density check	
		Moisture range	Compaction	Moisture	Compaction		1-Point	Curve

Remarks: ____

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Fig. 10B—In-place moisture-density determination: calibrated cylinder method test record (ASTM D2937) fine-grained soils–less than 5% + no. 4 sieve.

IN-PLACE MOISTURE-DENSITY DETERMINATION: CALIBRATED CYLINDER METHOD TEST DATA ASTM D2937

Fine-grained soils—less than 5% + no. 4 sieve

Volume Determination	Test No. 1	2	3	4
1. Volume of cylinder (volume of hole)				

Moisture Determination	Test No. 1	2	3	4
Sample tested using: direct heat ____ oven ___ microwave ___				
		Container No.		
2. Weight of moist sample plus container: _____________				
3. Weight of dry sample plus container: ______________				
4. Weight of moisture = [2] – [3]: __________				
5. Weight of container: ____________				
6. Weight of dry sample = [3] – [5]: ____________				
7. Moisture content = ([4] ÷ [6])*100: ________________ (%)				
8. Correction for ignition: _____________ (%)				
9. Corrected moisture content = [7] – [8]: _____________ (%)				

Density Determination				
10. Weight of moist sample plus cylinder: _____________				
11. Weight of cylinder: _________________________				
12. Weight of moist sample = [10] – [11]: _______________				
13. Wet density=[12] ÷[1]: ____________				
14. Fill dry density: [13] ÷ [1 + [9]/100]: ____________				
15. Maximum dry density: ____________				
16. Ratio[1] = ([14] ÷ [15])*100: ______________ (%)				

[1] Ratio of fill dry density to maximum dry density.
Indicate weight and volume units used in test.

Fig. 11B—In-place moisture-density determination: calibrated cylinder method test data (ASTM D2937), fine-grained soils–less than 5% + no. 4 sieve.

IN-PLACE MOISTURE-DENSITY DETERMINATION: TEMPLATE AND PLASTIC LINER METHOD TEST RECORD ASTM D5030

Location: ______________________ Site No. ______________

Project Name: ______________________

Contract No. ______________ Contractor: ______________________

Tested by: ______________ Computed by: ______________ Checked by: ______________

Test No.	Date	Location of test			Borrow source, location, and depth	Material classification
		Station	Centerline offset	Elevation		

Size of template: ______________________

Test No.	Date	Specified requirements		Test results		
		Moisture range (%)	Density (lb/ft^3)	Moisture (%)	Density (lb/ft^3)	Compaction (%)

Remarks: ______________________

____

____

____

Fig. 12B—In-place moisture-density determination: template and plastic liner method test record (ASTM D5030).

IN-PLACE MOISTURE-DENSITY DETERMINATION: TEMPLATE AND PLASTIC LINER METHOD TEST DATA ASTM D5030

Volume Determination		Test No. 1	Test No. 2	Test No. 3	Test No. 4
1. Weight of water plus container before filling template: ____________	( )				
2. Weight of water plus container after filling template: ____________	( )				
3. Weight of water required to fill template = [1] – [2]: ____________	( )				
4. Weight of water plus container before filling template and hole: _____	( )				
5. Weight of water plus container after filling template and hole: ______	( )				
6. Weight of water to fill template and hole = [4] – [5]: ____________	( )				
7. Net weight of water to fill hole = [6] – [3]: ____________	( )				
8. Volume = [7] ÷ [62.4] : ____________	( )				

Moisture Determination		Container No. 1	Container No. 2	Container No. 3	Container No. 4
Sample tested using: direct heat ____ oven ___ microwave ___					
9. Weight of moist sample and container: ____________	( )				
10. Weight of dry sample and container: ____________	( )				
11. Weight of moisture = [9] – [10]: ____________	( )				
12. Weight of container: ____________	( )				
13. Weight of wet sample = [9] – [12]: ____________	( )				
14. Weight of dry sample = [10] – [12]: ____________	( )				
15. Moisture content = ([11] ÷ [14]) 100: ____________	(%)				
16. Correction for ignition: ____________ (%)					
17. Corrected moisture content = [15] – [16]: ____________	(%)				

Density Determination		Test No. 1	Test No. 2	Test No. 3	Test No. 4
18. Total weight of soil removed from the hole: ____________	( )				
19. Total wet density = [18] ÷ [8]: ____________	( )				
20. Total dry density = [19] ÷ [1 + [17 ÷ 100]]: ____________	( )				
21. Required density = ____________	( )				
22. Ratio[1] = ____________	( )				

[1] Ratio of fill dry density to maximum dry density.
Indicate weight and volume units used in test.

