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FM 5-472
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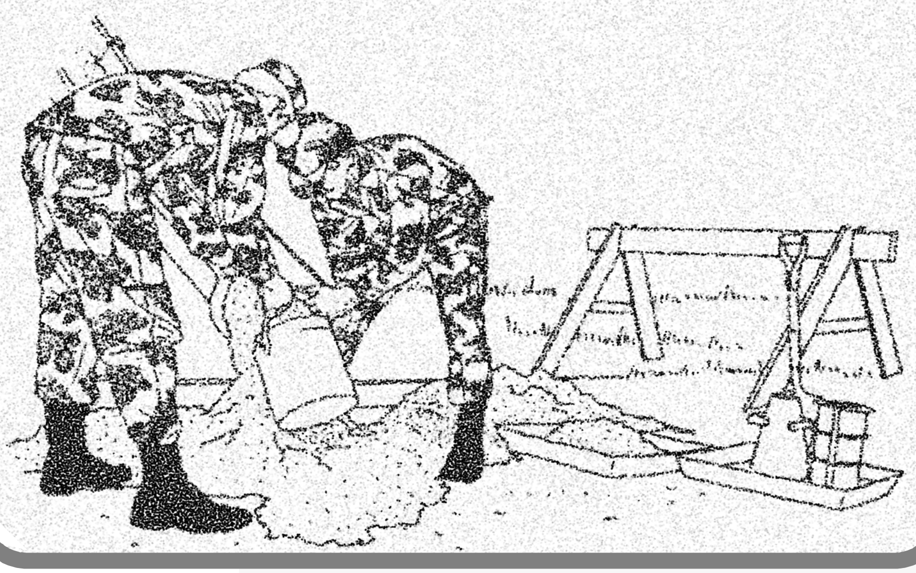
AFJMAN 32-1221(I)

Materials Testing



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FM 5-472
C2

Change 2

Headquarters
Department of the Army
Washington, DC, 1 July 2001

Materials Testing

1. Change FM 5-472, 27 October 1999, as follows:

Remove Old Pages	Insert New Pages
2-55 through 2-58	2-55 through 2-58
2-83 and 2-84	2-83 and 2-84
2-103 and 2-104	2-103 and 2-104
4-21 through 4-24	4-21 through 4-24
4-29 and 4-31	4-29 through 4-31
A-1	A-1

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FM 5-472
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Change 1

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2-65 and 2-66	2-65 and 2-66
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Foreword

This publication may be used by the US Army, US Navy, and US Air Force during training, exercises, and contingency operations.

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NAVFAC M0 330
AFJMAN 32-1221(I)

Headquarters
Department of the Army
Department of the Navy
Department of the Air Force
Washington, DC, 27 October 1999

MATERIALS TESTING

Table of Contents

Page

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*This publication supersedes FM 5-530, 17 August 1987.

Preface

Field Manual (FM) 5-472 provides the technical information necessary for military personnel to obtain samples and perform engineering tests and calculations on soils, bituminous paving mixtures, and concrete. These tests and calculations are required to achieve proper design with these materials and adequate control over their use in military construction.

This manual covers soils, aggregates, bituminous cements, bituminous paving mixtures, portland-cement concrete, and stabilized soil including stabilizing agents such as bitumens, cements, lime, fly ash, and chemical modifiers. The manual gives detailed instructions for taking adequate representative test samples and step-by-step procedures for making physical-properties tests and for recording, calculating, and evaluating test results. The manual explains methods for designing bituminous paving mixtures and for stabilizing soil. It also gives the procedures and tests required to control the manufacture of these mixtures. The manual describes the tools and equipment for performing these tests and contains general instructions for the care, calibration, and use of test equipment.

FM 5-472 is adopted for use by Navy personnel. Certain tests and procedures prescribed differ in principle or method and are more detailed than counterpart tests currently required by the Navy for new construction at Navy installations, including those in forward areas. Although this manual provides general guidance for materials and soils testing, the design of new structures and facilities will be based on the results obtained from methods and procedures outlined in Naval Facilities Engineering Command Design Manual (NAVFAC DM)-7.1 and NAVFAC DM-7.2. When methods and procedures prescribed by the Navy differ from those in this manual, the Navy's methods will take precedence.

The test procedures and terminology used in this manual conform to the latest methods and specifications of the American Society for Testing and Materials (ASTM), the American Concrete Institute (ACI), and the Portland Cement Association (PCA).

The tests listed in this manual also apply to arctic construction. However, cold-weather effects present different problems and additional tests will be required for correct evaluation of the materials. These additional tests and the considerations associated with arctic construction are in Technical Manual (TM) 5-349.

Appendix A contains an English-to-metric measurement conversion chart.

The proponent of this publication is HQ TRADOC. Send comments and recommendations on Department of the Army (DA) Form 2028 directly to Commander, USAES, ATTN: ATSE-TD-D, Fort Leonard Wood, Missouri 65473-6650.

Unless this publication states otherwise, masculine nouns and pronouns do not refer exclusively to men.

Chapter 1

Materials Testing Overview

Military engineers develop and maintain—

- Transportation facilities.
- Housing and special structures.
- Sanitary facilities.
- Military defenses.

Transportation facilities include roads, railways, airports and landing strips, pipelines, and harbor structures. These structures and facilities are built on and sometimes use the local soil, so engineers must know the type and characteristics of the soil at the site to design them. For example, in designing a road or an airfield, engineers must determine whether the soil can withstand the loads to be transported, including vehicle weights. They must also determine whether soil stabilization or paving will be needed. If the road or airfield requires pavement, then the engineers establish the suitability of available aggregate materials, since speed and efficiency of construction dictate the use of nearby sources. Bituminous paving mixtures and portland-cement concrete are made at or near the work site under the control of the local engineering officers. The engineers must design the mix and test the finished product's performance. They must have basic data concerning the properties of these materials to use them effectively in construction. Such data are obtained from the tests described in this manual.

MATERIALS TESTS

The properties of all materials are determined by their chemical composition and the physical structure in which the constituent compounds are arranged. Earth minerals and cementing materials are very complex, and the nature of the forces that bind them together is poorly understood on an atomic or molecular scale. However, the strength, stiffness, stability, and resistance to wear, erosion, or weathering can be determined by tests on the bulk material. Laboratory research related to field observation and experience with such materials enables engineers to establish limiting values of the measured properties to ensure satisfactory performance in service. Materials specifications based on this research give such limits. Tests of representative samples of a particular material available for engineering use are made, and the results are compared to the specifications to decide whether the material is adequate for the intended application. Materials tests also are used to identify or classify materials on the basis of their physical properties. These tests also provide basic data on the aggregates and cements necessary for the design of bituminous mixtures, stabilized soil, or portland-cement concrete.

SOIL PROPERTIES

A soil's physical characteristics determine its usefulness to support traffic or to serve as a subgrade or foundation material. These physical characteristics include the—

- Size and shape of the individual grains.
- Grain-size distribution.
- Specific gravity.
- Compaction characteristics.

Properties of many soils depend on moisture content. Tests for the moisture limits describe the soil's plasticity characteristics. Strength tests, such as the California Bearing Ratio (CBR) and the unconfined compression test, measure load-carrying capacity directly. Tests for these properties include expedient and deliberate testing procedures. The tests are used to identify and classify the type of soil represented by the samples. With the soil accurately tested and classified, its suitability for supporting traffic as a subgrade, base, or foundation material or as an aggregate, filler, or binder for mixtures can be evaluated. The construction of subgrades and bases for pavements, embankments, and other earth structures requires continual testing during the course of the work to adjust the mixtures and the construction methods that are needed.

CLASSIFICATIONS

Tests and evaluations of test results are more easily made by using a common reference or system that has a universal interpretation. Thus, no matter who performs the tests or the evaluation, the results can be understood by anyone familiar with the system. One reference for soils used by military engineers is the Unified Soil Classification System (USCS). Unified soil classification is found in ASTM D 2487-93. All soils are divided into three major categories, two of which are based on grain size. Further subdivision distinguishes between gravel, sand, silt, clay, and organic content and between well-graded and poorly graded soils. Each of these types is symbolized by a combination of two or four letters. A more detailed explanation of the USCS is in Chapter 2, Section I, of this manual and in Chapter 5 of FM 5-410.

DESIGN REQUIREMENTS

When engineers have the completed soil classification and all other information required for the proper design of an airfield or road, they can start the design. The design requirements are covered in FM 5-430-00-1 (for roads) and FM 5-430-00-2 (for airfields).

BITUMENS

Bituminous paving mixtures consist of aggregates, filler, and bitumen binder. Aggregate sources near the construction area are tested to give data on particle-size distribution and specific gravity. These data are used in designing the mix. Testing of bituminous cements received at the mixing plant may require identifying the material, determining its suitability as a binder, or providing data for determining what aggregates and fillers are required. The tests of bitumens described in this manual are field-identification procedures to expedite the use of the material until more detailed tests can be performed.

CONCRETE

Portland-cement concrete is a mixture of fine and coarse aggregates, portland cement, and water. The cement and water chemically react to form compounds that hold the aggregates in a strong, rock-like mass. Concrete is made in mixing plants, field mixers, or truck mixers near or at the construction site. The quality of the concrete produced depends on the proper mix design to achieve the desired workability of the fresh concrete and strength of the hardened material. This manual describes tests of aggregates and fresh and hardened concrete, with instructions for using test data in mix design and control. Detailed information about concrete can be found in FM 5-428.

STABILIZATION

An accurate soil description, determined from test data, is necessary to determine whether its properties must be improved by stabilization to make it adequate for supporting traffic or for use as a base course. Test results indicate the method of stabilization and materials to be used and verify the adequacy of the stabilized soil.

EQUIPMENT

The equipment for all materials tests given in this manual consists of three sets:

- The soil test set.
- The asphalt test set.
- The concrete test set.

The soil test set is considered the basic set; the other two sets are used with it. In selected operations (such as control testing at a concrete batch plant), the concrete test set can be used without the soil test set. Certain items listed as part of the sets are not issued with the sets but must be requisitioned separately. Some of the test methods (such as the specific gravity of solids, the hydrometer analysis, and the shrinkage limit) describe items of equipment that are not issued with the test sets but would be available locally or from commercial sources.

TEST SET, SOIL (SC 6635-98-CL-E02)

The soil test set is the basic set for performing soil tests including sieve analysis, moisture content, Atterberg limits, CBR, and soil trafficability. The separately packaged soil-trafficability test set (SC 6635-97-CL-E01) is included in this set. Items such as DA forms, pencils, tracing paper, labels and tags, towels, twine, and wax are not issued initially with the set but must be requisitioned separately by stock or form number.

TEST SET, ASPHALT (SC 6635-98-CL-E03)

The asphalt test set is issued in three chests:

- Laboratory-centrifuge chest.

- Miscellaneous-equipment chest.
- 1,1,1-trichloroethane chest.

This set is not complete within itself; therefore, it must be used with the soil test set. Items such as forms, brushes, cloths, and chemicals (alcohol, sodium hydroxide, and 1,1,1-trichloroethane) are not issued with the set but must be requisitioned separately. The 1,1,1-trichloroethane is a hazardous substance. Numerous substitute agents are available. Consult your installation environmental office to find out which substitute is available and best suited to your needs.

TEST SET, CONCRETE (SC 6635-98-CL-E04)

The concrete test set is issued in three chests:

- Beam-testing-machine chest.
- Collapsible-steel-forms chest.
- Miscellaneous-equipment chest.

This set normally is used with the soil test set. Chemicals, DA forms, and some other items are not issued with the set but must be requisitioned separately.

NUCLEAR MOISTURE DENSITY GAUGE

The nuclear moisture density gauge (national stock number [NSN] 6635-01-030-6896) is used to perform moisture and density tests. The gauge is issued in its case with all necessary equipment except test forms. Forms should be locally reproduced from the United States Army Engineer School (USAES) or from manufacturers' samples. This equipment requires special storage considerations and licensing of operators. The Nuclear Regulatory Commission (NRC) requires that any unit that owns this equipment must have a qualified local radiation protection officer (RPO) and a licensed operator before it can be operated. Information concerning this equipment should be addressed to the USAES or to the Tank-automotive and Armaments Command (TACOM).

SAFETY AND ENVIRONMENTAL CONSIDERATIONS

Safety and environmental awareness must be planned and integrated as part of all military operations to protect personnel and the environment. Some of the tests described in this manual will require special safety equipment that may not be included in the test sets. Some of the tests in this manual will require the use of environmentally hazardous materials. Special care must be taken to minimize the potential harmful effects of exposure to hazardous substances. Violation of environmental laws through improper storage, handling, or disposal of hazardous materials can result in severe penalties, including fines and imprisonment. Proper training is necessary to ensure that all personnel performing materials tests know how to properly handle and dispose of the substances listed in the test procedures. If you are unsure what materials are hazardous or how to dispose of materials properly, or if you need training, consult your unit or post environmental representative.

Chapter 2

Soils

The soil in an area is an important consideration in selecting the exact location of a structure. Military engineers, construction supervisors, and members of engineer reconnaissance parties must be capable of properly identifying soils in the field to determine their engineering characteristics. Because a military engineer must be economical with time, equipment, material, and money, site selection for a project must be made with these factors in mind.

SECTION I. FORMATION, CLASSIFICATION, AND FIELD IDENTIFICATION

The word soil has numerous meanings and connotations to different groups of professionals who deal with this material. To most soil engineers (and for the purpose of this text), soil is the entire unconsolidated earthen material that overlies and excludes bedrock. It is composed of loosely-bound mineral grains of various sizes and shapes. Due to its nature of being loosely bound, it contains many voids of varying sizes. These voids may contain air, water, organic matter, or different combinations of these materials. Therefore, an engineer must be concerned not only with the sizes of the particles but also with the voids between them and particularly what these voids enclose (water, air, or organic materials).

SOIL FORMATION

Soil formation is a continuous process and is still in action today. The great number of original rocks, the variety of soil-forming forces, and the length of time that these forces have acted all produce many different soils. For engineering purposes, soils are evaluated by the following basic physical properties:

- Gradation of sizes of the different particles.
- Bearing capacity as reflected by soil density.
- Particle shapes.

An engineer extends soil evaluation by considering the effect of water action on the soils. With a complete evaluation, an engineer can determine whether or not the soil is adequate for the project.

ROCKS

Soil forms when rocks that are exposed to the atmosphere disintegrate and decompose, either by mechanical action (wind, water, ice, and vegetation), chemical action, or both. The resulting material may remain where it is formed or it may be transported by water, glaciers, wind, or gravity and deposited at a distance from the parent rock.

Geologists classify rocks into three basic groups:

- Igneous (formed by cooling from a molten state).
- Sedimentary (formed by the accumulation and cementation of existing particles and remains of plants and animals).
- Metamorphic (formed from existing rocks subjected to heat and pressure).

STRATA

At a particular location are usually several layers (strata), one above the other, each composed of a different kind of soil. Strata may be a fraction of an inch or many feet thick. The upper layer is called the topsoil or agricultural soil since it supports plant growth. For an adequate soil evaluation for engineering uses, identify all strata to whatever depth may be affected by the construction. A vertical cross section through the earth, with the depths and types of soil indicated, is called a soil profile.

PHYSICAL PROPERTIES

A soil’s physical properties help determine the engineering characteristics. The following properties are the basis for the soil-classification system used in engineering identification of soil types. The discussion of the physical properties of soil focuses on the soil particles themselves. The terms *particle* and *grain* are used interchangeably.

- Grain size.
- Particle shape.
- Gradation.
- Density.
- Specific gravity.
- Moisture.
- Consistency.
- Organic soil.

Physical characteristics of soil particles include size and shape. The proportions of different-sized particles determine an aggregate's gradation. Density or compactness refers to the closeness of packing of soil particles; the closer the packing, the greater the compactness and the larger the soil weight per unit of volume. Plasticity characteristics of fine-grained soil components include the liquid limit (LL) and the plastic limit (PL); shrinkage ratios; dry strength; and unconfined, compressive strength. Specific gravity of soil particles aids in their identification. The presence of organic matter is important to the engineering use of soils. Color, texture, odor, structure, and consistency are readily observed factors that aid in soil description.

GRAIN OR PARTICLE SIZE

Soils are divided into groups based on the size of the particle grains in the soil mass. Common practice is to distinguish the sizes by using sieves. A sieve is a screen attached across the end of a shallow, cylindrical frame. The screen permits smaller particles to fall through and retains the larger particles on the sieve. Sieves with screen openings of different sizes (the largest on the top and the smallest at the bottom) separate the soil into particle groups based on size. The amount remaining on each sieve is measured and described as a percentage by weight of the entire sample. The size groups that are designated by the USCS are cobbles, gravels, sands, and fines (silt or clay), as shown in *Table 2-1*. Further discussion on these size groups can be found later in this chapter and in Appendix B.

Table 2-1. Grain-size groups

Size Group	Sieve Size	
	Passing	Retained On
Cobbles	No maximum size*	3 inches
Gravels	3 inches	No. 4
Sands	No. 4	No. 200
Fines (clay or silt)	No. 200	No minimum size

*** In military engineering, the maximum size of cobbles is accepted as 40 inches, based on the maximum jaw opening of a rock-crushing unit.**

GRAIN OR PARTICLE SHAPE

The shape of the particles influences a soil's strength and stability. Two general shapes are normally recognized—bulky and platy.

Bulky

The bulky shapes include particles that are relatively equal in all three dimensions. In platy shapes, one dimension is very small compared to the other two. For example, a thick book would be considered bulky, but a page of the book would be platy. Bulky shapes are subdivided into four groups: angular, subangular, subrounded, and well-rounded (see *Figure 2-1, page 2-4)*. These four subdivisions are dependent on the amount of weathering that has occurred. Cobbles, gravel, sand, and silt usually fall into this bulky-shape group. These groups are discussed in the order of desirability for construction.

Angular-shaped particles are those that have recently broken up. They are characterized by jagged projections, sharp ridges, and flat surfaces. The interlocking characteristics of angular gravels and sands generally make them the best materials for construction. These particles are seldom found in nature because weathering processes normally wear them down in a relatively short time. Angular material may be produced artificially by crushing, but because of the time and equipment required for such an operation, natural materials with other grain shapes are frequently used.

Subangular-shaped particles have been weathered to a point that the sharper points and ridges of their original angular shape have been worn off. These

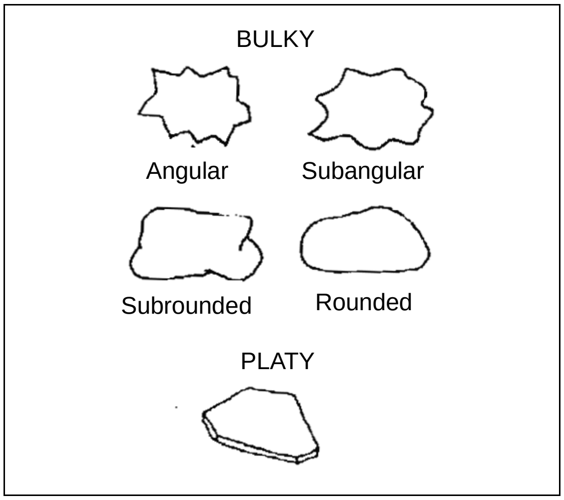


Figure 2-1. Shapes of soil grains

particles are still very irregular in shape with some flat surfaces and are excellent for construction.

Subrounded particles are those on which weathering has progressed even further. While they are still somewhat irregular in shape, they have no sharp corners and few flat areas. These particles are frequently found in streambeds. They may be composed of hard, durable particles that are adequate for most construction needs.

Rounded particles are those in which all projections have been removed and few irregularities in shape remain. The particles approach spheres of varying sizes. Rounded particles are usually found in or near streambeds, beaches, or dunes. Possibly the most extensive deposits exist at the beaches where repeated wave action produces almost perfectly rounded particles that may be uniform in size. They may also be found in arid environments due to wind action and the resulting abrasion between particles. They are not desirable for use in asphalt or concrete construction until the rounded shape is altered by crushing.

Platy

The platy shapes have one dimension relatively small compared to the other two. They have the general shape of a flake of mica or a sheet of paper. Particles of clay soil exhibit this shape, although they are too small to be seen with the naked eye. Coarse-grained soil particles are individually discernible to the naked eye; fine-grained particles with platy or bulky shapes are not.

GRADATION

Gradation describes the distribution of the different size groups within a soil sample. The soil may be well-graded or poorly graded.

Well-Graded Soils

Well-graded soils must have a good range of all representative particle sizes between the largest and the smallest. All sizes are represented, and no one size is either overabundant or missing (see *Figure 2-2*).

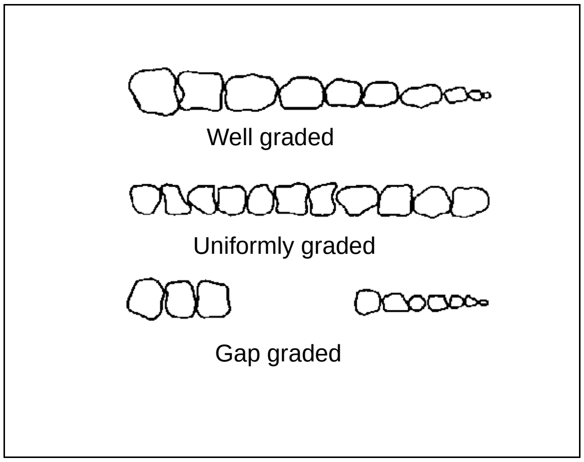


Figure 2-2. Soil gradation

Poorly Graded Soils

Poorly graded soils can be classified as either uniformly graded or gap graded. A uniformly graded soil consists primarily of particles of nearly the same size. A gap-graded soil contains both large and small particles, but the gradation continuity is broken by the absence of some particle sizes (see *Figure 2-2*).

DENSITY

The structure of the aggregate of soil particles may be dense (closely packed) or loose (lacking compactness). A dense structure provides interlocking of particles with smaller grains filling the voids between the larger particles. When each particle is closely surrounded by other particles, the grain-to-grain contacts are increased, the tendency for displacement of individual grains under a load is lessened, and the soil is capable of supporting heavier loads. Coarse materials that are well-graded usually are dense and have strength and stability under a load. Loose, open structures have large voids and will compact under a load, leading to settlement or disintegration under foundation or traffic loads.

SPECIFIC GRAVITY

The specific gravity is the ratio between the weight-per-unit volume of the material and the weight-per-unit volume of water at a stated temperature. There are three ways of determining and expressing specific gravity:

- Specific gravity of solids.
- Apparent specific gravity.
- Bulk specific gravity.

The specific gravity of solids is the method most widely used when testing soils. The apparent and bulk specific-gravity methods are used in testing fine and coarse aggregates. The specific gravity of solids is explained further in Section IV of this chapter, along with the test procedure.

MOISTURE

The term moisture content (w) is used to define the amount of water present in a soil sample. It is the proportion of the weight of water to the weight of the solid mineral grains (weight of dry soil) expressed as a percentage.

The moisture content of a soil mass is often the most important factor affecting the engineering behavior of the soil. Water may enter from the surface or may move through the subsurface layers either by gravitational pull, capillary action, or hygroscopic action. This moisture influences various soils differently and usually has its greatest effect on the behavior of fine-grained soils. The fine grains and their small voids retard the movement of water and also tend to hold the water by surface tension.

Many fine-grained soils made from certain minerals exhibit plasticity (putty-like properties) within a range of moisture contents. These soils are called clays, and their properties may vary from essentially liquid to almost brick hard with different amounts of moisture. Further, clays are basically impervious to the passage of free or capillary moisture. Coarse-grained soils with larger voids permit easy drainage of water. They are less susceptible to capillary action. The amount of water held in these soils is less than in fine-grained soils, since the surface area is smaller and excess water tends to drain off.

Surface Water

Surface water from precipitation or runoff enters the soil through the openings between the particles. This moisture may adhere to the different particles or it may penetrate the soil to some lower layer.

Subsurface Water

Subsurface water is collected or held in pools or layers beneath the surface by a restricting layer of soil or rock. This water is constantly acted on by one or more external forces.

Gravitational Pull

Water controlled by gravity (free or gravitational water) seeks a lower layer and moves through the voids until it reaches some restriction. This restriction may be bedrock or an impervious soil layer with openings or voids so small that they prevent water passage.

Capillary Action

Voids in soil may form continuous tunnels or tubes and cause the water to rise in the tubes by capillary action (capillary moisture). Since the smaller the tube, the stronger the capillary action, the water rises higher in the finer soils that have smaller interconnected voids. This area of moisture above the free water layer or pool is called the capillary fringe.

Adsorbed Water and Hygroscopic Moisture

In general terms, adsorbed water is water that may be present as thin films surrounding separate soil particles. When soil is in an air-dried condition, the adsorbed water present is called hygroscopic moisture. Adsorbed water is present because soil particles carry a negative electrical charge. Water is dipolar; it is attracted to the surface of a particle and bound to it. The water films are affected by the soil particle's chemical and physical structures and its relative surface area. The relative surface area of a particle of fine-grained soil, particularly if it has a platy shape, is much greater than for coarse soils composed of bulky grains. The electrical forces that bind adsorbed water to a soil particle also are much greater.

In coarse soils, the adsorbed layer of water on a particle is quite thin in comparison to the overall particle size. This, coupled with the fact that the contact area with adjacent grains is quite small, leads to the conclusion that the presence of the adsorbed water has little effect on the physical properties of coarse-grained soils. By contrast, for finer soils and particularly in clays, the adsorbed water film is thick in comparison to the particle size. The effect is very pronounced when the particles are of colloidal size.

Plasticity and Cohesion

Two important aspects of the engineering behavior of fine-grained soils are directly associated with the existence of adsorbed water films. These aspects are plasticity and cohesion.

Plasticity is a soil's ability to deform without cracking or breaking. Soils in which the adsorbed films are relatively thick compared to particle size (such as clays) are plastic over a wide range of moisture contents. This is presumably because the particles themselves are not in direct contact with one another. Coarse soils (such as clean sands and gravels) are nonplastic. Silts also are essentially nonplastic materials, since they are usually composed predominantly of bulky grains; if platy grains are present, they may be slightly plastic.

A plasticity index (PI) is used to determine whether soil is cohesive. Not all plastic soils are cohesive. Soil is considered cohesive if its PI is greater than 5. That is, it possesses some cohesion or resistance to deformation because of the surface tension present in the water films. Thus, wet clays can be molded into various shapes without breaking and will retain these shapes. Gravels, sands, and most silts are not cohesive and are called cohesionless soils.

In engineering practice, soil plasticity is determined by observing the different physical states that a plastic soil passes through as the moisture conditions change. The boundaries between the different states, as described by the moisture content at the time of changes, are called consistency limits or Atterberg limits, named after the Swedish scientist who defined them years ago.

CONSISTENCY

Atterberg established four states of consistency for fine-grained soils: liquid, plastic, semisolid, and solid. The dividing lines between these states are called the LL, the PL, and the shrinkage limit. These limits are quantified in terms of water content.

Liquid Limit

The LL is the moisture content at an arbitrary limit between a soil's liquid and plastic states of consistency. Above this value, a soil is presumed to be a liquid and flows freely under its own weight. Below this value, it will deform under pressure without crumbling, provided it exhibits a plastic state.

Plastic Limit

The PL is the moisture content at an arbitrary limit between the plastic and semisolid states. As a sample is dried, the semisolid state is reached when the soil is no longer pliable and crumbles under pressure. Between the LL and PL is the plastic range. The PI is the numerical difference in moisture contents between the two limits (PI = LL - PL). It defines a soil's range of moisture content in a plastic state.

Shrinkage Limit

The shrinkage limit is the boundary moisture content between the semisolid and solid states. This boundary is determined when a soil sample, upon being dried, finally reaches a limiting or minimum volume. Beyond this point, further drying will not reduce the volume, but may cause cracking.

ORGANIC SOIL

Soil having a high content of organic material is described as organic soil. It is usually very compressible and has poor load-maintaining properties.

EFFECTS OF SOIL CHARACTERISTICS

Soil characteristics are a measure of a soil's suitability to serve an intended purpose. Generally, a dense soil will withstand greater applied loads (having greater bearing capacity) than a loose soil. Particle size has a definite relation to this capacity. Empirical tests show that well-graded, coarse-grained soils generally can be compacted to a greater density than fine-grained soils because the smaller particles tend to fill the spaces between the larger ones. The shape of the grains also affects the bearing capacity. Angular particles tend to interlock and form a denser mass. They are more stable than the rounded particles which can roll or slide past one another. Poorly graded soils, with their lack of one or more sizes, leave more or greater voids and therefore a less-dense mass. Moisture content and consistency limits aid in describing a soil's suitability. A coarse-grained, sandy or gravelly soil generally has good drainage characteristics and may be used in its natural state. A fine-grained, clayey soil with a high PI may require considerable treatment, especially if used in a moist location.

SOIL CLASSIFICATION

Soil seldom exists separately as sand, gravel, or any other single component in nature. It is usually a mixture with varying proportions of different-sized particles. Each component contributes to the mixture’s characteristics. Once the principal characteristics are identified within this system (by both visual examination and laboratory tests), a descriptive name and letter symbol are assigned to the soil.

Before soil can be classified properly, it is necessary to establish a basic terminology for the various soil components and to define the terms used. As mentioned earlier, the USCS uses specific names to designate the size ranges of soil particles. These basic designations are cobbles, gravel, sand, and fines (silt or clay).

CLASSIFICATION

To start the classification process, become familiar with the USCS (see Appendix B) and the classification sheet used in the classification process (see *Table B-1*, *page B-2*). The first three columns of the classification sheet show the major divisions of the classification and the symbols that distinguish the individual soil types. Names of typical and representative soil types found in each group are shown in column 4. The field procedures for identifying soils by general characteristics and from pertinent tests and visual observations are shown in column 5. The desired descriptive information for a complete identification of a soil is presented in column 6. Column 7 presents the laboratory classification criteria by which the various soil groups are identified and distinguished.

CATEGORIES

In the USCS, soils are divided into three major soil divisions: coarse-grained, fine-grained, and highly organic. Coarse-grained soils are those having 50 percent or less material passing the number (No.) 200 sieve; fine-grained soils are those having more than 50 percent passing the No. 200 sieve. Highly organic soils can generally be identified by visual examination. This system recognizes 15 soil groups and uses names and letter symbols to distinguish between these groups. The letter symbols used are relatively easy to remember. They are derived either from the terms descriptive of the soil fractions, the relative value of the LL (high or low), or the relative gradation (well-graded or poorly graded). *Table 2-2* shows these individual letter symbols. The symbols are combined to form the group symbols that correspond to the names of typical soils as seen in columns 3 and 4 of the classification sheet. These symbols are also used in combination to describe borderline soils.

Table 2-2. Soil-classification symbols

Soil Groups	Symbol
Gravel	G
Sand	S
Silt	M
Clay	C
Organic (silts and clays)	O
Organic (peat)	Pt
Soil Characteristics	**Symbol**
Well graded	W
Poorly graded	P
Low LL (less than 50)	L
High LL (50 or greater)	H

Coarse-Grained Soils

Coarse-grained soils are subdivided into two divisions—

- Gravels and gravelly soils (G).
- Sands and sandy soils (S).

A coarse-grained soil is classified as a gravel if more than half the coarse fraction, by weight, is larger than a No. 4 sieve. It is a sand if more than half the coarse fraction, by weight, is smaller than a No. 4 sieve. In general, there is no clear-cut boundary between gravelly and sandy soils. As far as behavior is concerned, the exact point of division is relatively unimportant. Where a mixture occurs, the primary name is the predominant fraction, in percent by weight, and the minor fraction is used as an adjective. For example, a sandy gravel would be a mixture containing more gravel than sand, by weight. It is desirable to further divide coarse-grained soils into three groups based on the amount of fines (materials passing a No. 200 sieve) they contain.

NOTE: If fines interfere with free-draining properties (as may occur with plastic fines), use the double symbol (GW-GM, GW-GC, and so on) indicating that such soils will be classed with soils having from 5 to 12 percent fines.

Less-Than-5-Percent Nonplastic Fines

These soils may fall into the groups GW, GP, SW, or SP, where the shape of the grain-size-distribution curve determines the symbol’s second letter. The terms well graded (W) and poorly graded (P) have been discussed earlier. However, as noted above, if the fines interfere with the free-drainage properties, a dual or double symbol is used.

- GW and SW groups. The GW and SW groups include well-graded, gravelly soils and sandy soils with little or no nonplastic fines (less than 5 percent passing the No. 200 sieve). The presence of the fines must not noticeably change the strength characteristics of the coarse-grained fraction and must not interfere with its free-draining characteristics.
- GP and SP groups. The GP and SP groups include poorly graded gravels and sands with little or no nonplastic fines. These materials may be classed as uniform gravels, uniform sand, or gap-graded materials.

More-Than-12-Percent Fines

These soils may fall into the groups designated GM, GC, SM, and SC. The use of the symbols M and C is based on the plasticity characteristics of the material passing the No. 40 sieve. Use the LL and PI in specifying the laboratory criteria for these groups. The symbol M is used to indicate that the material passing the No. 40 sieve is silty in character. An M usually designates a fine-grained soil of little or no plasticity. The symbol C is used to indicate that the binder soil is predominantly clayey in character.

- GM and SM groups. The GM and SM groups comprise silty (M) gravels and silty sands with fines (more than 12 percent passing the No. 200 sieve) having low or no plasticity. For both of these groups, the LL and PI will plot below the A line on the plasticity chart or the PI will be less than 4. Both well-graded and poorly graded materials are included in these two groups. Normally these soils have little to no dry strength, but occasionally the fine or binder materials will contain a natural cementing agent that will increase dry strength.

- GC and SC groups. The GC and SC groups include gravelly or sandy soils with fines (more than 12 percent passing the No. 200 sieve) that are more clay-like and that range in plasticity from low to high. For both of these groups, the LL and PL will plot above the A line with a PI of more than 7 (see Section VI of this chapter).

Borderline Soils

Coarse-grained soils of which between 5 and 12 percent of material passes the No. 200 sieve are classed as borderline and are given a dual symbol (for example, GW-GM). Similarly, coarse-grained soils of which more than 12 percent of material passes the No. 200 sieve, and for which the limits plot in the shaded portion of the plasticity chart, are classed as borderline and require dual symbols (for example, SM-SC). It is possible, in rare instances, for a soil to fall into more than one borderline zone. If appropriate symbols were used for each possible classification, the result would be a multiple designation consisting of three or more symbols. This approach is unnecessarily complicated.

It is best to use only a double symbol in these cases, selecting the two that are believed to be most representative of the soil's probable behavior. In cases of doubt, the symbols representing the poorer of the possible groupings should be used. For example, a well-graded sandy soil with 8 percent passing the No. 200 sieve, with an LL of 28 and a PI of 9, would be designated as SW-SC. If the soil's LL and PI were plotted in the shaded portion of the plasticity chart (for example, an LL of 20 and a PI of 5), the soil would be designated either SW-SC or SW-SM, depending on the engineer's judgment from the standpoint of the climatic region.

Fine-Grained Soils

Fine-grained soils are subdivided into two divisions—

- Silts (M).
- Clays (C).

Fine-grained soils are not classified on the basis of grain-size distribution, but according to plasticity and consistency. Like the coarse-grained soils, laboratory classification criteria are based on the relationship between the LL and PI designated in the plasticity chart. The L groups, which have LLs less than 50, and the H groups, which have LLs greater than 50, are the two major groupings of fine-grained soils. The symbols L and H have general meanings of low and high LLs, respectively.

Fine-grained soils are further divided by their position above or below the plasticity chart's A line.

- ML and MH groups. Typical soils of the ML and MH groups are inorganic silts. Those that have an LL less than 50 are in the ML group; others are in the MH group. All of these soils plot below the A line. The ML group includes very fine sands; rock flours (rock dust); and silty or clayey, fine sands or clayey silts with slight plasticity. Loess-type soils usually fall into this group. Micaceous and diatomaceous soils generally fall into the MH group, but they may extend into the ML group when their LLs are less than 50. This is true of certain types of kaolin clays which have relatively low plasticity. Plastic silts fall into the MH group.

- CL and CH groups. In these groups, the symbol C stands for clay, while L and H denote low or high LLs. These soils plot above the A line and are principally inorganic clays. The CL group includes gravelly, sandy, silty, and lean clays. The CH group contains inorganic clays of medium to high plasticity including fat clays, the gumbo clays of the southern United States (US), volcanic clays, and bentonite. The glacial clays of the northern US cover a wide area in the CL and CH groups.
- OL and OH groups. The soils in these two groups are characterized by the presence of organic matter, hence the symbol O. All of these soils generally plot below the A line. Organic silts and organic silt-clays of low plasticity fall into the OL group. Organic clays of high plasticity plot in the OH zone of the plasticity chart. Many of the organic silts, silt-clays, and clays deposited by the rivers along the lower reaches of the Atlantic seaboard have LLs above 40 and plot below the A line. Peaty soils may have LLs of several hundred percent and will plot well below the A line due to their high percentage of decomposed vegetation. However, an LL test is not a true indicator where a considerable portion consists of other than soil matter.
- Borderline soils. Fine-grained soils with limits that plot in the shaded portion of the plasticity chart are borderline cases and are given dual symbols (for example, CL-ML). Several soil types that exhibit low plasticity plot in this general region on the chart where no definite boundary between silty and clayey soils exists.

Highly-Organic Soils

A special classification is reserved for the highly organic soils (Pt), such as peat, which have many characteristics undesirable for use as foundations and construction materials. No laboratory criteria are established for these soils, as they can be identified in the field by their distinctive color, odor, spongy feel, and fibrous textures. Particles of leaves, grass, branches, or other fibrous vegetable matter are common components of these soils.

IDENTIFICATION OF SOIL GROUPS

The USCS is designed so that most soils may be classified into the three primary or major divisions (coarse-grained, fine-grained, and highly organic) by means of visual inspection and simple field tests. Classification into the subdivisions can be made by visual examination only with some degree of success. More positive identification may be made by means of laboratory tests on the materials. However, in many instances a tentative classification determined in the field is of great benefit and may be all the identification that is necessary, depending on the purpose for which the soil is to be used.

Methods of general identification of soils are discussed in the following paragraphs as well as a laboratory testing procedure. It is emphasized that the two methods of identification are never entirely separated. Certain characteristics can only be estimated by visual examination, and in borderline cases it may be necessary to verify a classification by laboratory tests. The field methods are entirely practical for preliminary laboratory identification and may be used to advantage in grouping soils in such a way that only a minimum number of laboratory tests need to be conducted. The field methods of classification should never be used as the end product for performing design.

FIELD IDENTIFICATION

Field identification is an excellent tool when an engineer needs to have an idea of the general type of soil being dealt with. One excellent use of these procedures is during a preliminary construction-site analysis.

Several simple tests are used in field identification. The number of tests depends on the soil type and the experience of the individual employing them. Experience is the greatest asset in field identification, and learning the techniques from an experienced technician is the best way to acquire experience. Lacking such assistance, experience is gained during laboratory testing by systematically comparing the numerical test results for typical soils in each group with the “look” and “feel” of the material. An approximate identification can be made by examining a dry sample spread on a flat surface.

All lumps should be separated until individual grains are exposed. Individual grains, no matter how large, should not be broken to a smaller size since this changes the soil’s grain size and character. A rubber-faced or wooden pestle and a mixing bowl are recommended, but separating the sample underfoot on a smooth surface will suffice for an approximate identification. Examining the characteristics of the particles in the sample makes it possible to assign the soil to one of the three principal groups. Classification derived from these tests should be recognized as approximations.

An approximate identification of a coarse-grained soil is made by observing—

- Grain size.
- Gradation.
- Grain shape.
- Hardness.

Tests for identifying the fine-grained portions of a soil are performed on the portion of material that passes a No. 40 sieve. This is the same soil fraction used in the laboratory for the LL and PL tests. If this sieve is not available, a rough separation may be made by spreading the material on a flat surface and removing the gravel and larger sand particles. Fine-grained soils are examined primarily for characteristics related to plasticity.

Organic soils are identified by significant quantities of organic matter. When decayed roots, leaves, grasses, and other vegetable matter are present, they produce a highly organic soil, which is usually dark-colored when moist and has a soft, spongy feel and a distinctive odor of rotting organic matter. Partly-organic soils may contain finely divided organic matter detectable by color or odor.

TEST EQUIPMENT

Field tests may be performed with little or no equipment other than a small amount of water. However, accuracy and uniformity of results will be increased greatly by properly using the following equipment available in nearly all engineer units:

- A No. 4 and a No. 40 sieve. (A No. 200 sieve is useful but not required.)
- A digging instrument such as a pick, shovel, or entrenching tool. (A hand earth auger or posthole digger is useful in obtaining samples from depths a few feet or more below the surface.)
- A stirrer.
- A knife.
- Several sheets of heavy, nonabsorbent paper.
- A mixing bowl and a pestle (a canteen cup and a wooden dowel).
- A pan and a heating element.
- Scales or balances.

TEST FACTORS

The USCS considers three soil properties:

- The percentage of gravel, sand, or fines.
- The shape of the grain-size-distribution curve.
- The plasticity.

These are the primary factors to be considered, but other observed properties, whether made in the field or in the laboratory, should also be included in the soil description. The following information can be determined from field identification:

- Color (in moist condition).
- Grain size (estimated maximum grain size and estimated percent, by weight, of gravel, sands, and fines).
- Gradation (well or poorly graded).
- Grain shape (bulky or platy and angular, subangular, rounded, or subrounded).
- Plasticity (nonplastic, low, medium, or high).
- Predominant soil type.
- Secondary components.
- Identification or classification symbol.
- Organic, chemical, or metallic content.
- Compactness (dense or loose).
- Consistency.

- Cohesiveness (ability to hold together without cementation).
- Dry strength.
- Source (residual or transported).

SOIL-DESCRIPTION EXAMPLE

A complete description with proper classification symbols conveys more to the user of the data than the symbol of any isolated portion of the description. An example of a soil description, using the sequence and considering the properties, is—

- Dark brown to white.
- Coarse-grained (maximum particle size 3 inches; estimated 60 percent gravel, 36 percent sand, and 4 percent fines passing through the No. 200 sieve).
- Poorly graded (gap-graded, insufficient fine gravel).
- Subrounded to rounded gravel particles.
- Nonplastic.
- Predominantly gravel.
- Considerable sand and a small amount of nonplastic fines (silt).
- GP (identification symbol).
- Slightly calcareous, no dry strength, dense in the undisturbed state.

FIELD-IDENTIFICATION TESTS

The following tests produce observations that pertain to the USCS and permit field identification as well as classification. Tests appropriate to the given soil sample should be made. Some tests appear to yield duplicate results. The purpose of these tests is to get the best possible identification in the field. Thus, if a simple visual examination will define the soil type, only one or two of the other tests have to be made to verify the identification. When the results from a test are inconclusive, some of the similar tests should be tried to establish the best identification.

*Figure 2-3, pages 2-1*6 *and 2-17*, gives the suggested sequence of tests for identifying and classifying a soil sample using the hasty field procedures described in the following paragraphs.

VISUAL TEST

This test should establish the color, grain sizes, grain shapes of the coarse-grained portion, some idea of the gradation, and some properties of the undisturbed soil.

Determine the color, grain size, and grain shape of the material and estimate, if possible, the grain-size distribution by visual examination. The following paragraphs provide methods and information concerning identification of these properties.

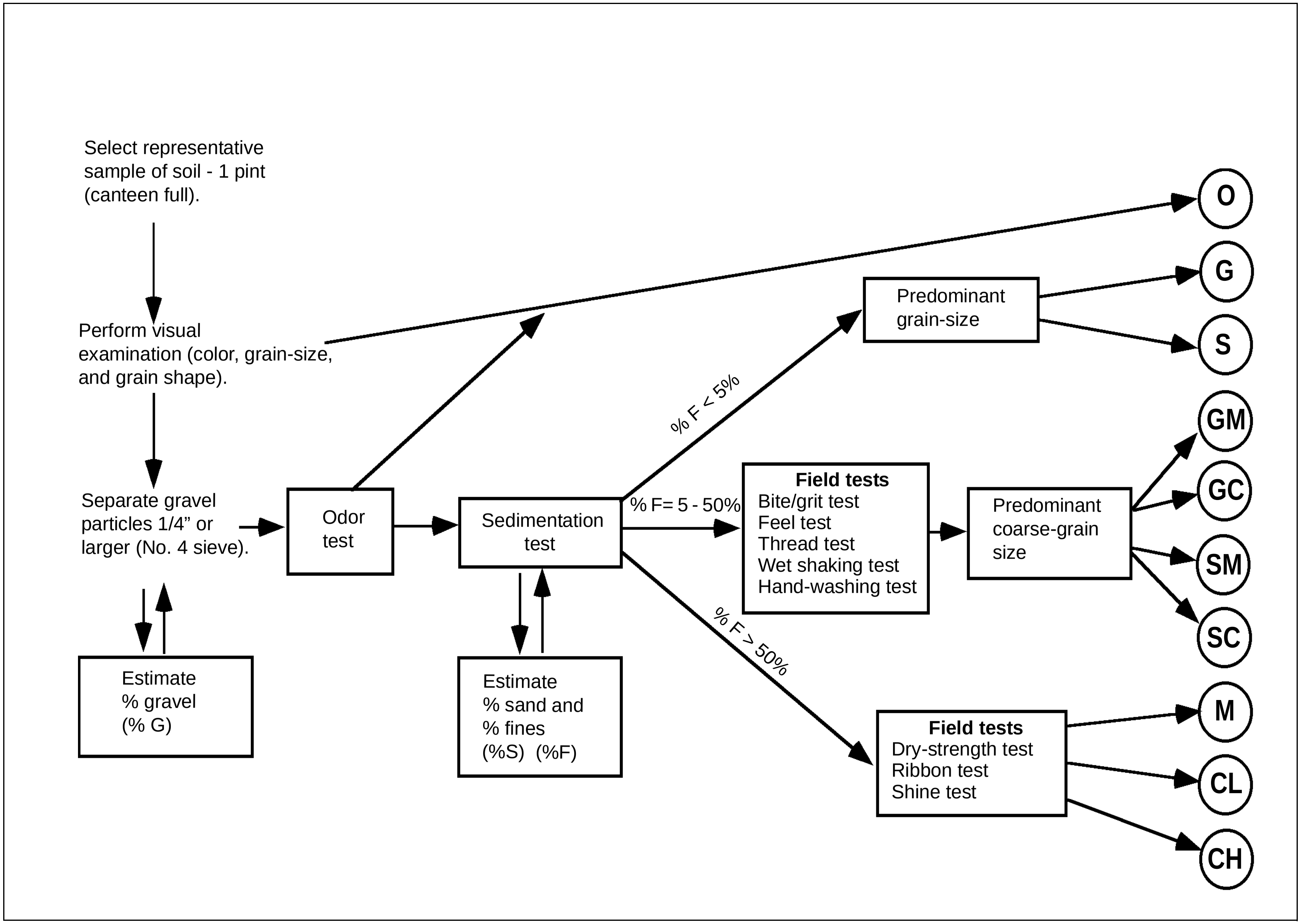


Figure 2-3. Suggested procedures for hasty field identification

1. Perform a visual examination of the sample.
   - a. Color.
   - b. Grain size.
   - c. Grain shape.
   - d. Contents— leaves, grass, and other possible organic material.

2. Separate the gravel.
   - a. Remove from the sample all particles larger than 1/4 inch in diameter (No. 4 sieve).
   - b. Estimate the % G.

3. Perform the odor test.
   - a. Heat the sample (less gravel) with a match or open flame.
   - b. If the odor becomes musty or foul smelling, there is a strong indication that organic material is present.

4. Perform the sedimentation test to determine the % S.
   - a. Place approximately 1 inch of the sample (less gravel) in a glass jar.
   - b. Mark the depth of the sample with a grease pencil.
   - c. Cover the sample with 5 inches of water with at least 1 inch space to the top of the jar.
   - d. Cover and shake the mixture for 3 to 4 minutes.
   - e. Place on a flat surface and allow sand particles to settle for 30 seconds.
   - f. Compare the settled material after 30 seconds to the grease-pencil mark and estimate the percent that has settled.
   - g. Determine % S for the overall sample.
     % S = (% settled) x (100% - % G)
   - h. Determine % F for the overall sample.
     % F = 100% - (% S + % G)

*5. Perform the bite or grit test. Place a pinch of the sample between the teeth and bite.
- If the sample feels gritty, the sample is silt (M).
- If the sample feels floury, the sample is clay (C).

*6. Perform the feel test. Rub a portion of dry soil over a sensitive part of the skin, such as the inside of the wrist.
- If the feel is harsh and irritating, the sample is silt (M).
- If the feel is smooth and floury, the sample is clay (C).

*7. Perform the roll or thread test.
   - a. Form a ball of moist soil (marble size).
   - b. Attempt to roll the ball into a thread 1/8 inch in diameter.
     - If a thread is easily obtained, it is clay (C).
     - If a thread cannot be obtained, it is silt (M).

*8. Perform the wet shaking test.
   - a. Place the pat of moist (not sticky) soil in the palm of your hand (the volume is about 1/2 cu in).
   - b. Shake the hand vigorously and strike it against the other hand.
   - c. Observe how rapidly water rises to the surface.
     - If it is fast, the sample is silty (M).
     - If there is no reaction, the sample is clayey (C).

*9. Perform the breaking or dry-strength test.
   - a. Form a moist pat 2 inches in diameter by 1/2 inch thick.
   - b. Allow it to dry with low heat.
   - c. Place the dry pat between the thumb and index finger only and attempt to break it.
     - If breakage is easy, it is a silt (M).
     - If breakage is difficult, it is a clay of low plasticity (CL).
     - If breakage is impossible, it is a clay of high plasticity (CH).

*10. Perform the ribbon test.
   - a. Form a cylinder of moist soil, approximately cigar shape and size.
   - b. Flatten the cylinder over the index finger with the thumb, attempting to form a ribbon 8 to 9 inches long, 1/8 to 1/4 inch thick, and 1 inch wide.
     - If 8 to 9 inches is obtained, it is (CH).
     - If 3 to 8 inches is obtained, it is (CL).
     - If less than 3 inches is obtained, it is silt (M).

*11. Perform the shine test. Draw a smooth surface, such as a knife blade or a thumbnail, over a pat of slightly moist soil.
- If the surface becomes shiny and lighter in texture, the sample is a highly plastic clay (CH).
- If the surface remains dull, the sample is a low plasticity clay (CL).
- If the surface remains very dull or granular, the sample is silt or sand (M).

***These tests are conducted only with material that passes the No. 40 sieve.**

Figure 2-3. Suggested procedures for hasty field identification (continued)

Color

Color helps in distinguishing between soil types and, with experience, aids in identifying the particular soil type. Color may also indicate the presence of certain chemicals or impurities. Color often varies with the soil's moisture content. Thus, the moisture content at the time of color identification should be included. Some of the more familiar color properties are stated below. Colors in general become darker as the moisture content increases and lighter as the soil dries. Some fine-grained soil (OL and OH) with dark, drab shades of brown or gray (including almost black) contain organic colloidal matter. In contrast, clean, bright shades of gray, olive green, brown, red, yellow, and white are associated with inorganic soils. Gray-blue or gray-and-yellow mottled colors frequently result from poor drainage. Red, yellow, and yellowish-brown colors result from the presence of iron oxides. White to pink may indicate considerable silica, calcium carbonate, or aluminum compounds.

Grain Size

The maximum particle size of each sample considered should always be estimated if not measured. This establishes the gradation curve's upper limit. Gravels range down to the size of peas. Sands start just below this size and decrease until the individual grains are just distinguishable by the naked eye. The eye can normally see individual grains about 0.07 millimeter in size, or about the size of the No. 200 sieve. Silt and clay particles, which are smaller than sands, are indistinguishable as individual particles.

Grain Shape

While the sample is examined for grain sizes, the shapes of the visible particles can be determined. Sharp edges and flat surfaces indicate an angular shape; smooth, curved surfaces indicate a rounded shape. Particles may not be completely angular or completely rounded. These particles are called subangular or subrounded, depending on which shape predominates.

Grain-Size Distribution

A laboratory analysis must be performed to determine accurate distribution; however, an approximation can be made during the visual examination. Perform the following steps to obtain the grain-size distribution:

Step 1. Separate the larger grains (+4 gravel or coarse grains and some sand particles) from the remainder of the soil by picking them out individually.

Step 2. Examine the remainder of the soil, and estimate the proportion of visible individual particles (larger than the No. 200 sieve) and the fines (smaller than the No. 200 sieve).

Step 3. Convert these estimates into percentages by weight of the total sample. If the fines exceed 50 percent, the soil is considered fine-grained (M, C, or O). If the coarse material exceeds 50 percent, the soil is coarse-grained (G or S).

Step 4. Examine coarse-grained soil for gradation of particle sizes from the largest to the smallest. A good distribution of all sizes means the soil is well graded (W). Overabundance or lack of any size means the material is poorly graded (P).

Step 5. Estimate the percentage of the fine-grained portion of the coarse-grained soil.

If less than 5 percent (nonplastic fines) of the total, the soil may be classified either as a GW, GP, SW, or SP type, depending on the other information noted above. If the fine-grained portion exceeds 12 percent, the soil is either M or C and requires further testing to identify.

Fine-grained portions, between 5 and 12 percent (nonplastic fines or fines not interfering with free drainage) are borderline and require double symbols such as GW-GM or SW-SM.

Fine-grained soils (M, C, or O) require other tests to distinguish them further. Grain-size distribution of fine portions is not normally performed in field identification. However, if necessary, make an approximation by shaking the fine portions in a jar of water and allowing the material to settle. The material will settle in layers of different sizes from which the proportion can be estimated. Gravel and sand settle into a much denser mass than either clay or silt settles.

Undisturbed Soil Properties

Using characteristics determined up to this point, it is possible to evaluate the soil as it appeared in place. Gravels or sands can be described qualitatively as loose, medium, or dense. Clays may be hard, stiff, or soft. The ease of difficulty with which the sample was removed from the ground is a good indicator. Soils that have been cultivated or farmed can be further evaluated as loose or compressible. Highly-organic soils can be spongy or elastic. In addition, the soil's moisture content influences the in-place characteristics. This condition should be recognized and reported with the undisturbed soil properties.

ODOR TEST

Organic soils (OL and OH) usually have a distinctive, musty, slightly offensive odor. The odor can help identify such materials. This odor is especially apparent from fresh samples but becomes less pronounced as the sample is exposed to air. The odor can be made stronger by heating a wet sample.

SEDIMENTATION TEST

From the visual-examination tests, it is easy to approximate the proportions of sand and gravel in a soil by spreading the dry sample out on a flat surface and separating the gravel particles by hand. Separating the fines from the sand particles however, is more difficult, although just as important. Smaller particles settle through water at a slower rate than large particles.

To perform this test, place about 1 inch of the fine fraction of soil (passing the No. 4 sieve) in a transparent cup or jar. Mark the height of the sample with a grease pencil. Place about 5 inches of water into the jar, ensuring that at least 1 inch is still remaining above the water line to the top of the jar. Cover and shake the water and soil mixture for 3 to 4 minutes. Place the jar on a flat surface. After 30 seconds compare the level of material that settled to the bottom with the height of the original sample (grease pencil line). This comparison should indicate the proportion of sand within the mixture as indicated in *Table 2-3, page 2-20*. For example, if the level of the settled material comes halfway from the bottom of the jar to the grease-pencil line after 30 seconds, then it can be estimated that the amount of sand in this fraction of the soil is about 50 percent.

Table 2-3. Sedimentation test

Approximate Time of Settlement Through 5 Inches of Water	Grain Diameter (mm)	Differentiates
2 seconds	0.400	Coarse sand, fine sand
30 seconds	0.072	Sand, fines
10 minutes	0.030	Coarse silt, fine sand
1 hour	0.010	Silt, clay

Once this determination is complete, estimating the amount of sand and fines of the overall sample is made easy, providing the approximate percentage of gravel was obtained from the visual examination. Use the following equation to determine the percent of sand for the overall sample:

$$\%\,S = \mathit{percent\ of\ sand\ in\ jar} \times (100\% - \%G)$$

where—

$\%\,S = \mathit{percent\ of\ sand}$

$\%\,G = \mathit{percent\ of\ gravel}$

Using the information from the example in the previous paragraph and given the percent gravel as 40 percent, this equation would yield the following information:

$$\%S = 50\% \times (100\% - 40\text{-}\%)$$

$$\%S = 30\%$$

Additionally, once this information is obtained, the percent of fines (% F) can be determined by subtracting the percent of gravel and the percent of sand from 100 percent as follows:

$$\%F = 100\% - (40\% + 30\%)$$

$$\%F = 30\%$$

The most important use of the sedimentation test is to differentiate the coarse (0.072 millimeter) fraction from the fine fraction of a soil. Since all of the particles of soil larger than 0.072 millimeter will have settled to the bottom of the cup or jar 30 seconds after the mixture has been agitated, it follows that the particles still remaining in suspension are fines. Alternatively, if the water containing the suspended fines is carefully poured into another jar 30 seconds after agitation, if more water is added to the cup or jar containing the coarse fraction, and if the procedure is repeated until the water-soil mixture becomes clear 30 seconds after mixing, then the cup or jar will contain the coarse fraction of soil only, and the jar containing the suspension will hold the fines. If the water can be wicked or evaporated off, the relative amounts of fines and sand can be determined fairly accurately. Otherwise, a direct measurement of the settled-out fines can be obtained as a guide. Thus, in a sense, the test acts like the No. 200 sieve.

Difficulty may be encountered with many clay soils because the clay particles often form small lumps (flocculate) that will not break up in water. Usually this condition can be detected by examining the coarse fraction of the soil after several repetitions of the test. If substantial amounts of clay are still present, the sand will feel slippery and further mixing and grinding with a good stick will be necessary to break up the lumps.

BITE OR GRIT TEST

The bite or grit test is a quick and useful test in identifying sand, silt, or clay. A small pinch of soil is ground lightly between the teeth.

The results of this test indicate the following:

- Sandy soils. The sharp, hard particles of sand grate very harshly between the teeth and are highly objectionable. This is true even of the fine sands.
- Silty soils. Silt grains are so much smaller than sand grains that they do not feel nearly as harsh between the teeth. They are not particularly gritty, although their presence is still quite unpleasant and easily detected.
- Clayey soils. Clay grains are not gritty, but feel smooth and powdery like flour between the teeth. Dry lumps of clayey soils stick when lightly touched with the tongue.

FEEL TEST

This is a general-purpose test requiring considerable experience and practice before reliable results can be expected. Its use will increase with growing familiarity with soils. Consistency and texture are two characteristics that can be determined.

Consistency

The natural moisture content is an indicator of the soil drainage which may affect this characteristic. For the consistency test, squeeze a piece of undisturbed soil between the thumb and forefinger to determine its consistency. Consistency is described by such terms as hard, stiff, brittle, friable, sticky, plastic, and soft. Remold the soil by working it between the hands and observe the results. This can indicate the natural water content. Clays that become fluid on remolding are probably near their LL. If they remain stiff and crumble when reworked, they are probably below their PL.

Texture

This term is applied to the soil's fine-grained portion and refers to the degree of fineness and uniformity. Rub a portion of the soil between the fingers, observe the texture, and describe it as floury, smooth, gritty, or sharp. To increase sensitivity, rub the soil on a more tender skin area, such as the inside of the wrist. Typical results are similar to the bite test—sand feels gritty; silts, if dry, dust readily and feel soft and silky to the touch; and clays powder only with difficulty but feel smooth and gritless like flour.

ROLL OR THREAD TEST

This test is performed only on the material passing the No. 40 sieve. Mix a representative portion of the sample with water until it can be molded or shaped without sticking to the fingers. This moisture content is referred to as being just below the sticky limit.

Prepare a nonabsorbent rolling surface by placing a sheet of glass or heavy waxed paper on a flat or level support, shape the sample into an elongated cylinder, and roll the prepared soil cylinder on the surface rapidly into a thread about 1/8 inch in diameter. The technique is shown in *Figure 2-4*.

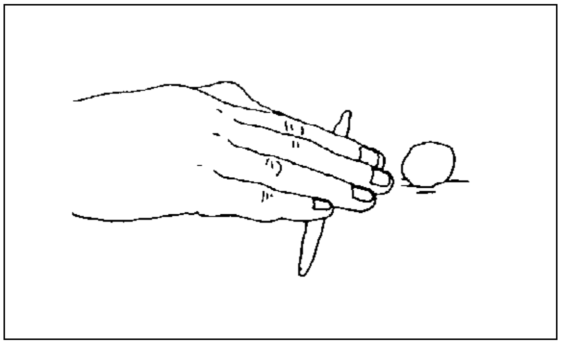


Figure 2-4. Roll or thread test

If the moist soil rolls into a thread, it has some plasticity. The number of times it can be rolled into a thread without crumbling is a measure of the soil's degree of plasticity. Materials that cannot be rolled in this manner are nonplastic or have a very low plasticity.

The results of this test indicate the following:

- If the soil can be molded into a ball or cylinder and deformed under very firm finger pressure without crumbling or cracking, it is of high plasticity (CH).
- If the soil can be molded but cracks or crumbles without breaking up, it is of low plasticity (CL, ML, or MH).
- If the soil forms a soft, spongy ball or thread when molded, it is organic material (OL or OH), also peat.
- If the soil cannot be rolled into a thread at any moisture content, it is nonplastic soil (ML or MH).

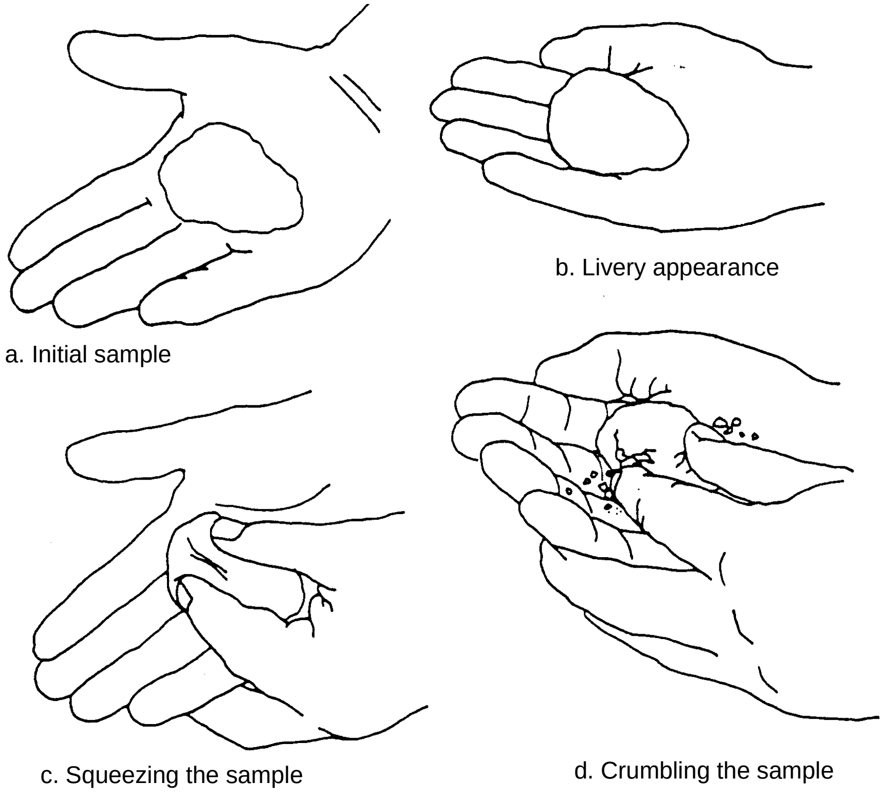
From the thread test, the cohesiveness of the material near the PL may also be described as weak, firm, or tough. The higher the soil is on the plasticity chart, the stiffer the threads are as they dry out and the tougher the lumps are if the soil is remolded after rolling.

WET SHAKING TEST

Perform the wet shaking test only on the material passing the No. 40 sieve. For this test, moisten enough material to form a ball of material about 3/4 inch in diameter. This sample should be just wet enough that the soil will not stick to the fingers when remolding or just below the sticky limit.

Smooth the soil pat in the palm of the hand with a knife blade or a small spatula. Shake it horizontally and strike the back of the hand vigorously against the other hand. The soil reacts to this test when, on shaking, water comes to the surface of the sample, producing a smooth, shiny appearance. This appearance is frequently described as livery (see *Figure 2-5a* and *2-5b*). Squeeze the sample between the thumb and forefinger of the other hand and the surface water will quickly disappear. The surface becomes dull (see *Figure 2-5c*) and the material becomes firm, resisting deformation. Cracks occur as pressure is continued, with the sample finally crumbling like a brittle material (see *Figure 2-5d*).

The vibration caused by the shaking of the soil sample tends to reorient the soil grains, decrease the voids, and force water within these voids to come to


a. Initial sample

b. Livery appearance

c. Squeezing the sample

d. Crumbling the sample

Figure 2-5. Wet shaking test

the surface. Pressing the sample between the fingers tends to disarrange the soil grains, increase the voids space, and draw the water into the soil. If the water content is still adequate, shaking the broken pieces will cause them to liquefy again and flow together, and the complete cycle may be repeated. This process can occur only when the soil grains are bulky in shape and noncohesive in character.

Very fine sands and silts fall into this category and are readily identified by the wet shaking test. Since it is rare that fine sands and silts occur without some amount of clay mixed with them, there are varying reactions to this test. Even a small amount of clay tends to greatly retard this reaction.

The results of this test indicate the following:

- A rapid reaction to the shaking test is typical of nonplastic, fine sands and silts.
- A sluggish reaction indicates slight plasticity (such as might be found from a test of some organic silts) or silts containing a small amount of clay.
- No reaction at all to this test does not indicate a complete absence of silt or fine sand.

BREAKING OR DRY-STRENGTH TEST

This test is performed only on the material passing the No. 40 sieve. It is used to measure the soil's cohesive and plastic characteristics. The test distinguishes between the clayey (C) and silty (M) soils.

Separate the selected soil sample on the No. 40 sieve and prepare a pat of soil about 2 inches in diameter and 1/2 inch thick by molding it in a wet, plastic state. Natural samples may be found in pats that are of the proper size but that may yield incorrect results. This is due to the variations in the natural drying and compaction processes. If natural samples are used, the results must be treated as approximations and verified later.

Allow the pat to dry completely, then grasp it between the thumbs and forefingers of both hands and attempt to break it. See *Figure 2-6* for the proper way to hold the pat. If the pat breaks, powder it by rubbing it between the thumb and forefinger of one hand.

The results of this test indicate the following:

- If the pat cannot be broken nor powdered by finger pressure, it is very highly-plastic soil (CH).
- If the pat can be broken with great effort, but cannot be powdered, it is highly-plastic soil (CL).
- If the pat can be broken and powdered with some effort, it is medium-plastic soil (CL).
- If the pat breaks easily and powders readily, it is slightly-plastic soil (ML, MH, or CL).
- If the pat has little or no dry strength and crumbles or powders when picked up, it is nonplastic soil (ML or MH) or (OL or OH).

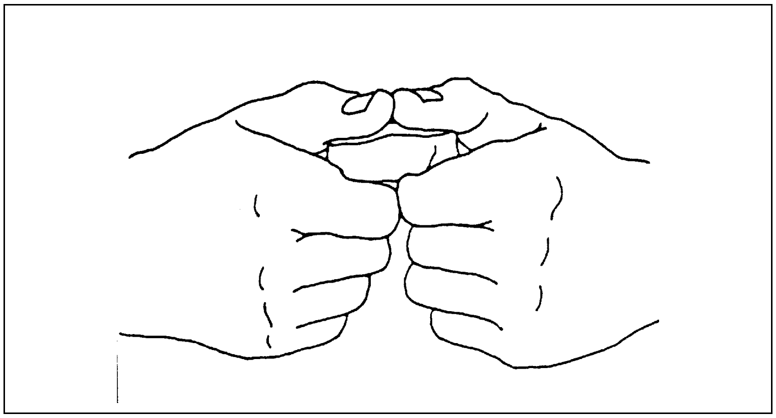


Figure 2-6. Breaking or dry-strength test

NOTE: Dry pats of highly-plastic clays often display shrinkage cracks. Breaking the sample along such a crack gives an indication of only a very small part of the soil's true dry strength. It is important to distinguish between a break along such a crack and a clean, fresh break that indicates the soil's true dry strength.

RIBBON TEST

The ribbon test is also performed only on material passing the No. 40 sieve. The sample is prepared as for the roll or thread test until the moisture content is just below the sticky limit. This test and the roll test complement each other and give a clearer picture of the soil.

Form a roll of soil 1/2 to 3/4 inch in diameter and 3 to 5 inches long. Lay the roll across the palm of one hand (palm up). Starting at one end, squeeze the roll between the thumb and forefinger over the edge of the hand to form a flat, unbroken ribbon about 1/8 to 1/4 inch thick. Allow the ribbon as formed to hang free and unsupported (see *Figure 2-7, page 2-26*). Continue squeezing and handling the roll carefully to form the maximum length of ribbon that can be supported only by the cohesive properties of the soil.

The results of this test indicate the following:

- If the sample holds together for a length of 8 to 10 inches without breaking, it is considered to be plastic having a high LL (CH).
- If the soil can be ribboned only with difficulty into 3- to 8-inch lengths, it is of low plasticity (CL).
- If the soil cannot be ribboned, it is nonplastic (ML) or (MH).

SHINE TEST

The shine test is another means of determining the soil's plasticity. A slightly moist or dry piece of highly-plastic clay (CH) produces a definite shine when rubbed with a fingernail or a smooth, metal surface such as a knife blade. Lean clay remains dull after this treatment (CL).

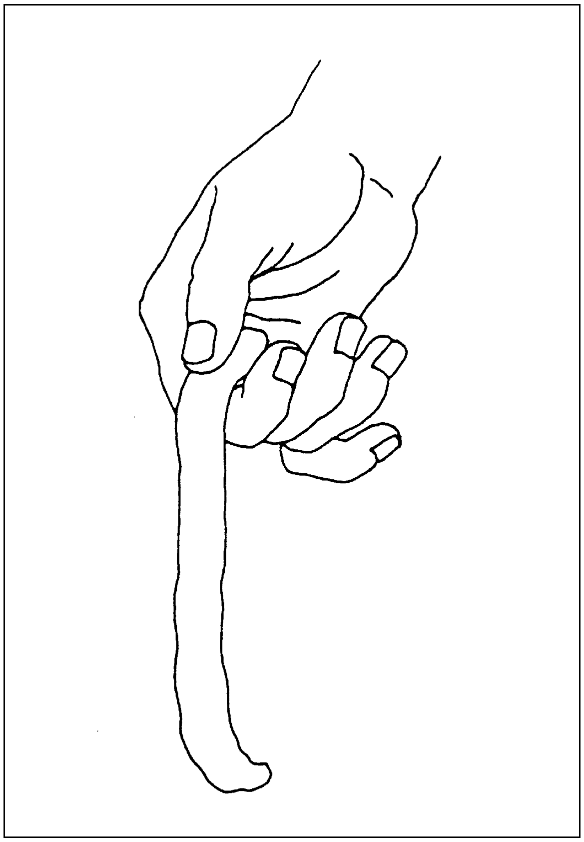


Figure 2-7. Ribbon test

SECTION II. SOIL SURVEYS AND SAMPLING

Surveying soil conditions at proposed military construction sites provides information about the nature, extent, and condition of soil layers; the position of the water table and drainage characteristics; and the sources of possible construction materials. A soil survey is vital to planning and executing military construction operations. The information obtained from a soil survey is the basis for a project's success.

TYPES OF SOIL SURVEYS

A soil survey consists of gathering soil samples for examining, testing, and classifying soils and developing a soil profile. The two types of soil surveys commonly associated with military construction are the hasty and deliberate surveys.

A hasty survey—made either under expedient conditions or when time is very limited—is a type of survey that usually accompanies a preliminary site analysis. A deliberate survey is made when adequate equipment and time are available. When possible, a hasty survey should be followed by a deliberate survey.

HASTY SURVEY

A hasty survey should be preceded by as careful a study of all available sources of information as conditions permit. If aerial observation is possible, a trained person may observe soil conditions in the proposed construction area. This gives a better overall picture, which is often difficult to secure at ground level because important features may be hidden in rough or wooded terrain. Rapid ground observation along the proposed road location or at the proposed airfield site also yields useful information. The soil profile may be observed along a stream's natural banks, eroded areas, bomb craters, and other exposed places. As construction proceeds, additional soil studies will augment the basic data gained through the hasty survey and will dictate necessary modifications in location, design, and construction.

DELIBERATE SURVEY

A deliberate survey does not dismiss the fact that the time factor may be important. Therefore, the scope of a deliberate survey may be limited in some cases. A deliberate survey is often performed while topographical data is being obtained so that the results of the soil survey may be integrated with other pertinent information. The principal method of exploration used in soil surveys for roads, airfields, and borrow areas is soil samples obtained either by using hand augers or by digging a test pit. Other methods that may be used are power-driven earth augers, sounding rods, or earthmoving equipment under expedient conditions to permit a hasty approach to the underlying soil.

OBJECTIVE OF A SOIL SURVEY

The objective of a soil survey, whether hasty or deliberate, is to explore and gather as much information of engineering significance as possible about the subsurface conditions of a specified area. The explorations are conducted to determine the—

- Location, nature, and classification of soil layers.
- Condition of soils in place.
- Drainage characteristics.
- Groundwater and bedrock.

LOCATION, NATURE, AND CLASSIFICATION OF SOIL LAYERS

Information regarding the location, nature, and classification of soil layers is required for adequate and economical earthwork and foundation design of a structure. By classifying the soils encountered, a prediction can be made as to the extent of problems concerning drainage, frost action, settlement, stability, and similar factors. An estimate of the soil characteristics may be obtained by field observations, but samples of the major soil types and the less-extensive deposits that may influence design should be obtained for laboratory testing.

CONDITION OF SOILS IN PLACE

Soil conditions, such as moisture content and density of a soil in its natural state, play an important part in design and construction. The moisture content may be so high in some soils in place that the selection of another site should be considered for an airfield or other structure. If the natural soil is dense enough to meet the required specifications, no further compaction of the subgrade is required. Very compact soils in cut sections may be difficult to excavate with ordinary tractor-scraper units, so scarifying or rooting may be needed before excavation.

DRAINAGE CHARACTERISTICS

Drainage characteristics in both surface and subsurface soils are controlled by a combination of factors, such as the void ratio, soil structure and stratification, the temperature of the soil, the depth to the water table, the height of capillary rise, and the extent of local disturbances by roots and worms. Remolding a soil also may change its drainage properties. Coarse-grained soils have better internal drainage than fine-grained soils. Observations of the soil should be made in both disturbed and undisturbed conditions.

GROUNDWATER AND BEDROCK

All structures must be constructed at such an elevation that they will not be adversely affected by the groundwater table. The grade line can be raised or the groundwater table lowered when a structure may be adversely affected by capillary rise or by the groundwater table itself. Bedrock within the excavation depth tremendously increases the time and equipment required for excavation. If the amount is very extensive, it may be necessary to change the grade or even the site location.

SOURCES OF INFORMATION

There are many sources of information available to soils engineers, and they should all be used to the fullest extent to eliminate as much detailed investigation as possible. These sources can be used to locate small areas within a large general area that are suitable for further investigation. Field information requires general observation of road cuts, stream banks, eroded slopes, earth cellars, mine shafts, and existing pits and quarries. A field party must obtain reliable data rapidly, since final decisions on site selection are based on field observations. These sources include—

- Intelligence reports.
- Local inhabitants.
- Maps.
- Aerial photographs.

INTELLIGENCE REPORTS

Intelligence reports that include maps and studies of soil conditions usually are available for areas in which military operations have been planned. Among the best and most comprehensive of these are the National Intelligence Surveys and Engineer Intelligence Studies. These reports are a source of information on geology, topography, terrain conditions, climate and weather conditions, and sources of construction materials.

LOCAL INHABITANTS

Local inhabitants (preferably trained observers), such as contractors, engineers, and quarry workers, may provide information to supplement intelligence reports or provide information about areas for which intelligence reports are unavailable. Data obtained from this source may include the possible location of borrow material, sand and gravel deposits, and peat or highly organic soils, as well as information on the area's climate and topography.

MAPS

Maps provide valuable information, especially when planning a soil survey. Maps showing the suitability of terrain for various military purposes, prepared by enemy or friendly foreign agencies, may be useful. Some of the maps that provide different types of information about an area under investigation are—

- Geological maps.
- Topographic maps.
- Agricultural soil maps.

Geological Maps

Geological maps and brief descriptions of regions and quadrangles are available from the *US Geological Survey, 1200 South Eads Street, Arlington, Virginia 22202*. Generally, the smallest rock unit mapped is a formation, and a geological map indicates the extent of the formation by means of symbolic letters, colors, or patterns. Letter symbols on the map also indicate the locations of sand and gravel pits. The rear of the map sheet sometimes has a brief discussion entitled "Mineral Resources" that describes the location of construction materials.

Topographic Maps

Ordinary topographic maps may be helpful in estimating soil conditions, but they give only a generalized view of the land surface, especially when the contour interval is 20 feet or more. Therefore, they should be used with geological maps. An inspection of the drainage pattern and slopes can provide clues to the nature of rocks, the depth of weathering, soil characteristics, and drainage. For example, sinkholes may indicate limestone or glacial topography; hills and mountains with gently rounded slopes usually indicate deeply weathered rocks; and parallel ridges are commonly related to steeply folded, bedded rock with hard rock along the ridges. Features such as levees, sand dunes, beach ridges, and alluvial fans can be recognized by their characteristic shapes and geographic location.

Agricultural Soil Maps

Agricultural soil maps and reports are available for many of the developed agricultural areas of the world. These studies are concerned primarily with surface soils usually to a depth of 6 feet and are valuable as aids in the engineering study of surface soils. For example, if the same soil occurs in two different areas, it can be sampled and evaluated for engineering purposes in one area, and the amount of sampling and testing can then be reduced in the second area. Maps are based on field survey factors that include the careful study of the soil horizons in test pits, highway and railway cuts, auger borings, and other exposed places. Information on topography, drainage, vegetation, temperature, rainfall, water sources, and rock location may be found in an agricultural report. Soil usually is classified according to its texture, color, structure, chemical and physical compositions, and morphology.

AERIAL PHOTOGRAPHS

Aerial photographs may be used to predict subsurface conditions and previous explorations for nearby construction projects. The photographs aid in delineating and identifying soils based on the recognition of typical patterns formed under similar conditions of soil profile and weathering. Principal elements that can be identified on a photograph and that provide clues to the identification of soils to a trained observer are—

- Landforms.
- Slopes.
- Drainage patterns.
- Erosion patterns.
- Soil color.
- Vegetation.
- Agricultural land use.

Landforms

The landform or land configuration in different types of deposits is characteristic and can be identified on aerial photographs. For example, glacial forms such as moraines, kames, eskers, and terraces are readily identifiable. In desert areas, characteristic dune shapes indicate areas covered by sands subject to movement by wind. In areas underlaid by flat-lying, soluble limestone, the air photograph typically shows sinkholes.

Slopes

Prevailing ground slopes usually represent the soil’s texture. Steep slopes are characteristic of granular materials, while relatively flat and smoothly rounded slopes may indicate more plastic soils.

Drainage Patterns

A simple drainage pattern is frequently indicative of pervious soils. A highly integrated drainage pattern frequently indicates impervious soils, which in turn are plastic and lose strength when wet. Drainage patterns also reflect the underlying rock structure. For example, alternately hard and soft layers of rock cause major streams to flow in valleys cut in the softer rock.

Erosion Patterns

Erosion patterns provide information from the careful study of gullies. The cross section or shape of a gully is controlled primarily by the soil’s cohesiveness. Each abrupt change in grade, direction, or cross section indicates a change in the soil profile or rock layers. Short, V-shaped gullies with steep gradients are typical of cohesionless soils. U-shaped gullies with steep gradients indicate deep, uniform silt deposits such as loess. Cohesive soils generally develop round, saucer-shaped gullies.

Soil Color

Soil color is shown on photographs by shades of gray, ranging from white to black. Soft, light tones generally indicate pervious, well-drained soils. Large, flat areas of sand are frequently marked by uniform, light-gray tones; a very flat appearance; and no natural surface drainage. Clays and organic soils often appear as dark-gray to black areas. In general, sharp changes in the tone represent changes in soil texture. These interpretations should be used with care.

Vegetation

Vegetation may reflect surface soil types, although its significance is difficult to interpret because of the effects of climate and other factors. To interpreters with local experience, both cultivated and natural vegetation cover may be reliable indicators of soil type.

Agricultural Land Use

Agricultural land use also facilitates soil identification. For example, orchards require well-drained soils, and the presence of an orchard on level ground would imply a sandy soil. Wheat is frequently grown on loess-type soils. Rice is usually found in poorly draining soils underlain by impervious soils, such as clay. Tea grows in well-draining soils.

FIELD INVESTIGATIONS

A field investigation consists of the sampling operation in the field.

SAMPLING METHODS

The extent and methods of sampling used depend on the time available. Military engineers obtain samples from—

- The surface.
- Excavations already in existence.
- Test pits.
- Auger borings or holes.

In a hasty survey, the number of test pits and test holes is kept to a minimum by using existing excavations for sampling operations. In a deliberate survey, where a more thorough sampling operation is conducted, auger borings or holes are used extensively and are augmented by test pits, governed by the engineer’s judgment. The following paragraphs describe this method of sampling.

Test Pit

A test pit is an open excavation large enough for a person to enter and study the soil in its undisturbed condition. This method provides the most satisfactory results for observing the soil’s natural condition and collecting undisturbed samples. The test pit usually is dug by hand. Power excavation by dragline, clamshell, bulldozer, backhoe, or a power-driven earth auger can expedite the digging, if the equipment is available. Excavations below the groundwater table require pneumatic caissons or the lowering of the water table. Load-bearing tests can also be performed on the soil in the bottom of the pit. Extra precaution must be taken while digging or working in a test pit to minimize potentially fatal earth slides or cave-ins. The walls must be supported or sloped to prevent collapse. A good rule of thumb for sloping the pit sides is to use a 1:1 slope. For additional guidance on excavation, refer to Engineering Manual (EM) 385-1-1, Section 23B.

Auger Boring

A hand auger is most commonly used for digging borings. It is best suited to cohesive soils; however, it can be used on cohesionless soils above the water table, provided the diameter of the individual aggregate particles is smaller than the bit clearance of the auger. The auger borings are principally used at shallow depths. By adding pipe extensions, the earth auger may be used to a depth of about 30 feet in relatively soft soils. The sample is completely disturbed but is satisfactory for determining the soil profile, classification, moisture content, compaction capabilities, and similar soil properties.

Table 2-4 shows methods of underground exploration and sampling in a condensed form.

Table 2-4. Methods of underground exploration and sampling

Method	Materials in Which Used	Method of Advancing Hole	Sampling Method	Value for Foundation Purposes
Auger boring	Cohesive soils and cohesionless soils above ground-water elevation	Augers rotated until filled with soil and then removed to surface	Samples recovered from materials brought up on augers	Satisfactory for highway exploration at shallow depths
Well drilling	All soils, rock, and boulders	Churn drilling with power machines	Bailed sample of churned material or clay socket	Clay socket samples are dry samples. Bailed samples are of no value.
Rotary drilling	All soils, rock, and boulders	Rotating bits operating in a heavy, circulating liquid	Samples recovered from circulating liquid	Samples are of no value.
Test pits	All soils—lowering of groundwater may be necessary	Hand digging or power excavation	Samples taken by hand from original position in ground	Materials can be inspected in natural condition and place.

PREPARING SAMPLES

The location of auger holes or test pits depends on the particular situation. In any case, the method described in the following paragraphs locates the minimum number of holes. The completeness of the exploration depends on the time available. A procedure is described for road, airfield, and borrow-area investigations. Make soil tests on samples representing the major soil types in the area.

First, develop a general picture of the subgrade conditions. Conduct a field reconnaissance to study landforms and soil conditions in ditches and cuts. Techniques using aerial photographs can delineate areas of similar soil conditions. Make full use of existing data in agricultural spill maps for learning subsurface conditions.

Next, determine subgrade conditions in the area to be used for runway, taxiway, and apron construction. This usually consists of preliminary borings spaced at strategic points. Arbitrary spacing of these borings at regular intervals does not give a true picture and is not recommended. Using these procedures (especially the technique of identifying soil boundaries from aerial photographs) permits strategic spacing of the preliminary borings to obtain the most information with the least number of borings. In theater-of-operations (TO) cut areas, extend all holes 4 feet below the final subgrade elevation. In TO fill areas, extend all holes 4 feet below the natural ground elevation. These holes usually result in borings below the depth of maximum frost penetration (or thaw in permafrost areas). Where the above requirements do not achieve this result, extend the borings to the depth of maximum frost (or thaw in permafrost areas).

Obtain soil samples in these preliminary borings. After classifying these samples, develop soil profiles and select representative soils for detailed testing. Make test pits (or large-diameter borings) to obtain the samples needed for testing or to permit in-place tests. The types and number of samples required depend on the characteristics of the subgrade soils. In subsoil investigations in the areas of proposed pavement, include measurements of the in-place water content, density, and strength. Use these to determine the depth of compaction and the presence of any soft layers in the subsoil.

In borrow areas, where material is to be borrowed from adjacent areas, make holes and extend them 2 to 4 feet below the anticipated depth of borrow. Classify and test samples for water content, density, and strength.

Select material and subbase from areas within the airfield site and within a reasonable haul distance from the site. Exploration procedures for possible sources of select material and subbase are similar to those described for subgrades since the select material and subbase usually are natural materials (unprocessed). Test pits or large borings put down with power augers are needed in gravelly materials.

Base and pavement aggregates are materials that generally are crushed and processed. Make a survey of existing producers plus other possible sources in the general area. Significant savings can be made by developing possible quarry sites near the airfield location. This is particularly important in remote areas where no commercial producers are operating and in areas where commercial production is limited.

RECORDING SAMPLES

The engineer in charge of the soil survey is responsible for properly surveying, numbering, and recording each auger boring, test pit, or other investigation. Keep a log of each boring, showing the elevation (or depth below the surface) of the top and bottom of each soil layer, the field identification of each soil encountered, and the number and type of each sample taken. Include other information in the log that relates to the density of each soil, the changes in moisture content, the depth to groundwater, and the depth to rock. A typical boring log (as recorded on Department of Defense [DD] Form 2464) is shown in *Figure 2-8*.

OBTAINING REPRESENTATIVE SOIL SAMPLES

Planning the general layout determines the extent of the various soil types (vertically and laterally) within the zone where earthwork may occur. Large cuts and fills are the most important areas for detailed exploration. See Chapter 4 for procedures on obtaining soil or aggregate samples from a stockpile.

Place borings at high and low spots, in places where a soil change is expected, and in transitions from cut to fill. There is no maximum or minimum spacing requirement between holes; however, the number of holes must be sufficient to give a complete and continuous picture of the soil layers throughout the area of interest. As a general rule, the number of exploration borings required on a flat terrain with uniform soil conditions is less than in a terrain where the soil conditions change frequently.

Conduct exploration borings at the point of interest and locate them in a manner to get the maximum value from each boring. This may require exploration borings in the centerline as well as edges of runways or roads, but no specific pattern should be employed except perhaps a staggered or offset pattern to permit the greatest coverage. Exploration borings may be conducted at the edge of existing pavements, unless these pavements have failed completely. In this case, find the reason for the failure.

Purpose

Conduct exploration borings to—

- Obtain individual soil samples and a composite sample by investigating a borehole/test pit to a minimum depth of 4 feet.
- Prepare soil and moisture-content samples of each soil layer encountered for transportation.
- Record all information detailing the soils encountered, such as their location in the pit and the pit location within the excavation site, in a project log book.

Before beginning, ensure that digging is permitted at the testing site.

Steps

Perform the following steps to obtain a representative soil sample:

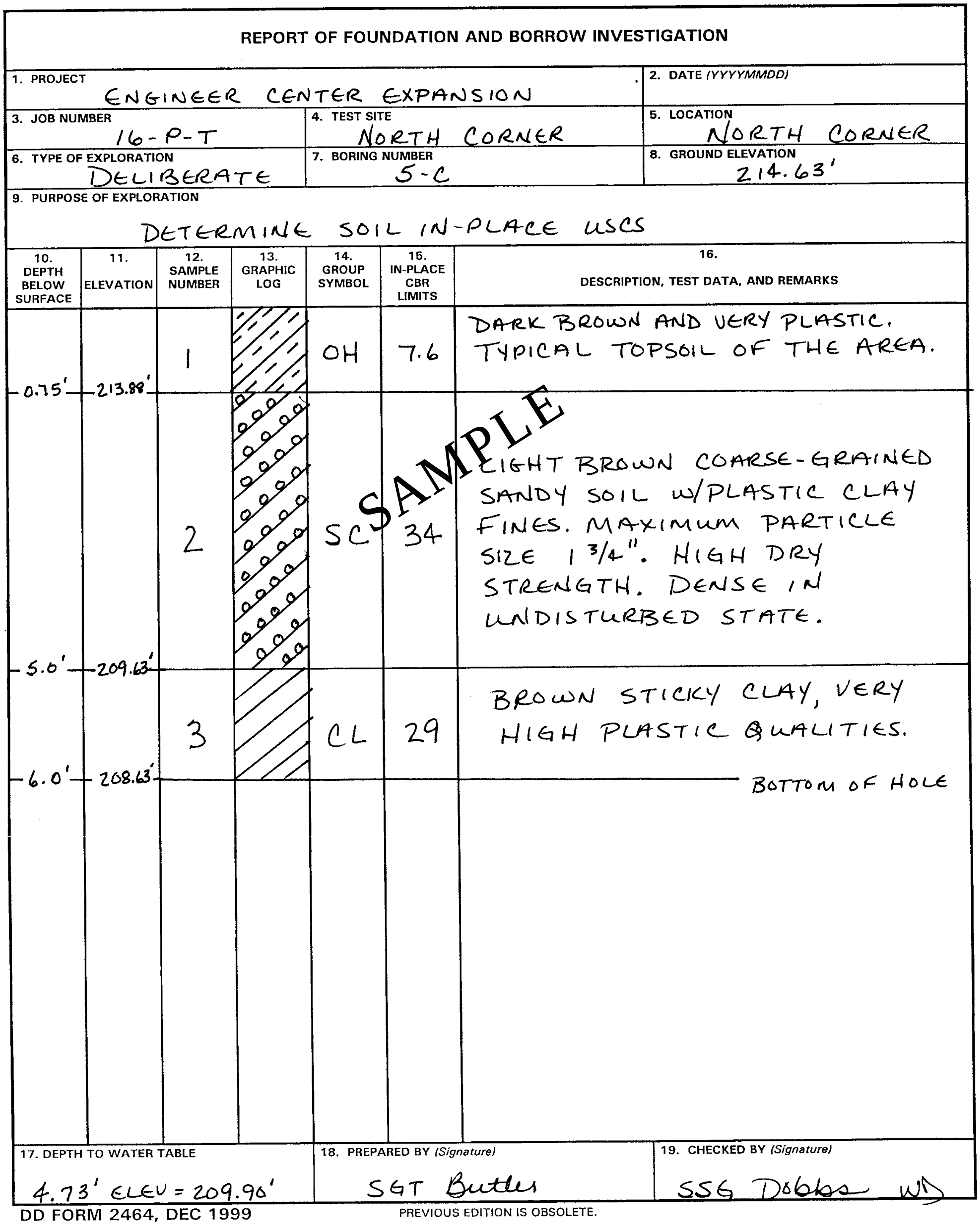
Step 1. Locate the boreholes or test pit. (The location of auger holes depends on the particular situation.)

a. Draw a site sketch recording the borehole’s location, elevation, azimuth, and distance from a benchmark or reference point.

b. Determine and record a number for each borehole and record it on the diagram.

Step 2. Dig the borehole or test pit.

a. Remove the overburden.



REPORT OF FOUNDATION AND BORROW INVESTIGATION

1. PROJECT ENGINEER CENTER EXPANSION		2. DATE *(YYYYMMDD)*
3. JOB NUMBER 16-P-T	4. TEST SITE NORTH CORNER	5. LOCATION NORTH CORNER
6. TYPE OF EXPLORATION DELIBERATE	7. BORING NUMBER 5-C	8. GROUND ELEVATION 214.63'
9. PURPOSE OF EXPLORATION DETERMINE SOIL IN-PLACE USCS		

10. DEPTH BELOW SURFACE	11. ELEVATION	12. SAMPLE NUMBER	13. GRAPHIC LOG	14. GROUP SYMBOL	15. IN-PLACE CBR LIMITS	16. DESCRIPTION, TEST DATA, AND REMARKS
		1		OH	7.6	DARK BROWN AND VERY PLASTIC. TYPICAL TOPSOIL OF THE AREA.
0.75'	213.88'					
		2		SC	34	LIGHT BROWN COARSE-GRAINED SANDY SOIL W/PLASTIC CLAY FINES. MAXIMUM PARTICLE SIZE 1 3/4". HIGH DRY STRENGTH. DENSE IN UNDISTURBED STATE.
5.0'	209.63'					
		3		CL	29	BROWN STICKY CLAY, VERY HIGH PLASTIC QUALITIES.
6.0'	208.63'					BOTTOM OF HOLE

17. DEPTH TO WATER TABLE 4.73' ELEV = 209.90'	18. PREPARED BY *(Signature)* SGT Butler	19. CHECKED BY *(Signature)* SSG Dobbs

DD FORM 2464, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

Figure 2-8. Typical boring log

b. Extend boreholes 4 feet below the final subgrade elevation in cut areas.

c. Extend boreholes 4 feet below the natural ground elevation in fill areas.

d. Make an effort to locate the groundwater table.

NOTE: The depth may be deeper depending on the depth of thaw penetration.

Step 3. Obtain an individual soil sample for each soil layer encountered (see *Figures 2-9* and *2-10*.)

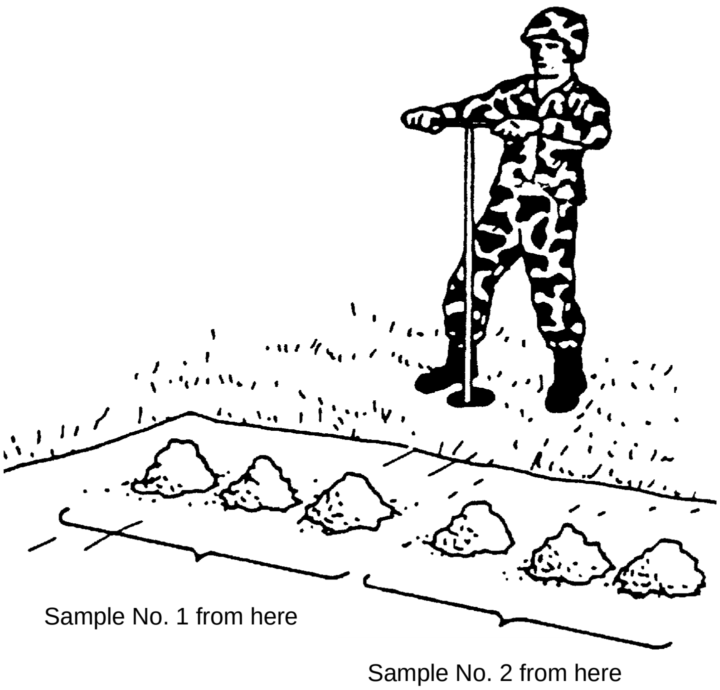


Figure 2-9. Obtaining individual bag samples

a. Place the soil from the borehole in a row in the order it was excavated, keeping soil layers separated for borings. If in a test pit, obtain samples from each layer encountered.

b. Obtain moisture-determination samples from each soil layer, ensuring that the moisture-tare sample number corresponds with the soil-sample bag number when labeled.

Step 4. Obtain data on the borehole and record the information.

a. Determine the elevation or depth below the surface of the top and bottom of each soil layer encountered.

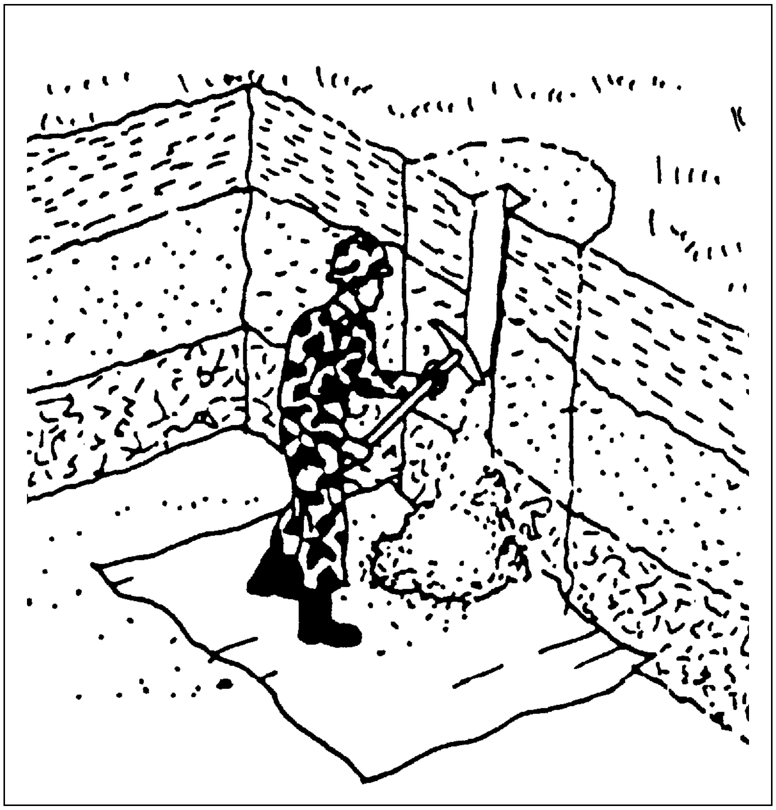


Figure 2-10. Taking a composite sample with an exposed face

b. Record a description of the type of soil encountered.

c. Record the depth of the water table, if encountered.

d. Record the depth of bedrock, if encountered.

e. Record other pertinent facts such as borehole number, date, noncommissioned officer in charge (NCOIC), project number, and project location.

Step 5. Bag individual soil samples for transportation.

a. Place each soil layer encountered in separate bags. For a deliberate survey, ensure that there is enough material to perform the testing required (at a minimum, the sieve-analysis, LL, PL, and compaction tests).

b. Label two shipping tags for each bag, indicating the project, borehole, and sample numbers (indicating the order in which it was obtained) and the total number of bags included in the sample.

c. Place one tag inside the bag and tie the other to the outside when the bag is secured (see *Figure 2-11, page 2-38*).

Step 6. Take a composite sample.

a. Remove any overburden or surface soil that is to be wasted.

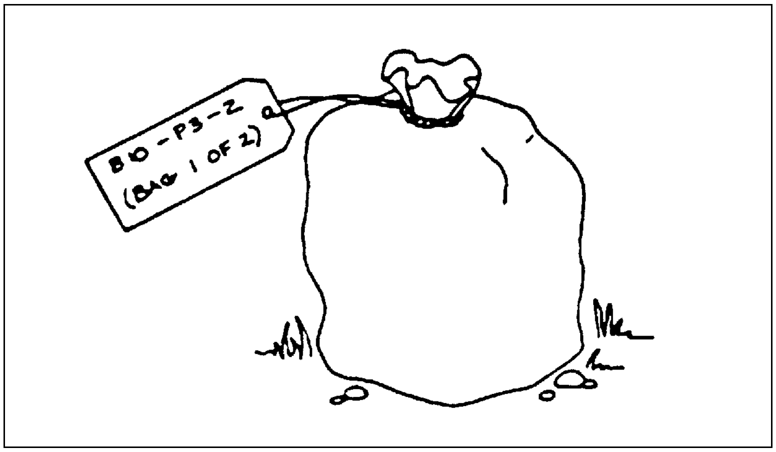


Figure 2-11. Labeling bag samples

b. Shave off any loose or dried material to provide a fresh face.

c. Spread a quartering cloth or tarpaulin at the toe of the bank.

d. Cut a channel of uniform cross section from top to bottom and deposit the soil on the canvas.

e. Bag all material that was removed to ensure that the sample contains the appropriate proportions.

NOTE: It is important that sample numbering be recorded carefully and accurately so that the diagram borehole number, sample bags, and moisture tares all correspond.

MOISTURE-CONTENT SAMPLES

Soil's natural moisture content is determined from samples taken in the field and placed in a container that is sealed to prevent moisture loss by evaporation. Natural moisture-content determinations are valuable in interpreting information obtained from test borings or pits, in drawing the soil profile, and in estimating the physical properties of soils encountered in the field. Generally, 100 grams of soil are enough to determine the moisture content of fine-grained soils. Larger samples are required for soils that contain gravel. The soil test set contains three sizes of metal dishes that have tight-fitting covers and do not require sealing if the test is made within one day after the sample is taken. If a longer time interval elapses between sampling and testing, the boxes may be sealed by the method shown in *Figure 2-12*. Other clean containers that can be sealed adequately may be used for moisture-content samples.

UNDISTURBED SAMPLES

Undisturbed soil samples are those in which the natural structures, void ratio, and moisture content are preserved as carefully as possible. They are cut, removed, and packed with the least possible disturbance. Samples of this type are used for determining the density (unit weight) of soil in the laboratory and for investigating the strength of undisturbed soils in the laboratory by the CBR or unconfined compression tests. These samples may be shipped to more

Figure 2-12. Sealing a container to retain a sample's moisture content

completely equipped laboratories for shear, consolidation, or other strength tests.

The types of undisturbed samples are—

- Chunk.
- Cylinder.

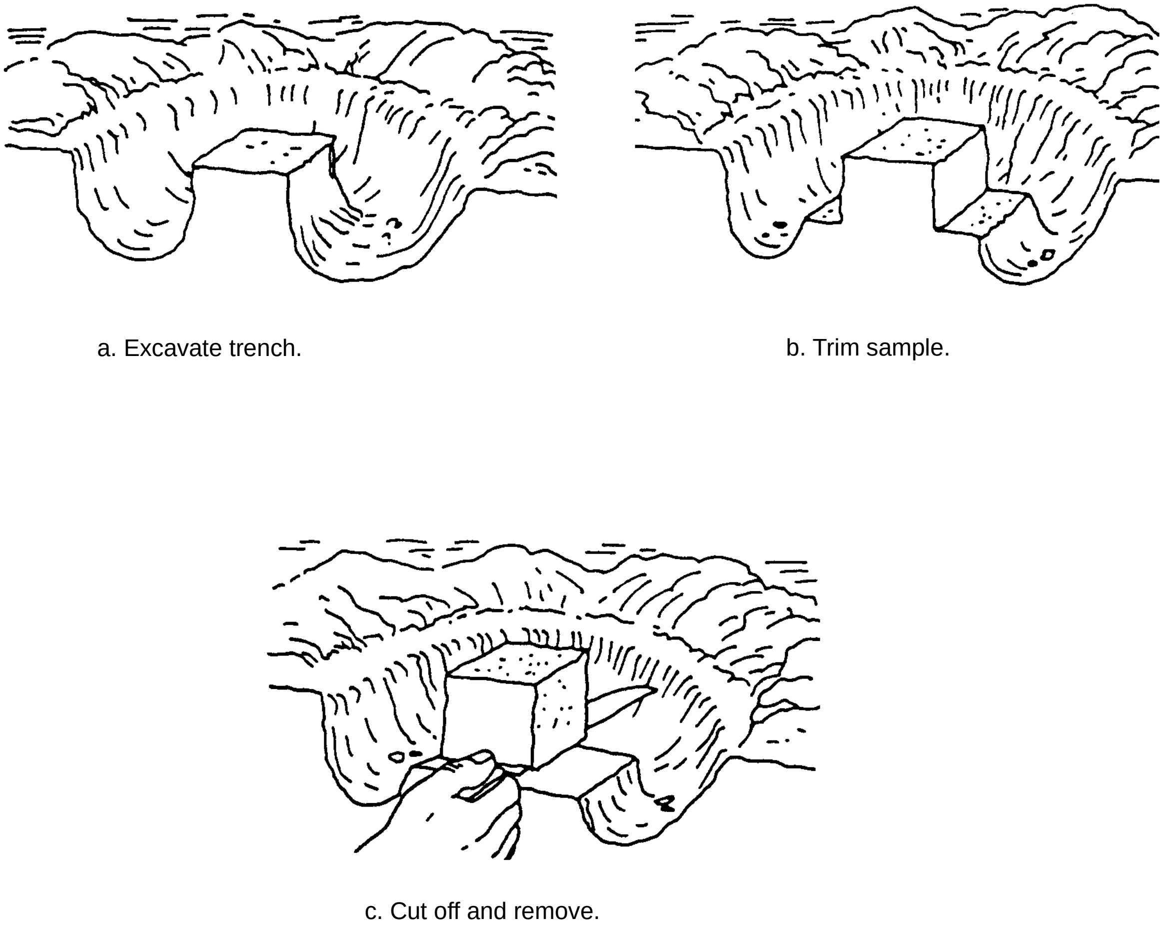
Choose the method of sampling based on the equipment available, the tests required, and the type of soil. Handle all undisturbed samples with care. Keep cohesionless soil samples in the container until ready for testing. Handle the container without jarring or vibration. Some soils are too hard or contain too many stones to permit sampling with the cylindrical samplers and can be sampled only by cutting out chunks by hand. Taking undisturbed samples frequently requires a great deal of ingenuity in adapting the sampling devices to job conditions and in devising schemes for their use. Whatever method is used, pack the sample in the container for shipment without allowing its structure to change. Protect the sample against change in moisture content during sampling and shipment.

CHUNK SAMPLES

Obtain the simplest type of undisturbed sample by cutting out a chunk of soil the desired size; cover it to prevent loss of moisture and breakage. Use this method only with soils that will not deform, break, or crumble while being removed. Cut chunk samples by hand with a shovel and knife.

The process of obtaining a chunk sample from a subgrade or other level surface, such as the bottom of a test pit, is shown in *Figure 2-13, page 2-40*. The first step is to smooth the ground surface and mark the outline of the chunk. Excavate a trench around the chunk (see *Figure 2-13a*), then deepen the excavation and trim the sides of the chunk with a knife (see *Figure 2-13b*). Cut off the chunk at the bottom with a knife, trowel, or hacksaw blade, and carefully remove it from the hole (see *Figure 2-13c*).

To obtain a chunk sample from the vertical face of a test pit or trench, carefully smooth the surface of the face and mark the outline of the chunk (see *Figure 2-14, page 2-41*). Excavate soil from the sides and back of the chunk



a. Excavate trench.

b. Trim sample.

c. Cut off and remove.

Figure 2-13. Taking a chunk sample from a level surface

(see *Figure 2-14a*). Shape the chunk with a knife (see *Figure 2-14b*), then cut off the chunk and carefully remove it from the hole (see *Figure 2-14c*).

Seal the chunk sample after removing it from the hole. One method is to apply three coats of melted paraffin (see *Figure 2-15*). Allow each coat to cool and become firm before applying the next coat. This gives adequate protection for strong samples that are to be used within a few days. Samples that are weak or that may not be used soon require additional protection. Wrap them with cheesecloth or other soft cloth and seal them in paraffin (see *Figure 2-16*, *page 2-42*). If cloth is not available, reinforce the sample with several loops of friction tape or twine, and apply three more coats of paraffin. Use extreme care to prevent damaging the sample while performing these operations.