Fig. 13B—In-place moisture-density determination: template and plastic liner method test data (ASTM D5030).

Project ____
Job number ____
Date ____
Taken by ____

NUCLEAR COMPACTION TEST DATA
FOR ASTM D6938

Test number	1	2	3	4	5	6	7	8	9	10
Station										
Offset										
Elevation										
Mode & depth										
Density count										
Wet density										
Moisture cnt.										
% Moisture										
Moisture corr.										
Dry density										
Std. density										
Opt. moisture										
% Compaction										

Test number	11	12	13	14	15	16	17	18	19	20
Station										
Offset										
Elevation										
Mode & depth										
Density count										
Wet density										
Moisture cnt.										
% Moisture										
Moisture corr.										
Dry density										
Std. density										
Opt. moisture										
% Compaction										

Density	Moisture

Remarks: ____

Fig. 14B—Nuclear compaction test data for ASTM D6938.

Moisture Content Determination
Summary Data Sheet for ASTM Methods

Kind of material ______________________ Location ______________________ Owner______________________

Project name____ Site no.______________

Contract no.______________________ Contractor______________________________________

Dried by oven ☐ direct heat ☐ microwave ☐ carbide ☐

Test no.	Date of test	Location of test (structure or station, offset)	Elevation	WW[1]	DW[1]	TW[1]	Moisture content (%)[2]				Tested by
							Oven D2216	Direct heat D4959	Microwave D4643	Carbide D7944	

[1] WW-Weight of moisture sample and container DW=Weight of dry sample and container TW=Weight of container

[2] Moisture content (%)=[(WW–DW)÷(DW–TW)]*100

Fig. 15B—Moisture content determination summary data sheet for ASTM methods.

ASTM D2216	Moisture Content Oven				Designation USBR 5300
PROJECT **Example Computations**			FEATURE		
TESTED BY DATE	COMPUTED BY DATE		CHECKED BY DATE		
SAMPLE NUMBER	1	2			UNITS
CONTAINER NUMBER	15	20			
DATE PLACED IN OVEN	9/8/86	9/8/86			☒ g
MASS OF CONTAINER + WET SPECIMEN	366.1	374.6			
MASS OF CONTAINER + DRY SPECIMEN	348.0	342.1			☐ Kg
MASS OF CONTAINER	129.4	118.0			
MASS OF WATER	18.1	32.5			☐ lbm
MASS OF DRY SPECIMEN	218.6	224.1			
MOISTURE CONTENT (%)	8.3	14.5			

ASTM D2216	Moisture Content Oven				Designation USBR 5300
PROJECT			FEATURE		
TESTED BY DATE	COMPUTED BY DATE		CHECKED BY DATE		
SAMPLE NUMBER					UNITS
CONTAINER NUMBER					
DATE PLACED IN OVEN					☐ g
MASS OF CONTAINER + WET SPECIMEN					
MASS OF CONTAINER + DRY SPECIMEN					☐ Kg
MASS OF CONTAINER					
MASS OF WATER					☐ lbm
MASS OF DRY SPECIMEN					
MOISTURE CONTENT (%)					

Fig. 16B—Moisture content oven.

ASTM D4643	Moisture Determination Using Microwave Oven				Designation USBR 5315
SAMPLE NUMBER 1		PROJECT **Example Computations**		FEATURE	
TESTED BY DATE		COMPUTED BY DATE		CHECKED BY DATE	
DISH NUMBER 36			MASS OF DISH (g) 146.30		
TIME IN OVEN (min)	TOTAL TIME IN OVEN (min)	MASS OF DISH SOIL (g)	MASS OF SOIL (g)	MASS OF WATER (g)	MOISTURE CONTENT (%)
0	0	231.62	—	—	—
3	3	217.75	71.45	13.87	19.4
1	4	216.22	69.92	15.40	22.0
1	5	215.72	69.42	15.90	22.9
1	6	215.48	69.18	16.14	23.3
1	7	215.32	69.02	16.30	23.6
1	8	215.22	68.92	16.40	23.8
1	9	215.19	68.89	16.43	23.8
1	10	215.19	68.89	16.43	23.8

Fig. 17B—Moisture content determination summary data sheet for ASTM methods.