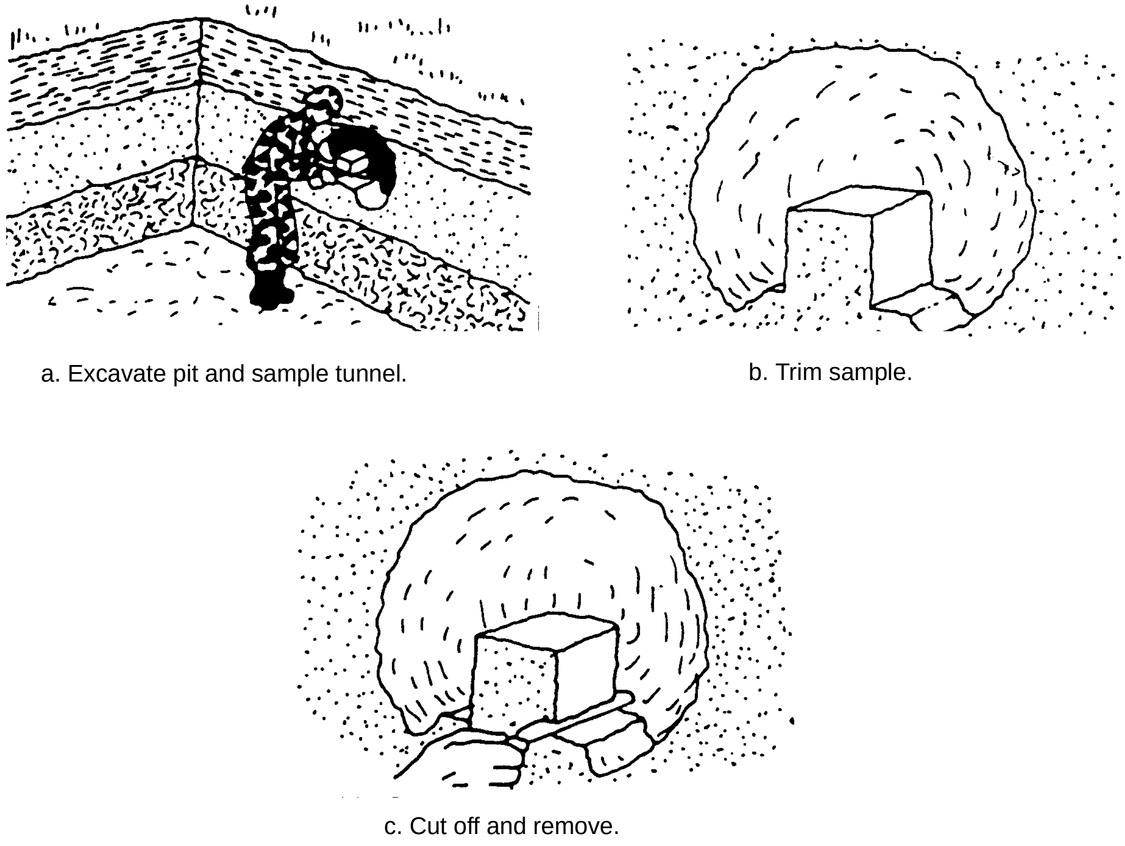


Figure 2-14. Taking a chunk sample from a vertical face

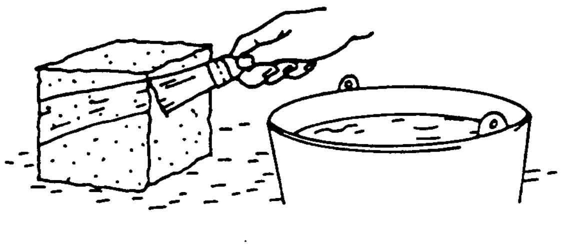


Figure 2-15. Applying paraffin to seal a chunk sample

An alternate method of sealing the chunk sample is to dip the entire sample in melted paraffin after the first brush coat has been applied and the sample has been wrapped (see *Figure 2-17, page 2-42*). This requires a large container and

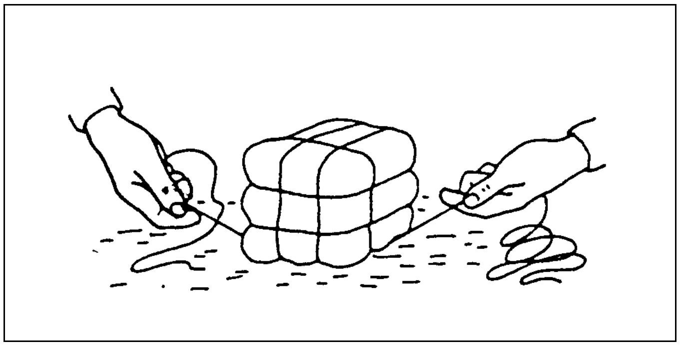


Figure 2-16. Wrapping a weak chunk sample before final sealing

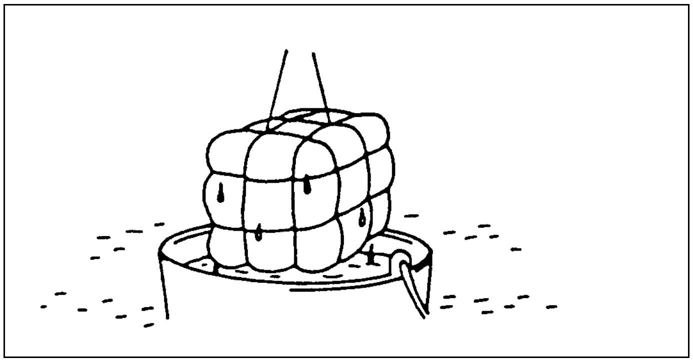


Figure 2-17. Dipping a chunk sample into melted paraffin

more paraffin but gives a more uniform coating. Build up the layer of paraffin to a minimum 1/8 inch thickness by dipping repeatedly. Provide additional protection for samples that are to be shipped by placing the chunk in a small box and packing (see *Figure 2-18*) or by applying many coats of cloth and paraffin.

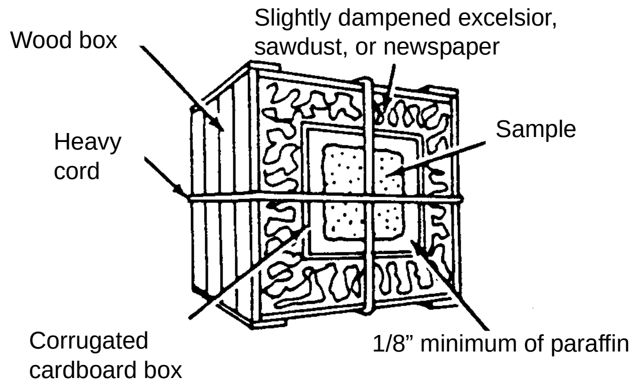


Figure 2-18. Packing a chunk sample for transportation or shipment to laboratory

CYLINDER SAMPLES

Obtain cylinder samples by using a cylindrical sampler or the CBR mold equipped with a sampling cutter. Expedient methods of obtaining cylinder samples are also used.

Soil-Trafficability Sampler

The soil-trafficability sampler consists of a cylindrical sample tube and an assembly to force the tube into the soil (see *Figure 2-19*). It is forced by hand pressure, not by blows from a hammer. A movable piston is fitted within the cylinder and attached to a rod that extends through the center of the drive tube and terminates in a flat dish or baseplate at the upper end. The outer drive tube is attached to the sample cylinder at the bottom and has two handles at the top. One of the handles is knurled and can be turned to lock the inner rod when the piston is in position. A long and a short spacer bar are bolted to the outer tube and used to establish the size of the sample core. The sampler should not be used for other than extremely soft and yielding soils. The walls of the cylinder are very thin and can be deformed if they come in contact with a hard object. Even hard or dry soil can damage the sampler. Its primary use is for samples to test the remolding characteristics in soils having initially low or very low supporting value. Additional information on the soil-trafficability sampler and soil-trafficability test set can be found in FM 5-430-00-1.

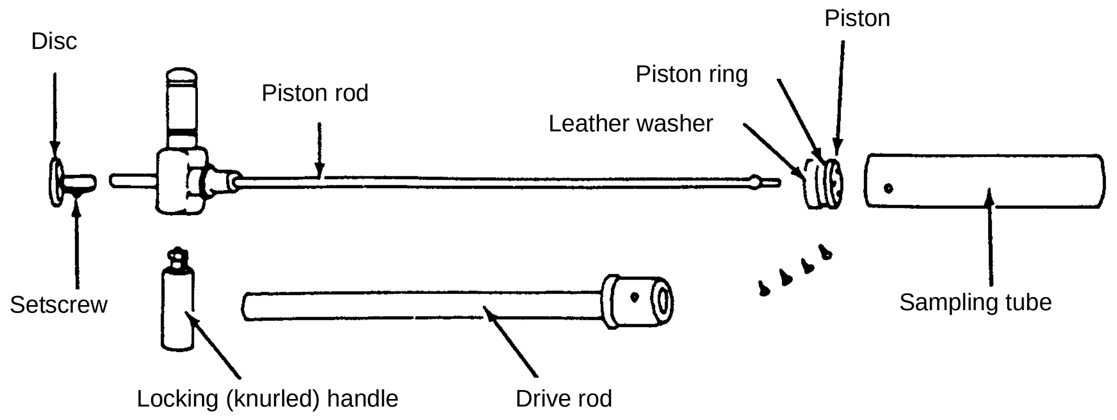


Figure 2-19. Soil-trafficability sampler

Perform the following steps for taking cylinder samples using a soil-trafficability sampler:

Step 1. Adjust the piston so it is flush with the sampler cylinder's cutting edge. Lock the knurled handle.

Step 2. Place the sampler firmly in contact with the soil to be sampled.

Step 3. Hold the disk at the top to prevent vertical movement, unlock the knurled handle, and force the sampler cylinder into the soil.

Step 4. Ensure that the cylinder is fully in the soil; then lock the knurled handle to clamp the piston, and keep the soil sample from slipping out due to the vacuum created.

Step 5. Rotate the entire sampler a half turn to shear the soil at the base of the cylinder; then carefully withdraw it from the hole and invert it so that the dish becomes a baseplate. There is a spud on the side of the sampler for releasing the side friction and the vacuum caused by withdrawing the sampler from the ground.

Step 6. Swing the longer spacer bar into position to act as a stop while the piston ejects the sample.

Step 7. Release the knurled handle, and push the drive tube until the spacer bar hits the baseplate and a portion of the sample is pushed up out of the cylinder.

Step 8. Cut and discard the portion of the sample flush with the cutting edge of the sampler. This amount of soil discard offsets any uneven shearing at the bottom of the hole and gives the soil sample a true cylindrical shape.

Step 9. Swing the short spacer bar into position and move the long bar out of the way.

Step 10. Eject the soil sample until the short bar stops the action. A portion of the soil sample will still be in the cylinder.

Step 11. Cut off the soil sample flush with the sampler’s cutting edge into a preformed plate made to fit around the cutter. Use the sample cutter (piano wire) for this operation. The sample is now exactly 1.87 inches in diameter and 3.45 inches long.

Step 12. Discard the remaining soil in the sampler.

This sampler can be used with a hand auger to obtain cores at depths up to 48 inches below the surface. The cores are sealed.

The soil-trafficability sampler requires proper maintenance and adjustment to produce consistent results. Keep the inside of the sampling tube, the piston tube, the piston ring, and the leather washer reasonably clean. After 5 to 25 samplings (depending on the type of soil), immerse the tube, first in water and then in fuel oil, and work the piston up and down five or six times in each liquid. After wiping off the excess fuel oil, squirt light machine oil into the tube. If the instrument becomes stiff and hard to work, remove the tube, disassemble and thoroughly clean the piston, and oil the leather washer. Take care in removing the tube to prevent its slipping from the head suddenly and bending the piston rod. The tube walls and cutting edges are relatively soft and should be handled with care.

Adjust the effective piston-rod length to keep the face of the piston flush with the tube’s cutting edge when the piston-rod handle (disk) is fully depressed. Do this by loosening the setscrew on the handle, screwing the handle up or down to the correct position, and retightening the setscrew.

CBR Mold

In soft, fine-grained soils, cylinder samples for undisturbed CBR or density tests may be taken directly in the CBR compaction cylinder by using the sampling collar (cutter) (see *Figure 2-20*). Perform the following steps using a CBR mold:

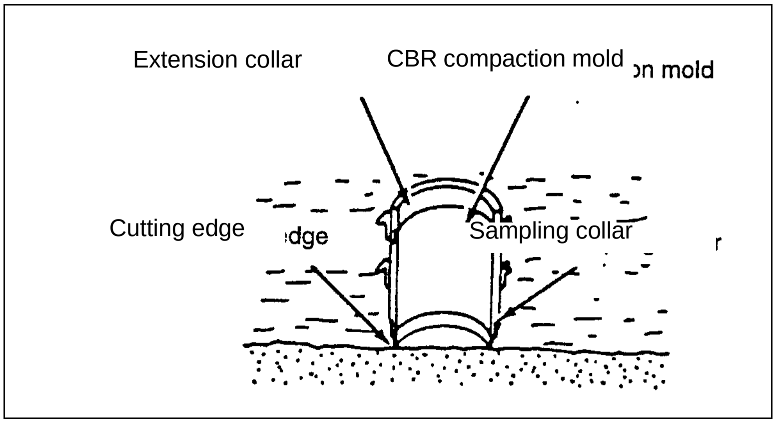


Figure 2-20. Section through CBR mold

Step 1. Smooth the surface of the ground and press the sampling collar and mold into the soil with moderate pressure.

Step 2. Excavate a trench around the cylinder (see *Figure 2-21*).

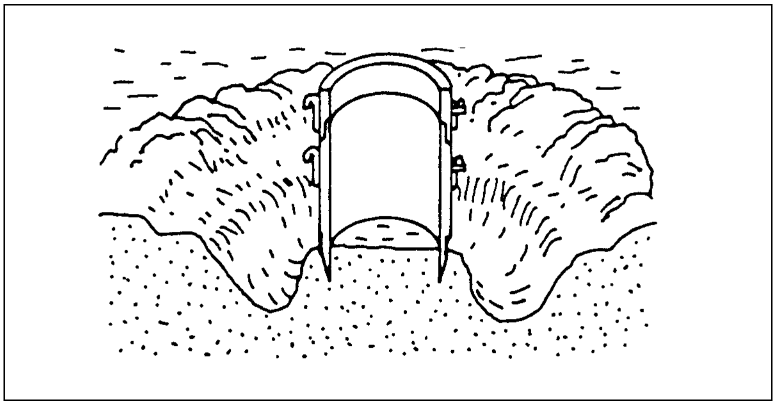


Figure 2-21. Trench excavated around cylinder

Step 3. Press the mold down firmly over the soil again, using the hand driver or loading bar if necessary (see *Figure 2-22, page 2-46*). A loading bar may be improvised from any piece of timber of suitable size.

Step 4. Trim the soil away from the sampling collar with a knife, cutting downward and outward to avoid cutting into the sample. The actual cutting to size is done with the sampling collar. The sampler may be forced down with the truck jack, if available. In either case, do not force the sampler down ahead of the trimming on the outside of the cylinder.

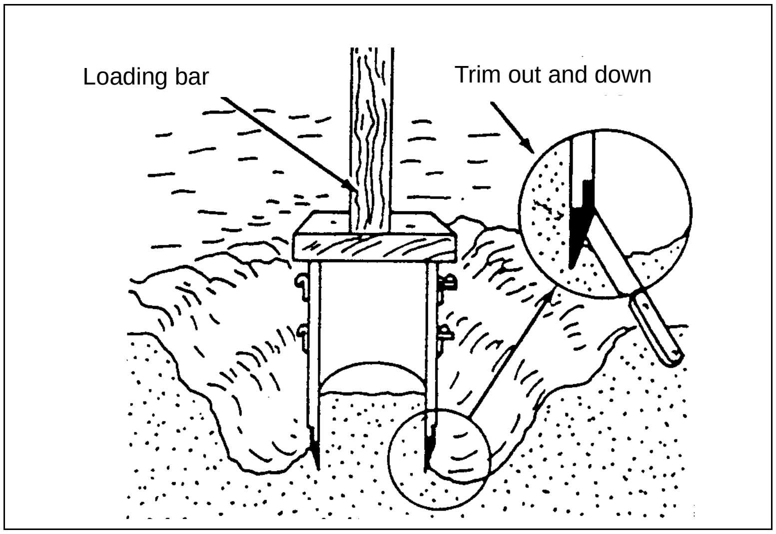


Figure 2-22. Using load bar to drive cylinder

Step 5. Excavate the trench deeper and repeat the process until the soil penetrates well into the extension collar (see *Figure 2-23*). If stones interfere, pick them out carefully and fill the space with soil. Record this fact in the log of the sample where it is pertinent.

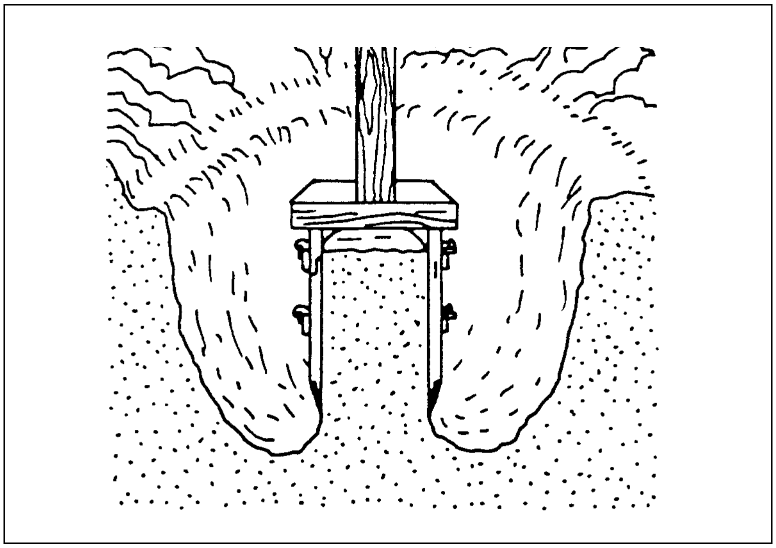


Figure 2-23. Cylinder in position before cutting sample

Step 6. Cut the sample off at the bottom of the mold using a shovel, knife, or wire saw (see *Figure 2-24*).

Step 7. Remove the mold and sample from the hole.

Step 8. Remove the upper collar, and trim the top surface of the sample about 1/2 inch down into the mold. Fill this recess with paraffin to seal the end of the sample (see *Figure 2-25*).

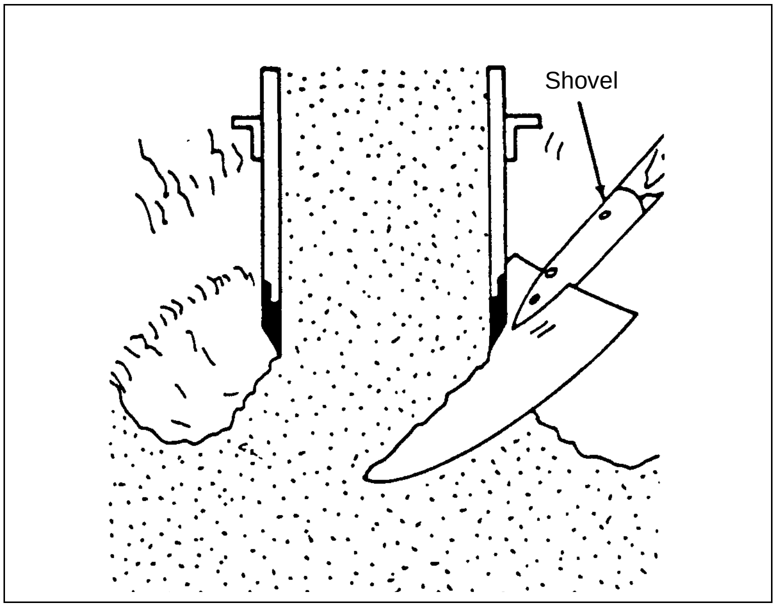


Figure 2-24. Cutting off cylindrical sample

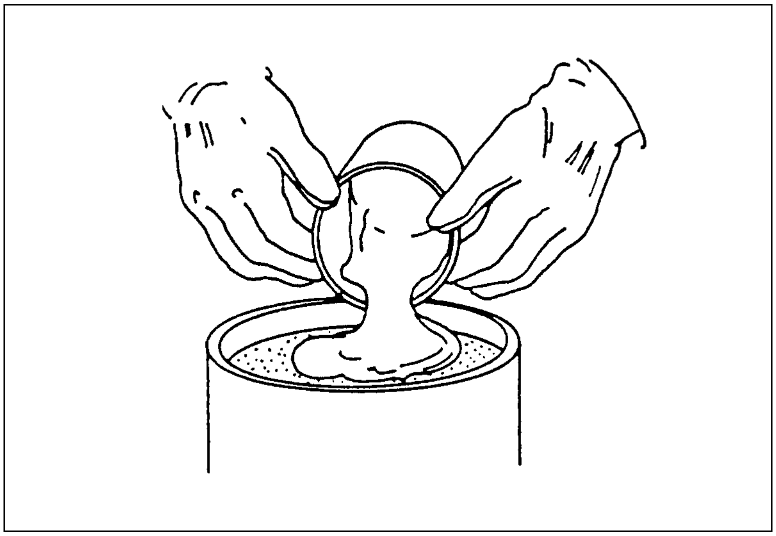


Figure 2-25. Cylinder in position before cutting sample

Step 9. Turn the mold over and remove the cutting edge. Trim this end down into the mold about 1/2 inch, as before, and fill the resulting space with paraffin. If the sample is to be handled very much before testing, overfill the ends with paraffin and then trim it exactly flush with a straightedge.

Step 10. Place boards over each end, and clamp them in place using bolts, wire, or string (see *Figure 2-26, page 2-48*).

Step 11. Wrap the samples in cloth, and soak them in paraffin layers if they must be transported some distance or if they have to be handled quite a bit before testing.

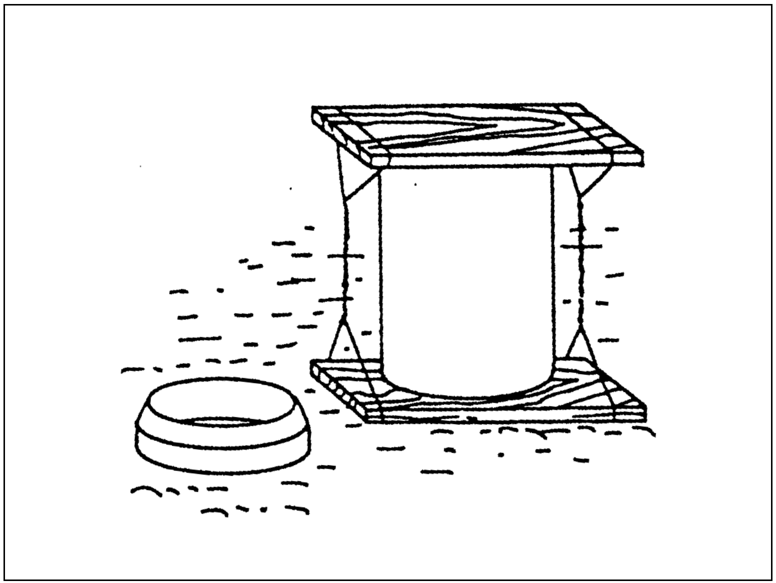


Figure 2-26. Protecting a sample in a CBR mold

QUARTERING SAMPLES

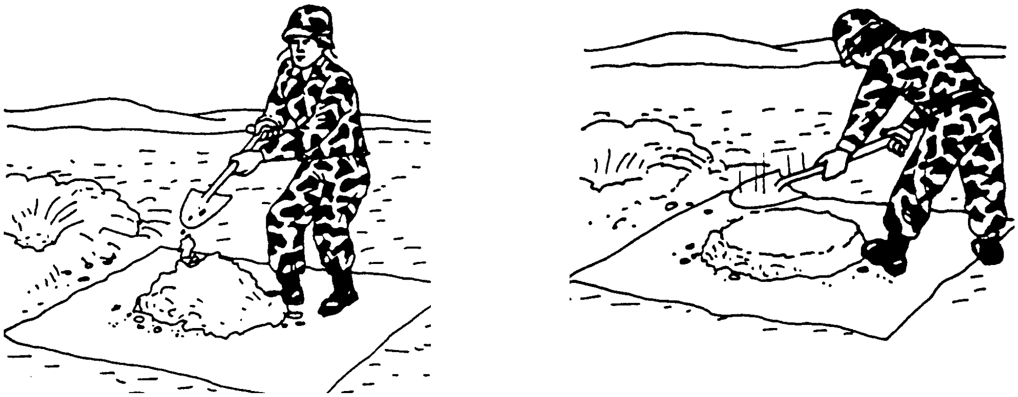
The process of reducing a representative sample to a convenient size, or of dividing a sample into two or more smaller samples for testing, is called quartering. The procedure to be used varies somewhat, depending on the size of the sample.

SAMPLES WEIGHING OVER 100 POUNDS

Quartering a sample in excess of 100 pounds is shown in *Figure 2-27*. First, mix the sample and pile it on the quartering canvas (see *Figure 2-27a*). Place each shovelful in the center of the cone so that the soil runs down evenly in all directions to mix the sample. Flatten the cone with the shovel, spreading the material to a circular layer of uniform thickness (see *Figure 2-27b*). Insert a stick or pipe under the center of the pile (under the canvas) and lift both ends of the stick, thus dividing the sample into two parts (see *Figure 2-27c*). Remove the stick, leaving a fold in the canvas. Insert the stick under the pile (this time at right angles to the first division) and lift again, dividing the sample into four parts (see *Figure 2-27d*). Discard the two diagonally opposite quarters and carefully clean the fines from the canvas. Remix the remaining material by taking alternate shovelfuls from each quarter. Repeat the quartering process as necessary to reduce the sample to the desired size.

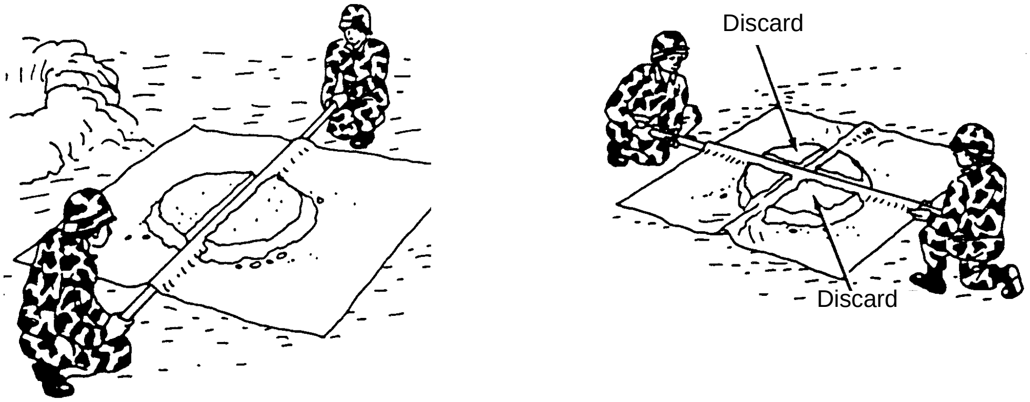
SAMPLES WEIGHING 25 TO 100 POUNDS

To quarter samples weighing 25 to 100 pounds, pile the soil on the canvas and mix it by alternately lifting the corners of the canvas and pulling over the samples as if preparing to fold the canvas diagonally, as illustrated in *Figure 2-28, page 2-50*. Then flatten and quarter the sample.



a. Mixing and piling a sample weighing more than 100 pounds

b. Flattening sample to uniform thickness



c. Halving the sample

d. Quartering the sample

Figure 2-27. Samples weighing more than 100 pounds

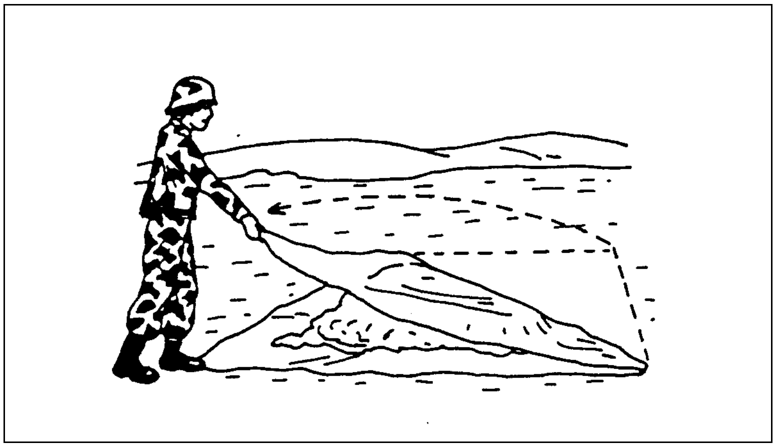


Figure 2-28. Mixing a sample weighing 25 to 100 pounds

SAMPLES WEIGHING LESS THAN 25 POUNDS

The process of quartering samples less than 25 pounds is similar to the process for 100 pounds and more. Place the sample on the canvas or a clean sheet of paper. Mix it thoroughly with a trowel and form it into a conical pile (see *Figure 2-29a)*. Flatten the cone by pressing downward with the trowel (see *Figure 2-29b*). Use the trowel to divide the sample into quarters. Discard diagonally opposite quarters (see *Figure 2-29c*). Repeat the process as necessary to reduce the size of the sample for testing.

THE SOIL PROFILE

Keep a detailed field log of each auger boring or test pit made during the soil survey. After completing the survey, consolidate the information contained in the separate logs. In addition to the classification and depth of soil layers recorded in each log, show the natural water contents of fine-grained soils along the side of each log. Also note the elevation of the groundwater table. Determine the elevation during the soil survey by observing the level at which free water stands in the borings. To get an accurate determination, cover holes and inspect them 24 hours after being dug. This allows the water to reach its maximum level. The soil profile is a graph of a vertical cross section of the soil layers from the surface of the earth downward (see *Figure 2-30, page 2-52*).

PURPOSE

The soil profile has many practical uses in locating, designing, and constructing roads, airfields, and structures. It has a great influence in the location of the grade line, which should be placed to take full advantage of the best soils available at the site. The profile shows whether soils to be excavated are suitable for use in embankments or if borrow soils are required. It may show the existence of undesirable conditions, such as peat or organic matter or bedrock close to the surface, which will require special construction measures. It aids in planning drainage facilities to take advantage of the presence of well-draining soils. It may indicate that special drainage installations will be needed with soils that are difficult to drain, particularly in areas where the

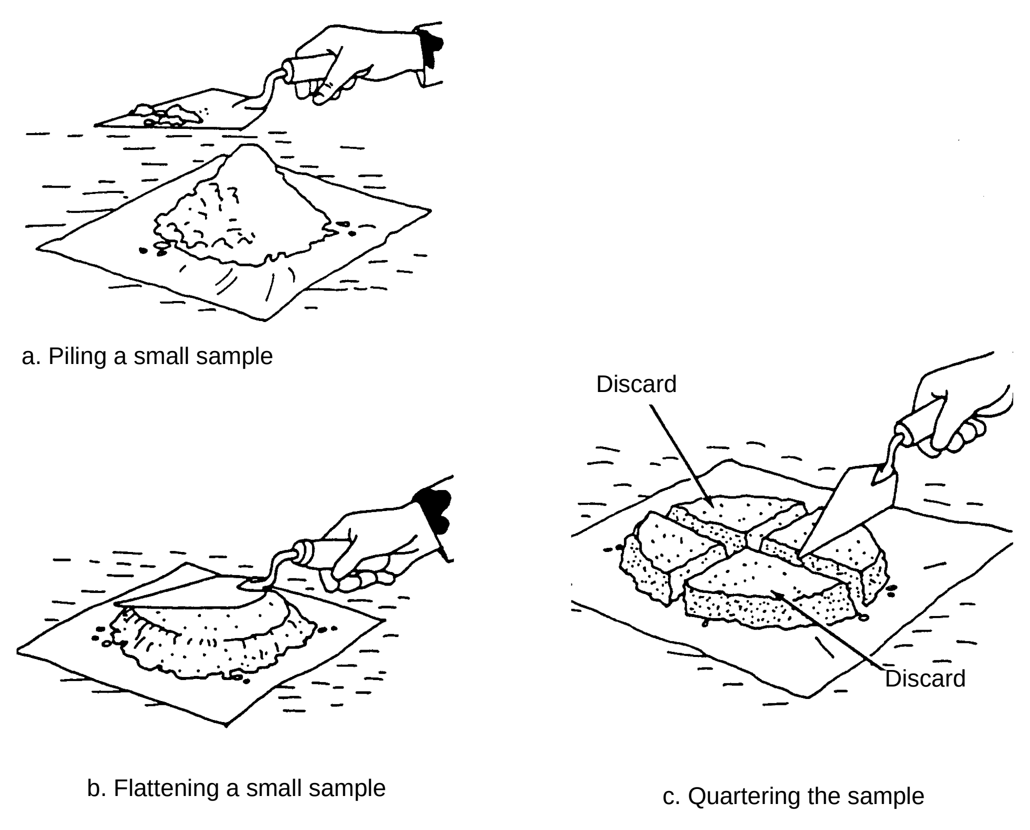


Figure 2-29. Mixing a sample weighing less than 25 pounds

water table is high. Considerations for capillary and frost action may be particularly important when frost-susceptible soils are shown on the profile.

The soil profile, including the legend, will show each soil layer, water table, and the relative elevation to within ± 1 foot. Locate the holes horizontally to within one half of the smallest dimension of the scale used. The boreholes will be sketched in with appropriate soil symbol hatchings for each layer.

EQUIPMENT

Use the following items in a laboratory environment to obtain a soil profile:

- Boring logs.
- Graph paper.
- Pencils.
- A straightedge.
- FM 5-430-00-1.

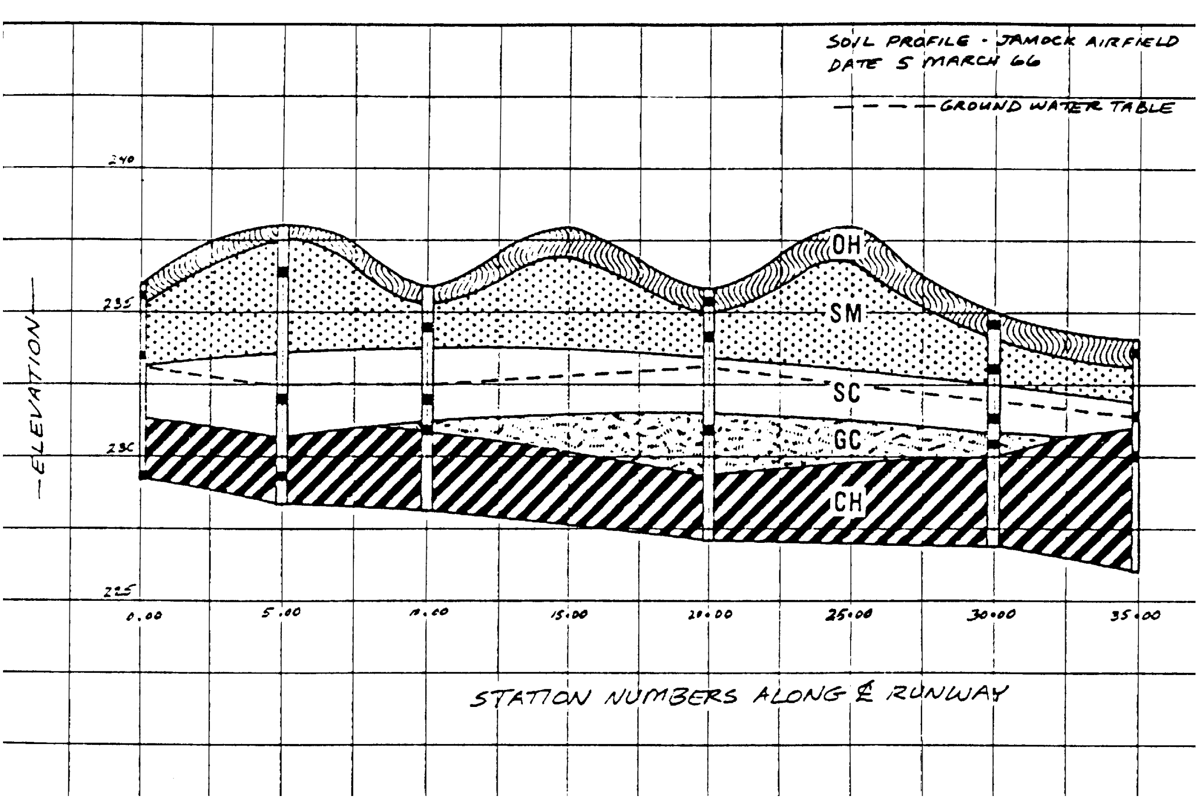


Figure 2-30. Typical soil profile

STEPS

Perform the following steps to obtain a soil profile:

Step 1. Determine the scales to be used (see *Figure 2-31*).

a. Determine and label (along the left side of the graph paper) the vertical scale representing the highest and lowest elevations found in the bore logs.

b. Determine and label (along the bottom of the graph paper) the horizontal scale representing the stations that cover the area where borings have been made.

Step 2. Plot the boreholes and mark the depth for each soil layer of each borehole.

Step 3. Draw the representing soil hatching symbol for each soil layer. The symbols taken from bore logs are located in FM 5-430-00-1.

Step 4. Connect the soil layers from borehole to borehole with a solid line. Connect the bottoms of the boreholes with a dashed line.

Step 5. Label each soil layer with a soil-group symbol in the USCS and a color (use the symbols from *Table B-2, pages B-6 and B-7*, or *Table B-3, pages B-16 and B-17*).

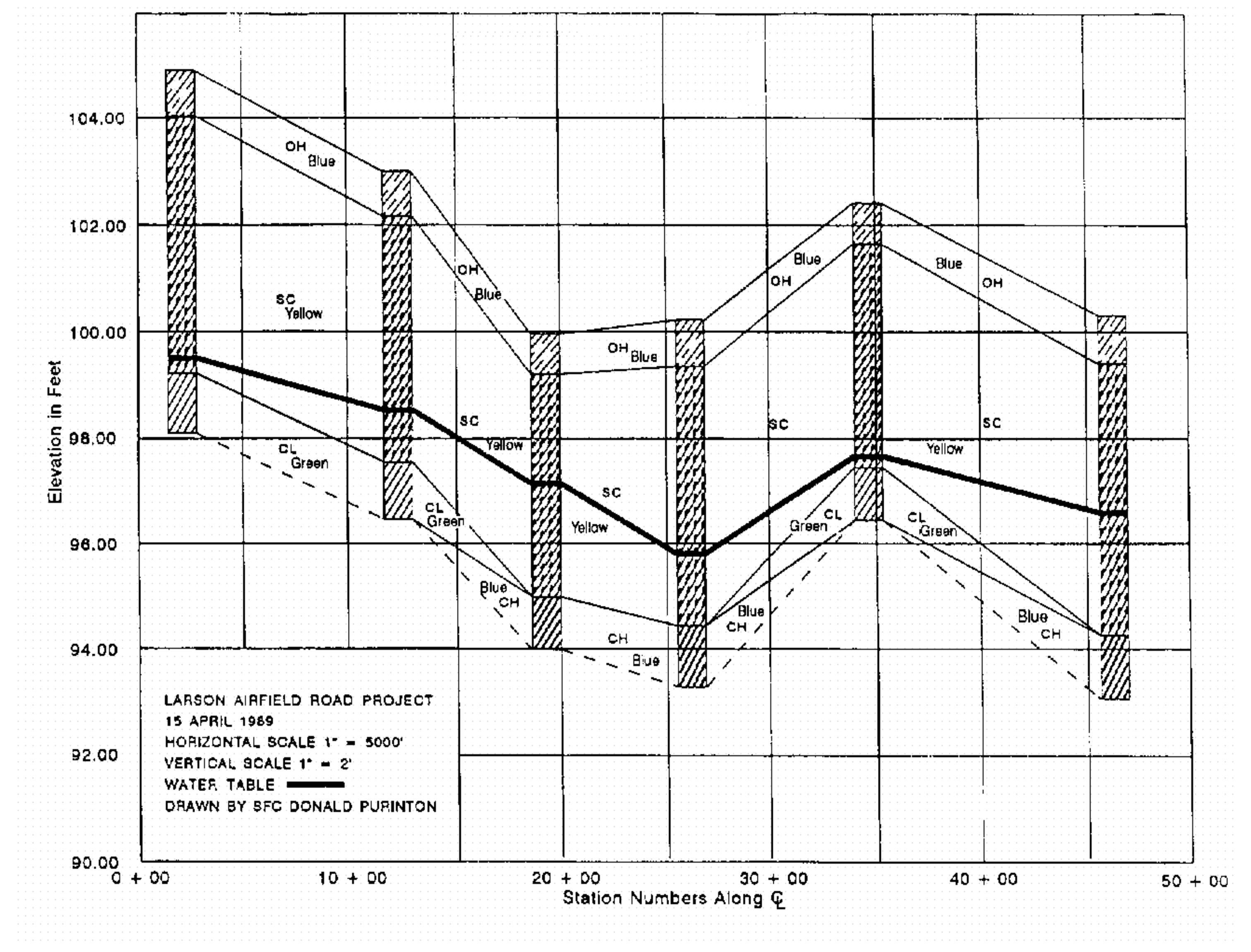


Figure 2-31. Soil-profile scales

Step 6. Plot the depth of water at each borehole and connect the points (with a solid, heavy line) together showing the water-table profile.

Step 7. Place the legend in a corner of the graph paper, including the following:

a. The horizontal and vertical scales.

b. The line symbol for the water table.

c. The project title and location.

d. The name of the preparer and the date prepared.

SECTION III. MOISTURE-CONTENT DETERMINATION

The soil's moisture content (also referred to as water content) is an indicator of the amount of water present in a soil. By definition, moisture content is the ratio of the weight of water in a sample to the weight of solids (oven-dried) in the sample, expressed as a percentage (w).

$$w = \frac{W_W}{W_S} \times 100$$

where—

w = moisture content of the soil (expressed as a percentage)

Ww = weight of water in the soil sample

Ws = weight of oven-dried-soil solids in the sample

With many soils, close control of moisture content during field compaction by rolling is necessary to develop a required density and strength in the soil mass. The amount of compaction effort that must be exerted to obtain a specified density depends on having the moisture content at or very close to optimum. Because the specified density is in terms of dry unit weight, the moisture content must be determined with the wet unit weight to determine whether moisture must be added or removed from the in-place soil to achieve the optimum moisture content (OMC). This is a necessary field procedure in constructing embankments and compacting highway subgrades, since moisture-content adjustments are known promptly and oven-drying time is not always afforded.

There are several methods of determining the moisture content of soils, including the–

- Oven-dry method (ASTM D 2216-90).
- Microwave-oven method (ASTM D 4643-87).
- Calcium-carbide-gas pressure method (American Association of State Highway and Transportation Officials [AASHTO] T 217-1986).
- Nuclear-moisture-and-density-gauge method (ASTM D 2922-96 and ASTM D 3017-96).

OVEN-DRY METHOD (ASTM D 2216-90)

The most accurate method of determining moisture content is the oven-dry method. This method uses an oven with a temperature or thermostatic control. For expedient determinations, soils are sometimes dried in a frying pan or container heated by an external source, either a stove or an exhaust manifold. However, heating most soils to excessive temperatures results in chemical changes that may lead to errors in moisture-content results. Hence, drying soils by an uncontrolled heat source is usually less accurate than drying them in a thermostatically controlled oven.

PURPOSE

Perform this test to determine the moisture content of a soil sample to within a desired percentage.

EQUIPMENT

The following items are necessary for this test method:

- A laboratory oven.
- Heat-resistant gloves.
- A calculator.
- Moisture-determination tares.
- A grease pencil.
- A balance scale sensitive to 0.01 gram.
- DD Form 1205.
- A pencil.

STEPS

Perform the following steps to determine the moisture content:

Step 1. Record all identifying information of the sample in blocks 1 through 5 of DD Form 1205 (see *Figure 2-32, page 2-56*).

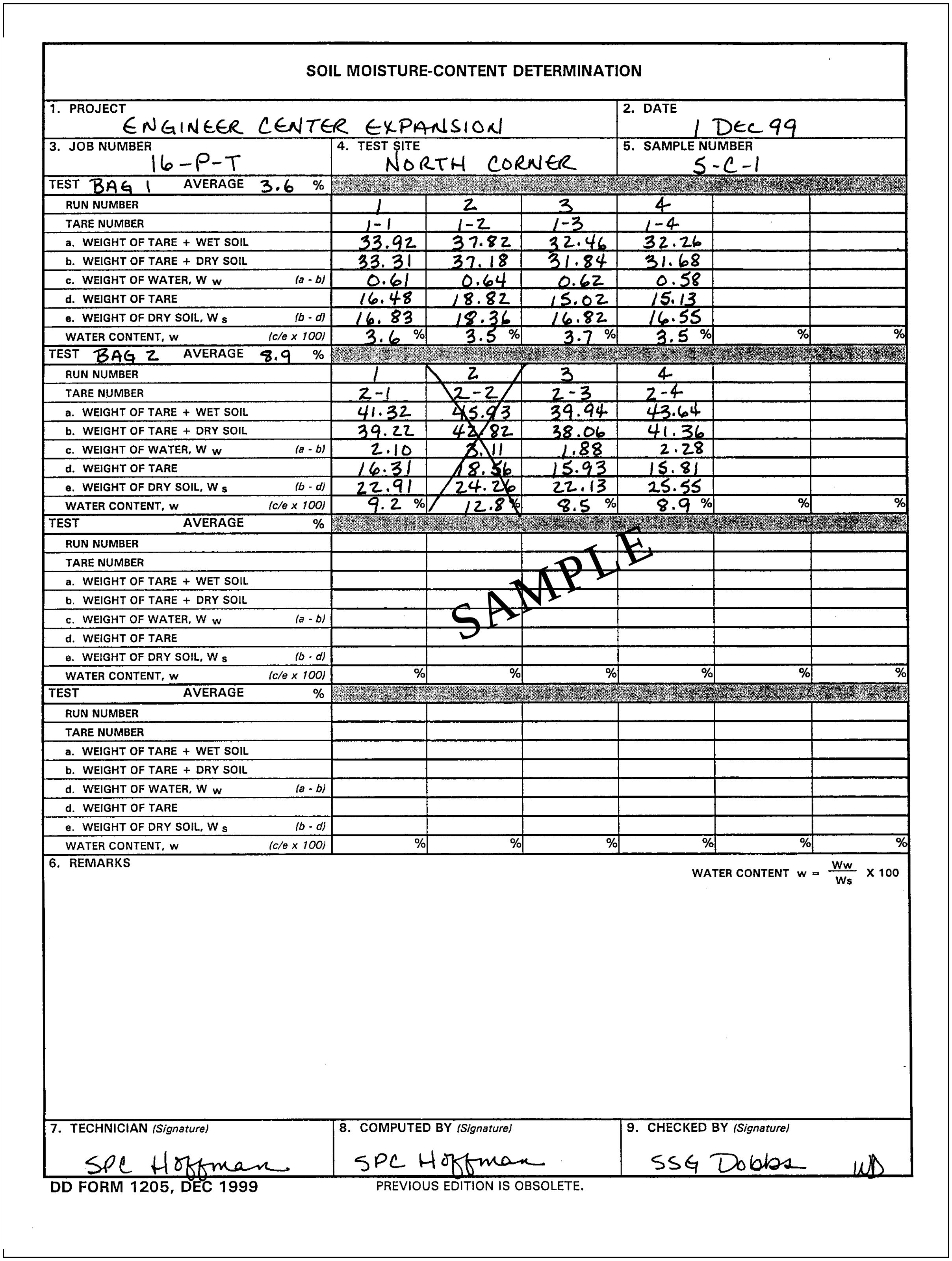
Step 2. Label and weigh the clean, dry moisture-determination tares, and record the weights on the form as the weight of the tare (line D).

Step 3. Obtain the required soil sample. Place it in the tare and cover it with the lid.

- When conducting this test as part of another test method, use the specimen mass stated in that test method.
- When conducting this test with no minimum specimen mass provided, use the values provided in *Table 2-5*, *page 2-57*, depending on the degree of accuracy of the reported water content.

Step 4. Weigh the soil sample and the tare to the nearest 0.01 gram. Record the weight on the form as the weight of the tare and the wet soil (line A).

Step 5. Oven-dry the sample, with the moisture-determination tare lid removed, at 110°C ± 5° until the sample weight becomes constant. Oven-drying



SOIL MOISTURE-CONTENT DETERMINATION

1. PROJECT: ENGINEER CENTER EXPANSION
2. DATE: 1 DEC 99
3. JOB NUMBER: 16-P-T
4. TEST SITE: NORTH CORNER
5. SAMPLE NUMBER: S-C-1

TEST BAG 1 AVERAGE 3.6 %						
RUN NUMBER	1	2	3	4		
TARE NUMBER	1-1	1-2	1-3	1-4		
a. WEIGHT OF TARE + WET SOIL	33.92	37.82	32.46	32.26		
b. WEIGHT OF TARE + DRY SOIL	33.31	37.18	31.84	31.68		
c. WEIGHT OF WATER, W_w *(a - b)*	0.61	0.64	0.62	0.58		
d. WEIGHT OF TARE	16.48	18.82	15.02	15.13		
e. WEIGHT OF DRY SOIL, W_s *(b - d)*	16.83	18.36	16.82	16.55		
WATER CONTENT, w *(c/e x 100)*	3.6 %	3.5 %	3.7 %	3.5 %	%	%
TEST BAG 2 AVERAGE 8.9 %						
RUN NUMBER	1	~~2~~	3	4		
TARE NUMBER	2-1	~~2-2~~	2-3	2-4		
a. WEIGHT OF TARE + WET SOIL	41.32	~~45.93~~	39.94	43.64		
b. WEIGHT OF TARE + DRY SOIL	39.22	~~42.82~~	38.06	41.36		
c. WEIGHT OF WATER, W_w *(a - b)*	2.10	~~3.11~~	1.88	2.28		
d. WEIGHT OF TARE	16.31	~~18.56~~	15.93	15.81		
e. WEIGHT OF DRY SOIL, W_s *(b - d)*	22.91	~~24.26~~	22.13	25.55		
WATER CONTENT, w *(c/e x 100)*	9.2 %	~~12.8 %~~	8.5 %	8.9 %	%	%
TEST AVERAGE %						
RUN NUMBER						
TARE NUMBER						
a. WEIGHT OF TARE + WET SOIL						
b. WEIGHT OF TARE + DRY SOIL						
c. WEIGHT OF WATER, W_w *(a - b)*						
d. WEIGHT OF TARE						
e. WEIGHT OF DRY SOIL, W_s *(b - d)*						
WATER CONTENT, w *(c/e x 100)*	%	%	%	%	%	%
TEST AVERAGE %						
RUN NUMBER						
TARE NUMBER						
a. WEIGHT OF TARE + WET SOIL						
b. WEIGHT OF TARE + DRY SOIL						
d. WEIGHT OF WATER, W_w *(a - b)*						
d. WEIGHT OF TARE						
e. WEIGHT OF DRY SOIL, W_s *(b - d)*						
WATER CONTENT, w *(c/e x 100)*	%	%	%	%	%	%

6. REMARKS

WATER CONTENT $w = \frac{W_w}{W_s} \times 100$

7. TECHNICIAN *(Signature)*	8. COMPUTED BY *(Signature)*	9. CHECKED BY *(Signature)*
SPC Hoffman	SPC Hoffman	SSG Dobbs

DD FORM 1205, DEC 1999

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Figure 2-32. Sample DD Form 1205

Table 2-5. Recommended minimum test specimen for reporting water content

Maximum Particle Size (100% Passing)	Standard Sieve Size	Minimum Moist Mass for Reporting to ± 0.1%	Minimum Moist Mass for Reporting to ± 1%
2.0 mm or less	No. 10	20.0 g	20 g*
4.75 mm	No. 4	100.0 g	20 g*
9.50 mm	3/8 in	500.0 g	50 g
19.00 mm	3/4 in	2.5 kg	250 g
37.50 mm	1 1/2 in	10.0 kg	1 kg
75.00 mm	3 in	50.0 kg	5 kg

*** To be representative, not less than 20 grams shall be used.**

time will vary depending on the type of soil, the size of the sample, and other factors. For routine water-content determination, oven-dry a sample consisting of clean sands and gravel for a minimum of 4 hours. For most other soils, a minimum drying time of 16 hours is adequate.

Step 6. Remove the sample from the oven and replace the moisture-determination tare lid. Allow the sample to cool until the tare can be handled comfortably with bare hands.

Step 7. Weigh the dried soil sample and the tare. Record the weight as the weight of the tare and dry soil (line B).

Step 8. Determine the weight of the water (W_w) by subtracting the weight of the tare and dry soil (line B) from the weight of the tare and wet soil (line a). Record the weight on the form (line C).

Step 9. Determine the weight of the dry soil (W_s) by subtracting the weight of the tare (line D) from the weight of the tare and dry soil (line B). Record the weight on the form (line E).

Step 10. Determine the water content (w), in percent, and record it using the following formula:

$$w = \frac{W_W}{W_S} \times 100$$

When determining the average water content, the individual tests must be within ± 1 percent. Any individual tests that do not meet this requirement will not be used (see *Figure 2-32*). If none of the individual tests meet this requirement, then additional testing is required.

CALCIUM-CARBIDE-GAS PRESSURE METHOD (AASHTO T 217-1986)

CAUTION

The chemical reaction of calcium carbide with water produces acetylene gas which is extremely flammable. Exercise extreme caution to avoid open flame when releasing the gas from the speedy moisture tester. Perform the test in a well-ventilated area, as asphyxiation could occur if performed in a confined area.

Use the calcium-carbide-gas pressure method to determine the moisture content of a soil sample using the 26-gram speedy moisture tester to within ± 0.5 percent. If another tester is to be used, consult the user's manual for the tester before conducting the moisture-content determination.

PURPOSE

Perform this test to determine the moisture content of a soil sample to within ± 0.5 percent.

EQUIPMENT

Use the following items for the calcium-carbide-gas pressure test:

- A calcium-carbide-pressure (speedy) moisture tester to hold a 26-gram soil specimen.
- A balance (readable to 0.1 gram).
- Two 1 1/4-inch steel balls.
- A cleaning brush and cloth.
- A scoop (for measuring calcium-carbide reagent).
- Calcium-carbide reagent.

STEPS

Perform the following steps to determine the soil's moisture content:

Step 1. Weigh the soil sample to be tested, ensuring that it weighs exactly 26 grams. Place the soil sample in the tester's body and add the two 1 1/4-inch steel balls.

Step 2. Place three scoops (about 24 grams) of calcium carbide into the cap of the tester and, with the pressure vessel in a horizontal position, insert the cap into the pressure vessel. Seal the unit by tightening the clamp, taking care that no carbide comes in contact with the soil until a complete seal is achieved.

Step 3. Raise the moisture tester to a vertical position so that the reagent in the cap will fall into the pressure vessel.

Step 4. Shake the instrument vigorously in a rotating motion so that all lumps are broken up to permit the calcium carbide to react with all available free moisture. Shake the instrument in a rotating motion so that the steel balls will not damage the instrument or cause soil particles to become embedded in the orifice leading to the pressure diaphragm. Continue shaking at least 1 minute for granular soils and up to 3 minutes for other soils to permit complete reaction between the calcium carbide and the free moisture. Allow time for the dissipation of the heat generated by the chemical reaction.

Step 5. Hold the instrument in a horizontal position at eye level. Read the dial when the needle stops moving. Record the dial reading as the percent of moisture by wet mass.

Step 6. Point the cap of the instrument away from the operator and release the gas pressure slowly. Empty the pressure vessel and examine the material for lumps. If the sample is not completely pulverized, repeat the test using a new sample. Clean the cap thoroughly of all carbide and soil before running another test.

The limit of the tester is 12 percent moisture for aggregate or 20 percent moisture for soil. If the limit is exceeded, then the test must be run again using a half-sized sample (13 grams) and the dial reading must be multiplied by 2.

CALCULATIONS

Determine the percentage of moisture by dry mass of the soil from the calibration curve (see *Figure 2-33*, *page 2-60*) or from the conversion chart (see *Figure 2-34*, *page 2-61*) as explained below. The calibration curves, moisture-content determination by the calcium-carbide-gas pressure method, are used for materials that need the pulverizing steel balls (see *Figure 2-33*). Using the highest reading obtained during the test (direct reading), read over to the curve and then down to the oven-dry moisture percentage. The curve to be used will depend on the test time.

Use the conversion chart (see *Figure 2-34*) to determine oven-dry moisture contents of materials that do not need the steel balls. If direct readings are not on the conversion chart, interpolate the difference between the two known direct readings.

EXAMPLE

A speedy test is performed on a sand. The highest reading obtained is 3.5 percent (wet weight). The difference between 3.0 percent (which has a known dry weight of 3.2 percent) and 4.0 percent (which has a known dry weight of 4.3 percent) must be interpolated for 3.5 percent.

Using *Figure 2-34*, the values in *Table 2-6* are known. To find the value of x, place the differences in the table values into a ratio. The ratio of differences is—

$$\frac{3.5-3.0}{4.0-3.0} = \frac{x-3.2}{4.3-3.2} \qquad or \qquad \frac{0.5}{1.0} = \frac{x-3.2}{1.1}$$

then—

cross multiplied, $x - 3.2 = 0.55$;

therefore, leaving $x = 3.2 + 0.55 = 3.75$.

So, at 3.5 percent wet weight, $x = 3.2 + 0.55 = 3.75$ percent comparable oven-dry weight.

Table 2-6. Determining the soil's moisture content

Percent Wet Weight	Percent Dry Weight
3.0	3.2
3.5	x
4.0	4.3

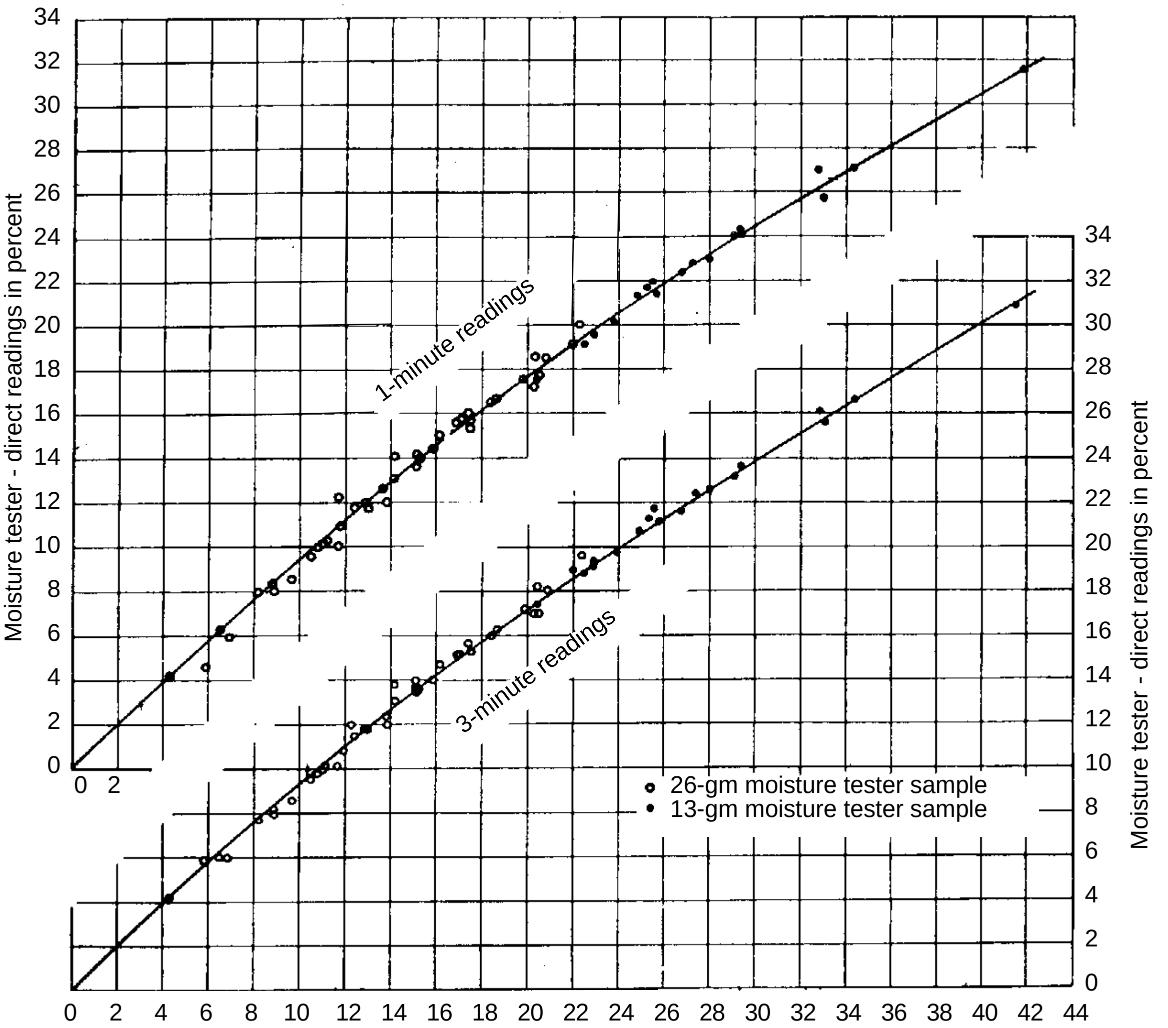


Figure 2-33. Speedy-moisture tester calibration chart, for use with the 1 1/4-inch steel balls

The curves and charts are usually supplied with the moisture tester. However, check each moisture tester for the accuracy of its gauge and the accuracy of the conversion curve. Check the accuracy of the tester gauge by using a calibration kit (obtained from the tester's manufacturer) equipped with a standard gauge. In case of discrepancy, adjust the gauge on the tester to conform with the standard gauge. For checking the accuracy of the conversion curve, make a calibration for meter readings versus oven-dry moisture contents using local soils. Also, additional testing may be necessary

Speedy Reading Wet Weight %	Dry Weight %	Speedy Reading Wet Weight %	Dry Weight %	Speedy Reading Wet Weight %	Dry Weight %
1.0	1.0	20.5	25.8	35.5	55.0
2.0	2.1	21.0	26.5	36.0	56.2
3.0	3.2	21.5	27.4	36.5	57.4
4.0	4.3	22.0	28.2	37.0	58.7
5.0	5.4	22.5	29.0	37.5	60.0
6.0	6.5	23.0	29.8	38.0	61.2
7.0	7.6	23.5	30.7	38.5	62.6
8.0	8.7	24.0	31.5	39.0	63.9
9.0	9.8	24.5	32.4	39.5	65.2
10.0	11.0	25.0	33.3	40.0	66.6
10.5	11.7	25.5	34.2	40.5	68.0
11.0	12.3	26.0	35.3	41.0	69.4
11.5	13.0	26.5	36.0	41.5	70.9
12.0	13.6	27.0	36.9	42.0	72.4
12.5	14.2	27.5	37.9	42.5	73.8
13.0	14.9	28.0	38.8	43.0	75.4
13.5	15.6	28.5	39.8	43.5	76.9
14.0	16.3	29.0	40.8	44.0	78.5
14.5	16.9	29.5	41.8	44.5	80.1
15.0	17.6	30.0	42.8	45.0	81.8
15.5	18.3	30.5	43.9	45.5	83.4
16.0	19.0	31.0	44.9	46.0	85.1
16.5	19.7	31.5	45.9	46.5	86.9
17.0	20.4	32.0	47.0	47.0	88.6
17.5	21.2	32.5	48.1	47.5	90.6
18.0	21.9	33.0	49.2	48.0	92.3
18.5	22.7	33.5	50.3	48.5	94.1
19.0	23.4	34.0	51.5	49.0	96.0
19.5	24.2	34.5	52.6	49.5	98.0
20.0	25.0	35.0	53.8	50.0	100.0

Figure 2-34. Speedy-moisture tester conversion chart, for use when not using the 1 1/4-inch steel balls

to extend the conversion curve beyond 44 percent moisture content (see *Figure 2-33*).

It may be more convenient for field use of the equipment to prepare a table of moisture-tester readings versus oven-dry moisture content.

SECTION IV. SPECIFIC-GRAVITY-OF-SOLIDS DETERMINATION (ASTM D 854-92)

The specific gravity of a solid substance is the ratio of the weight of a given volume of material to the weight of an equal volume of water (at 20°C). In effect, it tells how much heavier (or lighter) the material is than water. For exact analysis, the specifications require distilled or demineralized water and all measurements of water and solids should be made at stated temperatures. In dealing with soils testing, the value of specific gravity is necessary to compute the soil's void ratio and for determining the grain-size distribution in hydrometer analysis.

SPECIFIC GRAVITY OF SOIL OR SOLIDS

The term specific gravity of soil actually refers to the specific gravity of the solid matter of the soil, which is designated G_s. The specific gravity of solids is normally only applied to that fraction of a soil that passes the No. 4 sieve. Generally, geotechnical engineers need the soil's specific gravity to perform additional testing of that soil. In these cases there may be a different soil fraction used when performing this test. For example, the resulting specific-gravity value of soil from this test using a –10 sample is applicable to hydrometer analysis while the determination of the zero-air-voids curve in laboratory soils-compaction testing uses the specific gravity from the –4 sample.

A soil's specific gravity largely depends on the density of the minerals making up the individual soil particles. However, as a general guide, some typical values for specific soil types are as follows:

- The specific gravity of the solid substance of most inorganic soils varies between 2.60 and 2.80.
- Tropical iron-rich laterite, as well as some lateritic soils, usually have a specific gravity of between 2.75 and 3.0 but could be higher.
- Sand particles composed of quartz have a specific gravity ranging from 2.65 to 2.67.
- Inorganic clays generally range from 2.70 to 2.80.
- Soils with large amounts of organic matter or porous particles (such as diatomaceous earth) have specific gravities below 2.60. Some range as low as 2.00.

SPECIFIC-GRAVITY TEST

Take particular care to obtain representative samples for a specific-gravity test. It is easier to begin the test with an oven-dried sample. However, some soils, particularly those with a high organic content, are difficult to rewet. Test these at their natural water content and determine the oven-dried weight at the end of the test.

PURPOSE

Perform this test to determine the specific gravity of solids (which will be used to assist in the hydrometer-analysis test) and to calculate the zero-air-voids curve for compaction results.

EQUIPMENT

Perform the specific-gravity test in a laboratory environment, using the following items (see *Figure 2-35*):

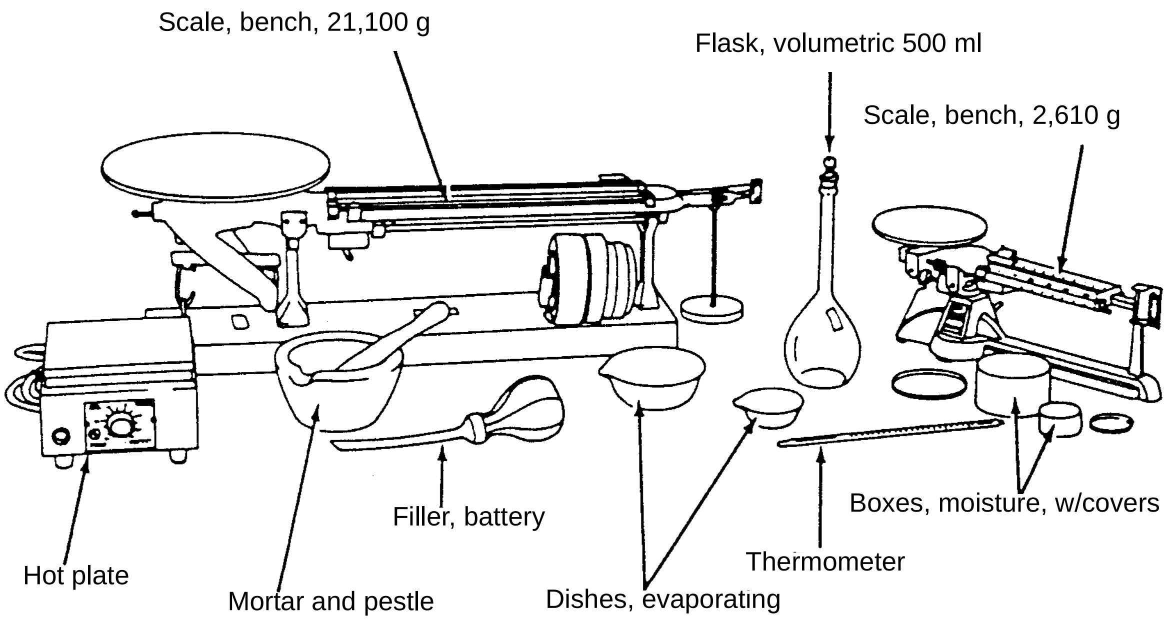


Figure 2-35. Apparatus for determining specific gravity of soils

- A pycnometer; volumetric flask (500-milliliter capacity).
- A laboratory oven.
- Heat-resistant gloves.
- Balance scales sensitive to 0.01 gram.
- Pudding pans.
- A waterbath.
- A thermometer.
- A battery-filler syringe.
- A No. 4 sieve (for general specific-gravity results).

- A No. 10 sieve (when results are used for hydrometer analysis).
- Graph paper.
- A pencil.
- A french curve.
- Cloth towels.
- Paper towels.
- A hot plate.
- DD Form 1208.
- A spatula.
- An evaporating dish.
- Distilled water.
- A calculator.
- A pail.

STEPS

Perform the following steps to determine the soil's specific gravity:

Step 1. Calibrate the pycnometer (volumetric flask). If a calibration graph has already been prepared for this pycnometer and will be used for the determination procedures, go to step 2.

a. Weigh a clean, dry pycnometer to the nearest 0.01 gram. Record this information, W_b, on DD Form 1208 (see *Figure 2-36*). Additionally, record the basic information concerning the specimens being tested and the pycnometer/flask number.

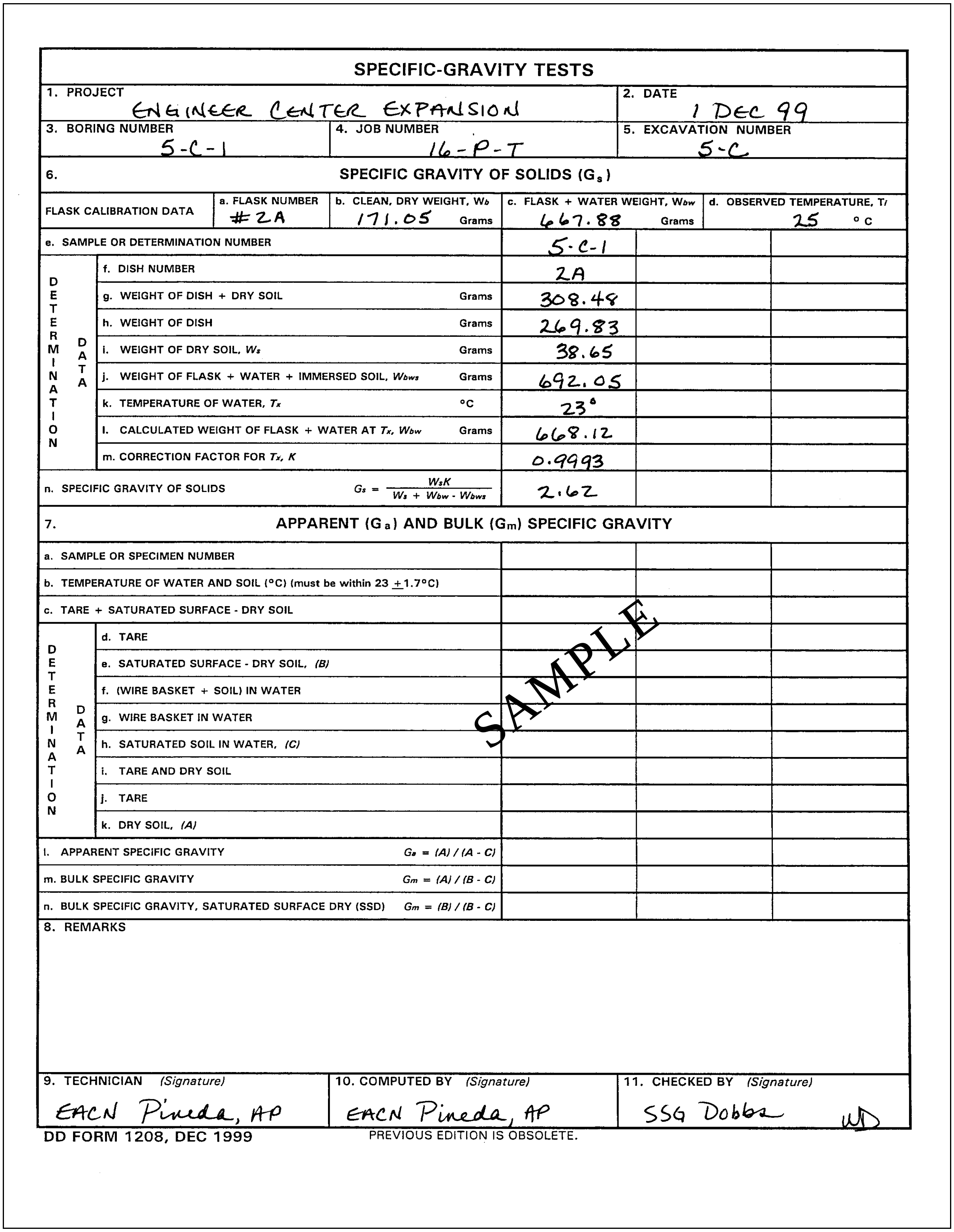
b. Fill the pycnometer with room-temperature distilled water. Ensure that the bottom of the meniscus is even with the calibration mark.

c. Weigh the pycnometer plus water. Record this information, W_{bw}, on the form.

d. Use the thermometer to determine the water temperature to the nearest whole degree, T_i. Record the temperature on the form.

e. Create a graph or table for the pycnometer being used (if additional specific-gravity determinations are to be made).

NOTE: This graph helps in determining values of W_{bw} for any desired water temperatures and eliminates the need to calibrate the pycnometer for each test. The graph can be developed by using the following equation for various temperatures, plotting the temperatures against the weight of the pycnometer and water, and drawing a smooth curve through the plotted points:



SPECIFIC-GRAVITY TESTS

1. PROJECT ENGINEER CENTER EXPANSION	2. DATE 1 DEC 99

3. BORING NUMBER 5-C-1	4. JOB NUMBER 16-P-T	5. EXCAVATION NUMBER 5-C

6. SPECIFIC GRAVITY OF SOLIDS (G_s)

FLASK CALIBRATION DATA	a. FLASK NUMBER #2A	b. CLEAN, DRY WEIGHT, W_b 171.05 Grams	c. FLASK + WATER WEIGHT, W_{bw} 667.88 Grams	d. OBSERVED TEMPERATURE, T_i 25 °C

e. SAMPLE OR DETERMINATION NUMBER			5-C-1		
DETERMINATION DATA	f. DISH NUMBER		2A		
	g. WEIGHT OF DISH + DRY SOIL	Grams	308.48		
	h. WEIGHT OF DISH	Grams	269.83		
	i. WEIGHT OF DRY SOIL, W_s	Grams	38.65		
	j. WEIGHT OF FLASK + WATER + IMMERSED SOIL, W_{bws}	Grams	692.05		
	k. TEMPERATURE OF WATER, T_x	°C	23°		
	l. CALCULATED WEIGHT OF FLASK + WATER AT T_x, W_{bw}	Grams	668.12		
	m. CORRECTION FACTOR FOR T_x, K		0.9993		
n. SPECIFIC GRAVITY OF SOLIDS		$G_s = \frac{W_sK}{W_s + W_{bw} - W_{bws}}$	2.62		

7. APPARENT (G_a) AND BULK (G_m) SPECIFIC GRAVITY

a. SAMPLE OR SPECIMEN NUMBER					
b. TEMPERATURE OF WATER AND SOIL (°C) (must be within 23 ± 1.7°C)					
c. TARE + SATURATED SURFACE - DRY SOIL					
DETERMINATION DATA	d. TARE				
	e. SATURATED SURFACE - DRY SOIL, *(B)*				
	f. (WIRE BASKET + SOIL) IN WATER				
	g. WIRE BASKET IN WATER				
	h. SATURATED SOIL IN WATER, *(C)*				
	i. TARE AND DRY SOIL				
	j. TARE				
	k. DRY SOIL, *(A)*				
l. APPARENT SPECIFIC GRAVITY		$G_a = (A) / (A - C)$			
m. BULK SPECIFIC GRAVITY		$G_m = (A) / (B - C)$			
n. BULK SPECIFIC GRAVITY, SATURATED SURFACE DRY (SSD)		$G_m = (B) / (B - C)$			

8. REMARKS

9. TECHNICIAN *(Signature)* EACN Pineda, AP	10. COMPUTED BY *(Signature)* EACN Pineda, AP	11. CHECKED BY *(Signature)* SSG Dobbs WD

DD FORM 1208, DEC 1999

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Figure 2-36. Sample DD Form 1208

$$W_{bw}\ (\text{for specified temperature}, T_x) = \left[\frac{\rho_w(T_x)}{\rho_w(T_i)} \times \left[(W_{bw}\ \text{at}\ T_i) - W_b\right]\right] + W_b$$

where—

ρ_w (T_x) = ***density of water identified by temperature (T_x) (see Table 2-7)***

ρ_w (T_i) = ***density of water identified by temperature (T_i) (see Table 2-7)***

W_{bw} = ***weight of pycnometer and water, in grams***

W_b = ***weight of pycnometer, in grams***

T_i = ***observed/recorded temperature of water, in °C***

T_x = ***any other desired temperature, in °C***

Table 2-7. Relative density of water and correction factor (K) at various temperatures

Temp °C	Relative Density	Correction Factor (K)
18.0	0.99862	1.0004
19.0	0.99843	1.0002
20.0	0.99823	1.0000
21.0	0.99802	0.9998
22.0	0.99780	0.9996
23.0	0.99757	0.9993
24.0	0.99733	0.9991
25.0	0.99708	0.9988
26.0	0.99682	0.9986
27.0	0.99655	0.9983
28.0	0.99627	0.9980
29.0	0.99598	0.9977
30.0	0.99568	0.9974
31.0	0.99537	0.9971
32.0	0.99505	0.9968

NOTE: Data obtained from ASTM. Correction factor, K, is found by dividing the relative density of water at the test temperature by the relative density of water at 20°C.

A completed graph using the above formula for the following data can be seen in *Figure 2-37*.

Calibration data:

W_{bw} = **656.43**

W_b = **158.68**

T_i = **24°C**

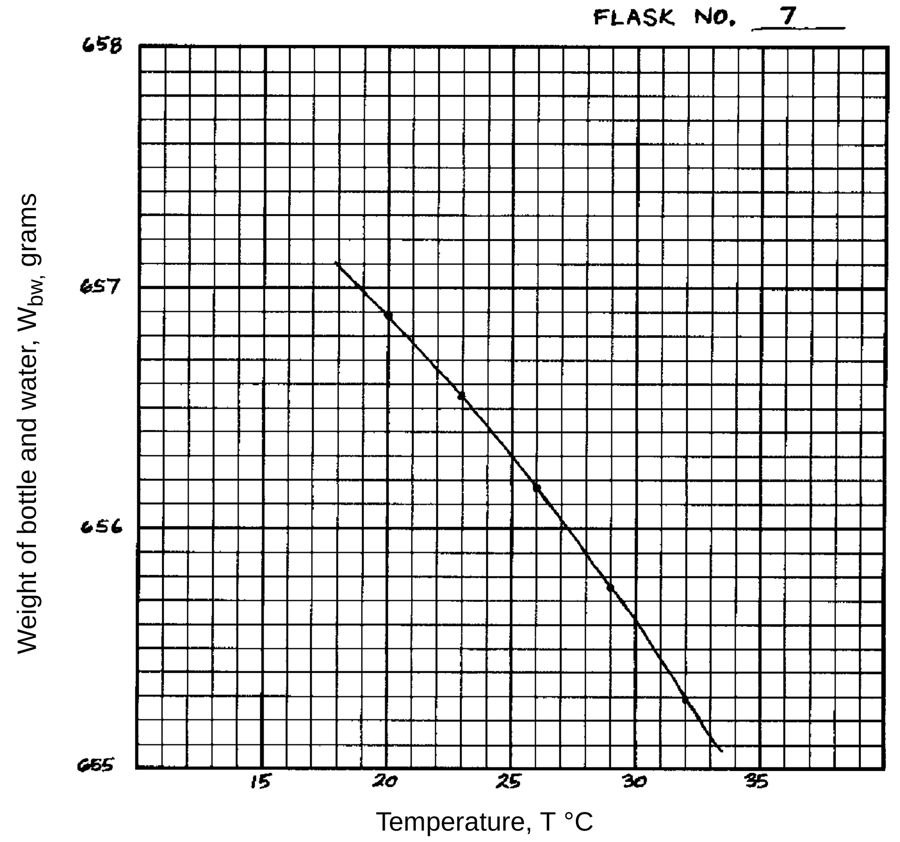


Figure 2-37. Calibration curve for a volumetric flask

Computed data:

T_x of 20°C yields W_{bw} of 656.88

T_x of 23°C yields W_{bw} of 656.55

T_x of 26°C yields W_{bw} of 656.17

T_x of 29°C yields W_{bw} of 655.75

T_x of 32°C yields W_{bw} of 655.29

A complete data table can be created from the formula above for each temperature expected to prevail during testing.

Step 2. Obtain a soil sample for testing. Separate the given sample over a No. 4 sieve to obtain at least a 100-gram sample passing the sieve or over a No. 10 sieve to obtain a 20-gram sample. Since this test method is only concerned with the sample passing the appropriate sieve used, discard the material retained on the sieve.

Step 3. Prepare the sample for testing.

NOTE: To determine the specific gravity of solids, the sample may be at its natural water content or oven-dried. Soils with a high organic content or with fines that are low compressible are difficult to rewet after having been oven-dried. These soils should be tested at their natural water content first and the oven-dried weight determined at the end of the test.

a. Record all identifying information about the sample on the form (see *Figure 2-36, page 2-65*).

b. Place the –4 or –10 sample into the evaporating dish.

c. Perform the following procedures for soil at natural water content or moisture; otherwise, go to step 3d:

(1) Add distilled water to the sample and mix to a slurry.

(2) Transfer the slurry to the pycnometer and add distilled water until the pycnometer is about three-fourths full.

d. Perform the following procedures for an oven-dried soil sample:

(1) Oven-dry the sample to a constant weight at a temperature of 110° ± 5°C. Allow the sample to cool and weigh it to the nearest 0.01 gram. Record the weight on the form as the weight of the dish and dry soil (in block 6g).

(2) Transfer the dried sample to the volumetric flask. Take care to avoid the loss of any particles.

(3) Fill the flask three-fourths full with distilled water and allow it to soak for 12 hours.

(4) Weigh the empty, dry evaporating dish. Record the weight on the form as the weight of the dish (block 6h).

Step 4. Process the sample through the test method.

a. Remove entrapped air by bringing the solution to a slow, rolling boil for 10 minutes while occasionally rolling the pycnometer to assist in the removal of the air (ensure that no loss of material occurs while boiling). Cool the sample to room temperature.

b. Fill the pycnometer with distilled water until the bottom of the meniscus is level with the calibration mark.

c. Dry the outside and thoroughly remove any moisture adhering to the neck of the pycnometer.

d. Weigh the pycnometer and its contents to the nearest 0.01 gram. Record this amount on the form as the weight of the flask and water and immersed soil (W_{bws}).

e. Shake the flask immediately after weighing (putting its contents in suspension) and determine the water temperature at middepth to the nearest whole degree, T_x. Record this amount on the form.

f. Determine the dry unit weight for soil processed at natural moisture content as follows:

(1) Transfer the soil solution from the flask to a preweighed pudding pan. Record the weight in block 6h. Use care when transferring all the grains of soil.

(2) Oven-dry the sample to a constant weight at a temperature of 110° ± 5°C. Allow the sample to cool. Weigh and record the weight on the form in block 6g as the weight of the dish and dry soil.

Step 5. Compute the results on DD Form 1208 (see *Figure 2-36*, *page 2-65*).

a. Compute the weight of the dry soil (W_s) by subtracting the weight of the dish from the weight of the dish and dry soil. Record it on the form.

b. Determine the weight of the flask and water (W_{bw}) by plotting the temperature of the water (T_x) obtained in step 4e (block 6k) on the calibration curve obtained in step 1. Record the result on the form. If the calibration curve and graph were not produced, use the formula as indicated in step 1e and record the result on the form.

c. Determine the correction factor (K) by locating the temperature of the water (T_x) (obtained in step 4e [block 6k]) in *Table 2-7, page 2-66*; read across to the correction factor column, and record it on the form as the correction factor K (for T_x).

d. Compute the specific gravity of solids (G_s) to two decimal places. Record the amount on the form using the following formula:

$$G_s = \frac{W_s K}{W_s + W_{bw} - W_{bws}}$$

APPARENT AND BULK SPECIFIC GRAVITY

The specific gravity of solids is not applied to coarse particles because they normally contain voids from which air cannot be displaced unless the particles are ground into finer particles so as to eliminate the voids. Thus, when dealing with coarser particles, it is more convenient to work with the apparent specific gravity of the particle mass or to determine the bulk specific gravity. Test methods for these determinations are listed in Chapters 3 and 4.

The apparent specific gravity is designated G_a and is the ratio of the weight in air of a unit volume of the impermeable portion of aggregate to the weight in air of an equal volume of distilled water, both at a stated temperature. The impermeable portion of a porous material, such as most large soil grains, includes the solid material plus impermeable pores or voids within the particles. This test method is applicable to the testing of fine and coarse aggregates (see Chapters 3 and 4).

The bulk specific gravity is designated G_m and is the ratio of the weight in air of a unit volume of aggregate (including permeable and impermeable voids in the particles, but not the voids between the particles) to the weight of an equal volume of distilled water at a stated temperature. This test method is applicable to the testing of soils with fine and coarse aggregates (see Chapters 3 and 4).

SECTION V. GRAIN-SIZE ANALYSIS AND DISTRIBUTION (ASTM D 422-63 AND ASTM 2217-85)

Soil particles, also referred to as grains, are discussed in Section I of this chapter, with some consideration of the effects of particle characteristics on the physical properties of soils. The use of grain size and grain-size distribution in soil classification and visual-manual tests—and their use for field identification—are also covered in Section I. Although estimates of grain size of coarser materials may be made in this way, the accurate determination of the grain-size distribution or gradation of coarse-soil fractions requires a grain-size analysis.

Grain-size analysis, which is among the oldest of soil tests, is used in soils classification and as part of the specifications of soil for airfields, roads, earth dams, and other soil-embankment construction. The standard grain-size-analysis test determines the relative proportions of different grain sizes as they are distributed among certain size ranges, which is referred to as particle-size or grain-size distribution. This is accomplished in two steps:

- A screening process (a sieve analysis, which is also called a mechanical analysis) for particle sizes retained on the No. 200 sieve.
- A sedimentation process (a hydrometer analysis) for particle sizes smaller than the No. 200 sieve.

NOTE: Previous test methods presented the sieve analysis and the hydrometer analysis as two separate test methods, and a combination of these analyses was referred to as a combined analysis. ASTM employs a different method for particle-size analysis which includes both methods (ASTM D 422-63). This single method also references the test method specific to wet preparation of soil samples (ASTM D 2217-85). These test methods provide for minor modifications to allow the end user to obtain results specific to the purpose of the test. The following test method is a product of this modification. It allows for easier identification of the USCS classification.

Performing just the sieve-analysis portion of this test method may yield sufficient information to classify a soil type and therefore not require the hydrometer analysis. However, the hydrometer analysis will ensure a more accurate depiction of the soil gradation as well as provide necessary information required to determine the soil's frost susceptibility.

SIEVE ANALYSIS (MECHANICAL ANALYSIS)

The accurate completion of the sieve-analysis test will produce the percent of gravel, sand, and fines of the material. The most accurate process for this test method is to wash the material over the sieves; this will give a more accurate percent of fines. It is possible the test will also provide sufficient information to calculate the coefficient of uniformity and the coefficient of curvature.

PURPOSE

Perform this test to determine the grain-size distribution or the gradation of a soil or aggregate for the portion of the material that is larger than the No. 200 sieve. The results of this test should assist in the soil-classification process.

EQUIPMENT

Use the following items to perform this test (see *Figure 2-38*):

- A calculator.
- DD Form 1206 or an equivalent form.
- DD Form 1207.
- Beam scales.

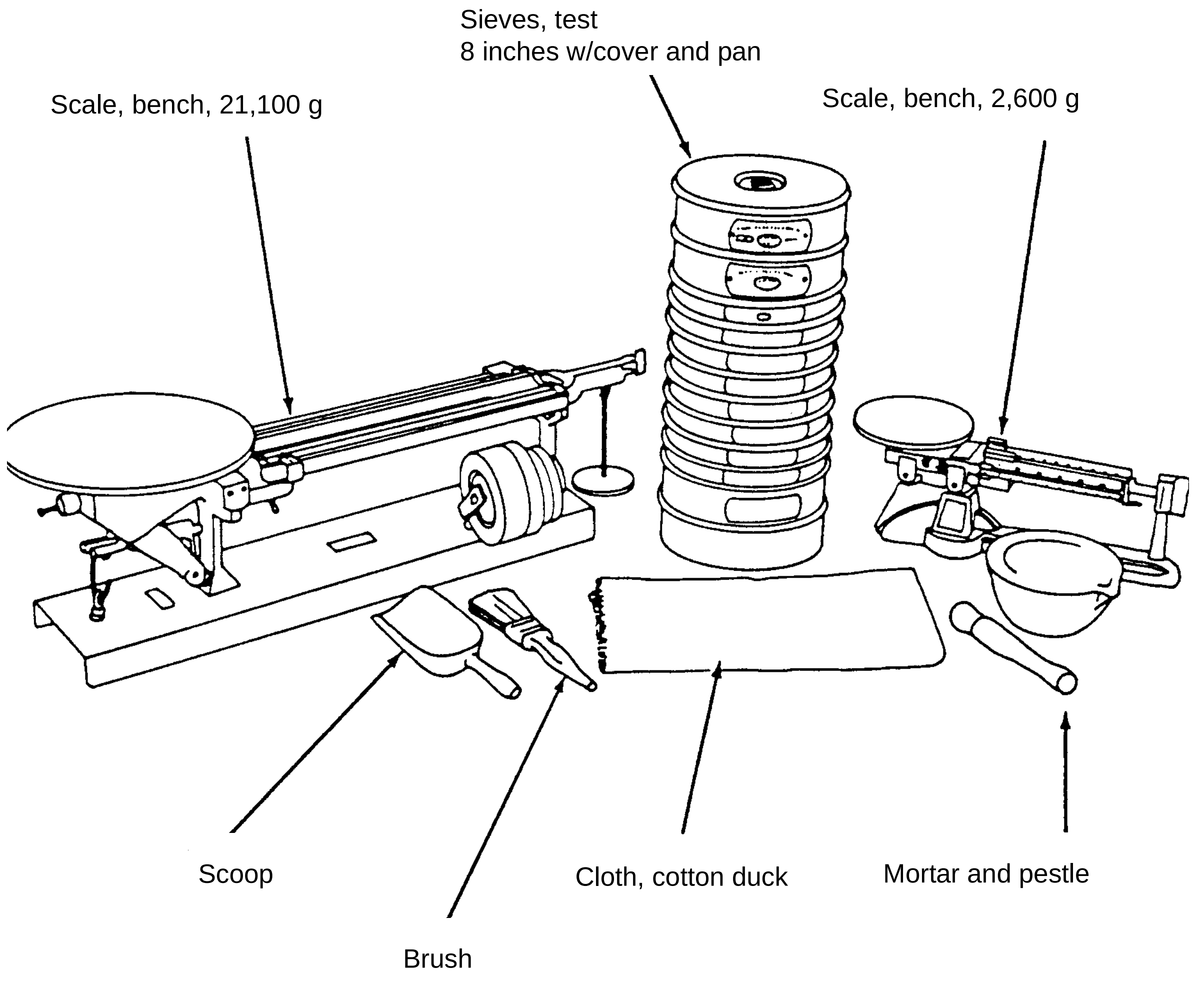


Figure 2-38. Equipment for sieve analysis

- Balance scales sensitive to 0.1 gram and 0.01 gram.
- A scoop.
- A brush.
- A sieve shaker.
- A nest of sieves including, as a minimum, the following sizes: 2 inches, 1.5 inches, 1 inch, 3/4 inch, 3/8 inch, No. 4, No. 10, No. 16, No. 30, No. 40, No. 50, No. 100, and No. 200.
- A pan.
- A cover.
- A mortar.
- A rubber-covered pestle.
- Pudding pans.
- Paper.
- A pencil.
- A french curve.
- A splitter (if available).
- Canvas (in a laboratory environment).
- A laboratory oven.
- Heat-resistant gloves.
- A battery-filler syringe.
- Distilled water.

STEPS

Perform the following steps to determine the grain-size distribution:

Step 1. Prepare the soil sample.

a. Spread out and air-dry the soil sample.

b. Break up the aggregate particles thoroughly with fingers or with the mortar and pestle.

c. Obtain a representative sample for testing by using a sample splitter or by quartering. The sample size recommended for sieve analysis depends on the particle size. Obtain the required minimum sample as listed in *Table 2-8*.

Step 2. Record all identifying information about the sample (such as the project name, excavation number, sample number, description of sample, and date [blocks 1 through 7]) on DD Form 1206 (see *Figure 2-39, page 2-74*).

Step 3. Oven-dry the material at 110°C ± 5° until a constant weight is obtained. Allow the sample to cool.

Step 4. Weigh the oven-dried sample and record the weight on the form (block 8) to the nearest gram as the weight of the original sample.

Table 2-8. Representative soil samples for grain-size analysis

Maximum Particle Size		Minimum Sample Weight (g)
in	mm	
3/8 (No. 4)	9.5 (No. 4)	500
3/4	19.0	1,000
1	25.4	2,000
1 1/2	38.1	3,000
2	50.8	4,000
3	76.2	5,000

Step 5. Check “No” in block 9 and enter 0 in blocks 10 and 11 if only a dry sieve is to be performed, then proceed to step 10. If the sample will be prewashed, check “Yes” in block 9 and proceed to step 6.

Step 6. Place the sample in a clean container and cover the sample completely with water. Allow the sample to soak until the adhering and lumpy particles are completely disintegrated. This process may take 2 to 24 hours.

Step 7. Wash the sample over a No. 200 sieve into a 2 x 2 concrete pan until all –200 material has been washed through. If the sample contains an appreciable amount of coarse particles, combine the No. 4 and No. 200 sieves. Take care not to overload the No. 200 sieve. If necessary, transfer the sample in increments (this process may take up to 6 different pans and as long as 8 hours).

Step 8. Process the +200 material. Oven-dry the washed +200 material at 110°C ± 5° until a constant weight is obtained and allow the material to cool. Record the weight on the form to the nearest tenth of a gram (block 10).

Step 9. Process the –200 material.

a. Allow the –200 material to settle in the pan until the surface water becomes clear (16 to 24 hours).

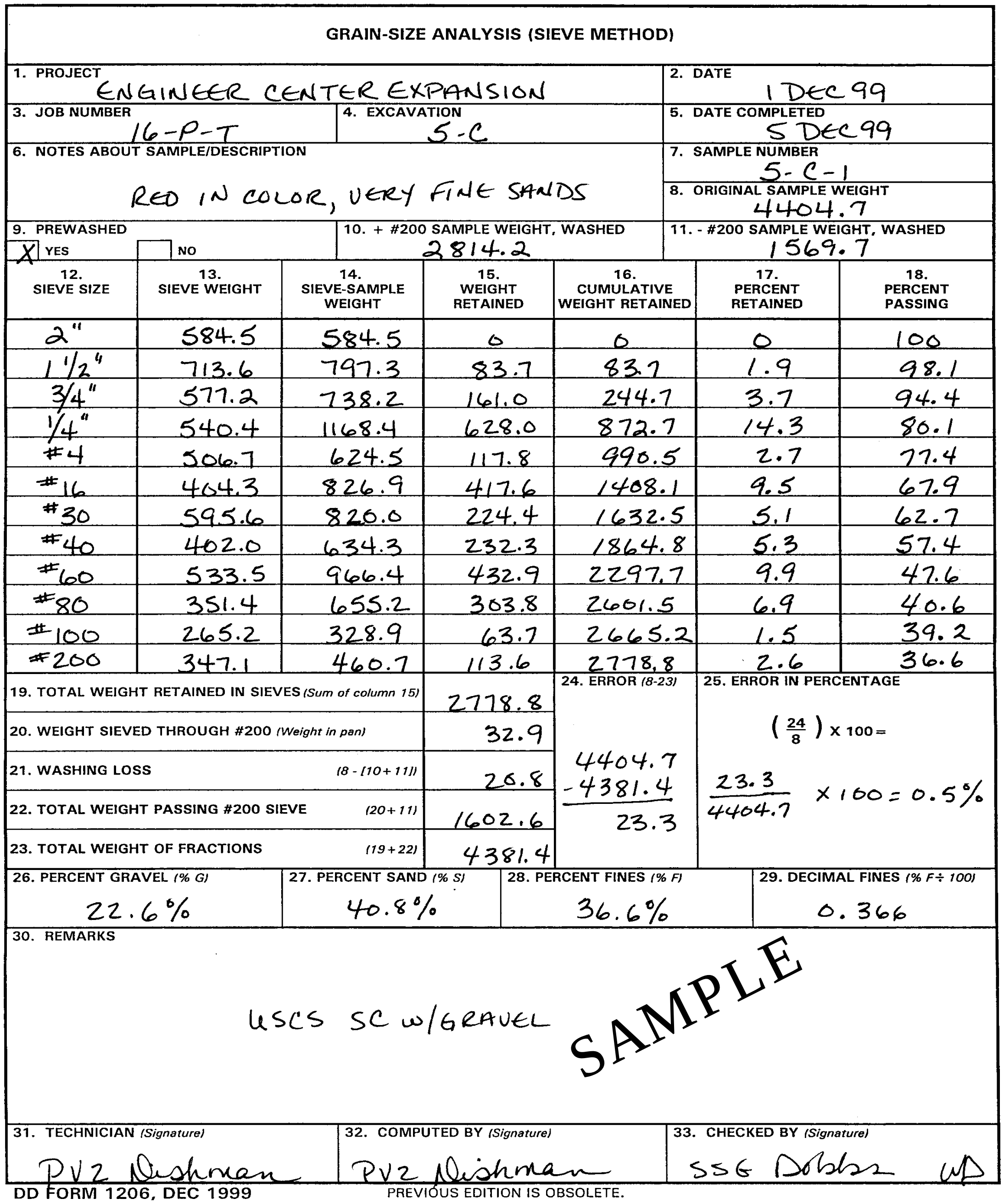
b. Decant the surface water (using a siphon or a syringe), ensuring that the settled material is not disturbed.

c. Use a trowel to transfer as much of the material as possible from the pan to the pudding pans.

d. Rinse the remainder of the material from the 2 x 2 pans to the pudding pans with as little water as possible.

e. Oven-dry the washed –200 material and determine the total –200 sample weight to the nearest tenth of a gram. Record this weight on the form (block 11). Retain this material for use in the hydrometer analysis.

Step 10. Select a nest of sieves to accommodate the largest particle size of the soil being tested, ensuring that all material will pass through the largest sieve. As a minimum, the following sieves sizes will be used (up to the largest



GRAIN-SIZE ANALYSIS (SIEVE METHOD)

1. PROJECT: ENGINEER CENTER EXPANSION
2. DATE: 1 DEC 99
3. JOB NUMBER: 16-P-T
4. EXCAVATION: 5-C
5. DATE COMPLETED: 5 DEC 99
6. NOTES ABOUT SAMPLE/DESCRIPTION: RED IN COLOR, VERY FINE SANDS
7. SAMPLE NUMBER: 5-C-1
8. ORIGINAL SAMPLE WEIGHT: 4404.7
9. PREWASHED: ☒ YES ☐ NO
10. + #200 SAMPLE WEIGHT, WASHED: 2814.2
11. - #200 SAMPLE WEIGHT, WASHED: 1569.7

12. SIEVE SIZE	13. SIEVE WEIGHT	14. SIEVE-SAMPLE WEIGHT	15. WEIGHT RETAINED	16. CUMULATIVE WEIGHT RETAINED	17. PERCENT RETAINED	18. PERCENT PASSING
2"	584.5	584.5	0	0	0	100
1 1/2"	713.6	797.3	83.7	83.7	1.9	98.1
3/4"	577.2	738.2	161.0	244.7	3.7	94.4
1/4"	540.4	1168.4	628.0	872.7	14.3	80.1
#4	506.7	624.5	117.8	990.5	2.7	77.4
#16	464.3	826.9	417.6	1408.1	9.5	67.9
#30	595.6	820.0	224.4	1632.5	5.1	62.7
#40	402.0	634.3	232.3	1864.8	5.3	57.4
#60	533.5	966.4	432.9	2297.7	9.9	47.6
#80	351.4	655.2	303.8	2601.5	6.9	40.6
#100	265.2	328.9	63.7	2665.2	1.5	39.2
#200	347.1	460.7	113.6	2778.8	2.6	36.6

19. TOTAL WEIGHT RETAINED IN SIEVES *(Sum of column 15)*	2778.8
20. WEIGHT SIEVED THROUGH #200 *(Weight in pan)*	32.9
21. WASHING LOSS *(8 - [10+11])*	26.8
22. TOTAL WEIGHT PASSING #200 SIEVE *(20+11)*	1602.6
23. TOTAL WEIGHT OF FRACTIONS *(19+22)*	4381.4

24. ERROR *(8-23)*: 4404.7 - 4381.4 = 23.3

25. ERROR IN PERCENTAGE: $(\frac{24}{8}) \times 100 =$ $\frac{23.3}{4404.7} \times 100 = 0.5\%$

26. PERCENT GRAVEL *(% G)*: 22.6%
27. PERCENT SAND *(% S)*: 40.8%
28. PERCENT FINES *(% F)*: 36.6%
29. DECIMAL FINES *(% F ÷ 100)*: 0.366

30. REMARKS: USCS SC w/GRAVEL

SAMPLE

31. TECHNICIAN *(Signature)*: PV2 Dishman
32. COMPUTED BY *(Signature)*: PV2 Dishman
33. CHECKED BY *(Signature)*: SSG Dobbs

DD FORM 1206, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

Figure 2-39. Sample DD Form 1206

particle size): 2 inches, 1.5 inches, 1 inch, 3/4 inch, 3/8 inch, No. 4, No. 10, No. 16, No. 30, No. 40, No. 50, No. 100, and No. 200.

Step 11. Record the weight of each sieve selected on the form to the nearest tenth of a gram (column 13), and arrange the sieves in a nest with the smallest sieve size on the bottom. Weigh and place a pan on the bottom.

Step 12. Cover the sample. If the sample was prewashed, place only the +200 material onto the top sieve of the nest and place a cover over it. If the sample was not prewashed, place the entire sample on the top sieve of the nest and place a cover over it.

Step 13. Place the nest of sieves and the sample in the sieve shaker and shake for 10 to 15 minutes (see *Figure 2-40*).

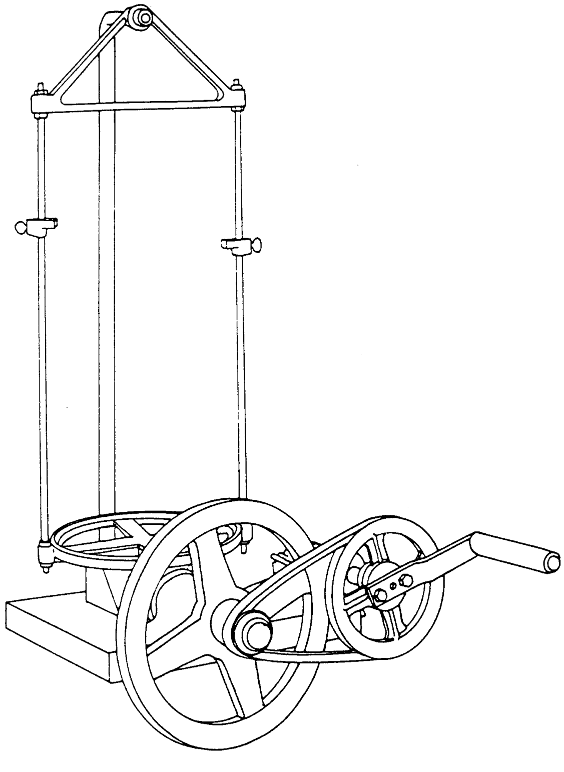


Figure 2-40. Hand-operated sieve shaker

Step 14. Remove the cover of the sieve nest after the shaking has been completed.

Step 15. Record the weight of each sieve with the retained sample (starting with the top sieve (see *Figure 2-41, page 2-76*) on the form (column 14).

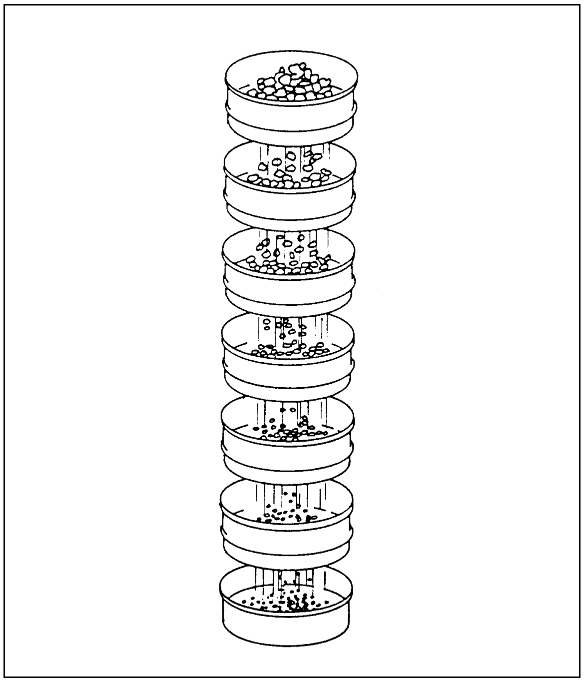


Figure 2-41. Testing sieves stacked large to small

Step 16. Determine the weight of the material retained on each sieve by subtracting the weight of the sieve from the weight of the sieve and retained sample (columns 14 through 13). Record this weight as the weight retained (column 15).

Step 17. Add the weights retained on all sieves and record as total weight retained in sieves (block 19).

Step 18. Weigh the pan with the material passing the No. 200 sieve. Subtract the weight of the pan (from step 11) and record this as the weight sieved through No. 200 (block 20).

Step 19. Complete blocks 21 through 25 of the form using the formulas provided on the sheet. If the error of percentage is 1 percent or greater, rerun the test.

Step 20. Compute the cumulative weight retained (column 16) for each sieve by adding the weight retained to the previous cumulative weight retained with the starting point being 0.

Step 21. Compute the percent retained (column 17) for each sieve by dividing the weight retained by the total weight of fractions as follows:

$$\frac{column\ 15}{block\ 23} \times 100$$

Step 22. Compute the percent passing for each sieve size by subtracting the cumulative weight retained from the total weight of fractions and dividing by the total weight of fractions as follows:

$$column\ 18 = \frac{block\ 23 - column\ 16}{block\ 23} \times 100$$

Step 23. Determine the percentages for gravel, sand, and fines. Record the information on the form.

- Gravel is the material retained on the No. 4 sieve.
- Sand is the material passing the No. 4 sieve and retained on the No. 200 sieve.
- Fines are the material passing the No. 200 sieve.

Step 24. Prepare DD Form 1207 (see *Figure 2-42, page 2-78*).

a. Record the identifying information for the sample in the remarks block.

b. Use the sieve-analysis data to plot (on DD Form 1207) the sieve size and the percentage passing the sieve.

c. Using a french curve, connect the plotted points to form a smooth, free-flowing curve (the grain-size distribution curve, *Figure 2-42*).

d. Determine the coefficient of uniformity (C_u).

NOTE: The grain size, in millimeters, which corresponds to 10 percent passing on the grain-size-distribution curve, is called Hazen's effective size. It is designated by the symbol D_{10}. If the grain-size-distribution curve extends to or below 10 percent passing, then the C_u can be determined. The uniformity coefficient is the ratio between the grain diameter, in millimeters, corresponding to 60 percent passing (D_{60}) and 10 percent passing on the curve. Use the following formula and record on the form:

$$C_u = \frac{D_{60}}{D_{10}}$$

If D_{10} cannot be determined using the data from the sieve analysis, a hydrometer analysis may be required to obtain information about the smaller size grains and to extend the distribution curve to make it more complete.

e. Determine the coefficient of curvature (C_c) by using D_{60} and D_{10} as previously discussed and D_{30}, the grain diameter, in millimeters, corresponding to 30 percent passing on the grain-size-distribution curve. These numbers are used in the following formula and recorded on the form:

$$C_c = \frac{(D_{30})^2}{(D_{60} \times D_{10})}$$

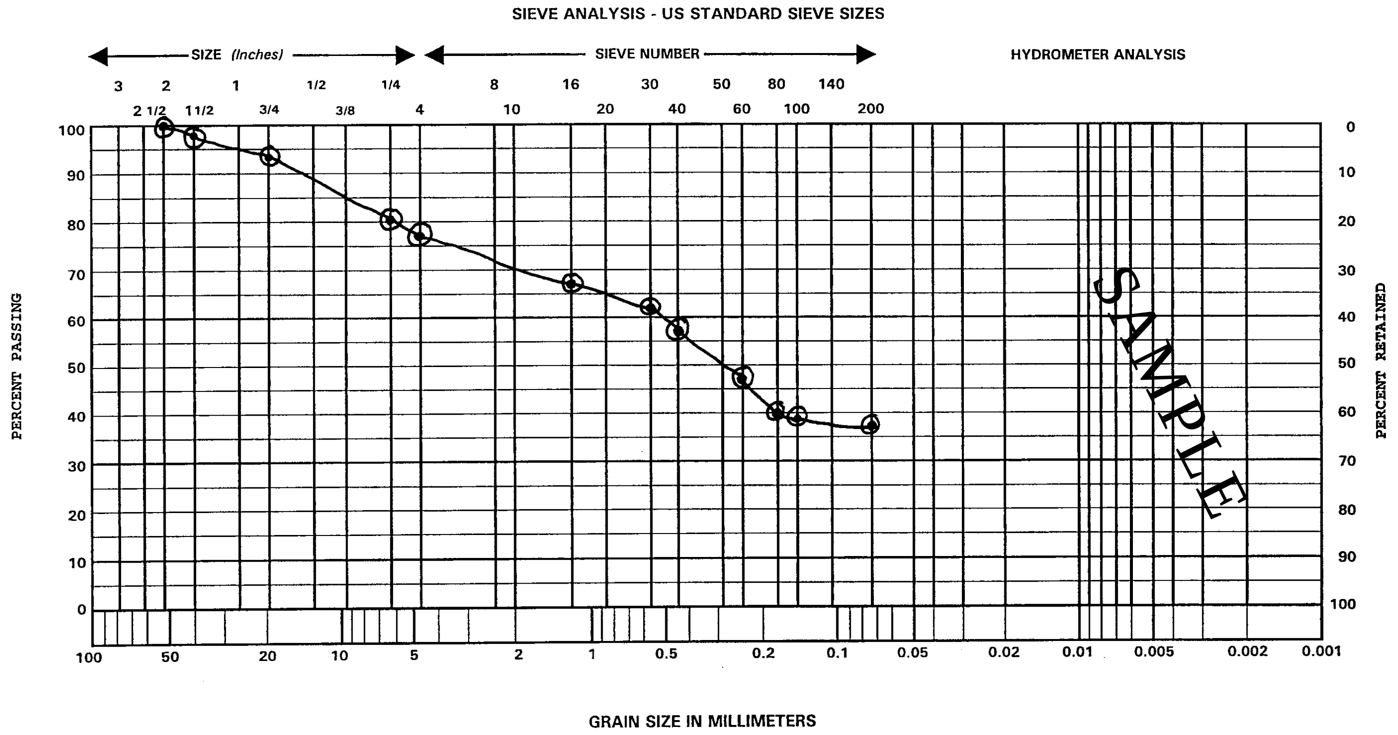
NOTE: The values for D_{60}, D_{10}, and D_{30} are obtained by going to the percent passing by weight on the left vertical scale, then moving horizontally across to the right until the grain-size-distribution curve is intercepted, and then vertically down to the horizontal axis where the diameter of the material is read in millimeters. See *Figure 2-42,* for the completed gradation chart.