Moisture Determination Using Direct Heat			ASTM D4959
TESTED BY DATE	COMPUTED BY DATE	CHECKED BY DATE	
Example Computations			
PAN NUMBER (g)	113	REMARKS:	
MASS OF PAN + WET SOIL (g)	282.82		
MASS OF PAN + DRY SOIL (g)	260.40		
MASS OF PAN (g)	165.95		
MASS OF WATER (g)	22.42		
MASS OF DRY SOIL (g)	94.45		
PERCENT MOISTURE (g)	23.7	NOTE: Correction may be needed for loss due to ignition of organic material.	

Moisture Determination Using Direct Heat			ASTM D4959
TESTED BY DATE	COMPUTED BY DATE	CHECKED BY DATE	
PAN NUMBER		REMARKS:	
MASS OF PAN + WET SOIL (g)			
MASS OF PAN + DRY SOIL (g)			
MASS OF PAN (g)			
MASS OF WATER (g)			
MASS OF DRY SOIL (g)			
PERCENT MOISTURE (g)		NOTE: Correction may be needed for loss due to ignition of organic material.	

Fig. 18B—Moisture determination using direct heat.

ASTM D 4944	**Determining Moisture Content of Soil Using the Calcium Carbide Method**	Designation USBR 5310

PROJECT **Example Computations**		FEATURE
SAMPLE NUMBER 60N	CLASSIFICATION SYMBOL ML	INSTRUMENT NUMBER 8225-14
CALIBRATION EQUATION y = 1.196 x + 0.02		DATE
TESTED BY	DATE	CHECKED BY DATE

SAMPLE NUMBER (1)	NOMINAL SPECIMEN SIZE (g) (2)*	DIAL GAUGE READING ON CCRD (3)	CORRECTED READING (4)**	MOISTURE CONTENT FROM CALIBRATION CURVE (%) (5)
60N-8	26	9.0	9.0	10.8
-12	26	10.5	10.5	12.6
-20	26	16.5	16.5	19.8
-24	26	17.7	17.7	21.2
-28	13 (half-size)	10.3	20.6	24.7
-30	13 (half-size)	11.3	22.6	27.0
60N-108	26	9.6	9.6	11.5
-112	26	11.4	11.4	13.7

*If the moisture content of the full specimen exceeds the limit of the gauge on the testing equipment, a half-sized specimen is used.
**If (2) = half-size specimen, (4) = (3) x 2.
If (2) = full-size specimen, (4) = (3).

Fig. 19B—Determining moisture content of soil using the calcium carbide method.

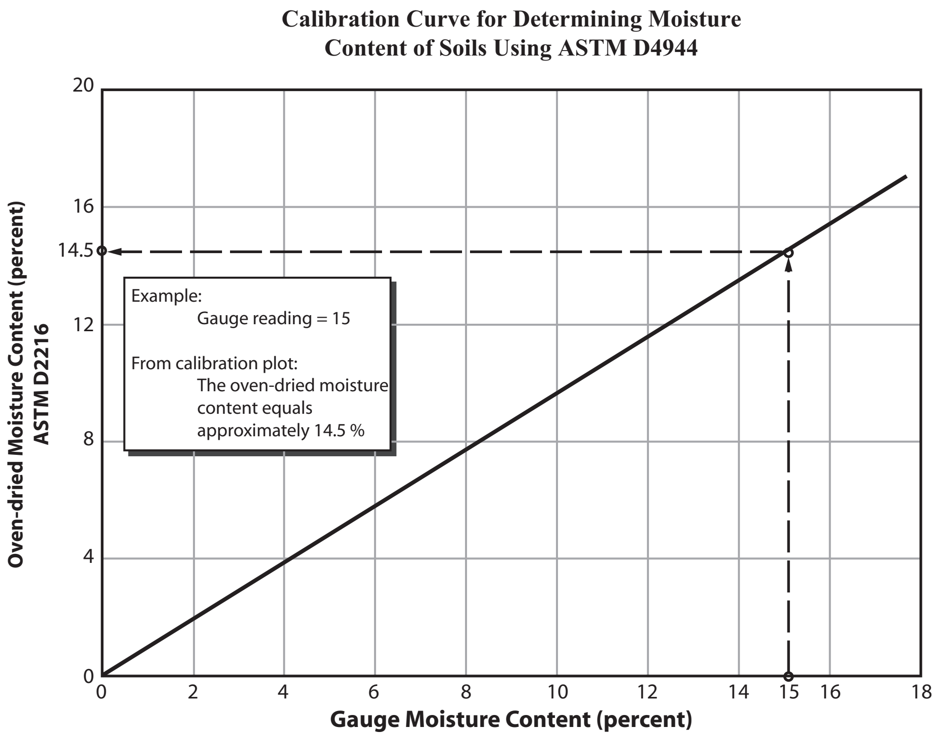

Fig. 20B—Calibration curve for determining moisture content of soils using ASTM D4944.

TEST FILL REPORT

Project Name: ______________________ Location: ______________________

Contract No. ______________ Contractor: ______________________

Inspector: ______________ Date: ______________ Time: ______________

Location of Test Fill: ______________________

Specified Lift Thickness (inches): ______ Specified Mass Density (pcf): ______ Specified Moisture Content: ____

Material:

Placing Method	Type of Fill	Unified Classification	% Passing ¾"	Maximum Particle Size (inches)

Test Fill Field Data:

Thickness of Fill (inches)	Length and Width (feet)	In-Place Dry Density of Mass (pcf)	Moisture Content of Test Fill (%)	No. of Test	Test Location

Equipment:

Type of Compaction Equipment	Operational Speed (mph)	(Number of Passes)

Remarks: ______________________

Signature of Inspector: ______________________ Date: ______________

Fig. 21B—Test fill report.

Appendix C

Fig. 1C—Traditional sheepsfoot roller—provides a kneading action for plastic soils.

Fig. 2C—Self-propelled tamping roller—provides the same kneading action for plastic soils.

Fig. 3C—Pheumatic roller—provides static rolling for soils of low to no plasticity.

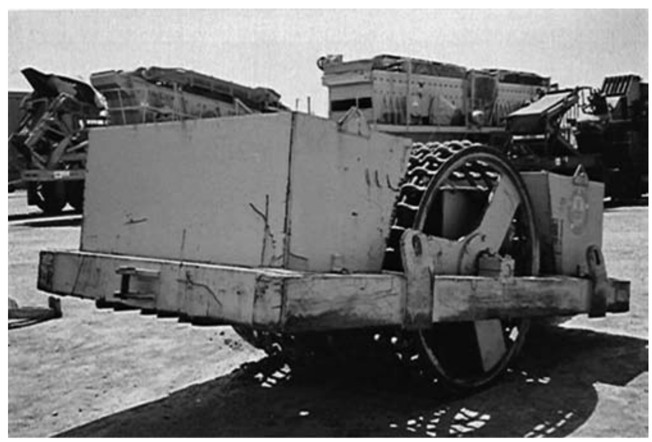

Fig. 4C—Grid roller—used to break down fill material such as shales.

Fig. 5C—Sand cone method (ASTM D1556).

Fig. 6C—Rubber balloon method (ASTM D2167).

Fig. 7C—Drive cylinder method (ASTM D2937).

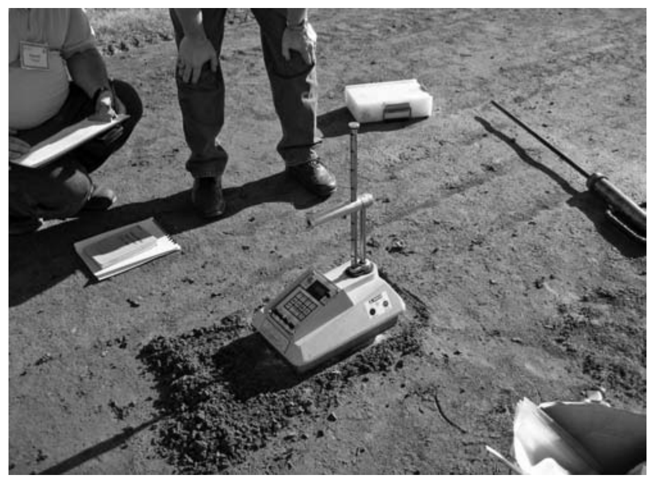

Fig. 8C—Nuclear method (ASTM D6938)

Fig. 9C—Nuclear method (ASTM D6938).

Fig. 10C—Calcium carbide gas pressure tester method (ASTM D4944).

Fig. 11C—Calcium carbide gas pressure tester method (ASTM D4944).

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ISBN: 978-0-8031-7023-0
Stock #: MNL70