Step 25. Determine the gradation by using the abbreviated information listed below. Record the information on the form.

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADATION CHART

1. PROJECT ENGINEER CENTER EXPANSION 16-P-T

2. DATE 6 DEC 99



EXCAVATION NUMBER	SAMPLE NUMBER	LL	PL	PI	Cu (D_{60}/D_{10})	Cc $(D_{30})^2/(D_{60} \times D_{10})$	SOIL DESCRIPTION/REMARKS	CLASSIFICATION (USCS)
5-C	5-C-1	20	10	10	N/A	N/A	RED IN COLOR, VERY FINE SANDS.	SC

3. TECHNICIAN (Signature) PV2 Dishman

4. PLOTTED BY (Signature) PV2 Dishman

5. CHECKED BY (Signature) SSG Dobbs WD

DD FORM 1207, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

Figure 2-42. Sample DD Form 1207

a. The soil is well-graded (W) when all of the following apply:

- C_u is greater than 4 if the soil is predominantly gravel and greater than 6 if the soil is predominantly sand.
- C_c is at least 1.0 but not more than 3.0 for both gravel and sand.

An indicator of a well-graded soil is a smooth curve plotted for grain-size distribution. The curve must not have any horizontal or vertical portions and must be continuous.

b. The soil is poorly graded (P) if any of the above criteria is not fulfilled.

HYDROMETER ANALYSIS

Hydrometer analysis is based on Stokes' law, which relates the terminal velocity of a free-falling sphere in a liquid to its diameter—or, in simpler terms, the larger the grain size, the greater its settling velocity in a fluid. It is assumed that Stokes' law can be applied to a mass of dispersed soil particles of various shapes and sizes. Larger particles settle more rapidly than smaller ones. The hydrometer analysis is an application of Stokes' law that permits calculating the grain-size distribution in silts and clays, where the soil particles are given the sizes of equivalent spherical particles.

The density of a soil-water suspension depends on the concentration and specific gravity of the soil particles. If the suspension is allowed to stand, the particles gradually settle out of the suspension and the density decreases. The hydrometer is used to measure the density of the suspension at a known depth below the surface. The density measurement, together with knowledge of specific gravity of the soil particles, determines the percentage of dispersed soil particles in suspension at the time and depth of measurement.

The depth at which the measurement is made is found by calibrating the hydrometer. Stokes' law is used to calculate the maximum equivalent particle diameter for the material in suspension at this depth and for the elapsed time of settlement. A series of density measurements at known depths of suspension and at known times of settlement give the percentages of particles finer than the diameters given by Stokes' law. Thus, the series of readings will reflect the amount of different sizes of particles in the fine-grained soils. The particle diameter (D) is calculated from Stokes' equation using the corrected hydrometer reading.

PURPOSE

Perform the hydrometer analysis to determine the grain-size distribution of the –200 material in a soil, to assist in determining the frost susceptibility of a soil, and to provide data needed to calculate the coefficient of uniformity and coefficient of curvature.

EQUIPMENT

Perform the hydrometer analysis in a laboratory environment using the following items:

- A hydrometer.
- A laboratory oven.

- Heat-resistant gloves.
- Balance scales sensitive to 0.1 gram and 0.01 gram.
- A thermometer.
- A timing device with a second hand.
- A No. 200 sieve.
- A battery-filler syringe.
- Distilled water.
- A dispersing agent.
- Pudding pans.
- Two graduated glass cylinders (1,000-milliliter) with cap.
- DD Form 1207.
- DD Form 1794.
- A calculator.
- Paper.
- A pencil.
- A grease pencil.
- Graph paper.
- A straightedge.
- A mechanically operated stirring device with a dispersion cup.

STEPS

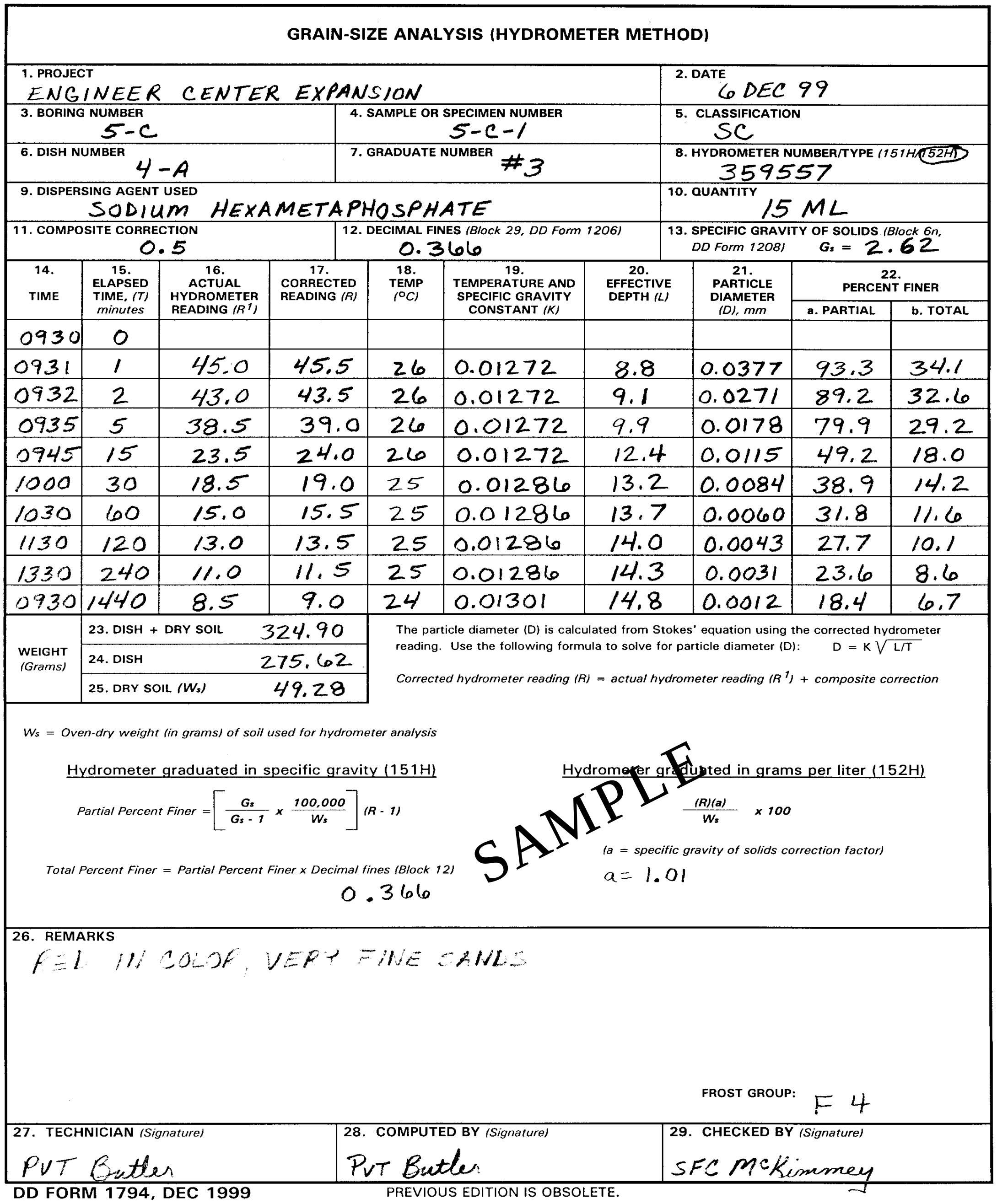
Perform the following steps for the hydrometer analysis:

Step 1. Prepare the sample.

a. Obtain the –200 sample as prepared in the sieve analysis. The size of the –200 sample varies according to the type of soil being tested. Approximately 100 grams are required for sandy soils and 50 grams for silty or clayey soils. Place the sample in a dish and add distilled water until the sample is submerged.

b. Determine the amount and type of dispersing agent that will be used during the test. Record it on DD Form 1794 (blocks 9 and 10) (see *Figure 2-43*). The dispersing agents shown in *Table 2-9, page 2-82,* are listed in approximate order of effectiveness. They have been found to be satisfactory for most types of soils. In most instances, 15 milliliters of a dispersing-agent solution is adequate to control flocculation (the adherence of fine soil grains to each other in clusters while in suspension). An additional 15 milliliters can be added a second or third time if flocculation continues.

c. Add the predetermined amount of dispersing agent to the soaking soil sample and allow the sample to soak at least 16 hours.



GRAIN-SIZE ANALYSIS (HYDROMETER METHOD)

1. PROJECT ENGINEER CENTER EXPANSION		2. DATE 6 DEC 99
3. BORING NUMBER 5-C	4. SAMPLE OR SPECIMEN NUMBER 5-C-1	5. CLASSIFICATION SC
6. DISH NUMBER 4-A	7. GRADUATE NUMBER #3	8. HYDROMETER NUMBER/TYPE *(151H/ [✓ 152H])* 359557
9. DISPERSING AGENT USED SODIUM HEXAMETAPHOSPHATE		10. QUANTITY 15 ML
11. COMPOSITE CORRECTION 0.5	12. DECIMAL FINES *(Block 29, DD Form 1206)* 0.366	13. SPECIFIC GRAVITY OF SOLIDS *(Block 6n, DD Form 1208)* G_s = 2.62

14. TIME	15. ELAPSED TIME, *(T)* *minutes*	16. ACTUAL HYDROMETER READING *(R^1)*	17. CORRECTED READING *(R)*	18. TEMP *(°C)*	19. TEMPERATURE AND SPECIFIC GRAVITY CONSTANT *(K)*	20. EFFECTIVE DEPTH *(L)*	21. PARTICLE DIAMETER *(D), mm*	22. PERCENT FINER a. PARTIAL	22. PERCENT FINER b. TOTAL
0930	0								
0931	1	45.0	45.5	26	0.01272	8.8	0.0377	93.3	34.1
0932	2	43.0	43.5	26	0.01272	9.1	0.0271	89.2	32.6
0935	5	38.5	39.0	26	0.01272	9.9	0.0178	79.9	29.2
0945	15	23.5	24.0	26	0.01272	12.4	0.0115	49.2	18.0
1000	30	18.5	19.0	25	0.01286	13.2	0.0084	38.9	14.2
1030	60	15.0	15.5	25	0.01286	13.7	0.0060	31.8	11.6
1130	120	13.0	13.5	25	0.01286	14.0	0.0043	27.7	10.1
1330	240	11.0	11.5	25	0.01286	14.3	0.0031	23.6	8.6
0930	1440	8.5	9.0	24	0.01301	14.8	0.0012	18.4	6.7

WEIGHT *(Grams)*	23. DISH + DRY SOIL	324.90
	24. DISH	275.62
	25. DRY SOIL *(W_s)*	49.28

The particle diameter (D) is calculated from Stokes' equation using the corrected hydrometer reading. Use the following formula to solve for particle diameter (D): $D = K\sqrt{L/T}$

Corrected hydrometer reading (R) = actual hydrometer reading (R^1) + composite correction

W_s = Oven-dry weight (in grams) of soil used for hydrometer analysis

Hydrometer graduated in specific gravity (151H)

Partial Percent Finer $= \left[\frac{G_s}{G_s - 1} \times \frac{100{,}000}{W_s}\right](R - 1)$

Total Percent Finer = Partial Percent Finer x Decimal fines (Block 12)

0.366

Hydrometer graduated in grams per liter (152H)

$\frac{(R)(a)}{W_s} \times 100$

(a = specific gravity of solids correction factor)

a= 1.01

SAMPLE

26. REMARKS

RED IN COLOR, VERY FINE SANDS

FROST GROUP: F 4

27. TECHNICAL *(Signature)* PVT Butler	28. COMPUTED BY *(Signature)* PVT Butler	29. CHECKED BY *(Signature)* SFC McKimmey

DD FORM 1794, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

Figure 2-43. Sample DD Form 1794

Table 2-9. Dispersing agents

Dispersing Agent	Stock Solution		Manufacturer
	Concentration	Grams Per Liter	
Sodium tripolyphosphate	0.4N	29	Blockson Chemical Co, Joliet, IL
Sodium polyphosphate	0.4N	36	Blockson Chemical Co, Joliet, IL
Sodium tetraphosphate (Quadrofos)	0.4N	31	Rumford Chemical Works, Rumford, NJ
Sodium hexametaphosphate (Calgon)	0.4N	41	Calgon Co, Pittsburgh, PA

Step 2. Determine the type of hydrometer. If the hydrometer scale ranges from 1.000 to 1.038, it is a Type 151H and measures specific gravity of the suspension. If the scale ranges from 0 to 60, it is a Type 152H and measures grams per liter of the suspension. The dimensions for both hydrometers are the same.

Step 3. Determine the composite correction.

NOTE: Before performing the hydrometer test, a composite correction for hydrometer readings must be determined to correct for items that tend to produce errors in the test.

The first of these items needing correction is the meniscus reading. Hydrometers are graduated by the manufacturer to be read at the bottom of the meniscus formed by the liquid on the stem. Since it is not possible to secure readings of soil suspensions at the bottom of the meniscus, readings must be taken at the top and a correction applied.

The second of these items needing correction is a result of using a dispersing agent in the water to control flocculation. This leads to errors in the analysis. While the dispersing agent assists in keeping the soil grains from adhering to each other, it also increases the specific gravity of the fluid used.

The net amount of the correction for the two corrections required is designated as the composite correction.

a. Place about 500 milliliters of distilled water in a graduated cylinder.

b. Place the amount of dispersing agent that was used in step 1 in the cylinder and mix well.

c. Add additional distilled water to the cylinder to reach the 1,000-milliliter mark.

d. Place the hydrometer in the cylinder and allow it to settle for 20 to 25 seconds. Read the hydrometer at the top of the meniscus formed on the stem. For the Type 151H hydrometer, the composite correction is the difference between this reading and 1. For the Type 152H hydrometer, the composite correction is the difference between the reading and 0.

e. Record the composite correction in block 11 of the form *(Figure 2-43*, *page 2-81)*.

f. Remove the hydrometer from the dispersing-fluid cylinder and place it in a second cylinder filled with distilled water.

NOTE: From this point forward, all hydrometer readings will be taken from the top of the meniscus.

Step 4. Perform the hydrometer test.

a. Record all identifying information for the sample, dispersing agent, quantity used, and composite correction on the form.

b. Obtain the decimal fines from the original soil sample from DD Form 1206. Record it on DD Form 1794 (block 12).

c. Obtain the specific gravity of solids (G_s) of the soil sample from DD Form 1208. Record it on DD Form 1794 (block 13).

d. Empty and thoroughly rinse the graduated cylinder containing the dispersing solution from step 3.

e. Transfer the soaked sample to a dispersion cup, using distilled water to wash any residue from the dish into the cup. Add distilled water to the cup until the water surface is 3 inches below the top of the cup. Place the cup in the dispersing machine and mix silts and sands for 5 minutes, low-plasticity clay for 7 minutes, and high-plasticity clay for 9 minutes.

f. Transfer the mixed solution to the clean 1,000-milliliter graduated cylinder, using distilled water to wash any residue from the cup into the cylinder. Add distilled water until the 1,000-milliliter volume mark is reached.

g. Place the rubber cap over the open end of the cylinder. Turn the cylinder upside down and back for a period of 1 minute to complete the agitation of the slurry.

NOTE: The number of turns during this minute should be about 60, counting the turn upside down and back as two turns. If any soil remains at the bottom of the cylinder during the first few turns, it should be loosened by vigorous shaking of the cylinder while it is in the inverted position.

h. After shaking the cylinder for 1 minute, place it on a level and sturdy surface where it will not be disturbed. Remove the cap and start the timer. Remove any foam that has formed during agitation by lightly touching it with a bar of soap.

i. Immerse the hydrometer slowly into the liquid 20 to 25 seconds before each reading. Take the actual hydrometer reading (R1) at 1 and 2 minutes of elapsed time. As soon as the 1- and 2-minute readings are taken, carefully remove the hydrometer and place it in the second cylinder of pure distilled water using a spinning motion. Record the reading on the form (block 16).

j. Place a thermometer in the solution. Record the temperature reading, in centigrade, to the nearest whole degree. Record on the form (block 18).

NOTE: It is extremely important to obtain accurate temperature readings. The soil hydrometer is calibrated at 20°C. Variations in temperature from this standard temperature produces inaccuracies in the actual hydrometer readings. These inaccuracies will be compensated for later during the computations.

k. Repeat steps 4i and 4j for the remainder of the required readings. Take readings at the following intervals: 5, 15, and 30 minutes and 1, 2, 4, and 24 hours. After each reading, remove the hydrometer, place it in the hydrometer of distilled water, and obtain the temperature reading. Record the information on the form for each reading.

Step 5. Determine the dry weight of the sample by carefully washing all of the sample into a preweighed pudding pan or dish (block 24). Oven-dry the sample, allow it to cool, and determine and record the weight of the sample and the pan or dish (block 23).

Step 6. Determine the weight of the dry soil by subtracting the weight of the pan from the weight of the pan and dry soil. Record this information on the form as the weight of the oven-dried soil (W_s) used for hydrometer testing (block 25).

Step 7. Compute the results on DD Form 1794 (see *Figure 2-43, page 2-81*).

a. Column 17. Obtain the corrected reading (R) by adding the actual hydrometer reading (column 16, R1) and the composite correction (block 11) and record the sum on the form.

$$R = R^1 + \text{composite correction}$$

b. Column 19. Obtain the temperature versus specific gravity constant (K) from *Table 2-10*. Record it on the form.

NOTE: Although typical specific-gravity values are listed in Table 2-10, there may be cases when a soil type falls above or below this range of values. In these situations the value of K must be computed using the following formula:

$$K = \sqrt{\frac{30\eta}{G_s - 1}}$$

where—

η = ***coefficient of viscosity of the liquid (water) in poises (varies with changes in temperature)***

G_s = ***Specific gravity of solids for the material being tested***

c. Column 20. Obtain the effective depth (L) for each corrected reading (column 17) by using *Table 2-11, page 2-86,* and record on the form.

Table 2-10. Values of K for use in Stokes' equation for computing particle diameter

Temp °C	Specific Gravity of Solids (G_S)								Coeff of Viscosity (η)
	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85	
16	0.01505	0.01481	0.01458	0.01435	0.01414	0.01394	0.01374	0.01355	0.00001133
17	0.01486	0.01462	0.01439	0.01417	0.01396	0.01376	0.01356	0.01338	0.00001104
18	0.01467	0.01443	0.01420	0.01399	0.01378	0.01358	0.01339	0.01321	0.00001076
19	0.01449	0.01426	0.01403	0.01382	0.01361	0.01342	0.01323	0.01305	0.00001050
20	0.01432	0.01408	0.01386	0.01365	0.01345	0.01326	0.01307	0.01289	0.00001025
21	0.01414	0.01391	0.01369	0.01348	0.01328	0.01309	0.01291	0.01273	0.00001000
22	0.01397	0.01374	0.01353	0.01332	0.01312	0.01294	0.01275	0.01258	0.00000976
23	0.01381	0.01358	0.01337	0.01316	0.01297	0.01278	0.01260	0.01243	0.00000953
24	0.01365	0.01342	0.01321	0.01301	0.01282	0.01263	0.01246	0.01229	0.00000931
25	0.01349	0.01327	0.01306	0.01286	0.01267	0.01249	0.01232	0.01215	0.00000910
26	0.01334	0.01312	0.01292	0.01272	0.01253	0.01235	0.01218	0.01201	0.00000890
27	0.01319	0.01298	0.01277	0.01258	0.01239	0.01221	0.01204	0.01188	0.00000870
28	0.01305	0.01283	0.01263	0.01244	0.01225	0.01208	0.01191	0.01175	0.00000851
29	0.01290	0.01269	0.01249	0.01230	0.01212	0.01194	0.01178	0.01162	0.00000832
30	0.01276	0.01255	0.01235	0.01217	0.01199	0.01181	0.01165	0.01149	0.00000814

d. Column 21. Determine the particle diameter (D) corresponding to a given hydrometer reading on the basis of Stokes' equation:

$$D = K\sqrt{\frac{L}{T}}$$

where—

D = diameter of the sphere, in millimeters

K = constant depending on temperature of suspension and specific gravity of soil particles; values of K can be obtained from Table 2-10 (entered in column 19)

L = distance from the surface of the suspension to the level at which the density of the suspension is being measured, in centimeters (effective depth) (entered in column 20)

T = interval of time from beginning of sedimentation to the taking of the reading, in minutes (entered in column 15)

e. Column 22a. Compute the partial percent finer. To compute the percent of particle diameters finer than that corresponding to a given hydrometer reading, use the following formulas based on the hydrometer type and record the results on the form:

(1) Hydrometer type 151H:

$$Partial\ percent\ finer\ =\ \frac{G_S}{G_S - 1} \times \frac{100,000}{W_S} \times (R - 1.0)$$

Table 2-11. Values of effective depth for hydrometer analysis

Hydrometer 151H				Hydrometer 152H					
Corr Hydro Reading	**Effective Depth, L, cm**	**Corr Hydro Reading**	**Effective Depth, L, cm**	**Corr Hydro Reading**	**Effective Depth, L, cm**	**Corr Hydro Reading**	**Effective Depth, L, cm**	**Corr Hydro Reading**	**Effective Depth, L, cm**
1.000	16.3	1.021	10.7	0	16.3	21	12.9	42	9.4
1.001	16.0	1.022	10.5	1	16.1	22	12.7	43	9.2
1.002	15.8	1.023	10.2	2	16.0	23	12.5	44	9.1
1.003	15.5	1.024	10.0	3	15.8	24	12.4	45	8.9
1.004	15.2	1.025	9.7	4	15.6	25	12.2	46	8.8
1.005	15.0	1.026	9.4	5	15.5	26	12.0	47	8.6
1.006	14.7	1.027	9.2	6	15.3	27	11.9	48	8.4
1.007	14.4	1.028	8.9	7	15.2	28	11.7	49	8.3
1.008	14.2	1.029	8.6	8	15.0	29	11.5	50	8.1
1.009	13.9	1.030	8.4	9	14.8	30	11.4	51	7.9
1.010	13.7	1.031	8.1	10	14.7	31	11.2	52	7.8
1.011	13.4	1.032	7.8	11	14.5	32	11.1	53	7.6
1.012	13.1	1.033	7.6	12	14.3	33	10.9	54	7.4
1.013	12.9	1.034	7.3	13	14.2	34	10.7	55	7.3
1.014	12.6	1.035	7.0	14	14.0	35	10.6	56	7.1
1.015	12.3	1.036	6.8	15	13.8	36	10.4	57	7.0
1.016	12.1	1.037	6.5	16	13.7	37	10.2	58	6.8
1.017	11.8	1.038	6.2	17	13.5	38	10.1	59	6.6
1.018	11.5			18	13.3	39	9.9	60	6.5
1.019	11.3			19	13.2	40	9.7		
1.020	11.0			20	13.0	41	9.6		

(2) Hydrometer type 152H:

$$\text{Partial percent finer} = \frac{R \times a}{W_S} \times 100$$

where—

G_s = *specific gravity of solids*

W_s = *oven-dried weight of soil (in grams) used for hydrometer analysis*

R = *Corrected hydrometer reading (with composite correction applied)*

a = *Correction factor from Table 2-12, to be applied to the reading for Type 152H*

Table 2-12. Specific-gravity correction factors applied to hydrometer 152H for computing partial percent finer

Specific Gravity	2.45	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85	2.90	2.95
Correction Factor	1.05	1.03	1.02	1.01	1.00	0.99	0.98	0.97	0.96	0.96	0.94

f. Compute the total percent finer for each hydrometer reading and record it on the form using the formula—

Total percent finer = partial percent finer x decimal fines (block 12)

PRESENTATION OF RESULTS

Plot the grain-size distribution on DD Form 1207 using the particle diameters (D, grain-size, in millimeters) and the total percent finer (percent passing) and connect the plotted points with a smooth curve (see *Figure 2-44, page 2-88*).

Read the curve on the form and determine if 3 percent or more of the particles are smaller than 0.02 millimeter in diameter; if so, the soil is frost susceptible.

Frost-susceptible soils are listed in four groups in the order of increasing susceptibility (see *Table 2-13, page 2-89*).

Soils in group F-4 have high frost susceptibility. Record the frost-susceptibility group for the soil type in block 27 of DD Form 1794 (see *Figure 2-43, page 2-81*).

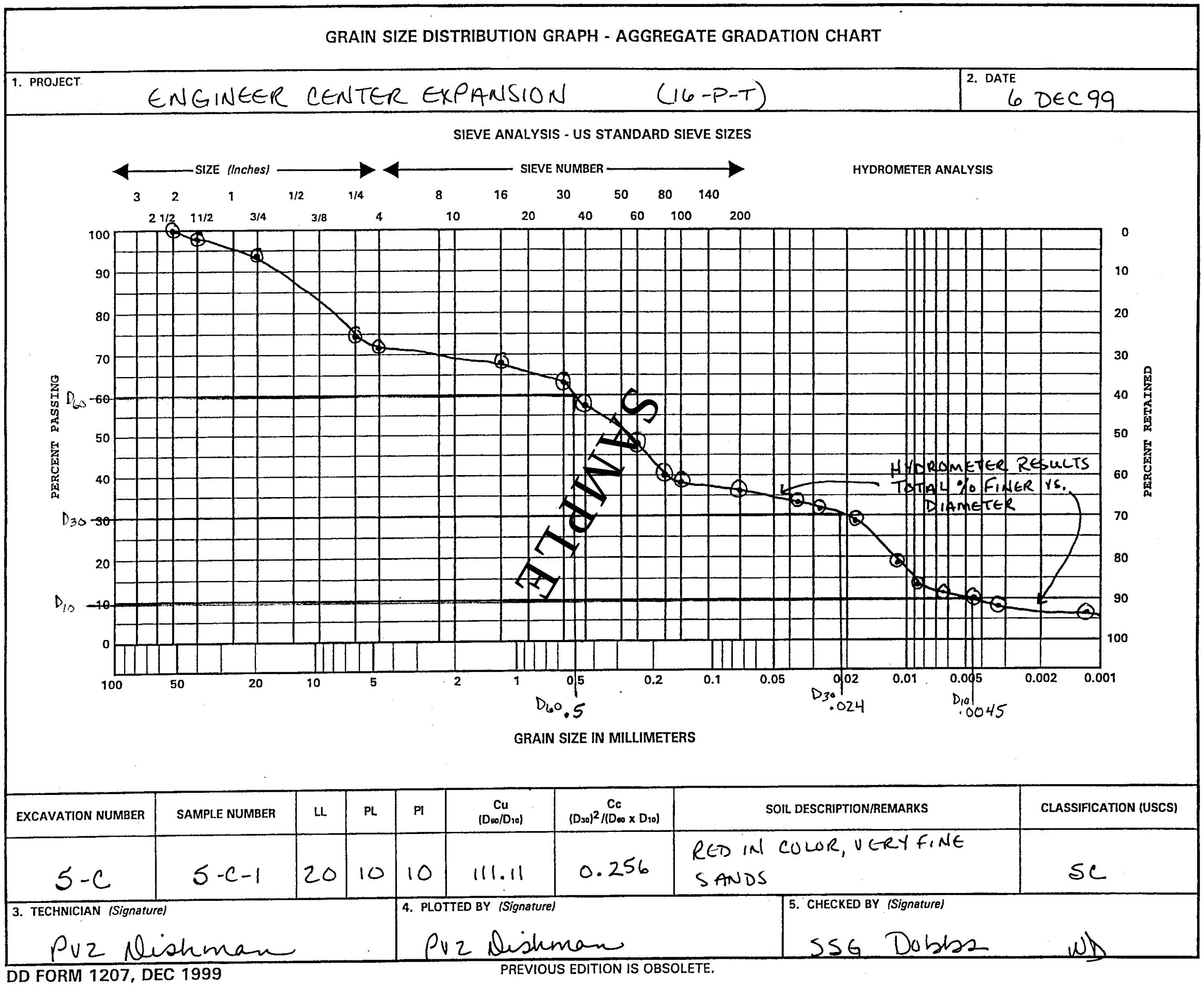
This curve can be used to determine the coefficient of uniformity (C_u) and the coefficient of curvature (C_c).

The data in the example shown on DD Form 1794 (*Figure 2-43, page 2-81*) is plotted on DD Form 1207 to give an example of such a curve for a mixed soil (see *Figure 2-44*). For this soil, the diameter corresponding to 60 percent passing (D_{60}) is 0.5 millimeter. The diameter corresponding to 10 percent passing (D_{10}) is 0.0045 millimeter. Hence, the coefficient of uniformity is as follows:

$$C_U = \frac{D_{60}}{D_{10}} = \frac{0.5}{0.0045} = 111.11$$

The diameter for 30 percent passing (D30) is 0.024 millimeters. Thus, the coefficient of curvature is as follows:

$$C_C = \frac{(D_{30})^2}{D_{60} \times D_{10}} = \frac{(0.024)^2}{0.5 \times 0.0045} = \frac{0.000576}{0.00225} = 0.256$$



GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADATION CHART

1. PROJECT ENGINEER CENTER EXPANSION (16-P-T)	2. DATE 6 DEC 99

EXCAVATION NUMBER	SAMPLE NUMBER	LL	PL	PI	Cu (D_{60}/D_{10})	Cc $(D_{30})^2/(D_{60} \times D_{10})$	SOIL DESCRIPTION/REMARKS	CLASSIFICATION (USCS)
5-C	5-C-1	20	10	10	111.11	0.256	RED IN COLOR, VERY FINE SANDS	SC

3. TECHNICIAN *(Signature)* PVZ Dishman	4. PLOTTED BY *(Signature)* PVZ Dishman	5. CHECKED BY *(Signature)* SSG Dobbs WD

DD FORM 1207, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

Figure 2-44. Sample DD Form 1207

Table 2-13. Frost-susceptibility groups for typical soil types

Frost Group	Kind of Soil	Percentage Finer Than 0.02 mm by Weight	Typical Soil Types Under USCS
NFS	(a) Gravels (e ≥ 0.25) crushed stone or rock	0 to 3	GW, GP
	(b) Sands (e ≤ 0.30)	0 to 3	SW, SP
	(c) Sands (e > 0.30)	3 to 10	SP
S-1	(a) Gravels (e < 0.25) crushed stone or rock	0 to 3	GW, GP
	(b) Gravelly soils	3 to 6	GW, GP, GW-GM, GP-GM, GW-GC, GP-GC
S-2	Sandy soils (e ≤ 0.30)	3 to 6	SW, SP, SW-SM, SP-SM, SW-SC, SP-SC
F-1	Gravelly soils	6 to 10	GW-GM, GP-GM, GW-GC, GP-GC
F-2	(a) Gravelly soils	10 to 20	GM, GC, GM-GC
	(b) Sands	6 to 15	SM, SC, SW-SM, SP-SM, SW-SC, SP-SC, SM-SC
F-3	(a) Gravelly soils	Over 20	GM, GC, GM-GC
	(b) Sands, except very fine silty sands	Over 15	SM, SC, SM-SC
	(c) Clays, PI > 12		CL, CH, ML-CL
F-4	(a) All silts		ML, MH, ML-CL
	(b) Very fine sands	Over 15	SM, SC, SM-SC
	(c) Clays, PI < 12		CL, ML-CL
	(d) Varved clays and other fine-grained, banded sediments		CL or CH layered with ML, MH, SM, SC, SM-SC, or ML-CL

NOTE: e = void ratio

SECTION VI. LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX DETERMINATION (ASTM D 4318-95A)

Clays and some other fine-grained soils exhibit plasticity if the proper amount of water is present in the soil. A plastic soil is one that can be deformed beyond the point of recovery without cracking or change in volume. Such soils can be remolded. The LL is the greatest water content that the material may contain and still remain plastic. More water causes it to become a thick liquid. The PL is the lowest water content that the material may contain for plastic behavior. With less water, the soil becomes brittle and breaks into fragments if remolding is attempted. The PI is the numerical difference between the LL and the PL:

$$PI = LL - PL$$

A large PI indicates a very plastic soil; a small PI denotes a soil having little plasticity. As water content decreases below the PL, the soil mass shrinks and becomes stiffer. The shrinkage limit is the water content where, with further drying, shrinkage stops. Since there is no sharp distinction between the liquid, plastic, and brittle solid states of consistency, standardized procedures have been established for determining the LL and the PL. These consistency limits, as well as the shrinkage limit, are called the Atterberg limits. Since the primary tests in this section determine only the LLs and PLs and do not include tests for the shrinkage limits, they are not identified as the Atterberg limits.

Research with large numbers of clay soils was used to establish the soil plasticity chart for laboratory classification of fine-grained soils. The LL and PI values are coordinates that locate a particular soil sample on the chart. The region on the chart in which the sample falls gives the classification based on the behavioral characteristics of the particular soil.

Take particular care when performing the test methods described below. Some soils, particularly those with a high organic content, can provide inconsistent readings or drastic differences between an oven-dried sample and a sample at natural moisture content. Conduct the test below on samples of natural moisture content. Determine the moisture content at the end of the test.

LL DETERMINATION

A soil's LL is the water content expressed as a percentage of the weight of the oven-dried soil at the boundary between the liquid and the plastic states and reported as a whole number. This boundary is arbitrarily defined by a standard test method. The test is performed on two halves of a prepared soil specimen in an LL device. The LL is determined when the soil halves flow together along a distance of 13 millimeters when the cup is dropped or jarred exactly 25 times from a height of 1 centimeter. This rate of drop is 1.9 to 2.1 drops per second.

PURPOSE

Perform this test to assist in classifying the soil by determining the LL from three moisture-content samples.

EQUIPMENT

Perform this test in a laboratory environment using the following items (see *Figure 2-45*):

- A balance scale sensitive to 0.01 gram.
- An LL device and a grooving tool (see *Figure 2-46, page 2-92*).
- A No. 40 sieve.
- Pudding pans.
- A ground-glass plate (at least 30 centimeters square by 1 centimeter thick) for mixing soil and rolling PL threads.
- A plastic bag.
- A calculator.
- Moisture-determination tares.

- Paper.
- A pencil.
- A grease pencil.
- DD Form 1209.
- Gummed labels.
- A spatula.
- A straightedge.
- A mortar with a rubber-tipped pestle.
- A laboratory oven.
- Heat-resistant gloves.
- Distilled water.

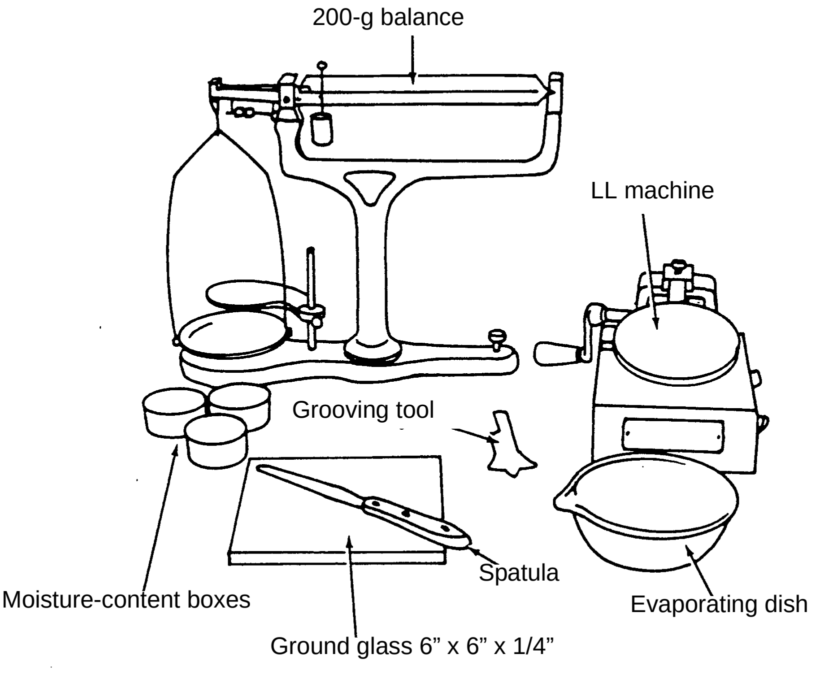


Figure 2-45. Equipment for the LL and PL tests

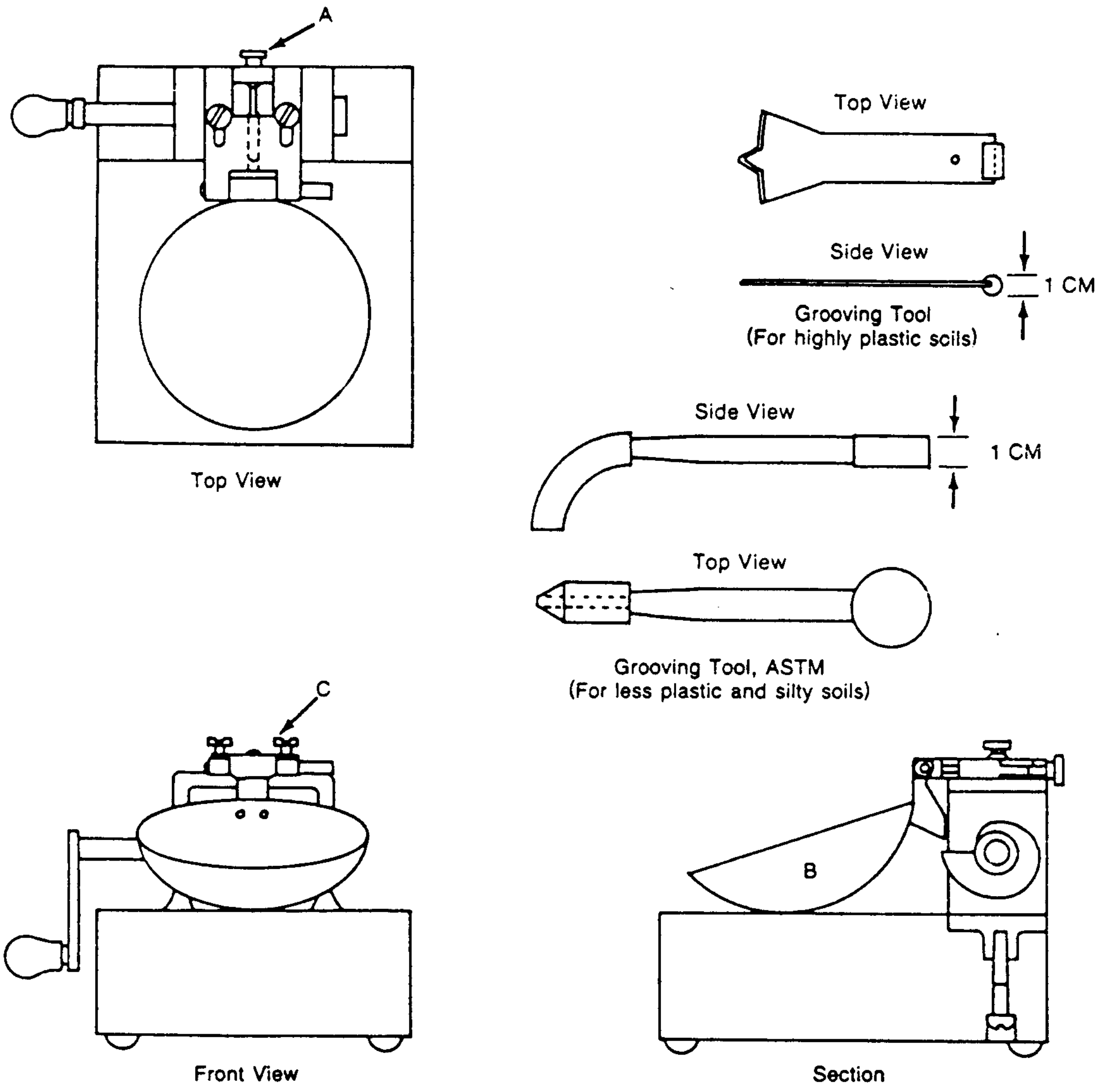


Figure 2-46. LL device with grooving tools

STEPS

Perform the following steps to determine the LL:

Step 1. Prepare the soil sample.

a. Sieve the soil sample (at natural moisture content) over the No. 40 sieve to obtain a sufficient quantity of at least 250 grams.

b. Perform the following steps if little or no material is retained on the No. 40 sieve, otherwise go to step 1c:

(1) Collect 200 to 250 grams of –40 material for testing.

(2) Mix the material with distilled water until the water content is slightly below the LL or about a peanut butter consistency. The goal is to have the material fall in the 25- to 35-blow range for the first test.

(3) Place the mixture in a plastic bag, making it airtight for at least 16 hours (overnight) so the moisture content can become consistent throughout the sample. Remix the material thoroughly before testing.

c. Perform the following steps if material is retained on the No. 40 sieve:

(1) Place the –40 material in a plastic bag, making it airtight to maintain its natural moisture content.

(2) Soak the coarse material retained on the No. 40 sieve (the soaking time is variable).

(3) Rub the colloidal material from the surfaces of the large particles until they are clean, placing the fines in suspension.

(4) Pour off the suspended fines slowly into another pan, being careful not to pour off the coarse material.

(5) Add clean water to the coarse material and repeat the wash process until the water poured off is sufficiently clear to indicate that the majority of fines that were put in suspension have been poured off.

(6) Remove the excess water from the pan containing the suspended fines after the fines have settled by decantation and evaporation. Do not oven-dry or add chemical substances to speed dry or hasten the settlement.

(7) Oven-dry the coarse material that has been soaked and washed.

(8) Sieve the oven-dried coarse material over the No. 40 sieve.

(9) Combine the –40 material obtained from steps 1c(1) and 1c(8) with the decanted material from step 1c(6). If the combined material is too moist, air-dry it until the water content is slightly below the LL. If the combined material is too dry, add small quantities of water until the water content is slightly below the LL (peanut butter consistency).

(10) Place the combined mixture in a plastic bag, making it airtight for at least 16 hours (overnight) so the moisture content can become consistent throughout the sample. Remix the material thoroughly before testing.

Step 2. Inspect the LL device before testing.

a. Ensure that the pin connecting the cup is not worn (which would permit side play).

b. Ensure that the screws connecting the cup to the hanger arm are tight.

c. Check the cup for wear. If a groove has developed from use, replace it.

d. Check the contact between the cup and the base. If a dent can be felt in the base or flat on the cup, replace or repair it.

e. Check the grooving tool for wear.

f. Check the height of the drop of the cup so that the point on the cup that comes in contact with the base (not the lowest point of the cup) rises to a height of 1 centimeter. Use the gauge on the handle of the grooving tool to assist in this measurement. The height of the drop must be 1 centimeter. Use the thumbscrew at the rear of the device to make an adjustment.

Example: The following is one procedure which could be used to aid in checking and adjustments:

1. Place a piece of masking tape across the outside bottom of the cup parallel with the axis of the cup-hanger pivot. The edge of the tape away from the cup hanger should bisect the spot on the cup that contacts the base. For new cups, place a piece of carbon paper on the base and allow the cup to drop several times to mark the contact spot.

2. Attach the cup to the device, and turn the crank until the cup is raised to its maximum height.

3. Slide the height gauge under the cup from the front, and observe whether the gauge contacts the cup or the tape. If the tape and cup are both contacted, the height of drop is approximately correct. If not, adjust the cup until simultaneous contact is made.

4. Check the adjustment by turning the crank at 2 revolutions per second while holding the gauge in position against the tape and cup. If a faint ringing or clicking sound is heard without the cup rising from the gauge, the adjustment is correct. If no ringing is heard or if the cup rises from the gauge, readjust the height of drop. If the cup rocks on the gauge during this step, the cam-follower pivot is excessively worn and the worn parts should be replaced.

5. Remove the tape after completion of adjustments.

Step 3. Perform the LL test.

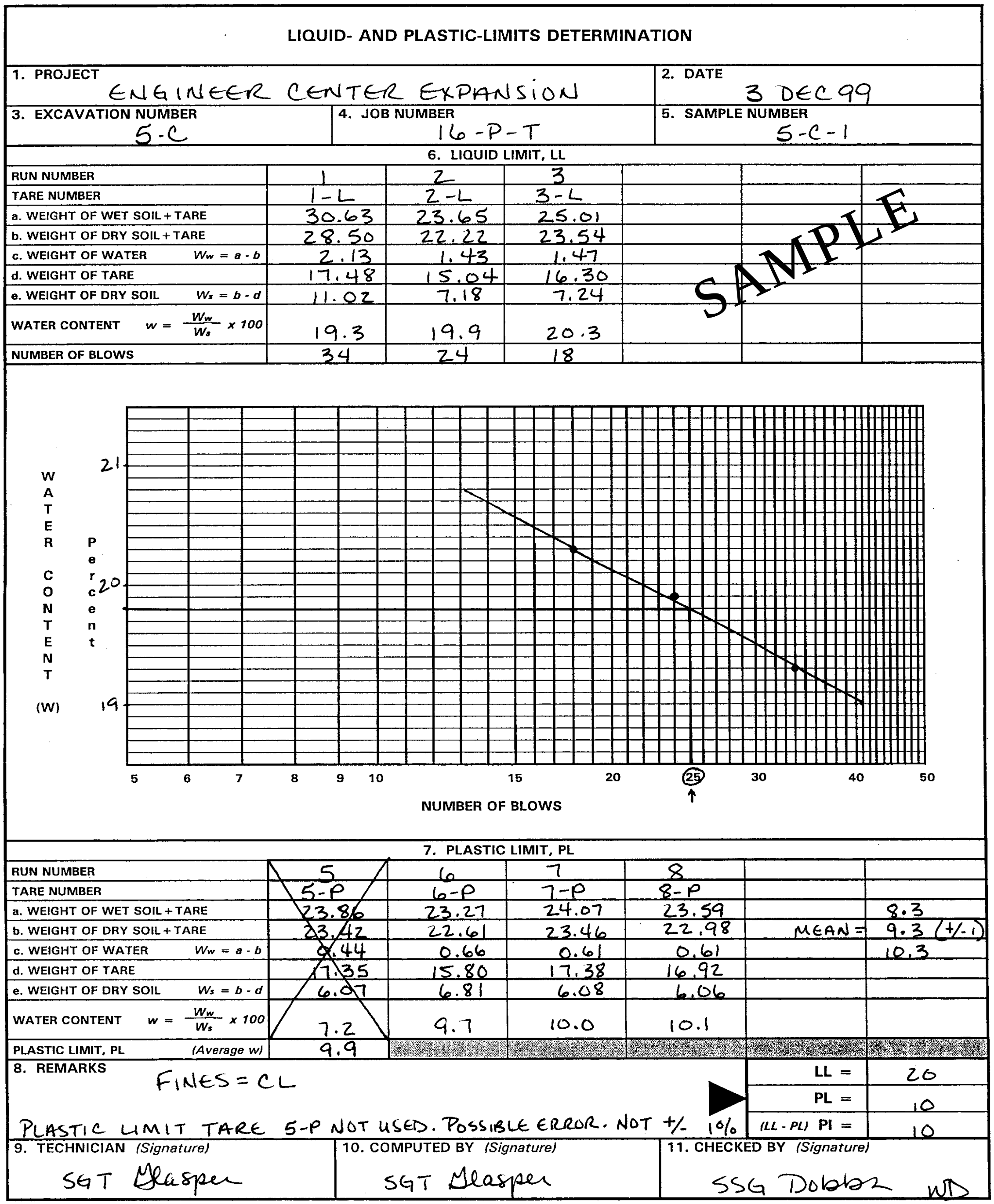
a. Obtain about 50 grams of the 200- to 250-gram prepared sample, and place in an airtight container for use in the PL test.

b. Record all identifying information for the sample on DD Form 1209 (see *Figure 2-47*).

c. Label and preweigh three empty moisture-determination tares. Record the weight on the form as the weight of the tare.

d. Place 20 to 25 grams of the thoroughly mixed sample into the brass cup, and level it off with a maximum depth of 1 centimeter (see *Figure 2-48, page 2-96*).

e. Divide the soil sample in the cup with a grooving tool so that a clean, sharp groove is formed. Hold the cup with the cam follower upward and draw the grooving tool, with the beveled edge forward, through the specimen downward away from the cam follower (see *Figure 2-49, page 2-96)*. Use more than one stroke to make the groove, but no more than six, cleaning the grooving tool's cutting edge after each stroke. Avoid tearing the side of the groove. Replace the soil sample in the cup, and regroove if



LIQUID- AND PLASTIC-LIMITS DETERMINATION

1. PROJECT ENGINEER CENTER EXPANSION

2. DATE 3 DEC 99

3. EXCAVATION NUMBER 5-C

4. JOB NUMBER 16-P-T

5. SAMPLE NUMBER 5-C-1

6. LIQUID LIMIT, LL

RUN NUMBER	1	2	3			
TARE NUMBER	1-L	2-L	3-L			
a. WEIGHT OF WET SOIL + TARE	30.63	23.65	25.01			
b. WEIGHT OF DRY SOIL + TARE	28.50	22.22	23.54			
c. WEIGHT OF WATER $W_w = a - b$	2.13	1.43	1.47			
d. WEIGHT OF TARE	17.48	15.04	16.30			
e. WEIGHT OF DRY SOIL $W_s = b - d$	11.02	7.18	7.24			
WATER CONTENT $w = \frac{W_w}{W_s} \times 100$	19.3	19.9	20.3			
NUMBER OF BLOWS	34	24	18			

7. PLASTIC LIMIT, PL

RUN NUMBER	~~5~~	6	7	8		
TARE NUMBER	~~5-P~~	6-P	7-P	8-P		
a. WEIGHT OF WET SOIL + TARE	~~23.86~~	23.27	24.07	23.59		8.3
b. WEIGHT OF DRY SOIL + TARE	~~23.42~~	22.61	23.46	22.98	MEAN =	9.3 (+/-1)
c. WEIGHT OF WATER $W_w = a - b$	~~0.44~~	0.66	0.61	0.61		10.3
d. WEIGHT OF TARE	~~17.35~~	15.80	17.38	16.92		
e. WEIGHT OF DRY SOIL $W_s = b - d$	~~6.07~~	6.81	6.08	6.06		
WATER CONTENT $w = \frac{W_w}{W_s} \times 100$	~~7.2~~	9.7	10.0	10.1		
PLASTIC LIMIT, PL (Average w)	9.9					

8. REMARKS FINES = CL

PLASTIC LIMIT TARE 5-P NOT USED. POSSIBLE ERROR. NOT +/- 1%

LL =	20
PL =	10
(LL - PL) PI =	10

9. TECHNICIAN (Signature) SGT Glasper

10. COMPUTED BY (Signature) SGT Glasper

11. CHECKED BY (Signature) SSG Dobbz WD

DD FORM 1209, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

Figure 2-47. Sample DD Form 1209

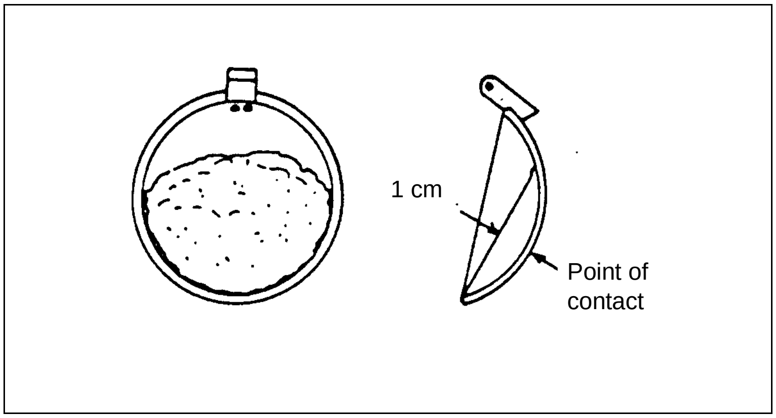


Figure 2-48. Leveling sample in the cup

the side tears. With some sandy and highly organic soils, it is impossible to draw the grooving tool through the specimen without tearing the sides of the groove. In such cases, the groove should be made with a spatula, using the grooving tool only for a final check of the groove (see *Figure 2-50).*

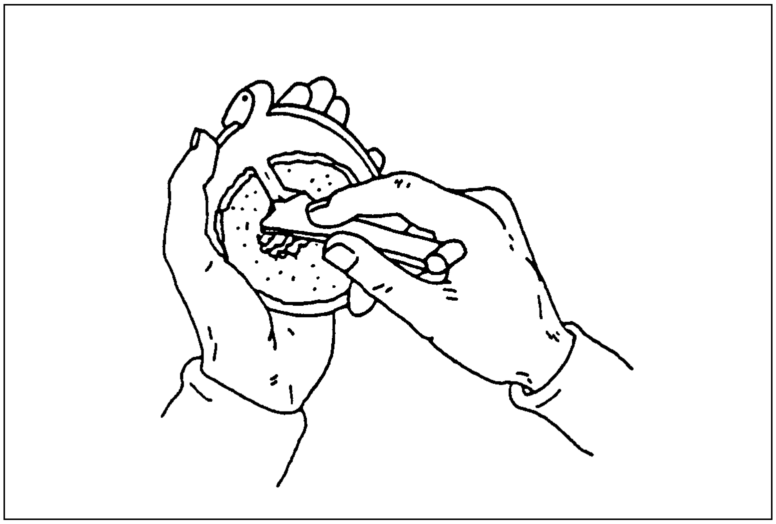


Figure 2-49. Holding cup and grooving tool

f. Attach the cup to the device; ensure that the height of the drop is 1 centimeter.

g. Turn the crank of the device at a rate of two revolutions per second. Count the blows until the two halves of the soil make contact at the bottom of the groove along a distance of 13 millimeters (see *Figure 2-51*).

h. Record the number of blows to close the groove for 13 millimeters.

i. Obtain 5 to 10 grams of soil from the cup to determine the moisture content. Take the sample perpendicular to the groove from the edge of the cup and through the portion that has closed in the bottom of the groove. Place the sample in the preweighed moisture-determination tare, and

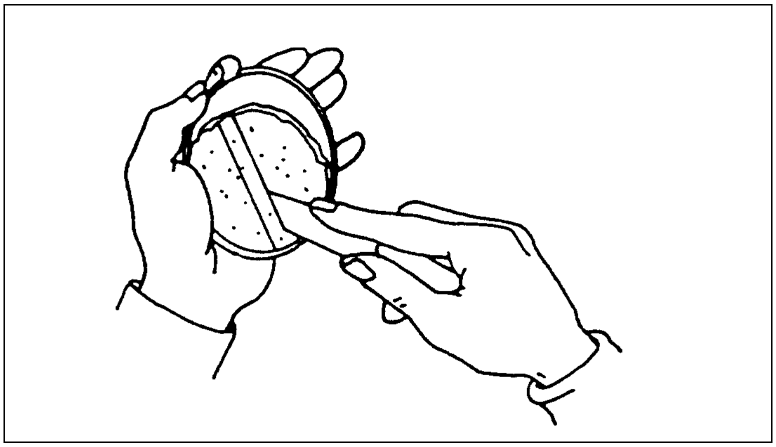


Figure 2-50. Cutting groove with spatula in sandy soil

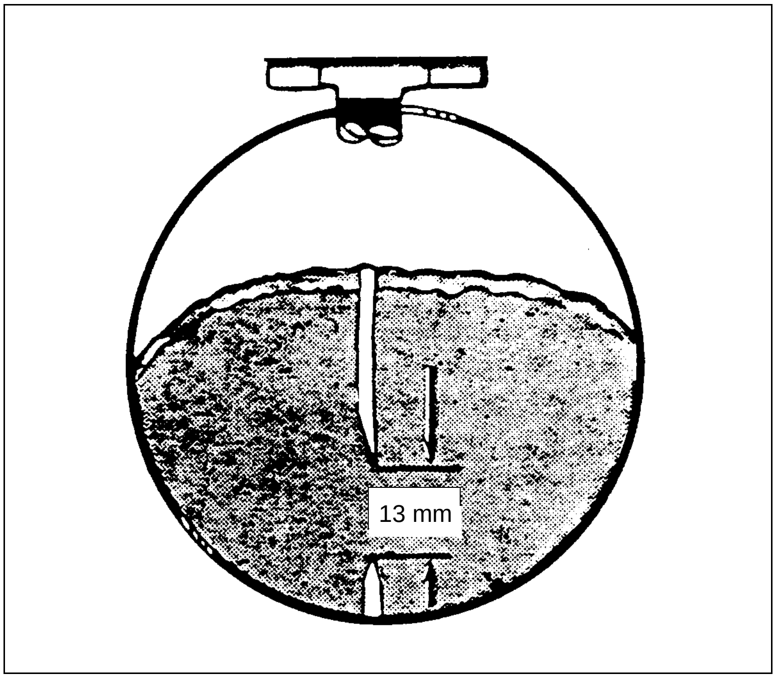


Figure 2-51. Soil coming into contact

cover it with a lid. Weigh it and record the weight on the form as the weight of the wet soil and the tare.

j. Transfer the soil remaining in the cup to the mixing dish. Wash and dry the cup and the grooving tool.

NOTE: It is recommended that one of the trials be for a closure requiring 25 to 35 blows, one for a closure between 20 and 30 blows, and one for a closure requiring 15 to 25 blows.

k. Remix the entire soil specimen, adding a little water to increase the water content of the soil and decrease the number of blows required to close the groove. Repeat steps 3d through 3j for at least two additional trials producing a successively lower number of blows to close the groove.

l. Oven-dry the water-content samples, allow them to cool, and reweigh them. Record the weight on the form as the weight of the dry soil and the tare.

m. Compute the weight of the water (W_w) by subtracting the weight of the dry soil and the tare from the weight of the wet soil and the tare. Record the weight on the form.

n. Compute the weight of the dry soil (W_s) by subtracting the weight of the tare from the weight of the dry soil and the tare. Record the weight on the form.

o. Record the water content for each specimen by computing the formula—

$$w = \frac{W_w}{W_s} \times 100$$

NOTE: All weighing should be accurate to 0.01 gram and water contents computed in percent to one decimal place.

p. Plot the water-content points on the semilog graph on the form (water versus number of blows) and draw a straight line (flow line) representative of the three or more points.

q. Determine the LL by interpreting the graph where the flow line intersects the 25-blow line. Record the LL to the nearest whole number.

PL DETERMINATION

The PL of a soil is the water content, expressed as a percentage of weight of oven-dried soil, at which the soil begins to crumble when rolled into a thread 3.2 millimeters in diameter. About 50 grams of material is required for the PL test. Prepare the sample and set it aside while preparing for the LL test.

PURPOSE

Perform this test to assist in classifying the soil by determining the PL moisture content to within ± 1 percent.

EQUIPMENT

Perform this test in a laboratory environment using the same equipment listed in the LL determination test.

STEPS

Perform the following steps to determine the PL:

Step 1. Label and preweigh two empty moisture-determination tares. Record the weight on DD Form 1209 as the weight of the tare.

Step 2. Obtain the 50-gram sample set aside during step 3a of the LL test. Reduce the water content, if required, to obtain a consistency with which the soil can be rolled without sticking to the hands by spreading or mixing continuously on the glass plate. The drying process may be accelerated by air-drying only.

Step 3. Select a portion of about 2 grams (marble size) from the 50-gram mass and form the test specimen into an ellipsoidal mass. Roll it on a finely-ground glass plate with the fingers or palm of the hand to a uniform thread diameter of 3.2 millimeters, taking no more than 2 minutes (see *Figure 2-52*).

NOTE: The rate of rolling should be between 80 to 90 strokes per minute, counting a stroke as one complete motion of the hand forward and back to the starting position. This rate of rolling may have to be decreased for very fragile soil.

Step 4. Remold the sample and roll it again to 3.2 millimeters diameter, repeating the rolling and remolding process until the total sample crumbles, before reaching the 3.2-millimeters-diameter thread (see *Figure 2-53, page 2-120*).

NOTE: All of the sample may not crumble at the same time. If the thread breaks into smaller lengths, roll each of these lengths to 3.2 millimeters. Continue the rolling and remolding process until the sample can no longer be remolded and rolled to the 3.2-millimeter thread without totally breaking up.

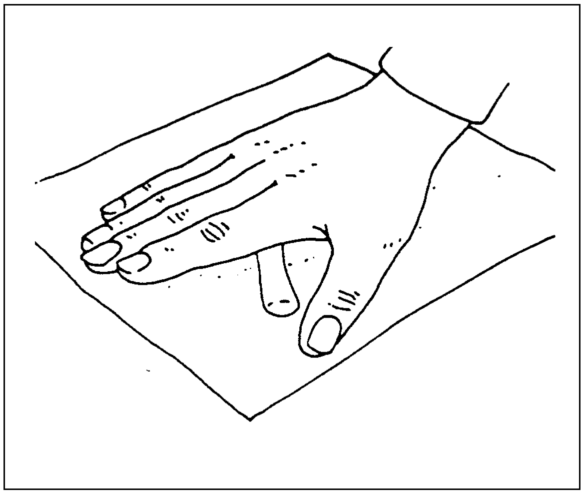


Figure 2-52. Rolling a soil specimen, PL test

Step 5. Collect and place the crumbled portions into a preweighed moisture-determination tare and cover it with the lid.

Step 6. Repeat steps 3 through 5 until the crumbled threads in the moisture-determination tare weigh at least 6 grams.

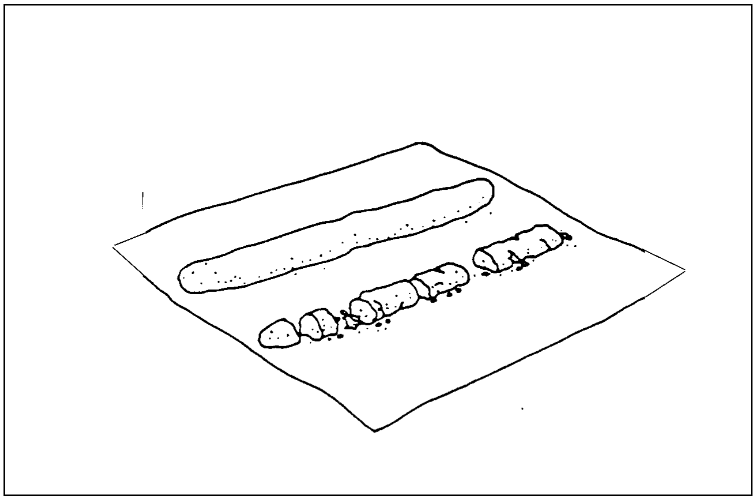


Figure 2-53. Rolled threads, crumbled and uncrumbled

Step 7. Repeat steps 3 through 6 to obtain a second moisture-determination tare of at least 6 grams of material.

Step 8. Weigh the moisture-determination tares with the crumbled threads, and record the weights on the form as the weight of the wet soil and the tare.

Step 9. Determine the water content by following steps 3l through 3o (page 2-118) of the LL test.

Step 10. Determine the average water content of the samples and record to the nearest tenth as the PL. When determining the average water content, the individual tests must be within ± 1 percent of the mean. Any individual tests that do not meet this requirement will not be used. If none of the individual tests meet this requirement, then additional testing is required.

PI DETERMINATION

Compute the PI and record it on the form using the following formula:

$$PI = LL - PL$$

Classify the soil by plotting the LL versus the PI on the plasticity chart as follows (see *Figure 2-54*):

- The material plotted on or above the A line is classified as clay, and the material plotted below the A line is classified as silt.
- The material plotted on or to the right of the 50 percent line has a high LL (H), and the material plotted to the left of the 50 percent line has a low LL (L).
- The upper, or U, line is an approximate upper boundary. Although not impossible, any results plotted above this line should be considered suspect and the tests should be rechecked.

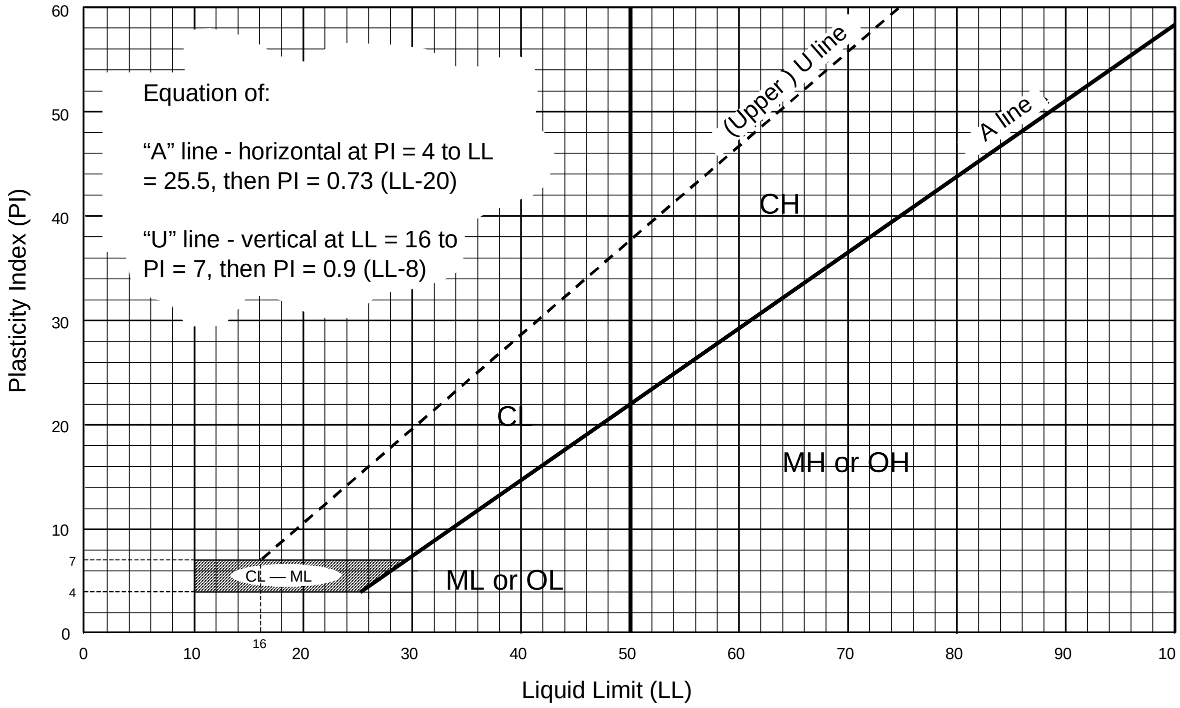


Figure 2-54. USCS plasticity chart

SECTION VII. LABORATORY COMPACTION CHARACTERISTICS OF SOIL USING MODIFIED EFFORT (COMPACTION TEST) (ASTM D 1557-91)

Compaction is one of the basic construction procedures involved in building subgrades and bases for roads and airport pavements, embankments, earthen dams, and similar structures. Compaction is the process of increasing the amounts of solids per unit volume of soil by mechanical means. This increase in density has an important effect in improving such soil properties as strength, permeability, and compressibility.

The amount of compaction is quantified in terms of the soil's density (dry unit weight). Usually, soil can be compacted best (and thus a greater density achieved) if only a certain amount of water is added. In effect, water acts as a lubricant, allowing soil particles to be packed together better. However, if too much water is added, a lesser density will result because the excess water separates the soil particles. Therefore, for a given compactive effort, there is a particular moisture content at which dry density is greatest and compaction is best. This moisture content is the OMC, and the associated dry density is called the maximum dry density (MDD).

COMPACTION TEST

In the field, compaction is accomplished by rolling or tamping with special equipment or by the passing of construction equipment. Laboratory compaction usually is accomplished by placing the soil in a cylinder of known volume and dropping a tamper of known weight onto the soil from a known height for a given number of blows. The amount of work done to the soil per unit volume of soil in this dynamic compaction procedure is called compactive effort. Each compactive effort for a given soil has its own OMC. As the compactive effort is increased, the maximum density usually increases and the OMC decreases.

Before performing the compaction test, the grain-size analysis must be determined (see Section V). This test method provides three alternative testing procedures. The procedure used shall be as indicated in the project specifications for the type of material being tested. If no procedure is clearly specified, the selection should be based on follow-on testing requirements (such as the CBR) and the material gradation.

PURPOSE

Perform compaction tests in the laboratory to determine such soil properties as the effect of varying percentages of water on dry density, the maximum density obtainable under a given compactive effort, and the OMC.

- Procedure A. This procedure uses the 4-inch mold on only the soil passing the No. 4 sieve when the overall representative sample has no more than 20 percent of the material by weight retained on the No. 4 sieve. The number of blows per layer for this procedure is 25 and the number of layers is 5. **NOTE: Materials that meet these gradations may also be tested using procedures B or C.**
- Procedure B. This procedure uses the 4-inch mold on only the soil passing the 3/8-inch sieve when the overall representative sample has more than 20 percent of the material by weight retained on the No. 4 sieve and 20 percent or less is retained on the 3/8-inch sieve. The number of blows per layer for this procedure is 25 and the number of layers is 5. **NOTE: Materials that meet these gradations may also be tested using procedure C.**
- Procedure C. This procedure uses the 6-inch mold on only the soil passing the 3/4-inch sieve when the overall representative sample has more than 20 percent of the material by weight retained on the 3/8-inch sieve and less than 30 percent is retained on the 3/4-inch sieve. The number of blows per layer for this procedure is 56 and the number of layers is 5.

NOTE: Previous testing methods incorporated a mix of standards that have since been rescinded. The current standard for this test is ASTM D 1557-91. This test procedure standardizes the use of the 4-inch (Proctor) and 6-inch (CBR) molds. The compaction effort at 56 blows is about the same as used in previous test methods (CE 55). This test method is applicable only to soils containing 30 percent or less by weight of material retained on the 3/4-inch sieve.

EQUIPMENT

Perform the compaction test using the following items:

- Cylinder molds (use one of the following molds, depending on the soil sample being processed):
  - Proctor mold; 4-inch (4.0-inch inside diameter and 4.584-inch inside height having an internal volume of 0.0333 cubic foot), having an extension collar (2.375 inches high) and a detachable metal baseplate.
  - CBR mold; 6-inch (6-inch inside diameter and 7-inch inside height), having an extension collar (2 inches high) and detachable metal baseplate. The mold should also have a metal spacer disk (5.94-inch inside diameter and 2.416 inches thick) for use as a false bottom in the mold during testing. When the spacer disk is in place in the bottom of the mold, the internal volume of the mold (excluding extension collar) shall be 0.075 cubic foot.
- A compacting hammer or tamper. A sliding-weight type compacting tamper, having a 2-inch-diameter steel striking face, a 10-pound mass, and an 18-inch fall.
- A No. 4 sieve.
- A 3/8-inch sieve.
- A 3/4-inch sieve.
- A balance scale sensitive to 0.01 gram.
- A balance scale sensitive to 1.0 gram.
- Moisture tares.
- A soils oven.
- Filter paper.
- A large spoon.
- A large knife.
- A steel straightedge.
- A calculator.
- DD Form 1210.
- DD Form 1211.

The amount of material (field sample) required for the compaction test depends on the test procedure being used and the field sample’s moisture content. The following are guidelines for the amount of soil required for the test procedures:

- Procedures A and B: Use about 35 pounds of dry soil or at least 50 pounds of moist soil.

- Procedure C: Use about 75 pounds of dry soil or at least 100 pounds of moist soil.

STEPS

Perform the following steps for the compaction test:

Step 1. Determine the test procedure to be used.

- If the CBR design and tests are to be developed for this project, do not use this method. See Section IX for procedures to be used for CBR.
- If CBR is not a factor, determine the test procedure by evaluating the gradation criteria of the procedures listed above (A, B, or C) with column 17 (percent retained) on DD Form 1206.

Step 2. Prepare the soil sample.

a. Dry the sample until it can be easily crumbled under a trowel. Drying may be done by air-drying or by using a drying apparatus, provided the temperature of the sample does not exceed 60°C.

b. Break up the sample thoroughly, but not in such a manner as to reduce the size of the individual particles.

c. Sieve the sample over a No. 4 (procedure A), 3/8-inch (procedure B), or 3/4-inch sieve (procedure C). When preparing the material by passing it over the 3/4-inch sieve for compaction in the 6-inch mold, break up aggregates sufficiently to at least pass the 3/8-inch sieve. This facilitates the distribution of water throughout the soil in later mixing.

d. Separate from the sample 5 equal portions representing each point desired on the compaction curve. The size of each sample for one mold is about 2,700 grams for procedures A and B or 6,800 grams for procedure C. Retain all excess soil sample.

Step 3. Adjust the water content.

NOTE: The water-content adjustments in this step are designed to provide approximations of the OMC. In no way should these approximations be used for or be interpreted as the actual moisture content. Exact moisture determinations will be conducted in a later step.

a. Establish the assumed or approximate OMC.

(1) Place exactly 100 grams of the excess soil sample in a dish.

(2) Add 5 milliliters of water to the sample and mix thoroughly. The approximate OMC is typically achieved so that when the soil is squeezed in the palm, it will adhere together on its own but it will break cleanly into two separate pieces without either piece shattering when bent. Usually this will be slightly less than the PL.

(3) Add small amounts of water (in milliliters), remembering to record the amounts added, until the approximate OMC is achieved. Do not confuse the approximate OMC with the actual moisture content of this soil, which will be determined in a later step. For purposes of conducting the test method, the approximate OMC will be the amount

of water, in milliliters, added to the sample. (For example, if only 8 milliliters of water was added to achieve the approximate OMC, then the approximate OMC is 8.0 percent. This works in approximating because 1 milliliter of water is about equal to 1 gram. By adding weight to an original sample of 100 grams, no mathematical calculations are required.)

b. Determine the moisture-content range. This range is the approximated OMC ±4. (For example, if you have determined that the approximated OMC is 8.0 percent, then the -4 is 4.0 percent and the +4 is 12.0 percent. This identifies the moisture-content range as 4.0 to 12.0 percent.)

c. Use the following formula to determine the amount of water to add to each of the 5 samples to obtain the desired approximate (-4, -2, OMC, +2, +4) moisture contents:

$$\text{water (in milliliters) to add} = \text{weight of sample (in grams)} \times \text{desired percent (decimal format)}$$

For example, to determine the water to add to obtain the approximated OMC for a sample of 6,804 grams (using procedure C)—

$$6,804 \times 0.08 = 544.3 \text{ milliliters}$$

Perform the same calculations to determine the water to add for the remaining samples for the required moisture-content range. The examples below illustrate this calculation for the remaining samples, taking into consideration that not all the sample weights will be exactly the same (6,804 grams):

(-4) 4.0% moisture for a sample at 6,815 grams: $6,815 \times 0.04 = 272.6$ *milliliters*

(-2) 6.0% moisture for a sample at 6,800 grams: $6,800 \times 0.06 = 408.0$ *milliliters*

(+2) 10.0% moisture for a sample at 6,822 grams: $6,822 \times 0.10 = 682.2$ *milliliters*

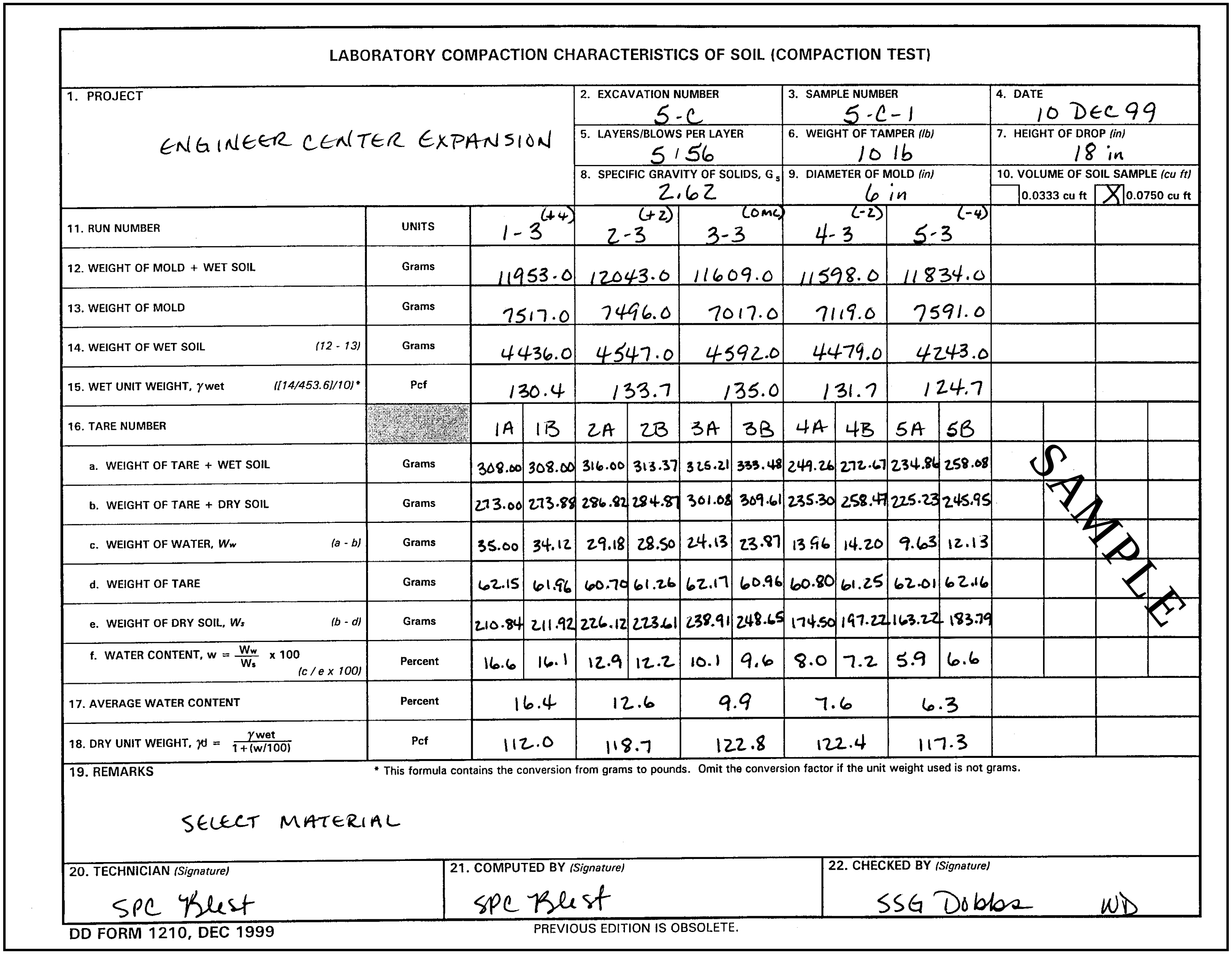
(+4) 12.0% moisture for a sample at 6,810 grams: $6,810 \times 0.12 = 817.2$ *milliliters*

d. Add the water figured from the formulas for each of the 5 desired moisture contents (-4, -2, OMC, +2, and +4) and mix thoroughly to ensure even distribution of water throughout the sample.

e. Place each sample in an airtight container and allow to stand for the minimum period of time indicated below:

- For GW, GP, SW, and SP soil types, there is no minimum standing period of time.
- For GM and SM soil types, a minimum of 3 hours standing time is required.
- For all other soil types, a minimum of 16 hours standing time is required.

Step 4. Record all identifying information such as the project, the excavation number, and other pertinent data on DD Form 1210 (see *Figure 2-55, page 2-106*).



LABORATORY COMPACTION CHARACTERISTICS OF SOIL (COMPACTION TEST)

1. PROJECT ENGINEER CENTER EXPANSION
2. EXCAVATION NUMBER 5-C
3. SAMPLE NUMBER 5-C-1
4. DATE 10 DEC 99
5. LAYERS/BLOWS PER LAYER 5 / 56
6. WEIGHT OF TAMPER *(lb)* 10 lb
7. HEIGHT OF DROP *(in)* 18 in
8. SPECIFIC GRAVITY OF SOLIDS, G_s 2.62
9. DIAMETER OF MOLD *(in)* 6 in
10. VOLUME OF SOIL SAMPLE *(cu ft)* ☐ 0.0333 cu ft ☒ 0.0750 cu ft

11. RUN NUMBER	UNITS	1-3 (+4)		2-3 (+2)		3-3 (OMC)		4-3 (-2)		5-3 (-4)					
12. WEIGHT OF MOLD + WET SOIL	Grams	11953.0		12043.0		11609.0		11598.0		11834.0					
13. WEIGHT OF MOLD	Grams	7517.0		7496.0		7017.0		7119.0		7591.0					
14. WEIGHT OF WET SOIL *(12 - 13)*	Grams	4436.0		4547.0		4592.0		4479.0		4243.0					
15. WET UNIT WEIGHT, γwet *([14/453.6]/10)**	Pcf	130.4		133.7		135.0		131.7		124.7					
16. TARE NUMBER		1A	1B	2A	2B	3A	3B	4A	4B	5A	5B				
a. WEIGHT OF TARE + WET SOIL	Grams	308.00	308.00	316.00	313.37	325.21	333.48	249.26	272.67	234.86	258.08				
b. WEIGHT OF TARE + DRY SOIL	Grams	273.00	273.88	286.82	284.87	301.08	309.61	235.30	258.47	225.23	245.95				
c. WEIGHT OF WATER, W_w *(a - b)*	Grams	35.00	34.12	29.18	28.50	24.13	23.87	13.96	14.20	9.63	12.13				
d. WEIGHT OF TARE	Grams	62.15	61.96	60.70	61.26	62.17	60.96	60.80	61.25	62.01	62.16				
e. WEIGHT OF DRY SOIL, W_s *(b - d)*	Grams	210.84	211.92	226.12	223.61	238.91	248.65	174.50	197.22	163.22	183.79				
f. WATER CONTENT, $w = \frac{W_w}{W_s} \times 100$ *(c / e x 100)*	Percent	16.6	16.1	12.9	12.2	10.1	9.6	8.0	7.2	5.9	6.6				
17. AVERAGE WATER CONTENT	Percent	16.4		12.6		9.9		7.6		6.3					
18. DRY UNIT WEIGHT, $\gamma d = \frac{\gamma wet}{1+(w/100)}$	Pcf	112.0		118.7		122.8		122.4		117.3					

SAMPLE

* This formula contains the conversion from grams to pounds. Omit the conversion factor if the unit weight used is not grams.

19. REMARKS

SELECT MATERIAL

20. TECHNICIAN *(Signature)* SPC Blest
21. COMPUTED BY *(Signature)* SPC Blest
22. CHECKED BY *(Signature)* SSG Dobbs WD

DD FORM 1210, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

Figure 2-55. Sample DD Form 1210

Step 5. Prepare the mold and moisture-determination tares.

a. Lightly oil a mold (4- or 6-inch, depending on the procedure selected).

b. Weigh the mold.

- For the 4-inch mold, weigh the mold with the baseplate to the nearest gram. Record this weight on the form as the weight of the mold (block 13). Do not include the collar.
- For the 6-inch mold, weigh the mold with the baseplate and spacer disk to the nearest gram. Record this weight on the form as the weight of the mold (block 13). Do not include the collar.

c. Attach the collar to the mold. If using the 6-inch mold, place a coarse filter paper on top of the spacer disk.

d. Record the volume of the mold as 0.0333 cubic foot for the 4-inch mold or 0.075 cubic foot for the 6-inch mold with the spacer disk.

e. Mark and weigh 2 moisture-determination tares for each mold prepared. Record as the weight of tare.

Step 6. Place sufficient soil in the mold (about 1 1/2 to 2 inches) to obtain about a 1-inch compacted layer. After compaction of all 5 layers, each layer should be about equal in thickness. The fifth compacted layer will slightly extend into the collar but will not exceed 1/4 inch above the top of the mold.

Step 7. Apply compactive effort.

a. Hold the 10-pound compaction tamper within 5 degrees of vertical, placing its face on top of the soil.

b. Raise the handle until it reaches the top (18 inches) and release it, allowing the weight to fall freely onto the soil.

c. Change the position of the guide and tamper, and repeat the process until the soil layer has received the prescribed 25 blows for procedures A and B or 56 blows for procedure C. Apply the blows at a uniform rate of about 25 blows per minute. The height of fall of the tamper must be controlled carefully and the blows distributed evenly over the specimen's surface (see *Figure 2-56, page 2-108*).

Step 8. Trim the compacted layer. After compacting each layer (except the fifth layer), use a knife to trim any soil adjacent to the mold walls that has not been compacted or that extends above the compacted surface. Include the trimmed soil with the additional soil for the next layer.

Step 9. Repeat steps 6, 7, and 8 until five layers have been compacted in the mold. Each compacted layer should be about equal in thickness (just under 1 inch). Adjust each layer accordingly to ensure that the fifth compacted layer will slightly extend into the collar, but will not exceed 1/4 inch above the top of the mold.

Step 10. Remove the collar from the mold.

a. Cut around the inside edge of the collar to prevent shearing the compacted soil when removing the collar.

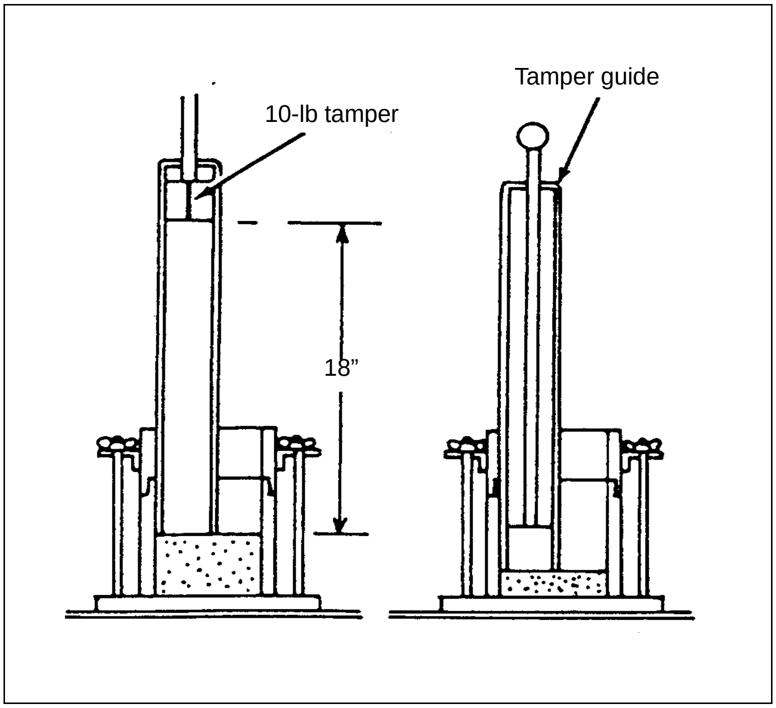


Figure 2-56. Soil compaction in a mold

b. Trim and smooth the compacted soil flush with the top of the mold (see *Figure 2-57*). Use a sawing motion with the straightedge to trim the excess soil. Start at the center of the mold and work outward, first to one side and then to the other. Fill any holes with unused or trimmed soil from the specimen, press in with the fingers, and again scrape the surface with the straightedge.

Step 11. Weigh and record the data. Weigh the complete mold with the baseplate and compacted specimen (including the spacer disk for the 6-inch mold) to the nearest gram. Record the weight on the form as the weight of the mold and wet soil (block 12). Do not include the collar.

Step 12. Prepare the specimen for moisture-content determination.

a. Remove the specimen from the mold.

b. Slice the compacted specimen axially through the center and remove about 250 grams of material from one side of the cut and place it in one of the moisture-determination tares and cover. Remove about 250 grams from the other side of the cut, and place it in the other moisture-determination tare and cover.

Step 13. Repeat steps 5 through 12 for the remaining molds at the different moisture contents described in step 3c.

Step 14. Determine the moisture content of the material in the moisture-determination tares by performing moisture-content testing of the tares as described in Section III.

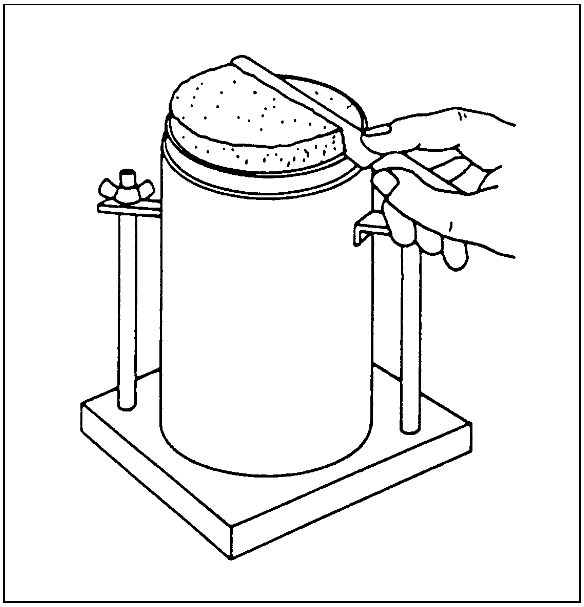


Figure 2-57. Trimming the compacted soil sample

Step 15. Perform calculations. Record the weight of the dry soil and tare on the form. Compute the weight of water, weight of dry soil, and the water content and record it on the form. Compute the average water content from the two tares and record it on the form. Calculate and record the dry unit weight as follows:

$$\text{dry unit weight} = \frac{\text{wet unit weight}}{1 + \dfrac{\text{percent water content}}{100}}$$

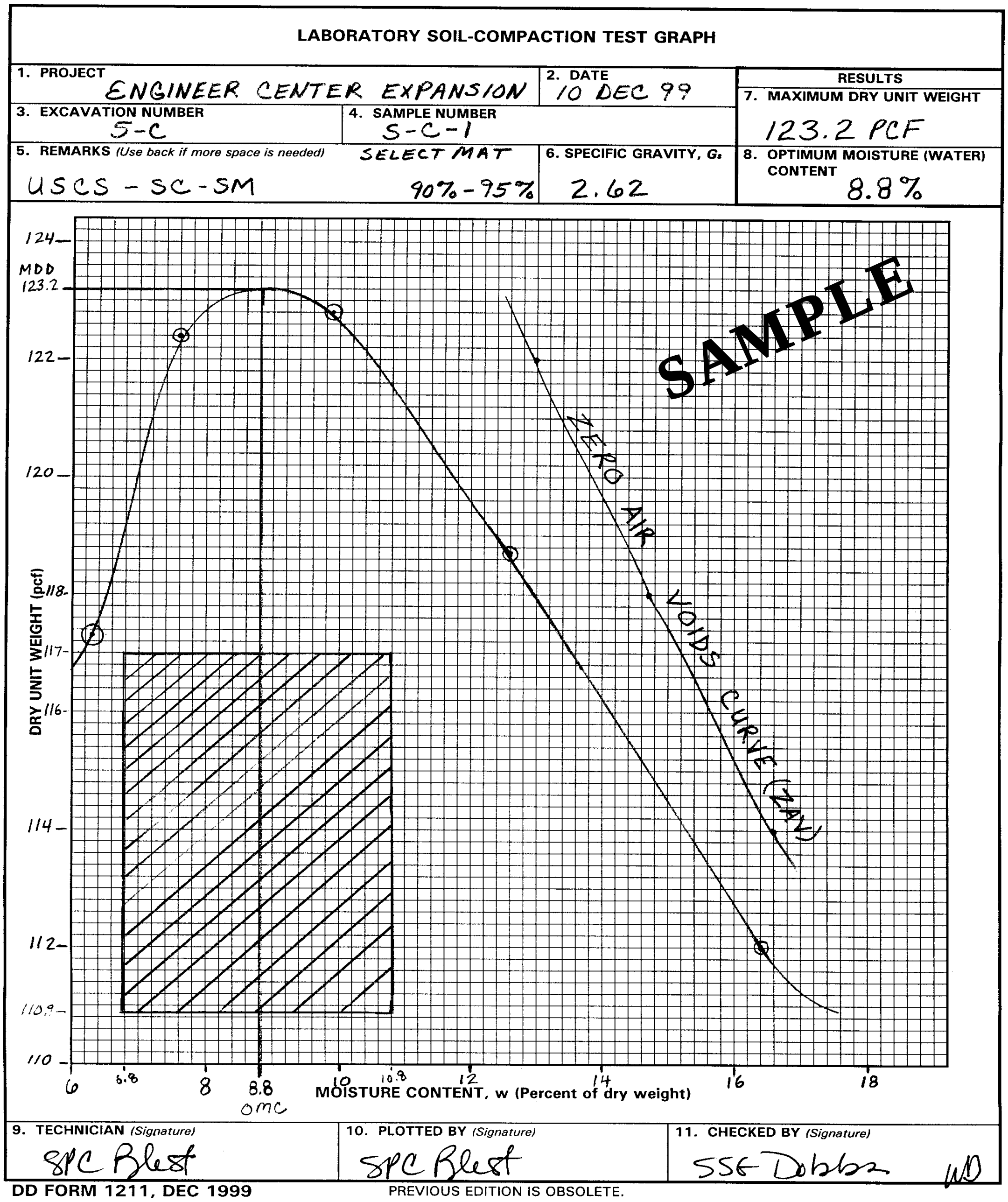
COMPACTION-TEST GRAPH—PRESENTATION OF RESULTS

The soil compaction-test graph (DD Form 1211) is an important part of presenting the data from the compaction test. It is used to plot a compaction curve. This curve is needed to determine the MMD and OMC as part of the compaction-test procedures. It also includes a zero-air-voids curve and a compaction-specification block.

COMPACTION CURVE

The compaction curve is obtained by plotting moisture content versus dry unit weight for each test on the soil-compaction-test graph (see *Figure 2-58, page 2-110*). To construct an acceptable curve, at least two of the plotted points should fall on each side of the OMC. It is important to remember that during the testing period, the only density that can be determined is the wet soil density or wet unit weight. To compute the dry unit weight, the moisture content must be determined. This can take up to 24 hours.

For a typical cohesive soil, dry density increases to a certain point (the OMC) as the moisture in the soil increases. Once the OMC is achieved, the dry density begins to decrease with increasing moisture content. The primary reason for performing the compaction test is to determine the moisture



LABORATORY SOIL-COMPACTION TEST GRAPH

1. PROJECT ENGINEER CENTER EXPANSION

2. DATE 10 DEC 99

RESULTS

7. MAXIMUM DRY UNIT WEIGHT 123.2 PCF

3. EXCAVATION NUMBER 5-C

4. SAMPLE NUMBER S-C-1

5. REMARKS *(Use back if more space is needed)* SELECT MAT USCS - SC-SM 90%-95%

6. SPECIFIC GRAVITY, G. 2.62

8. OPTIMUM MOISTURE (WATER) CONTENT 8.8%

9. TECHNICIAN *(Signature)* SPC Blest

10. PLOTTED BY *(Signature)* SPC Blest

11. CHECKED BY *(Signature)* SSG Dobbs WD

DD FORM 1211, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

Figure 2-58. Sample DD Form 1211

content at which the MDD can be obtained. After plotting the compaction curve, it is possible to determine the moisture content that will give the MDD for that particular soil directly from the plotted curve.

The compaction curve shows the OMC or the moisture content at which the MDD is obtained for a given compaction effort. By determining the highest point on the compaction curve (apex) and dropping vertically to the horizontal moisture scale or line, the OMC for this particular soil is found to be 8.8 percent, as shown in *Figure 2-58*.

The compaction curve also shows MDD (100 percent compaction). The MDD of 100 percent effort may be obtained by running a tangent from the highest point on the compaction curve for the particular soil to the vertical dry-density scale (see *Figure 2-58*); in this case, 123.2 pounds per cubic foot (pcf).

A compaction curve is not complete without the zero-air-voids curve, which acts as a control to the compaction curve.

ZERO AIR VOIDS AND SATURATION

The zero-air-voids curve represents theoretical values that are practically unattainable—no matter what degree of compactive effort—because it is not possible to remove all the air contained in the voids of a soil by compaction alone. Typically, at moisture contents beyond optimum, the actual compaction curve closely parallels the theoretically-perfect compaction curve. Any values of dry density that plot to the right of the zero-air-voids curve are in error. The error may be in the test measurement, the calculations, or the specific gravity.

At complete saturation, the voids in the solid mass are completely filled with water. That is, no air is present and the degree of saturation (S) is equal to 100 percent. The zero-air-voids curve (100 percent saturation) for the soil tested is shown on the graph in *Figure 2-58*. This curve is obtained by plotting dry densities corresponding to complete saturation at different moisture contents using the following formula:

$$w = S \times \left(\frac{62.43}{g_d} - \frac{1}{G_s} \right)$$

where—

w = *water content, in percent*

S = *degree of saturation, in percent*

62.43 = unit weight of water, in pcf

g_d = *dry unit weight of soil, in pcf*

G_s = *specific gravity of solids for this soil (block 6, DD Form 1211)*

Use this equation to compute points for plotting the zero-air-voids curve. Plot at least three points for drawing the curve. Use points within the range of the compaction-curve dry unit weights. Select three dry unit weights in this range and calculate as listed above.

Example: The upper plotted point of the zero-air-voids curve for the soil represented in *Figure 2-58* is a whole number just under maximum density (122 pcf). The specific gravity of solids (G_S) was determined to be 2.62. At 100 percent saturation for this density, the corresponding moisture content would be 13 percent. This is determined by the zero-air-voids formula:

$$100 \times \left(\frac{62.43}{122} - \frac{1}{2.62}\right) = 13$$

Any other plotted points are also determined using this formula as follows:

$$100 \times \left(\frac{62.43}{118} - \frac{1}{2.62}\right) = 14.7$$
$$100 \times \left(\frac{62.43}{114} - \frac{1}{2.62}\right) = 16.6$$

PERCENT MOISTURE

To obtain the maximum dry unit weight or density in the field, it is necessary to maintain the construction soil's moisture content as close as possible to the optimum determined from the laboratory compaction test. If the moisture content is not close to the OMC, it will require extra time and equipment effort to obtain the MDD. The limits for moisture contents should be outlined in the specifications for each job. If not specified, the limits should be established as ± 2 percent of the OMC. Using *Figure 2-58*, where the OMC is 8.8 percent, the moisture limits would range from 6.8 to 10.8 percent. This provides the limits for a workable and practical specification block.

PERCENT COMPACTION

Some soils will not or cannot be compacted to 100 percent at a reasonable equipment effort, regardless of the combination. In those cases, it is not mandatory to compact to 100 percent. For each job, the specifications will state the percent of compaction required for the particular loadings. Assume that the specifications require 90 percent of MDD. To find the dry density required, multiply the MDD (100 percent), regardless of its value, by 0.90. This will give the density limit. If the specifications state between 90 and 95 percent, the 90 percent density will constitute the lower limit with 95 percent as the upper limit. A specification block can now be constructed (see *Figure 2-58, page 2-110*).

COMPACTION-SPECIFICATION BLOCK

The compaction-specification block shows a determination range based on the project specifications. If no specifications are given, refer to the minimum compaction specification requirements as listed in FMs 5-410, 5-430-00-1, or 5-430-00-2. Once the range is determined, the specification block is plotted on the compaction curve (see *Figure 2-58*), covering the specified compaction-range requirement in percent for the dry unit weight within a 4 percent range of the OMC (± 2 OMC). The block is then lightly cross-hatched so as to not interfere with the compaction curve. If the field results fall within this block, the job is meeting specifications.

Equipment

Use the following to complete the compaction-specification block:

- DD Form 1211 with compaction curve and zero-air-voids curve.
- A calculator.
- The proposed use of the soil.
- The project's compaction specifications.

Steps

Perform the following steps to complete the soil compaction-specification block (DD Form 1211):

Step 1. Determine the compaction-specification requirements as discussed above.

Step 2. Draw the compaction-specification block. Establish the upper and lower limits of the block as the specification range (in percent) of the dry unit weight. Establish the left and right limits of the block as ± 2 of OMC.

The example in *Figure 2-58, page 2-110*, represents the compaction data from DD Form 1210 (see *Figure 2-55, page 2-106*), the zero-air-voids curve plotted from a specific gravity value of 2.62, and a compaction specification of 90 to 95 percent. Notice the compaction-specification block ranges vertically from 117 to 110.9 pcf and horizontally from 6.8 to 10.8 percent in moisture content. The computation used to achieve the upper and lower limits is based on the MDD (in this case, 123.2 pcf) and the specification range of 90 to 95 percent as follows:

$$123.2 \times 0.95 = 117.0$$

$$123.2 \times 0.90 = 110.9$$

Once the block has been established and drawn, the inside of the block can be lightly cross-hatched to easily identify the range. Do this so as not to interfere with the curve or any other data plotted on the chart.

EFFECT OF WATER ON DENSITY

Figure 2-59, page 2-114, demonstrates that as the moisture content is varied, the dry density also varies. As water is added to an oven-dried soil, the dry density increases until the OMC is reached. The dry density then begins to decrease using a constant compactive effort.

After adding small increments of water to a completely air-dry soil, subsequent compaction with a constant compactive effort causes a small increase in the soil's dry unit weight. During the initial hydration phase, the water being added to the soil is absorbed on the surface of the soil grains. This water does not aid compaction by acting as a lubricant since it is firmly attached to the surface of the soil particles. Adding additional water brings the soil to a point where a slight change in moisture begins to produce a large increase in density. This rapid increase indicates that the lubrication phase of the compaction curve has been reached.

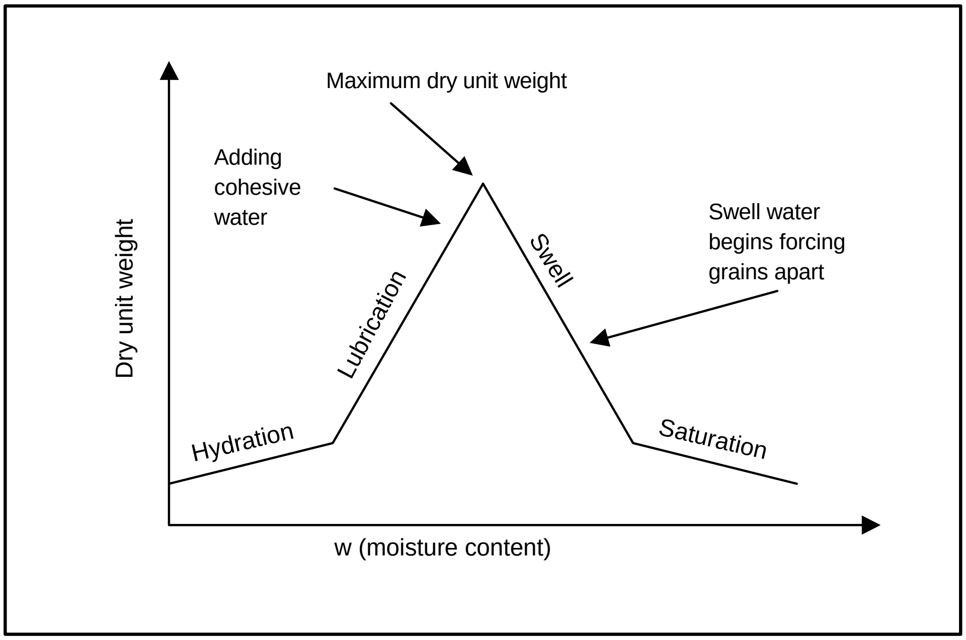


Figure 2-59. Effect of water on density

The first portion of the compaction curve to the right of OMC is known as the swell phase. The addition of water increases the film around the soil particles, forcing the soil particles apart and decreasing the dry density. With further increases in moisture content, free water added to the soil will fill the void spaces. This is known as the saturation phase. In the swell and saturation phases, the water begins to take the place of the solids, thus decreasing the dry density.

EFFECT OF DIFFERENT COMPACTIVE EFFORTS ON DENSITY

The mass-per-unit volume of a soil varies directly with the amount of energy expended to compact that soil. Therefore, the greater the compactive effort, the greater the amount of solids per unit volume. This results in a stronger and more stable soil.

As the compactive effort is increased, the dry density of the soil increases. This means that if more energy is used to compact a soil, the increased energy will cause the particles to be rearranged to a greater extent, thus increasing the mass of soil particles per unit volume. If the compactive effort is decreased, the particles will not be rearranged as much, thus decreasing the dry density. *Figure 2-60* shows how the dry density varies with the compactive effort.

The OMC varies inversely with respect to compactive effort. If the compactive effort is increased, the soil does not have to be as wet to obtain the MDD. In other words, the OMC will be decreased with increasing compactive effort (see *Figure 2-60*).

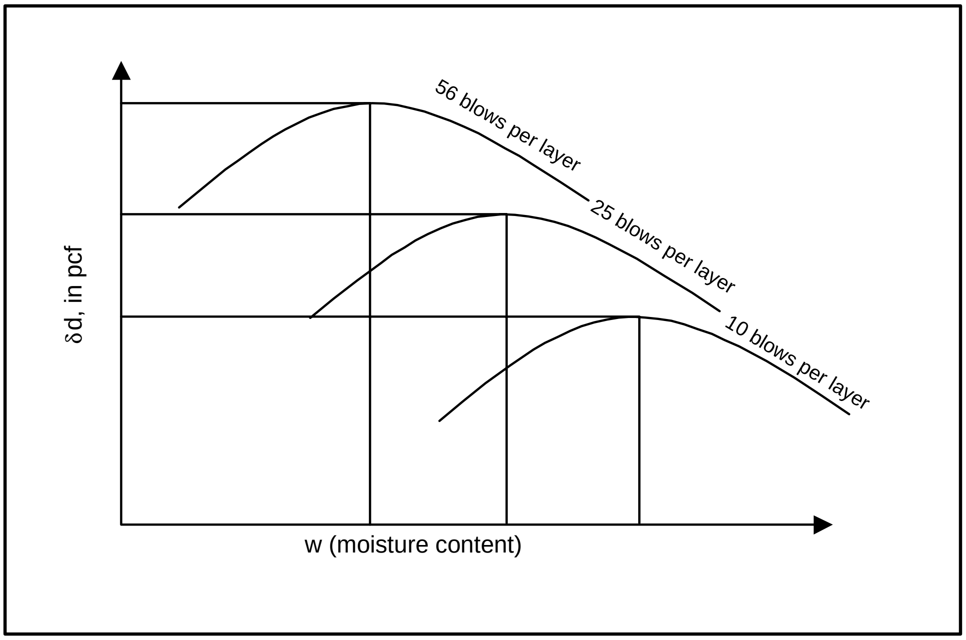


Figure 2-60. Effect of different compactive effort on density

EFFECT OF DIFFERENT TYPES OF SOILS ON DENSITY

Different soils have varying compactive characteristics. Gravelly and sandy soils have a lower OMC and higher densities under the same compactive effort compared with silty and clay soils (see *Figure 2-61, page 2-116*). The sharpness of the curves indicates that moisture content is much more critical in obtaining maximum density for coarse-grained soils than for fine-grained soils.

COMPACTION EQUIPMENT

Equipment normally available to the military engineer for soil compaction includes sheepsfoot, pneumatic-tired, and steel-wheeled rollers. Other construction equipment and load-hauling units may also be used. Crawler-type tractor units are efficient in compacting free-draining sands and gravels that should be kept wet during the compaction process. This equipment is not efficient for compacting cohesive soils. Compaction equipment use is covered in FM 5-434.

For compaction equipment to be used efficiently, the moisture content at which maximum compaction can be obtained (the OMC) and the maximum density to which the soil can be compacted are required. This data is obtained by performing the laboratory compaction test. The stability or strength of the base course in the field can only be obtained if the moisture content allows proper compaction and if compaction is obtained at or above the amount specified.

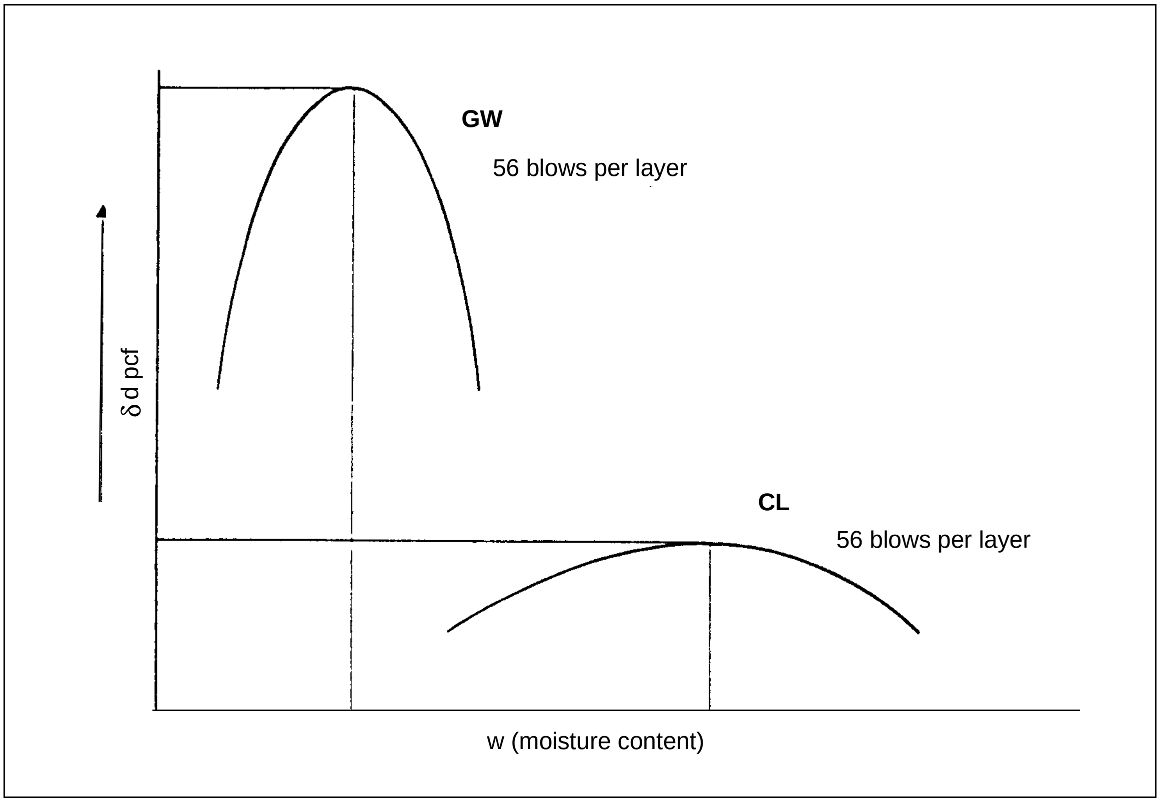


Figure 2-61. Effect of different soil types on density

OTHER COMPACTIVE EFFORTS

Under some circumstances, it may be necessary to use a compactive effort in the laboratory. Usually this is done to study the effect of variation in density on some property of the soil, such as the CBR. In this case, samples are compacted using the procedures previously described, except that the variation in density is achieved by varying the number of blows applied to each layer as described in ASTM D 1883-94. In unusual circumstances, the laboratory compaction procedure may be changed to produce a compactive effort that more closely resembles the energy that can be put into the soil using available rolling equipment. In the CBR test, the compaction procedure calls for 10, 25, or 56 blows per layer. This is explained in Section IX of this chapter.

SECTION VIII. IN-PLACE DENSITY DETERMINATION

Proper field control is essential in earthwork construction. The control tests are conducted on the soil at the jobsite as construction proceeds. If at any time a test indicates that operations are not producing a soil condition specified by the design tests, take immediate action to remedy the situation.

A soil’s stress-deformation characteristics are directly related to the soil's moisture content and density, allowing specifications to be set for a given soil as construction proceeds. Densities obtained are compared with minimum density requirements established for the particular job. Water contents are compared with the OMC previously established to see that compaction is taking place within the desired range or to permit its adjustment.

An undisturbed sample of known or measurable dimensions provides information for computing the soil's density and moisture content. If the soil is not in proper condition during construction to remove an undisturbed sample, the density may be determined by measuring the volume of the hole after the sample is removed. The procedure consists of filling the hole with a measured quantity of a known density material (such as sand, oil, or water) and computing the volume of soil removed. The soil's moisture content and density are then determined.

The method for in-place density depends on the type of soil encountered and the equipment available. On moist, cohesive, fine-grained soils, undisturbed samples taken by samplers may be sufficient. Coarse-grained or cohesionless soils make it difficult to obtain an undisturbed sample. In these soils, density determination may require the displacement method. Sand displacement may be used on any type of base course or subgrade material. Oil displacement cannot be used on highly pervious soils, crushed stone, or slag base courses. If the pavement to be used is asphaltic concrete, the residual oil and spillage will tend to soften the asphalt. Displacing water requires the use of a balloon to contain the water and can be used on any type of soil.

If the density determined by the methods described in the following paragraphs is equal to or greater than that required, compaction may be judged to be satisfactory and the placing of another lift may proceed. If the density is lower than that required, additional rolling may be necessary or the moisture content may have to be adjusted. If these methods fail, the weight of the roller may have to be increased, the thickness of lift reduced, or other methods used to obtain adequate compaction. The possibility that the soil being compacted in the field is not the same as the one tested in the laboratory should never be overlooked. Under normal field conditions (the work is proceeding smoothly and uniform soils are being compacted), the number of density and moisture checks required should be limited after the initial period of compacting. If adequate densities are being obtained and the proper moisture content is being maintained, inspections may be performed to determine and verify the number of passes and the combination of rollers to achieve the desired result with minimum effort. Where conditions are more variable, density and moisture checks may be needed more often for a fill of even moderate length. The exact number of checks needed should be determined by the engineer in charge of the job.

SAND-CONE OR SAND-DISPLACEMENT METHOD (ASTM D 1556-90)

The sand-cone or sand-displacement method may be used in either fine- or coarse-grained materials. Calibrated sand is used to determine the volume of the hole from which a sample has been taken.

The sand-cone or sand-displacement test consists of digging out a sample of the material to be tested, determining the volume of the hole, and determining the dry weight of the sample. There are three requirements that must be met for this test.

The volume of the sample should be as close to the same volume at which the sand was calibrated; however, the sample's maximum particle size will determine the volume of sample to be tested. Sample requirements are as follows:

- Material with a maximum aggregate size of 1/2 inch requires a minimum test-hole volume of 0.05 cubic foot.
- Material with a maximum aggregate size of 1 inch requires a minimum test-hole volume of 0.075 cubic foot.
- Material with a maximum aggregate size of 2 inches requires a minimum test-hole volume of 0.10 cubic foot.

A double-cone cylinder must be used. This permits calibrating the sand for each test performed. The sand must be clean, dry, and free-flowing with a constant moisture content during the test. The sand should pass the No. 10 sieve and should have less than 3 percent passing the No. 60 sieve. The ideal solution to ensure that the gradation requirements are met is to obtain Ottawa sand, which generally ranges from the No. 20 to No. 40 sieve sizes.

PURPOSE

Perform this test to determine the in-place density of a soil to within ± 2 pcf.

EQUIPMENT

Use the following items to perform this test in a field environment (see *Figure 2-62*):

- A template with a 6-inch hole.
- A 6-inch soil-density tester.
- A CBR mold.
- A 2 1/2-inch spacer disk.
- Filter paper.
- A knife.
- A hammer.
- Nails, twentypenny (20d).
- A steel straightedge.
- A varnish brush.
- A balance scale sensitive to 1.0 gram with a 20-kilogram capacity.
- A balance scale sensitive to 0.01 gram with a 500-gram capacity.
- A ruler.
- A speedy moisture test set.

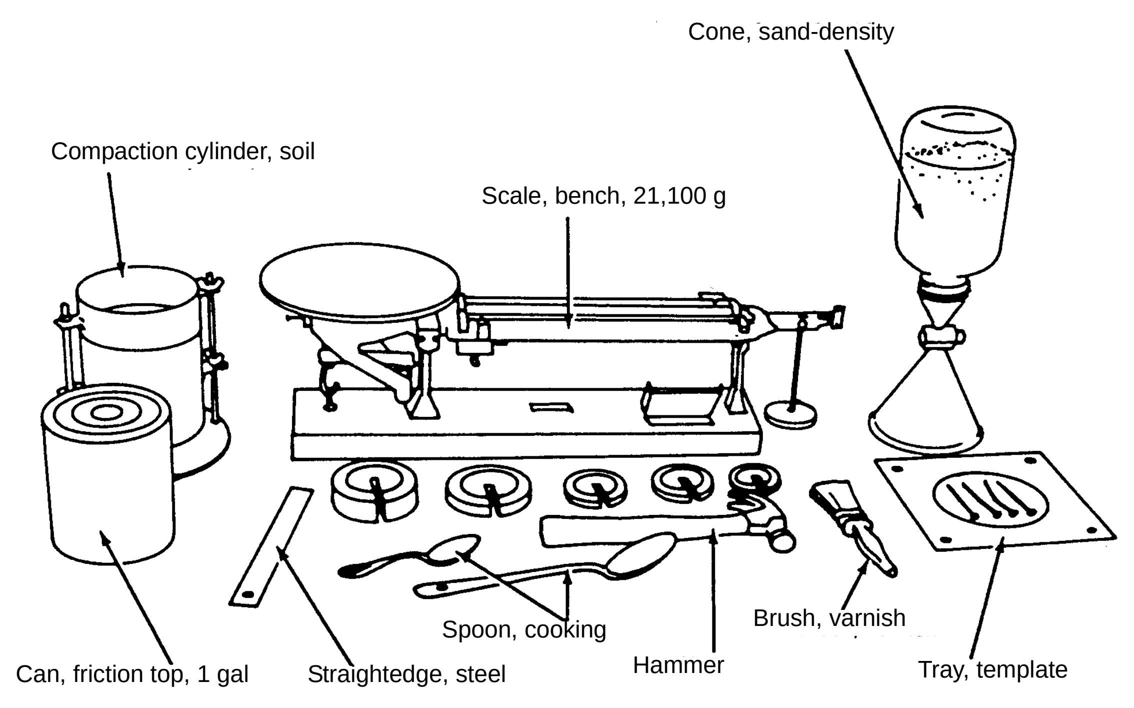


Figure 2-62. Sand-displacement-method apparatus

- A laboratory oven.
- Heat-resistant gloves.
- A spoon.
- A chisel.
- Friction-top cans.
- Sand, well-rounded, passing the No. 10 sieve with less than 3 percent passing the No. 60 sieve.
- DD Form 1215.
- Paper.
- A pencil.
- A calculator.
- A grease pencil.
- Moisture-determination tares.

STEPS

Perform the following steps for the sand-cone or sand-displacement method:

Step 1. Determine the sand's density.

a. Weigh the mold, baseplate, spacer disk, and filter paper. Record this weight (in grams) on line 8 of DD Form 1215 (see *Figure 2-63*).

b. Attach the collar to the mold and place the filled sand-cone apparatus, with the valve closed, on top of the mold.

c. Open the valve and allow sand to fall at its own rate into the mold. Do not jar the apparatus while the sand is falling.

d. Close the valve when the sand stops running into the mold and the collar.

e. Remove the sand cone and collar carefully from the mold.

f. Use a straightedge to strike off the excess sand remaining in the top of the mold.

g. Brush off any sand adhering to the outside of the mold.

h. Weigh the mold full of sand (in grams) and record it on line 7 on the form.

i. Determine the weight (in grams) of the material by subtracting line 8 from line 7 and record the weight on line 9 of the form.

j. Enter the known volume of the mold (in cubic feet) on line 10.

k. Determine the unit weight of the material. Convert the weight of the materials (line 9) from grams to pounds, divide this quotient by the known volume (line 10), and record the weight on line 11.

$$\text{unit weight of material (line 11)} = \frac{\left(\dfrac{\text{weight of sand}}{453.6}\right)}{\text{volume of mold}}$$

l. Repeat steps a through k at least two more times. The unit weight of sand used in the calculations shall be the average of at least three determinations. Record the average unit weight of material on line 12. The maximum variation between any one determination and the average should not exceed 1 percent.

Step 2. Prepare the site for sand-cone testing.

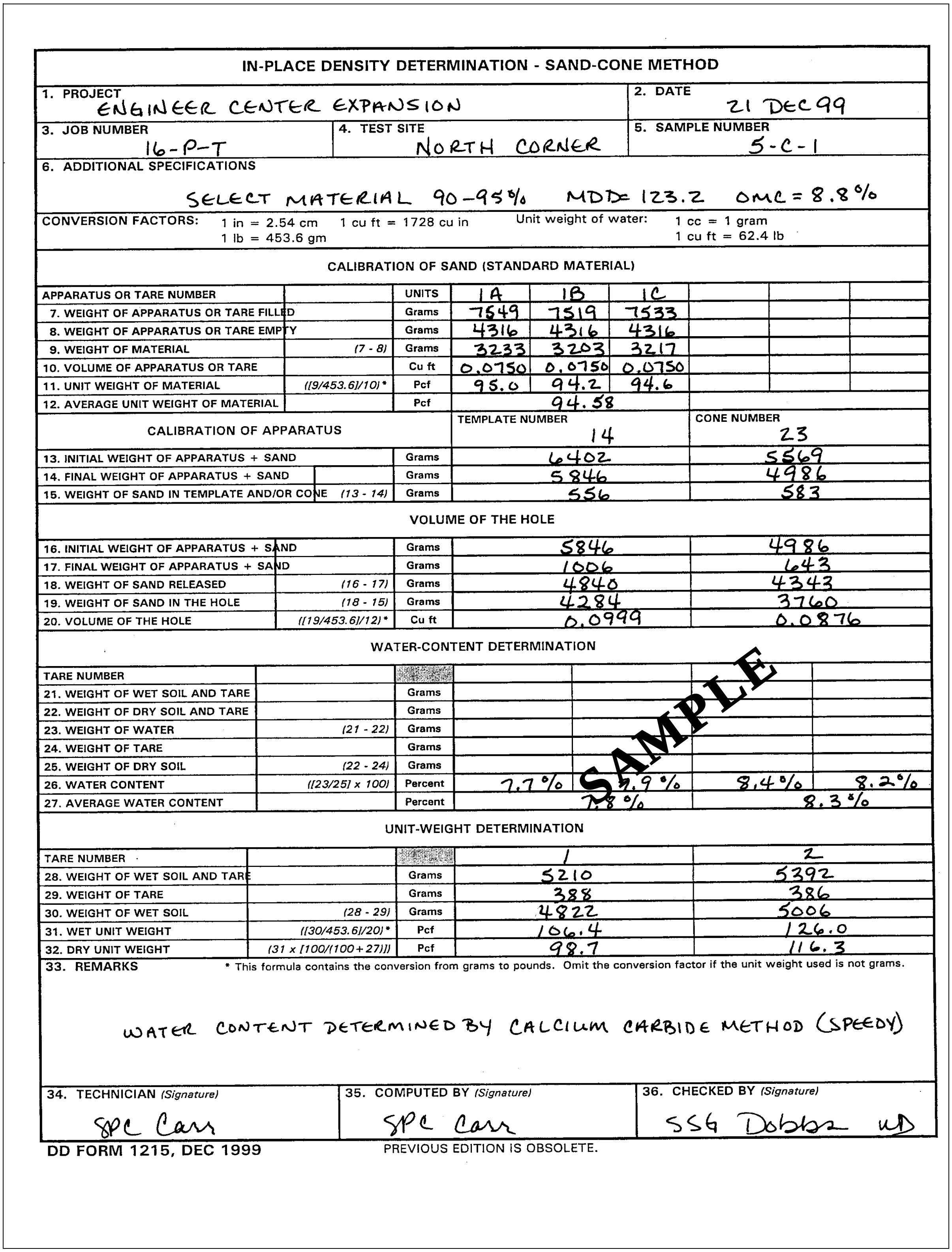
a. Clear the overburden and seat the template tray flush on the surface. Fasten it in place with nails.

b. Seal the spaces on the inside edge under the template using soil from the preparation site.

Step 3. Determine the surface calibration.

a. Weigh the sand-cone apparatus filled with sand. Record the weight (in grams) on line 13 of the form.

b. Turn the sand-cone apparatus over with the valve closed and place it on the template.



IN-PLACE DENSITY DETERMINATION - SAND-CONE METHOD

1. PROJECT: ENGINEER CENTER EXPANSION
2. DATE: 21 DEC 99
3. JOB NUMBER: 16-P-T
4. TEST SITE: NORTH CORNER
5. SAMPLE NUMBER: 5-C-1
6. ADDITIONAL SPECIFICATIONS: SELECT MATERIAL 90-95% MDD= 123.2 OMC = 8.8%

CONVERSION FACTORS: 1 in = 2.54 cm 1 cu ft = 1728 cu in
1 lb = 453.6 gm
Unit weight of water: 1 cc = 1 gram
1 cu ft = 62.4 lb

CALIBRATION OF SAND (STANDARD MATERIAL)

APPARATUS OR TARE NUMBER		UNITS	1A	1B	1C			
7. WEIGHT OF APPARATUS OR TARE FILLED		Grams	7549	7519	7533			
8. WEIGHT OF APPARATUS OR TARE EMPTY		Grams	4316	4316	4316			
9. WEIGHT OF MATERIAL	(7 - 8)	Grams	3233	3203	3217			
10. VOLUME OF APPARATUS OR TARE		Cu ft	0.0750	0.0750	0.0750			
11. UNIT WEIGHT OF MATERIAL	([9/453.6]/10)*	Pcf	95.0	94.2	94.6			
12. AVERAGE UNIT WEIGHT OF MATERIAL		Pcf	94.58					

CALIBRATION OF APPARATUS

			TEMPLATE NUMBER 14	CONE NUMBER 23
13. INITIAL WEIGHT OF APPARATUS + SAND		Grams	6402	5569
14. FINAL WEIGHT OF APPARATUS + SAND		Grams	5846	4986
15. WEIGHT OF SAND IN TEMPLATE AND/OR CONE	(13 - 14)	Grams	556	583

VOLUME OF THE HOLE

16. INITIAL WEIGHT OF APPARATUS + SAND		Grams	5846	4986
17. FINAL WEIGHT OF APPARATUS + SAND		Grams	1006	643
18. WEIGHT OF SAND RELEASED	(16 - 17)	Grams	4840	4343
19. WEIGHT OF SAND IN THE HOLE	(18 - 15)	Grams	4284	3760
20. VOLUME OF THE HOLE	([19/453.6]/12)*	Cu ft	0.0999	0.0876

WATER-CONTENT DETERMINATION

TARE NUMBER						
21. WEIGHT OF WET SOIL AND TARE		Grams				
22. WEIGHT OF DRY SOIL AND TARE		Grams				
23. WEIGHT OF WATER	(21 - 22)	Grams				
24. WEIGHT OF TARE		Grams				
25. WEIGHT OF DRY SOIL	(22 - 24)	Grams				
26. WATER CONTENT	([23/25] x 100)	Percent	7.7%	7.9%	8.4%	8.2%
27. AVERAGE WATER CONTENT		Percent	7.8%		8.3%	

SAMPLE

UNIT-WEIGHT DETERMINATION

TARE NUMBER			1	2
28. WEIGHT OF WET SOIL AND TARE		Grams	5210	5392
29. WEIGHT OF TARE		Grams	388	386
30. WEIGHT OF WET SOIL	(28 - 29)	Grams	4822	5006
31. WET UNIT WEIGHT	([30/453.6]/20)*	Pcf	106.4	126.0
32. DRY UNIT WEIGHT	(31 x [100/(100+27)])	Pcf	98.7	116.3

33. REMARKS

* This formula contains the conversion from grams to pounds. Omit the conversion factor if the unit weight used is not grams.

WATER CONTENT DETERMINED BY CALCIUM CARBIDE METHOD (SPEEDY)

34. TECHNICIAN *(Signature)* SPC Carr
35. COMPUTED BY *(Signature)* SPC Carr
36. CHECKED BY *(Signature)* SSG Dobbs WD

DD FORM 1215, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

Figure 2-63. Sample DD Form 1215

c. Open the valve and allow the sand to fall at its own rate. Do not jar the apparatus while the sand is falling. Close the valve when the sand stops running.

d. Remove the sand-cone apparatus from the template and weigh it. Record this weight (in grams) on line 14 of the form.

e. Determine the weight of the sand in the cone by subtracting line 14 from line 13. Record the difference on line 15 of the form.

Step 4. Recover as much sand as possible from the template and brush the remaining sand lightly from the hole, being careful not to disturb the soil surface.

Step 5. Dig the sample.

a. Predetermine the weight of a friction-top can. Record this weight on line 29 of the form.

b. Dig a hole through the center of the template. The hole should be 6 inches deep or to the bottom of the lift and about the same diameter as the hole in the template. The sides should be as smooth as possible.

c. Place all soil particles from the hole into the preweighed can, keeping the lid on the can as much as possible to prevent excessive moisture loss.

d. Weigh the wet soil and can. Record the weight on line 28 of the form.

Step 6. Determine the volume of the hole.

a. Refill the jar if it appears there is not enough sand to fill the hole and record the weight of the jar on line 16; otherwise, transfer the weight from line 14 to line 16.

b. Turn the sand-cone apparatus over with the valve closed and place it on the template.

c. Open the valve and allow the sand to fall at its own rate into the prepared hole. Do not jar the apparatus while the sand is falling.

NOTE: If additional sand is needed due to requirements for hole volume, ensure that these jars with sand have been properly weighed and will be on hand during the testing procedures. If used, ensure that the weight of the additional jar plus sand is included on line 16 and that line 17 also includes the final weight of the additional jar.

d. Close the valve when the sand stops running and remove the sand-cone apparatus. Weigh it and record this weight on line 17 of the form.

e. Determine the weight of the material released by subtracting line 17 from line 16. Record this difference on line 18 of the form.

f. Determine the weight of the material in the hole by subtracting line 15 from line 18. Record this difference on line 19 of the form.

g. Recover as much sand as possible from the hole.

h. Compute the volume of the hole. Convert the weight of the material in the hole (line 19) from grams to pounds, divide this quotient by the average unit weight of material (line 12), and record the weight on line 20 of the form using the following formula:

$$\text{volume of the hole (line 20)} = \frac{\left(\dfrac{\text{weight of sand in hole}}{453.6}\right)}{\text{average unit weight of sand}}$$

Step 7. Determine the average water content of the soil removed from the hole (step 5) using the oven-laboratory method or speedy moisture tester, and record it on line 27 of the form. If using the speedy moisture tester, enter the results on line 27; otherwise, make appropriate entries on lines 21 through 27 using at least two moisture-determination tares according to Section III of Chapter 2.

Step 8. Determine the unit weight (density).

a. Compute the weight of the wet soil by subtracting the weight of the tare (line 29) from the weight of the wet soil and the tare (line 28). Record the difference on line 30 of the form.

b. Compute the wet unit weight (density). Convert the weight of the wet soil (line 30) from grams to pounds, divide this quotient by the volume of the hole (line 20) and record it on line 31 of the form using the following formula:

$$\text{wet density (line 31)} = \frac{\left(\dfrac{\text{weight of wet soil}}{453.6}\right)}{\text{volume of hole}}$$

c. Compute the dry unit weight (density), and record it on line 32 of the form using the following formula:

$$\text{dry density (line 32)} = \text{wet unit weight} \times \frac{100}{100 + \text{average water content}}$$

NUCLEAR MOISTURE-AND-DENSITY TESTER

Use this method to determine the soil’s dry density and moisture content. Individual models of equipment vary in the specific procedures. Radiation protection programs vary as well from service to service. It is for this reason that the procedures are not discussed here. The procedures for this test method must be as prescribed by the individual equipment’s manufacturer’s manual, the requirements prescribed by the service holding the piece of equipment, and the Nuclear Regulatory Commission licensing agreement with that service.

The testers contain sources of radioactive material, typically cesium and a combination of americium mixed with beryllium powder. The cesium emits gamma radiation that the detector in the tester can count when it is passed through the soil. This count can be translated into density. The americium/beryllium emits neutrons following collisions with hydrogen which are moderated and detected by the tester. The moisture content can be determined by counting the hydrogen atoms in the soil.

WATER-DISPLACEMENT METHOD

Measure the volume of the hole from which a soil-density sample is taken by placing a rubber balloon in the hole and observing the volume of water required to fill the balloon. A water-balloon device is a watertight container with a float attached to a calibrated scale, graduated directly in cubic feet. A balloon is attached to the bottom of the device to make the test. Fill the cylinder in the device with water and place the apparatus over the area where the sample is to be removed. Allow the balloon to fill with water and take an initial reading. Remove the sample from the ground and replace the device over the hole in the original position. Allow the water to flow by gravity into the balloon in the hole. Blow through the hose attached to the device to increase the air pressure on the water surface and force the water-filled balloon in the hole to conform to all the contours of the hole. Observe the scale attached to the float for a reading of the water volume left in the device. Subtract this value from the original reading. The result is the volume of the hole, in cubic feet.

SECTION IX. CBR TESTS

The CBR test is a relatively simple test used to obtain an indication of the strength of a subgrade soil, subbase, and base-course material for use in road and airfield pavements. The test is used primarily to determine empirically required thicknesses of flexible pavements for highways and airfield pavements. The test was developed by the California Division of Highways in 1929 and adopted by the Corps of Engineers for use in the design of flexible pavements for airfields in locations where frost action is not the controlling factor.

NOTE: The current employed standard for CBR testing is ASTM D 1883-94.

The test procedure determines the CBR of pavement subgrade, subbase, and base-course materials from laboratory-compacted specimens that can be used in the design of a specific airfield. It consists of two steps—

- Preparing the soil test specimens.
- Performing the penetration test on the prepared soil samples.

Although one standardized procedure has been established for the penetration portion of the test, it is not possible to establish one procedure for the preparation of test specimens since soil conditions and construction methods vary widely. The soil test specimens are prepared to duplicate the soil conditions existing (or expected to occur later) in the field.

The method of preparing the test specimens and the number of specimens depend on the type of airfield, the soils encountered at the site, and other factors. Test the soil sample in the laboratory at a density comparable to the density required on the construction site. There are situations where moisture conditions are favorable and the subgrade will not accumulate moisture approaching a saturated condition. Test samples at a moisture content approximating the actual moisture conditions expected during the time the road or airfield is used. In all other conditions, samples are laboratory tested in a saturated condition.

Although penetration tests are most frequently performed on laboratory specimens, they may also be performed on undisturbed soil samples or in the field on the soil in place.

CBR OF LABORATORY-COMPACTED SOILS (ASTM D 1883-94)

The basic operations for conducting the CBR test are the same regardless of variations in soil conditions and types of construction. The test essentially measures the soil's shearing resistance under controlled moisture and density conditions. The CBR for soil is the ratio obtained by dividing the penetration stress required to cause a 3-inch two-area piston to penetrate 0.10 inch into the soil by a standard penetration stress of 1,000 pounds per square inch (psi). This standard penetration stress is roughly what is required to cause the same piston to penetrate 0.10 inch into a mass of crushed rock (limestone). The CBR value may be thought of as the strength of the soil relative to that of crushed rock.

Minor variations in the CBR test will cause wide variations in the results. For this reason, step-by-step procedures are detailed. Difficulties may still arise. Material with gravel or stones does not yield entirely satisfactory results. A number of tests must be conducted to establish a reasonable average value.

The CBR values range from as low as 3 to as high as 80, depending on the type of soils. The fine-grained soils vary from 3 for organic clays to 15 for micaceous or diatomaceous silts and sands. The sand-silt-clay coarse-grained combinations range from 10 for the clayey mixtures to 40 for the gravelly and silty sands. Gravelly soils range from 20 for the clayey group to 80 for the well-graded gravels and gravel-sand mixtures. *Table B-3, pages B-16 and B-17*, lists the typical range for soils classified under the USCS.

PURPOSE

Perform this test is to determine the CBR of a soil to ± 3 percent and determine the best moisture-content range to ± 4.

EQUIPMENT

Use the following items in a laboratory environment to perform this test (see *Figure 2-64, page 2-126*):

- A 225-pound soil sample with known classification, PI, and OMC.
- A CBR laboratory test set consisting of—
  - 15 CBR molds.
  - Surcharge weights.
  - A penetration piston.
  - Two dial gauges reading to 0.001 inch.
- Pails.
- Plastic bags.

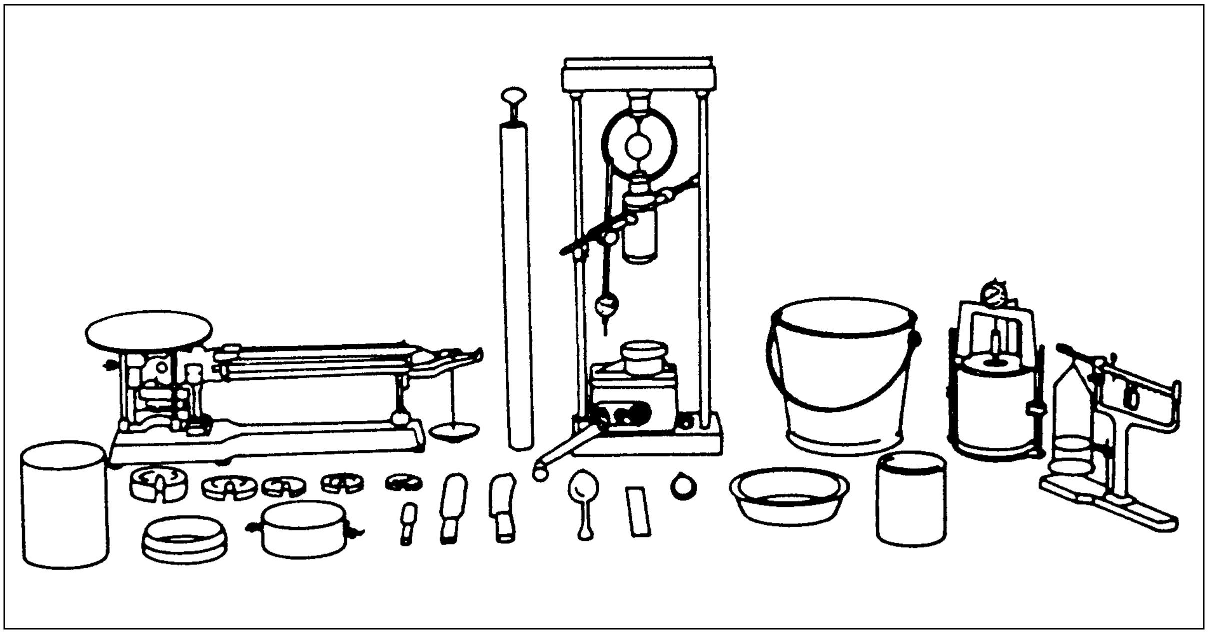


Figure 2-64. Laboratory CBR test-set apparatus

- Shipping tags.
- A compaction tamper.
- Filter paper.
- A recorder.
- Moisture-determination tares.
- Balance scales sensitive to 0.01 gram and 0.1 gram.
- A steel straightedge.
- A mixing pan.
- A spoon.
- A spatula.
- A graduated cylinder (100-milliliter).
- A laboratory oven.
- Heat-resistant gloves. Asbestos gloves should not be used for any materials-testing procedures. If your unit has asbestos gloves, turn them in through your supply system for proper disposal. Order heat-resistant gloves to replace them.
- A pencil.
- Paper.
- A grease pencil.
- A calculator.

- DD Forms 1210, 1211, and 1212.
- A 3/4-inch sieve.
- A No. 4 sieve.
- Pudding pans.
- A stopwatch.
- Plans and specifications.

STEPS

Perform the following steps for the CBR test:

Step 1. Determine the moisture-content range of investigation (± 4 percent of OMC). If the OMC has not yet been definitively established for this soil type, perform the laboratory compaction test as described in Section VIII to obtain the OMC.

Step 2. Prepare the soil sample. The size of the total sample will be about 225 pounds, which should provide enough material for the required 6,800 grams for each of the 15 molds.

a. Dry the soil sample until it can be easily crumbled under a trowel. Drying may be done by air-drying or by using a drying apparatus, provided the temperature of the sample does not exceed 60°C.

b. Break up the sample thoroughly, but not in such a manner as to reduce the size of the individual particles.

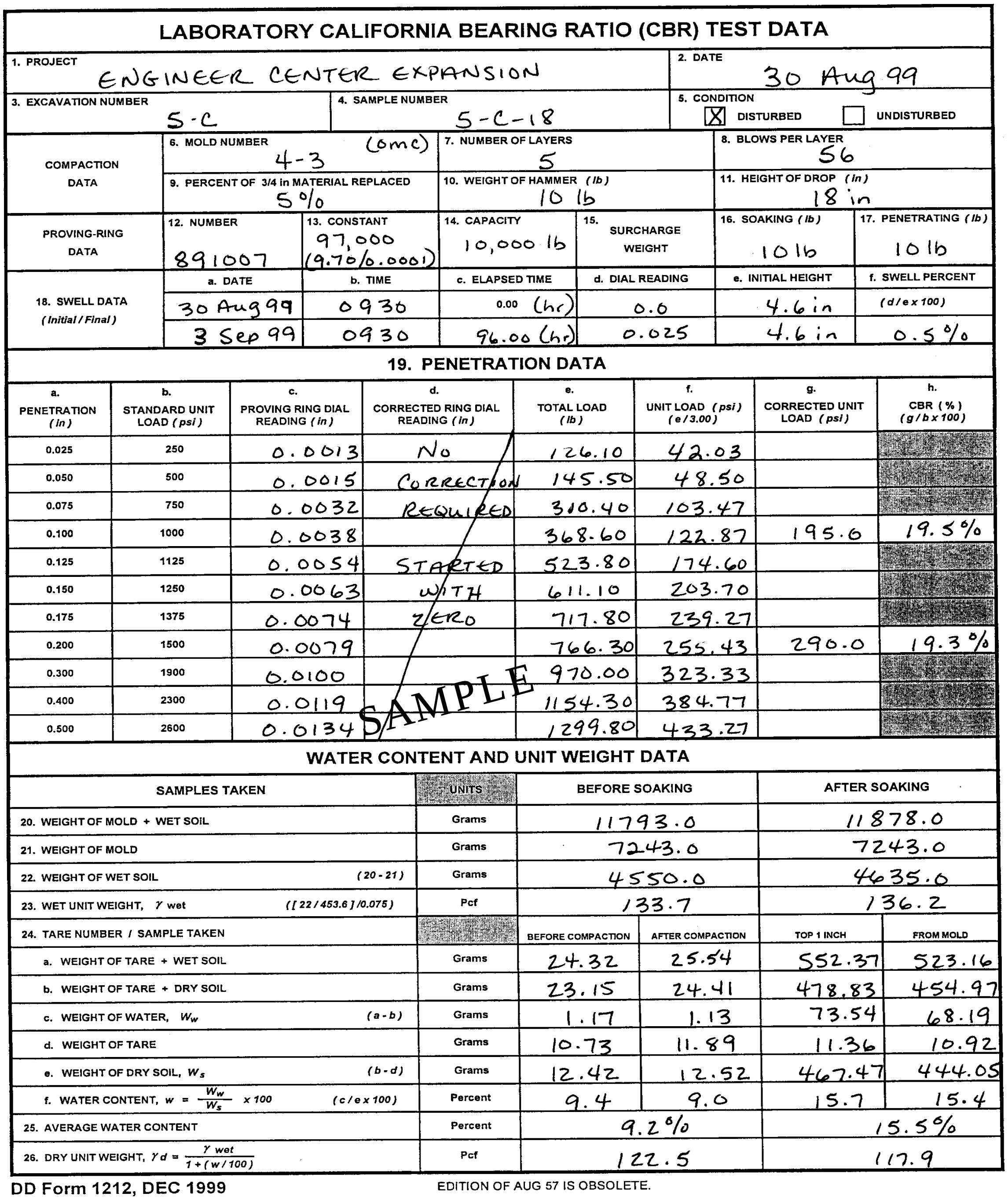
c. Sieve the sample over a 3/4-inch sieve. If all material passes the 3/4-inch sieve, use the entire gradation without modification. If there is material retained on the 3/4-inch sieve, remove it and replace it with an equal amount (by weight) of material passing the 3/4-inch sieve but retained on the No. 4 sieve by separation from portions of the sample not otherwise used for testing. (The CBR test is not conducted on any material retained on the 3/4-inch sieve.) This amount will be recorded later on DD Form 1212 (see *Figure 2-65, page 2-128 and 2-129*). .

d. Divide the entire sample into 15 smaller samples of about 6,800 grams each and place them into separate plastic bags. Seal the bags to maintain the current (or floor) moisture content.

e. Determine the moisture content from a 20-gram sample of the remaining material. Record this on a sheet of paper as the moisture content of the floor sample.

Step 3. Prepare and label a 6-inch compaction mold for each water content to be used (-4, -2, OMC, +2, and +4) at the compactive effort of 56 blows per layer as described in step 5 of Section VIII. In addition to the 5 molds required for the compactive effort of 56 blows per layer, prepare a mold for each water content using the other required compactive efforts of 10 and 25 blows per layer. Prepare a minimum of 15 molds. Prepare a DD Form 1212 for each mold and record the weight of the mold with the baseplate (to the nearest gram) on the form before continuing.

Step 4. Prepare the samples at the adjusted water contents.



LABORATORY CALIFORNIA BEARING RATIO (CBR) TEST DATA

1. PROJECT ENGINEER CENTER EXPANSION	2. DATE 30 Aug 99

3. EXCAVATION NUMBER 5-C	4. SAMPLE NUMBER 5-C-18	5. CONDITION ☒ DISTURBED ☐ UNDISTURBED

COMPACTION DATA	6. MOLD NUMBER 4-3 (omc)	7. NUMBER OF LAYERS 5	8. BLOWS PER LAYER 56
	9. PERCENT OF 3/4 in MATERIAL REPLACED 5%	10. WEIGHT OF HAMMER *(lb)* 10 lb	11. HEIGHT OF DROP *(in)* 18 in

PROVING-RING DATA	12. NUMBER 891007	13. CONSTANT 97,000 (9.70/0.0001)	14. CAPACITY 10,000 lb	15. SURCHARGE WEIGHT	16. SOAKING *(lb)* 10 lb	17. PENETRATING *(lb)* 10 lb

18. SWELL DATA *(Initial / Final)*	a. DATE	b. TIME	c. ELAPSED TIME	d. DIAL READING	e. INITIAL HEIGHT	f. SWELL PERCENT *(d / e x 100)*
Initial	30 Aug 99	0930	0.00 (hr)	0.0	4.6 in	
Final	3 Sep 99	0930	96.00 (hr)	0.025	4.6 in	0.5%

19. PENETRATION DATA

a. PENETRATION *(in)*	b. STANDARD UNIT LOAD *(psi)*	c. PROVING RING DIAL READING *(in)*	d. CORRECTED RING DIAL READING *(in)*	e. TOTAL LOAD *(lb)*	f. UNIT LOAD *(psi)* *(e / 3.00)*	g. CORRECTED UNIT LOAD *(psi)*	h. CBR (%) *(g / b x 100)*
0.025	250	0.0013	No	126.10	42.03		▨
0.050	500	0.0015	CORRECTION	145.50	48.50		▨
0.075	750	0.0032	REQUIRED	310.40	103.47		▨
0.100	1000	0.0038		368.60	122.87	195.6	19.5%
0.125	1125	0.0054	STARTED	523.80	174.60		▨
0.150	1250	0.0063	WITH	611.10	203.70		▨
0.175	1375	0.0074	ZERO	717.80	239.27		▨
0.200	1500	0.0079		766.30	255.43	290.0	19.3%
0.300	1900	0.0100		970.00	323.33		▨
0.400	2300	0.0119		1154.30	384.77		▨
0.500	2600	0.0134		1299.80	433.27		▨

WATER CONTENT AND UNIT WEIGHT DATA

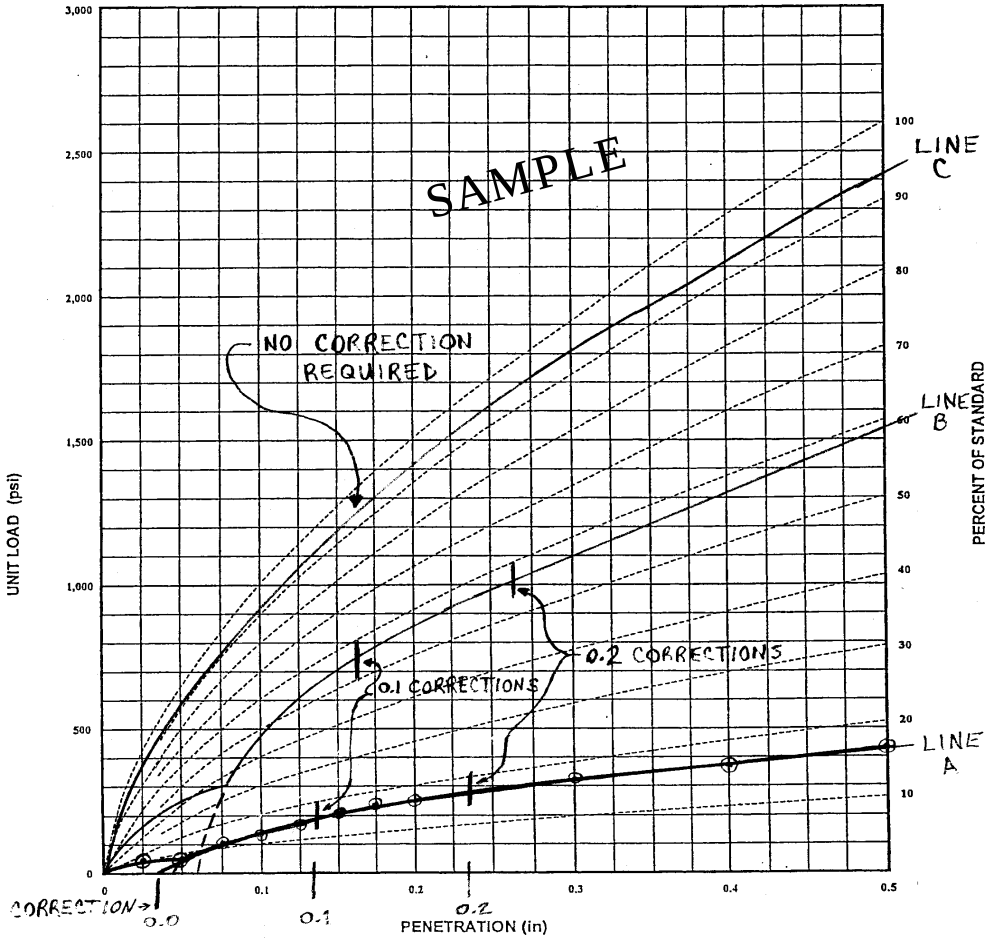
SAMPLES TAKEN	UNITS	BEFORE SOAKING		AFTER SOAKING	
20. WEIGHT OF MOLD + WET SOIL	Grams	11793.0		11878.0	
21. WEIGHT OF MOLD	Grams	7243.0		7243.0	
22. WEIGHT OF WET SOIL *(20 - 21)*	Grams	4550.0		4635.0	
23. WET UNIT WEIGHT, γ wet *([22 / 453.6] / 0.075)*	Pcf	133.7		136.2	
24. TARE NUMBER / SAMPLE TAKEN	▨	BEFORE COMPACTION	AFTER COMPACTION	TOP 1 INCH	FROM MOLD
a. WEIGHT OF TARE + WET SOIL	Grams	24.32	25.54	552.37	523.16
b. WEIGHT OF TARE + DRY SOIL	Grams	23.15	24.41	478.83	454.97
c. WEIGHT OF WATER, W_w *(a - b)*	Grams	1.17	1.13	73.54	68.19
d. WEIGHT OF TARE	Grams	10.73	11.89	11.36	10.92
e. WEIGHT OF DRY SOIL, W_s *(b - d)*	Grams	12.42	12.52	467.47	444.05
f. WATER CONTENT, $w = \frac{W_w}{W_s} \times 100$ *(c / e x 100)*	Percent	9.4	9.0	15.7	15.4
25. AVERAGE WATER CONTENT	Percent	9.2%		15.5%	
26. DRY UNIT WEIGHT, $\gamma d = \frac{\gamma\ wet}{1 + (w / 100)}$	Pcf	122.5		117.9	

DD Form 1212, DEC 1999 EDITION OF AUG 57 IS OBSOLETE.

Figure 2-65. Sample DD Form 1212

CBR TEST GRAPH

(Plot test curve below to obtain corrected unit load)



27. REMARKS

- CURVE A (ABOVE) IS PLOTTED FROM PAGE 1 OF DD FORM 1212, FIGURE 2-65.
- CURVE C DOES NOT REQUIRE A CORRECTION.
- CURVES A AND B (SHOWN ABOVE) ILLUSTRATE CURVES WITH CORRECTIONS MADE.

28. TECHNICIAN *(Signature)* PFC Bazonha	29. COMPUTED BY *(Signature)* PFC Bazonha	30. CHECKED BY *(Signature)* SSG Dobbs

DD Form 1212 (BACK), DEC 1999

Figure 2-65. Sample DD Form 1212 (continued)

a. Determine the amount of water to add to each sample using the following procedure:

$$\text{water to add (in milliliters)} =$$
$$\text{weight of sample (in grams)} \times (\text{desired percent} - \text{floor moisture content})$$

For example, to determine the amount of water to add to a sample to obtain the determined OMC of 8 percent,

$$6,800 \times (0.08 - 0.02) = 6,800 \times 0.06 = 408.0 \text{ milliliters}$$

where—

weight of sample = 6,800 grams

OMC = 8 percent (0.08)

floor moisture content = 2 percent (0.02)

To determine the water to add for the remaining samples for the required moisture-content range, perform the same calculation. The example below illustrates this calculation for the remaining samples, taking into consideration that not all of the sample weights will be exactly the same (6,800 grams):

$$\text{6.0\% moisture for a sample at 6,800 grams: } 6,800 \times (0.06 - 0.02) = 272.2 \text{ milliliters}$$

$$\text{4.0\% moisture for a sample at 6,815 grams: } 6,815 \times (0.04 - 0.02) = 136.3 \text{ milliliters}$$

$$\text{10.0\% moisture for a sample at 6,822 grams: } 6,822 \times (0.10 - 0.02) = 545.8 \text{ milliliters}$$

$$\text{12.0\% moisture for a sample at 6,810 grams: } 6,810 \times (0.12 - 0.02) = 681.0 \text{ milliliters}$$

b. Add the water figured from the formulas for each of the 5 desired moisture contents (-4, -2, OMC, +2, and +4), and mix thoroughly to ensure an even distribution of water throughout the sample.

c. Place each sample in an airtight container, and allow to stand for the minimum period of time indicated.

- For GW, GP, SW, and SP soil types, there is no minimum standing period of time.
- For GM and SM soil types, a minimum of 3 hours standing time is required.
- For all other soil types, a minimum of 16 hours standing time is required.

Step 5. Mark and weigh 2 moisture-determination tares for each mold prepared. Record each on DD Form 1212 as the weight of tare.

Step 6. Remove a small quantity of material (20 grams) from each sample and place it in one of the marked tares upon completion of the standing time. Weigh it in preparation for moisture determination and record the results on the form.

Step 7. Place sufficient soil in the mold (about 1 1/2 to 2 inches) to obtain a 1-inch compacted layer. After compaction of all 5 layers, each layer should be about equal in thickness. The fifth compacted layer will slightly extend into the collar but will not exceed 1/4-inch above the top of the mold.

Step 8. Apply compactive effort.

a. Hold the 10-pound compaction tamper within 5 degrees of vertical, placing its face on top of the soil.

b. Raise the handle until it reaches the top (18 inches) and release it, allowing the weight to fall freely onto the soil.

c. Change the position of the guide and tamper and repeat the process until the soil layer has received the prescribed number of blows for the compactive effort required. Apply the blows at a uniform rate of about 25 blows per minute. The height of fall of the tamper must be controlled carefully and the blows distributed evenly over the specimen's surface.

Step 9. Trim the compacted layer. After compacting each layer (except the fifth layer), use a knife to trim any soil adjacent to the mold walls that has not been compacted or that extends above the compacted surface. Include the trimmed soil with the additional soil for the next layer.

Step 10. Repeat steps 7, 8, and 9 until five layers have been compacted in the mold. Each compacted layer should be about equal in thickness (just under 1 inch). Adjust each layer accordingly to ensure that the fifth compacted layer will slightly extend into the collar, but will not exceed 1/4 inch above the top of the mold.

Step 11. Remove the collar from the mold.

a. Cut around the inside edge of the collar to prevent shearing the compacted soil when removing the collar.

b. Trim and smooth the compacted soil flush with the top of the mold. Use a sawing motion with the straightedge to trim the excess soil. Start at the center of the mold and work outward, first to one side and then to the other. Fill any holes with unused or trimmed soil from the specimen, press in with the fingers, and again scrape the surface with the straightedge.

Step 12. Take another small sample from the remaining material after compacting each mold and place it in the other moisture-determination tare and weigh it in preparation for moisture determination. Record the results on the form.

Step 13. Place a disk of coarse filter paper on top of the compacted specimen. Release the mold from the baseplate and while slightly lifting, slide the mold with the spacer disk off the baseplate and onto the edge of the table or countertop. Invert the baseplate and place it on top of the mold against the filter paper and reattach. Raise the mold and baseplate to allow the spacer disk to slide out. Invert the mold with baseplate and place it flat on the table or countertop.

Step 14. Weigh and record the data. Weigh the complete mold with the baseplate and compacted specimen (without the spacer disk) to the nearest gram. Record as the weight of the mold and wet soil (block 12). Do not include the collar.

Step 15. Repeat steps 7 through 14 for each of the compactive efforts required (10, 25, and 56 blows per layer). Compact a minimum of 5 molds for each compactive effort.

Step 16. Soak the samples and measure the swell (see *Figure 2-66*).

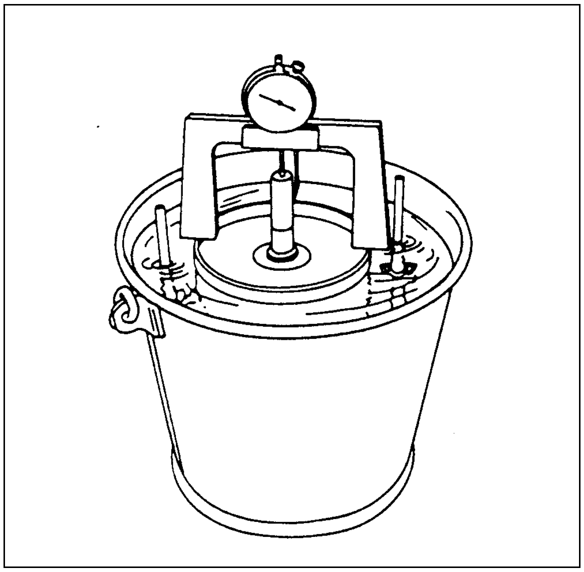


Figure 2-66. Soaking the CBR sample

a. Place surcharge weights on the perforated plate and adjustable stem assembly and carefully lower into the mold onto the filter paper and compacted soil specimen. Ensure that the surcharge applied is equal to the weight of the base material and pavement within 2.27 kilograms, but never use a total weight of less than 4.54 kilograms. If no pavement weight is specified, use 4.54 kilograms. Record the surcharge weight on the form.

b. Immerse the mold in water.

c. Calibrate the adjustable stem so that the tripodal dial reads 100 and can then travel in either direction. Obtain the initial dial reading and record it on the form.

d. Soak the sample for 96 hours.

e. Assemble the penetration apparatus (see *Figure 2-67*).

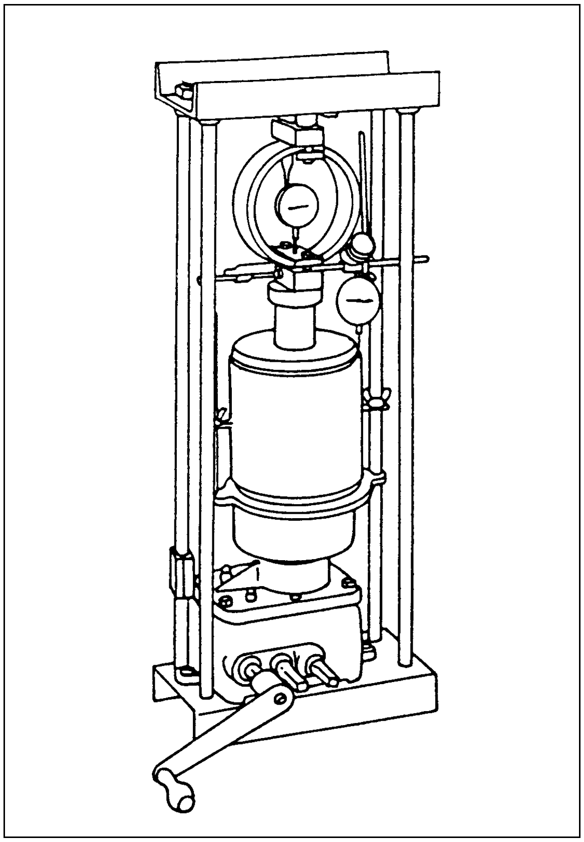


Figure 2-67. Assembled CBR-test penetration apparatus

(1) Attach the jack to the frame.

(2) Attach the proving ring and dial indicator to the frame. Record the proving-ring data on the form.

f. Take the dial reading at 96 hours and record on the the form.

g. Determine the amount of swell, in inches, by subtracting the initial dial reading from the final dial reading. Record the data on the form.

h. Determine the percent of swell and record it on the form using the following formula:

$$\frac{\text{swell in inches (at 96 hours)}}{\text{original sample height}} \times 100$$

Step 17. Drain the CBR mold.

a. Remove the immersed mold from the water and remove the free water. Allow the specimen to drain downward for 15 minutes. Do not disturb the specimen’s surface while removing the water. It may be necessary to tilt the specimen to remove surface water.

b. Remove the surcharge weights, the perforated plate, and the filter paper from the mold. Do not disturb the specimen's surface while removing these items.

c. Determine the mass and record it on the form as the weight of the soaked sample.

d. Return a 5-pound surcharge weight to the specimen.

Step 18. Perform the CBR penetration.

a. Prepare the components.

(1) Place the mold on the jack.

(2) Attach and adjust the piston to the jack, then zero the dial indicator.

(3) Lower the penetration piston until it is in contact with the sample with sufficient pressure to cause the load dial to register a load of 1 pound.

(4) Replace the remainder of the surcharge weights required for the mold.

(5) Attach the adjustable arm with the dial indicator to the jack assembly, adjusting the position until the dial-indicator plunger is resting on the mold's projecting rim.

(6) Turn both dial indicators to 0.

b. Apply the load.

(1) Crank the jack to lower the piston at a rate of 0.05 inch per minute.

(2) Read the proving-ring dial indicator when the piston has reached penetration depths of 0.025, 0.050, 0.075, 0.100, 0.125, 0.150, 0.175, 0.200, 0.300, 0.400, and 0.500 inch. Take the first eight readings at 30-second intervals and the remaining three at 2-minute intervals.

(3) Record the proving-ring dial readings on the form.

c. Determine the average moisture of the soaked samples.

(1) Remove the top 1 inch of soil from the mold. For fine-grained soils, place at least 100 grams of soil in a moisture-determination tare. For granular soils, place at least 500 grams of soil in a moisture determination tare.

(2) Weigh the tare and record on the form.

(3) Remove an additional sample from the remaining contents of the mold for moisture determination. For fine-grained soils, place at least 100 grams of soil in a moisture-determination tare. For granular soils, place at least 500 grams of soil in a moisture-determination tare.

(4) Weigh the tare and record on the form.

(5) Perform the moisture-content determination of the tares. Record the results and average on the form.

Step 19. Solve the computations for each reading, and record the results on the form.

a. Determine the total load, in pounds, by multiplying the proving-ring dial reading (block 19c) by the proving-ring constant (block 13). Enter this number in block 19e of the form. The corrected ring dial readings need not be determined as long as the dial indicators have been zeroed before penetration. If the dials were not adjusted to 0 before penetration, determine the corrected ring dial readings and enter them in block 19d. Calculate the total load from the corrected reading instead of the observed reading.

b. Determine the unit load (in psi) by dividing the total load by 3. Enter this number in block 19f of the form. The value of 3 is determined by the area of the penetrating piston in square inches.

c. Determine the corrected unit load (in psi) by plotting (on the reverse side of the form) the unit load (in psi) against the depth of penetration (in inches). If the curve has an initial concave upward shape between 0 and 0.1, then the zero point must be adjusted. This occasionally happens with some soil types under certain conditions and it is necessary to obtain true penetration loads. Adjust the zero point of the curve as indicated in the example in *Figure 2-65, pages 2-128 and 2-129*. Once the zero-point correction has been made, the 0.100- and 0.200-inch points are moved to the right on the curve the same distance as the zero point. Obtain corrected unit-load values from the corrected graphs at 0.100- and 0.200-inch penetrations and enter in block 19g of the form. If no corrections were made, the numbers entered into block 19g will be the same as block 19f.

d. Calculate the CBR (in percent) for penetration at 0.100 and 0.200 inches using the following formula:

$$\frac{\textit{corrected unit load}}{\textit{standard unit load}} \times 100$$

Since the standard unit load for each penetration is given (block 19b of the form), perform the following computations for each penetration:

$$\textit{CBR for 0.100} = \frac{\textit{corrected unit load}}{1,000} \times 100$$

$$\textit{CBR for 0.200} = \frac{\textit{corrected unit load}}{1,500} \times 100$$

NOTE: The CBR value of the mold is computed at 0.100- and 0.200-inch penetrations. The bearing ratio normally reported is that of the 0.100-inch penetration. When the ratio at 0.200-inch penetration is greater, the test must be verified by another test. If the test is verified with similar results, use the bearing ratio at the 0.200-inch penetration.

Step 20. Complete DD Form 1212. Ensure that any other information concerning the soil sample is indicated in the remarks block.

UNDISTURBED SAMPLE TESTING

Tests on undisturbed samples are used when the base design calls for uncompacted soil, such as highly compressible clay that loses strength upon remolding, or when correlating field in-place tests to the design-moisture condition. For this latter condition, duplicate samples should be tested to determine the correction necessary for the in-place tests. The reduction that occurs from four days of soaking is applied as a correction to the field in-place test.

Care and patience are necessary to maintain the relatively undisturbed samples in this condition. If proper lateral support is not given on the sides of the samples, erroneous CBR values will result. In fine-grained materials, molds or metal jackets are satisfactory. With samples cut or trimmed from a pedestal, use a mixture of 10 percent resin and 90 percent paraffin to fill the annular space and offer support. For gravelly soils, the box method is desirable. Use wax paper or paraffin to cover the sample and prevent moisture loss while transporting it to the laboratory.

Perform soaking and penetration tests after removing the paper or paraffin from the end of the specimen and after leveling the surface (use a thin layer of sand, if necessary).

IN-PLACE FIELD CBR TESTING

To overcome some of the shortfalls associated with older in-place field CBR test methods, the Corps of Engineers Waterway Experimentation Station has developed the dual-mass dynamic cone penetrometer (DCP) (see *Figure 2-68*) for evaluating the load-carrying capability of military roads and airfields. The results from using the DCP are reported in terms of index values which can be converted to CBR values. Three correlations currently exist for this conversion, each dependent on the soil type being tested.

The procedures for testing with the DCP and the correlation of CBR values can be found in the user's manual for the equipment or in FM 5-430-00-2, Annex J.

PRESENTATION AND ANALYSIS OF CBR DATA

The CBR value, molding water content, and dry density for each specimen can be presented in several ways that facilitate analysis. The individual test programs used to present this data are relative to the type of soil encountered and are discussed in the following paragraphs.

TEST PROGRAM FOR NONSWELLING SOILS

The test program for nonswelling soils applies to the majority of soils used in construction. As *Table 2-14* indicates, soils that fall into this group might be used as compacted subgrade, select, or subbase materials depending on their strengths and location regarding the construction site. The compaction requirements can then be determined as listed in *Table 2-15, page 2-138*.

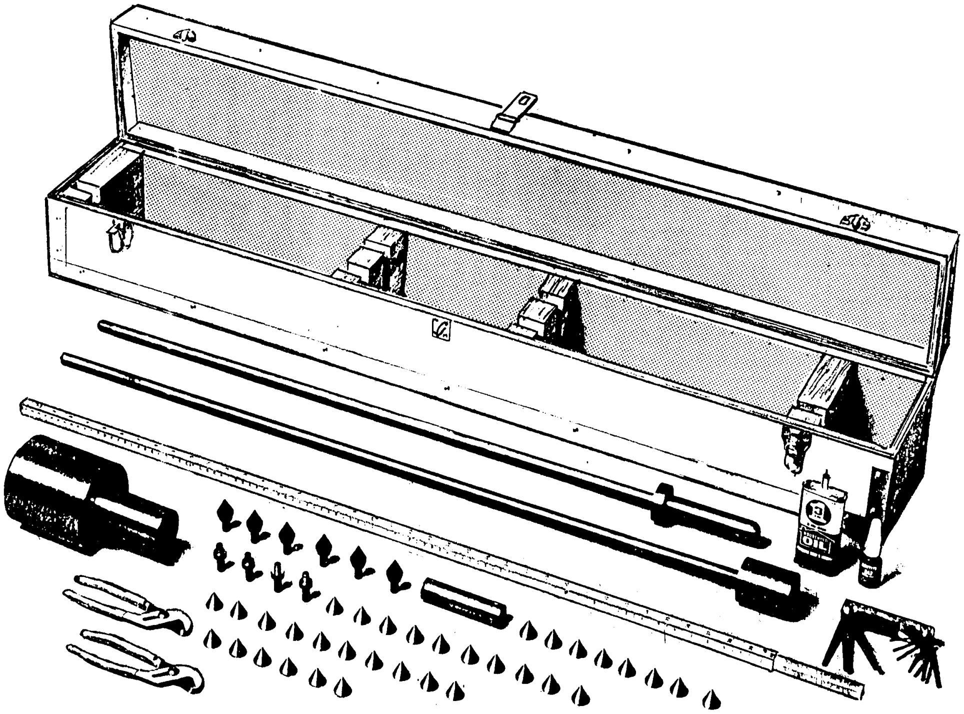


Figure 2-68. DCP test kit

Table 2-14. Summary of remolded CBR laboratory test programs

Test Program	Type of Soil Normally Tested*	Compaction Blows Per Layer	Probable Use of Test Results
Swelling soils	CH, MH, and OH	10, 25, and 56	Low-quality compacted subgrades
Free-draining soils	GW, GP, SW, and SP	25, 56, and 72	Compacted subgrade, select, and subbase materials
Other soils	All except CH, MH, OH, GW, GP, SW, and SP	For CBR >20: 25, 56, and 72 For CBR <20: 10, 25, and 56	Compacted subgrade, select, and subbase materials
***This categorization is intended to serve as a guide for planning laboratory activities. Deviations may be noted in the initial stages of a test program which will dictate adjustments.**			

Table 2-15. Summary of compaction requirements

Material	**Percentage of Compaction of Materials With Design CBR Values of 20 and Above**
Base course	No less than 100% of CE 56 maximum density
Subbase and subgrade	No less than 100% of CE 56 maximum density
Material	**Percentage of Compaction of Materials With Design CBR Values Below 20**
Select material and subgrade in fills	Cohesionless fill will not be placed at less than 95% of CE 56 maximum density. Cohesive fill will not be placed at less than 90% of CE 56 maximum density

Example

To illustrate the methods of evaluating the design CBR, the data given on the DD Form 2463 for the Engineer Center expansion road (see *Figure 2-69, pages 2-139 through 2-143*) will be used. The data was taken from the subgrade along a proposed road alignment. The object of the following analysis is to determine a soil-placement moisture-content range for a specified level of compactive effort which gives the greatest assured design CBR. This technique for determining a design CBR provides for a strength measure of at least 15.0 when the associated density and moisture-content ranges are followed. Greater strengths will be realized within the specified limits, but the value obtained allows the engineer to size the structure for the worst condition. Notice that for this soil and the limits used, the greatest assured strength occurs for the 4 percent moisture-content range centered on the OMC. This may not always be the case. Also note that the analysis is based on an initial selection of density limits. You may find it better to evaluate other density limits that meet the minimum requirements to see if an adjustment to these limits yields greater strengths.

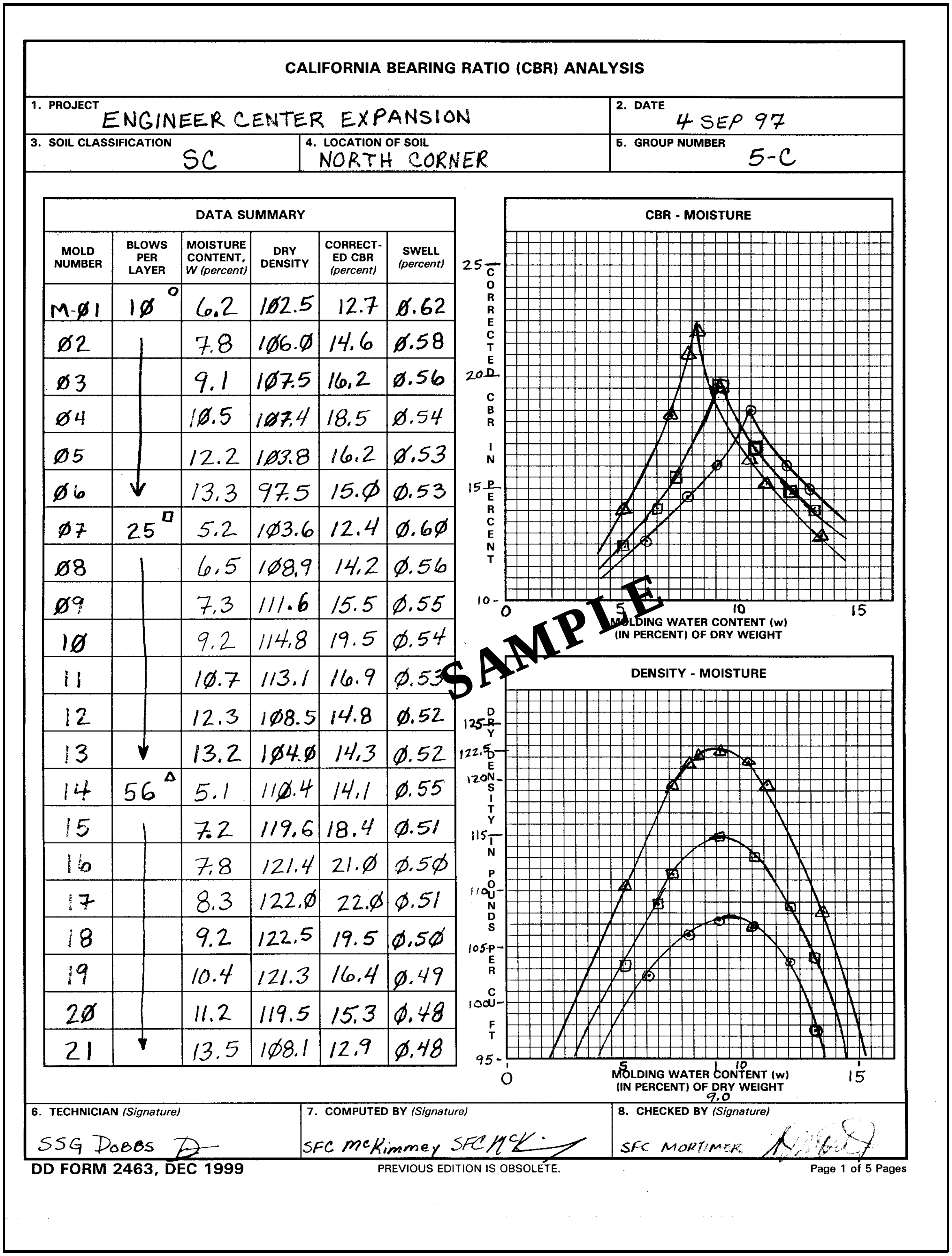
Steps

Perform the following steps for CBR testing of a nonswelling soil:

Step 1. Establish the OMC of the soil at 56 blows per layer by using the data collected from the compaction test as outlined in Section VII (see *Figure 2-58, page 2-110*). For this example, OMC equals 8.8 percent.

Step 2. Establish a moisture range for CBR investigation. The moisture range generally used for nonswelling soils is OMC ± 4 percent. This is a time-saving guide, as experience shows that the maximum CBR normally occurs at compaction moisture contents within this range and that testing soils beyond these limits is wasted effort. For this example, the moisture-content investigation range is 5 to 13 percent.

Step 3. Compact the samples within the moisture-content investigation range at different levels of compactive effort as described earlier in this section. This allows for evaluation of soil strength when field placement is other than 100 percent of the compaction test's maximum density.



CALIFORNIA BEARING RATIO (CBR) ANALYSIS

1. PROJECT ENGINEER CENTER EXPANSION		**2. DATE** 4 SEP 97
3. SOIL CLASSIFICATION SC	**4. LOCATION OF SOIL** NORTH CORNER	**5. GROUP NUMBER** 5-C

DATA SUMMARY

MOLD NUMBER	BLOWS PER LAYER	MOISTURE CONTENT, *W (percent)*	DRY DENSITY	CORRECTED CBR *(percent)*	SWELL *(percent)*
M-01	10 ○	6.2	102.5	12.7	0.62
02	↓	7.8	106.0	14.6	0.58
03		9.1	107.5	16.2	0.56
04		10.5	107.4	18.5	0.54
05		12.2	103.8	16.2	0.53
06		13.3	97.5	15.0	0.53
07	25 □	5.2	103.6	12.4	0.60
08	↓	6.5	108.9	14.2	0.56
09		7.3	111.6	15.5	0.55
10		9.2	114.8	19.5	0.54
11		10.7	113.1	16.9	0.53
12		12.3	108.5	14.8	0.52
13		13.2	104.0	14.3	0.52
14	56 △	5.1	110.4	14.1	0.55
15	↓	7.2	119.6	18.4	0.51
16		7.8	121.4	21.0	0.50
17		8.3	122.0	22.0	0.51
18		9.2	122.5	19.5	0.50
19		10.4	121.3	16.4	0.49
20		11.2	119.5	15.3	0.48
21		13.5	108.1	12.9	0.48

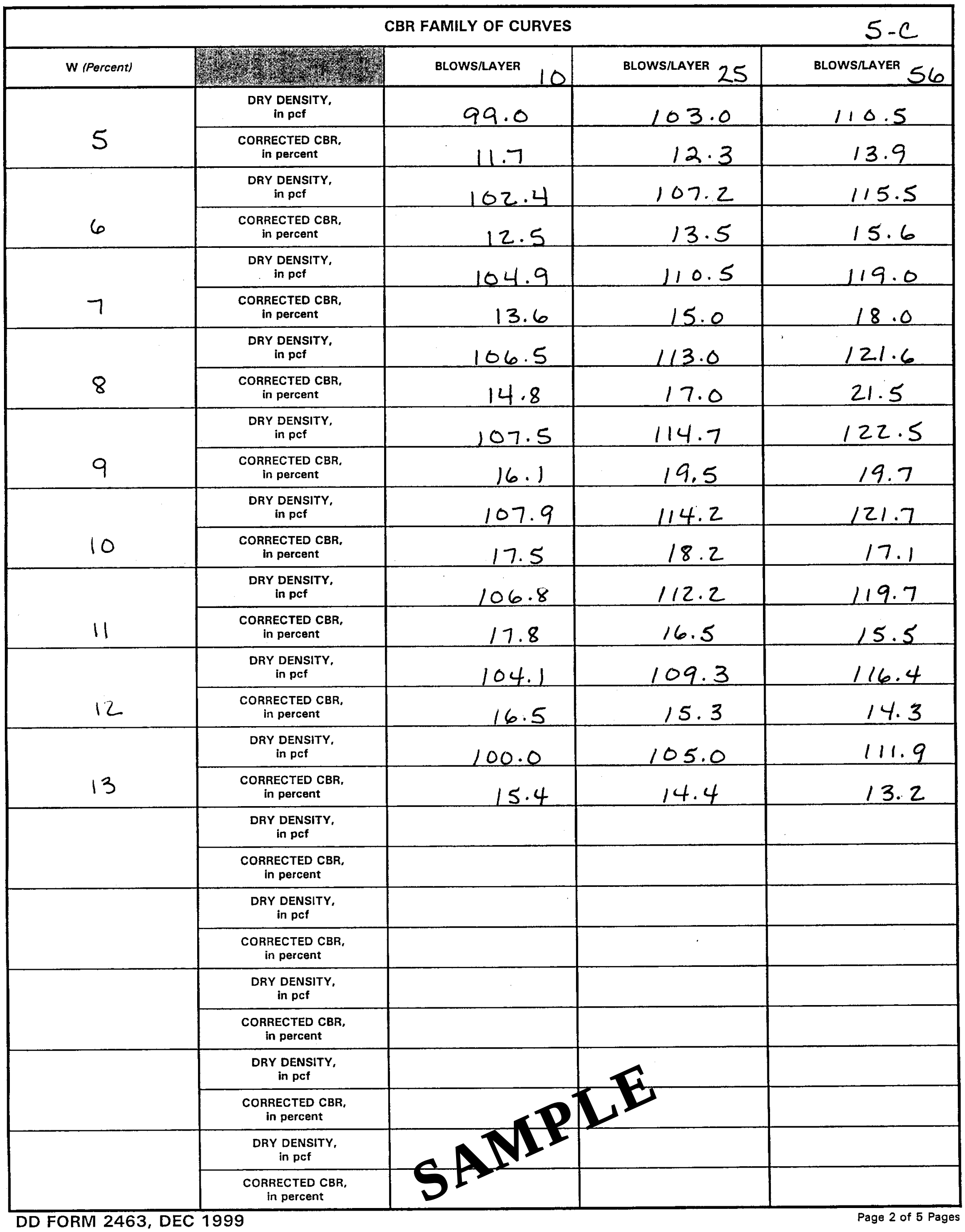
CBR - MOISTURE

DENSITY - MOISTURE

6. TECHNICIAN *(Signature)* SSG DOBBS	**7. COMPUTED BY** *(Signature)* SFC McKimmey SFC McK	**8. CHECKED BY** *(Signature)* SFC MORTIMER

DD FORM 2463, DEC 1999 PREVIOUS EDITION IS OBSOLETE. Page 1 of 5 Pages

Figure 2-69. Sample DD Form 2463, page 1



CBR FAMILY OF CURVES 5-C

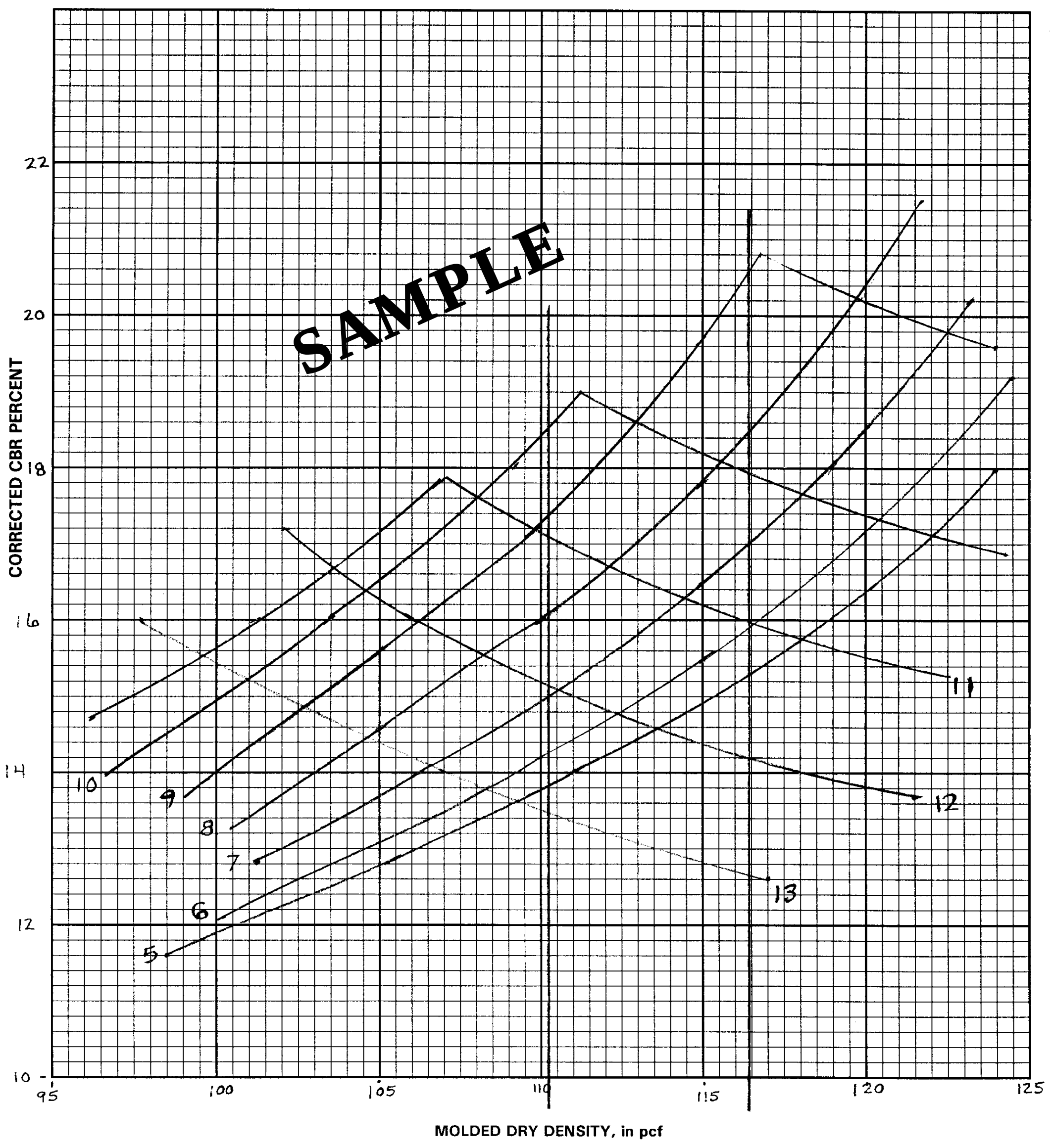
W (Percent)		BLOWS/LAYER 10	BLOWS/LAYER 25	BLOWS/LAYER 56
5	DRY DENSITY, in pcf	99.0	103.0	110.5
	CORRECTED CBR, in percent	11.7	12.3	13.9
6	DRY DENSITY, in pcf	102.4	107.2	115.5
	CORRECTED CBR, in percent	12.5	13.5	15.6
7	DRY DENSITY, in pcf	104.9	110.5	119.0
	CORRECTED CBR, in percent	13.6	15.0	18.0
8	DRY DENSITY, in pcf	106.5	113.0	121.6
	CORRECTED CBR, in percent	14.8	17.0	21.5
9	DRY DENSITY, in pcf	107.5	114.7	122.5
	CORRECTED CBR, in percent	16.1	19,5	19.7
10	DRY DENSITY, in pcf	107.9	114.2	121.7
	CORRECTED CBR, in percent	17.5	18.2	17.1
11	DRY DENSITY, in pcf	106.8	112.2	119.7
	CORRECTED CBR, in percent	17.8	16.5	15.5
12	DRY DENSITY, in pcf	104.1	109.3	116.4
	CORRECTED CBR, in percent	16.5	15.3	14.3
13	DRY DENSITY, in pcf	100.0	105.0	111.9
	CORRECTED CBR, in percent	15.4	14.4	13.2
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			

DD FORM 2463, DEC 1999

Page 2 of 5 Pages

Figure 2-69. Sample DD Form 2463, page 2 (continued)

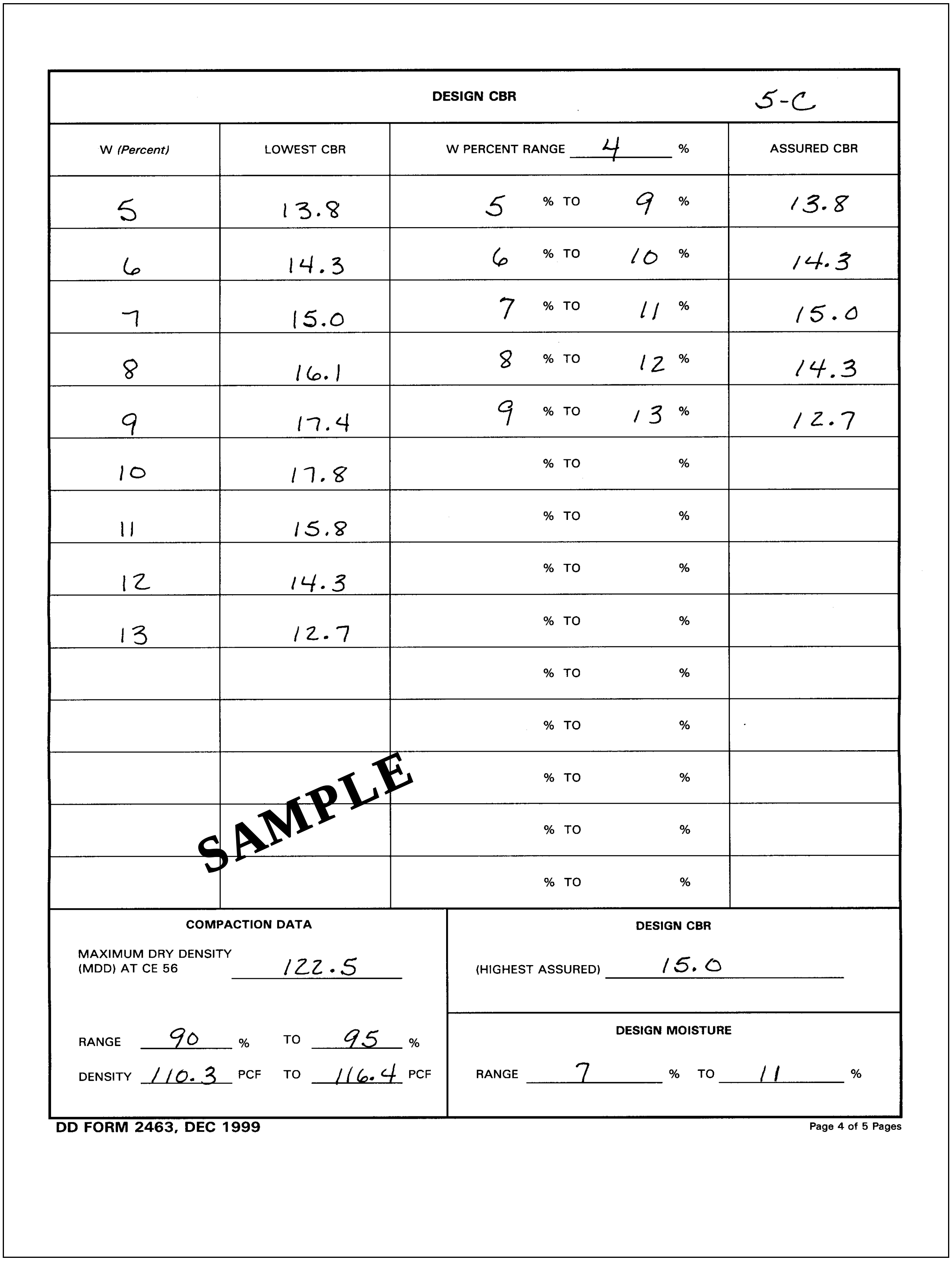
CBR FAMILY OF CURVES



DD FORM 2463, DEC 1999

Page 3 of 5 Pages

Figure 2-69. Sample DD Form 2463, page 3 (continued)



DESIGN CBR 5-C

W *(Percent)*	LOWEST CBR	W PERCENT RANGE 4 %	ASSURED CBR
5	13.8	5 % TO 9 %	13.8
6	14.3	6 % TO 10 %	14.3
7	15.0	7 % TO 11 %	15.0
8	16.1	8 % TO 12 %	14.3
9	17.4	9 % TO 13 %	12.7
10	17.8	% TO %	
11	15.8	% TO %	
12	14.3	% TO %	
13	12.7	% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	

COMPACTION DATA

MAXIMUM DRY DENSITY (MDD) AT CE 56 122.5

RANGE 90 % TO 95 %

DENSITY 110.3 PCF TO 116.4 PCF

DESIGN CBR

(HIGHEST ASSURED) 15.0

DESIGN MOISTURE

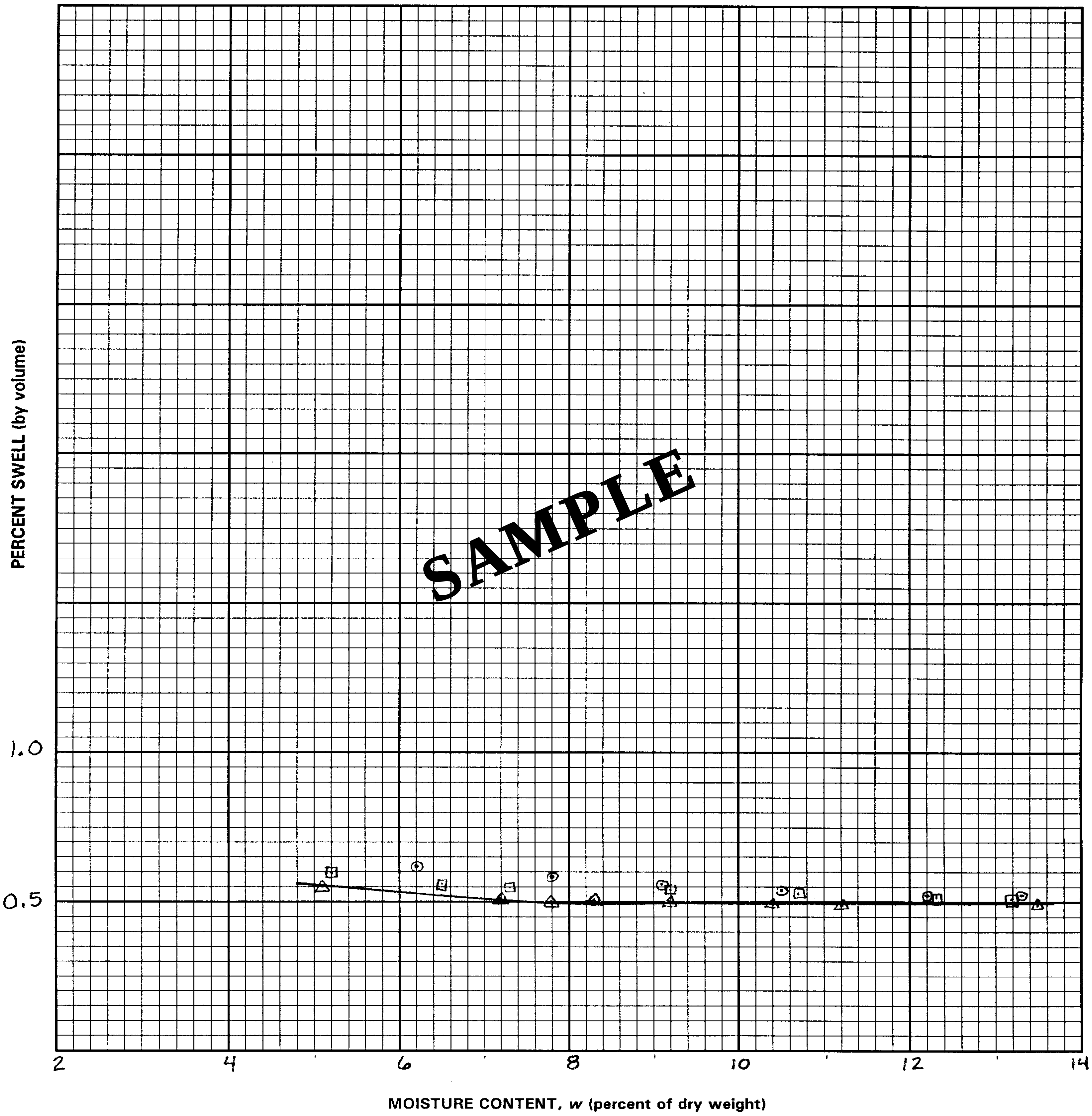
RANGE 7 % TO 11 %

DD FORM 2463, DEC 1999 Page 4 of 5 Pages

Figure 2-69. Sample DD Form 2463, page 4 (continued)

CBR SWELL DATA

5-C



NOTE: Maximum acceptable swell is 3 percent.

○ - 10 blows/layer
□ - 25 blows/layer
△ - 56 blows/layer

DD FORM 2463, DEC 1999

Page 5 of 5 Pages

Figure 2-69. Sample DD Form 2463, page 5 (continued)

The levels of compactive effort selected in the laboratory are based on the compaction requirements. If the soil is to be used as select material or subgrade in fills (CBR < 20), it may be placed at less than 100 percent of the compaction test's maximum density for CE 56, as indicated in *Table 2-15, page 2-138*. Additionally, laboratory compactive efforts of 10, 25, and 56 blows per layer are usually selected for this soil. If the soil is a very high-quality subgrade or a subbase (CBR ≥ 20), the laboratory tests should include samples compacted in excess of 56 blows per layer. Normally 25, 56, and 72 blows per layer are adequate. The only stipulations placed on the levels of compactive effort are that a 56-blow-per-layer compaction curve be obtained and that data be developed at two other levels of compactive effort encompassing the specified placement densities.

In this example, the soil type is SC and the DD Form 1211 from the compaction test displays a bell-shaped compaction curve. This is indicative of a either a swelling or a nonswelling soil. A U-shaped compaction curve indicates a free-draining soil. An assumption is made then that this is a nonswelling soil. This information is based on typical soil characteristics for soils of type SC as found in *Table B-3, pages B-16 and B-17*, and *Table 2-14, page 2-137*. Using *Table 2-14* and column 15 of *Table B-3* as a guide, the samples were compacted at 10, 25, and 56 blows per layer. The DD Form 1212 for this sample compacted at 56 blows per layer at approximate OMC can be found in *Figure 2-65, pages 2-128 and 2-129*.

Step 4. Soak the samples and measure the swell as outlined in the previous section.

Step 5. Perform CBR penetration tests and determine the corrected CBR for each sample according to the technique discussed in the previous section. Note that accumulating the required data involves a considerable amount of work. At a minimum, 15 molds (5 per level of compactive effort) must be made. In this example, 21 molds were compacted, soaked, and then penetrated.

Step 6. Transfer the results of the 21 tests from each DD Form 1212 onto the data summary section of DD Form 2463, page 1 (see *Figure 2-69, page 2-139*).

Step 7. Plot the data on the graphs of dry density versus molding moisture content and corrected CBR versus molding moisture content on DD Form 2463, page 1 (see *Figure 2-69*).

Step 8. Determine the moisture range for the CBR family of curves. For a nonswelling soil, the moisture content that corresponds to the MDD is the OMC. The moisture range is OMC ± 4 percent. In this example, the OMC discovered from DD Form 2463 is 9 percent; therefore, the range for the family of curves is 5 to 13 percent.

Step 9. Transcribe the data points from the graphs onto page 2 of the DD Form 2463 (see *Figure 2-69, page 2-140*) for each whole moisture percentage in the range determined from step 8. This step is performed to identify values of dry density and CBR for whole-integer moisture contents within the range of investigation at each level of compactive effort. Some of the data may be transferred directly from the data-summary table. The remainder must be interpolated from the CBR/dry-density versus molding moisture-content graphs.

Step 10. Plot the CBR family of curves on page 3 of DD Form 2463 (see *Figure 2-69, page 2-141*). This step places the laboratory data in a form that lends itself to analysis. The trends of strength variation are determined as the moisture content and dry density change. Drawing a CBR family of curves involves considerable practice, numerous attempts, and subsequent adjustments. The example (*Figure 2-69*) shows that the available data has been plotted on a graph of corrected CBR versus molded dry density for whole moisture contents. For low molding water contents (from 5 to 8 percent), there was an increase in strength with dry density. At high moisture contents (12 and 13 percent) the reverse was true. For the intermediate moisture contents, there was an increase in strength to some point and then a decrease as the dry density increased.

Step 11. Proceed with an engineering analysis. After all the testing has been completed, it is the engineer's responsibility to ensure that the CBR data is properly obtained and presented. The engineer must analyze the data and understand how the results affect the design and economic factors. Before considering the details of the analysis, it is possible to observe two points of interest about the soil being used for the example. First, the maximum strength of this soil does not occur at OMC but at 1 percent drier (see corrected CBR versus molding water content). Second, the CBR family of curves shows that for some moisture contents the soil loses strength as the dry density or level of compactive effort increases. The impact of these two factors is important. If maximum strength from this soil is desired, that strength may in some soils be achieved at a moisture content less than OMC and at some level of compactive effort less than CE 56. The reasons for these two phenomena are highly speculative. Soil placement at OMC and the most compactive effort possible are not always the answers to good construction.

Step 12. Establish a density range at which soil will be placed in the field. The TO standard compaction range is 5 percent, unless otherwise stated. The flow chart in *Figure 2-70, page 2-146,* provides the information necessary for determining the density and moisture ranges for nonswelling, swelling, and free-draining soils. For nonswelling soils, the following information is normally used to assist in determining a density range.

To facilitate construction, it is common to specify a reasonable range of densities that can be economically obtained and then examine the strength values that would occur without that range. Establishing the density range depends greatly on economics. The more latitude given to the builder, the better the chances of placing the soil within established limits. However, if an extreme range is stipulated, the CBR value allowed for design might be reduced and thicker pavement structure could be required. Another factor is the actual field compaction experience obtained from either a test strip or prior construction. Such data can be accumulated by measuring the in-place dry densities for different numbers of passes with the available compaction equipment and determining the point where additional passes give little increase in density. For this example, the soil type was determined to be SC with a PI of 10, as shown on DD Form 1209 (see *Figure 2-47, page 2-95*). With this information following the flow chart, the soil must be placed to at least 90 percent of MDD.

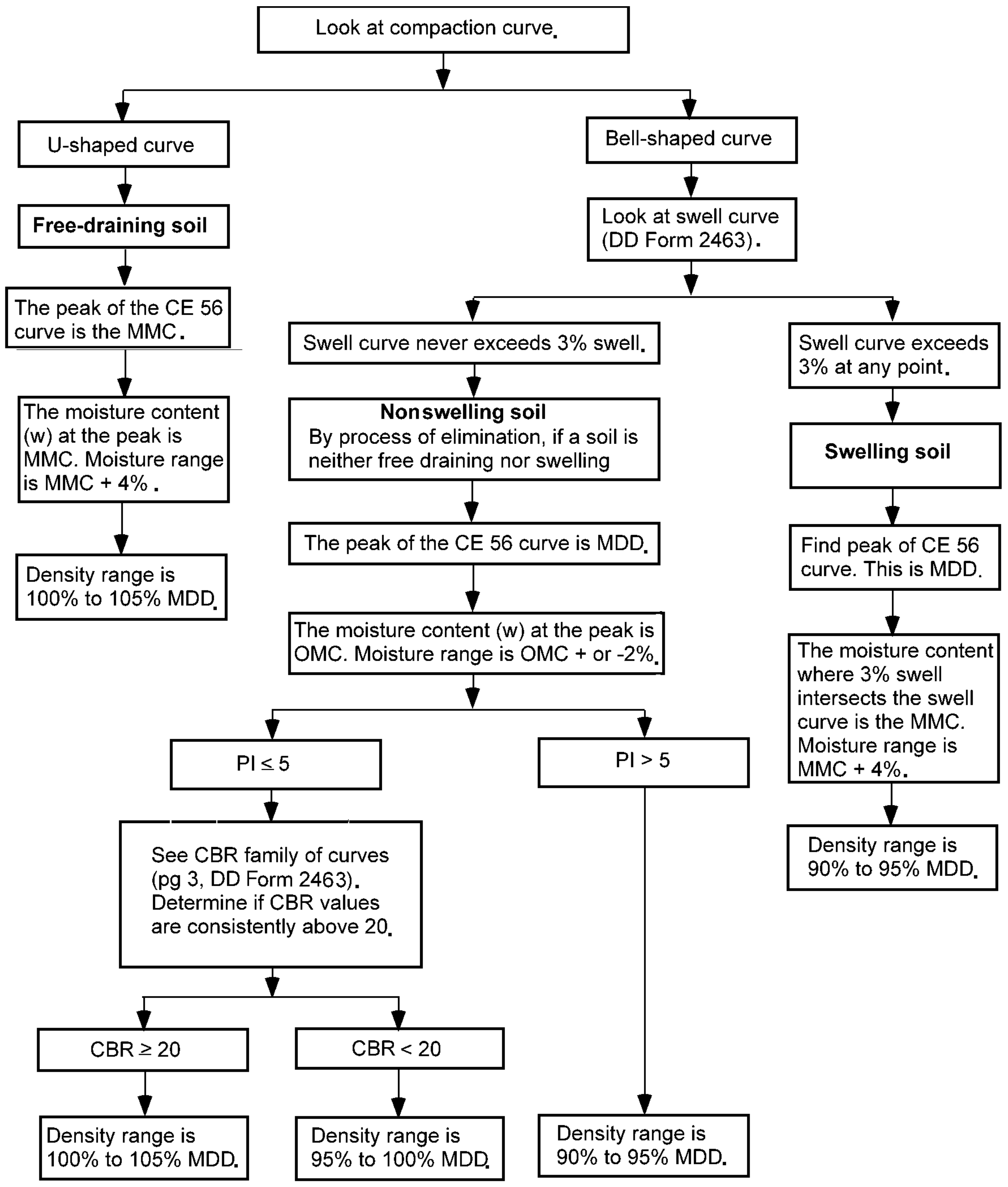


Figure 2-70. Density and moisture requirements using the CBR design method

This would provide a density range, in pcf, from 110.3 to 116.4. This is calculated in the following manner:

$$0.90 \times 122.5 = 110.3\ pcf$$

$$0.90 \times 122.5 = 116.4\ pcf$$

Using the range between 90 and 95 percent, these limits can be imposed on the CBR family of curves by drawing two vertical lines, one at 110.3 and the other at 116.4 pcf.

Step 13. Determine the assured CBR values between the specified density limits. If the builder is allowed to place the soil between 110.3 and 116.4 pcf, this step involves determining the CBR values obtained for each moisture content. The change in CBR for any specific moisture-content line between the two density limits shows that a range of strengths is possible. Since the builder will be allowed to place the soil anywhere between the established density limits, the CBR value selected as a potential design strength should represent the worst case. Using 8 percent moisture as an example, the CBR at 110.3 pcf is 15.6. At 116.4 pcf, the CBR is a maximum of 18.0. Of the two, 15.6 is the minimum strength for the specified density range. This procedure was followed for the remaining moisture contents in the example, and the results are recorded on page 4 of DD Form 2463 (see *Figure 2-69, page 2-142*).

Step 14. Determine the CBR values for potential moisture-content specification ranges. Like the density range, a moisture-content range that can be economically achieved in the field is desired. Within the overall range of investigation (OMC ± 4 percent), a smaller specification range giving the greatest assured CBR will be determined. Experience shows that a 4 percent range (± 2 percent) is a reasonable requirement; however, this span is not intended to represent an absolute rule. A smaller range may be specified to achieve a larger design CBR and a reduction in pavement thickness requirements. This savings in pavement materials may be offset by increased costs associated with the difficulties in meeting the more stringent requirements. Conversely, for some soils an expanded moisture-content specification may have little effect on the design CBR. The sample problem uses a 4 percent specification range. One possible range is 6 to 10 percent. If the engineer specified that the soil be placed within these limits, the worst possible strength would be a CBR of 14.4. A continued analysis can be done for the other possible 4 percent ranges, as shown on page 4 of DD Form 2463 (see *Figure 2-69*).

Step 15. Select the moisture-content range that gives the greatest assured CBR. In steps 13 and 14, CBR values were selected assuming that the builder will be allowed to place the soil anywhere between potential moisture content and density specification limits. Now the desired set of limits is selected. The tabulation in step 14 shows that the soil, if placed between 7 and 11 percent moisture, will give the largest of the possible CBR values. Thus, 15.0 becomes the design CBR. To ensure that a CBR of 15.0 is achieved, compaction must be 90 to 95 percent MDD or 110.3 to 116.4 pcf.

TEST PROGRAM FOR SWELLING OR EXPANSIVE SOILS

There is a small group of soils which expand objectionably after being compacted and saturated. This presents a problem in pavement design as this expansion or swell can damage the structure through reverse settlement. The measure of swell is expressed as a percentage of the initial sample height. Objectionable swell is defined as that in excess of 3 percent of the initial sample height. A soil's expansive nature is mainly due to the type of clay minerals present. For example, montmorillonitic clay consists of the smallest and most highly charged particles found in nature. The combination of large surface-area-per-unit volume and high surface charge causes a tremendous affinity for water and the ability to expand or shrink as water is taken in or removed from the soil.

Experience shows that the PI is an excellent indicator of expansive soils. Although a high PI does not guarantee that the soil is expansive, critical soils should be checked more closely for swell tendencies. See *Table 2-16* for suggested guidelines. *Table 2-14, page 2-137*, indicates that the potentially expansive soils—by USCS classification—are CH, MH, and OH. The test procedure to determine a design CBR for an expansive soil is similar to that discussed for nonswelling soils, but the objective is different. For nonswelling soils, the object is to find the greatest assured CBR value for some range of densities and moisture contents. The object of the test program for expansive soils is to find the moisture-content ranges that will prevent objectionable swell and provide the highest-soaked CBR. Generally, the minimum swell and the highest-soaked CBR occur at a molding moisture content higher than the OMC.

Table 2-16. Swell potential

PI	Expansiveness
0 to 14	Not expansive
14 to 25	Marginal
25 to 40	Critical
40	Highly critical

Example

The following is an example of such a test for a CH soil. As most of the steps are similar to those developed for the case of nonswelling soils, only differences will be discussed.

Steps

Step 1. Establish the soil's OMC. Determine a moisture range for CBR investigation. The OMC ± 4 percent range of investigations as used for the nonswelling soils may not apply. It will be necessary to prepare samples over a wider range of moistures with most of the work being done on samples higher than the OMC. Prepare samples over a range of the OMC ± 8 percent. As illustrated later, much of the laboratory work done on samples dry of the OMC is not essential.

Step 2. Compact the samples within the moisture-content range of investigation at different levels of compactive effort. As the type of soil being tested will generally be cohesive and will have a CBR less than 20, *Table 2-15, page 2-138,* indicates that compaction must be at least 90 percent MDD. An upper limit can be established as expansive soils are very difficult to compact at levels greater than 100 percent of MDD. Laboratory compactive efforts of 10, 25, and 56 blows per layer are adequate for nearly all cases.

Step 3. Soak the samples and measure the swell. This step deviates from the same step for the nonswelling soil. For each sample, measure the expansion, compute the percent of swell, and plot against the molding water content. As an example, the triangular data point at 10 percent moisture on the swell data curve (see *Figure 2-69, page 2-139*) was obtained as follows:

- Initial sample height = 4.60 inches.
- Final dial reading = 0.025 inches.
- Molding water content = 10.0 percent.
- Level of compactive effort = blows per layer.
- Free-swell index or percentage of swell = (0.025/4.60) x 100 = 0.5 percent.

Step 4. Plot the points on page 5 of the DD Form 2463 (see *Figure 2-69, page 2-143*).

NOTE: The example of page 5, DD Form 2463 provided in *Figure 2-69* is based on data from the nonswelling soil type as explained in the nonswelling program section. A plot of data for a swelling soil may look slightly different than this example, as indicated in *Figure 2-71*.

Once the points are plotted, a curve is then usually fit to only the CE 56 data points. The curve for a swelling soil (see *Figure 2-71, page 2-150*) shows that this soil, if placed and compacted at molding moisture contents of 14 percent or greater, will swell 3 percent or less.

Step 5. Perform CBR penetration tests and determine the corrected CBR for each sample.

Step 6. Plot the data on graphs of dry density versus molding moisture content and corrected CBR versus molding moisture content.

Step 7. Reformat the data on DD Form 2463, page 2 (*Figure 2-69, page 2-140*).

Step 8. Plot the CBR family of curves (DD Form 2463, page 3 (*Figure 2-69, page 2-141).*

Step 9. Establish a density range at which soils will be placed in the field. If no prior experience or benefit of a test strip is available, then the flow chart in *Figure 2-70, page 2-146,* may be used. In this case, the minimum level of compaction is 90 percent of MDD. Assuming a reasonable specification range of 5 percent, the upper limit will be set at 95 percent MDD.

The actual density range is calculated as discussed in the previous example for nonswelling soils. These calculated limits are then placed on page 3 of DD Form 2463 by drawing two vertical lines on the CBR family of curves.

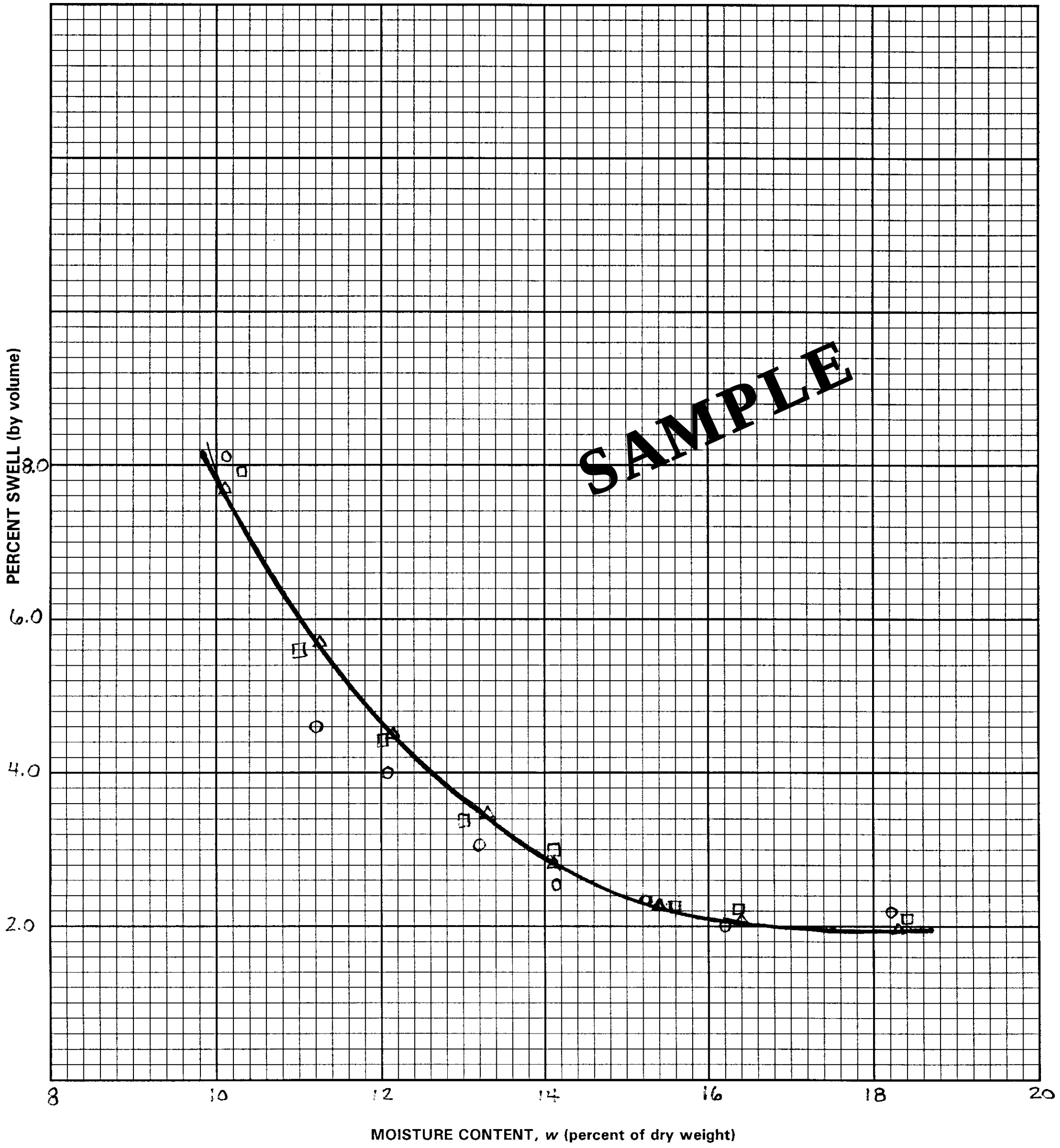
For example, if the MDD was determined to be 110, then the following calculation would be performed to establish the actual density limits:

$$110 \times 0.90 = 99 \text{ pcf}$$

$$110 \times 0.95 = 104.5 \text{ pcf}$$

CBR SWELL DATA

3-A
BORROW PIT A



NOTE: Maximum acceptable swell is 3 percent.

o - 10 blows/layer
□ - 25 blows/layer
△ - 56 blows/layer

DD FORM 2463, DEC 1999

Page 5 of 5 Pages

Figure 2-71. Sample DD Form 2463, page 5

Step 10. Determine the assured CBR values between the specified density limits. As more than 3 percent swell is not acceptable, evaluation of the CBR values at moisture contents less than 14 percent is needless. The CBR values of the applicable moisture contents should range from 14 through 20. The CBR values of the applicable moisture contents were determined to be the amounts shown in *Table 2-17.*

Table 2-17. Determining CBR values for moisture-content percentages for swelling soils

w	CBR
14	3.4
15	4.2
16	3.2
17	2.2
18	1.3
19	0.9
20	0.4

Step 11. Determine the CBR values for potential moisture-content specification ranges. For this example the values shown in *Table 2-18* were derived.

Step 12. Select the moisture-content range that gives the greatest design CBR. Step 11 shows that the 14 to 18 percent range provides the greatest CBR value. Thus, the design CBR value is 1.3. This value is obtained when the soil is placed at a moisture content between 14 and 18 percent and a density of between 99.0 and 104.5 pcf.

Table 2-18. Determining CBR values for potential moisture-content ranges for swelling soils

w (range, in percent)	CBR
14 to 18	1.3
15 to 19	0.9
16 to 20	0.4

Step 13. Analyze the results. Note that this design value was obtained at the expense of strength. This technique does not provide for drying the soil to a moisture content less than the amount of placement. Should such extreme drying take place, excessive shrinkage and pavement failure might be expected. However, it takes considerable effort to remove water from expansive soils and such soils are normally protected from drying by the overlying pavement. The first thing to consider when encountering an expansive soil is testing another location. However, this is not always feasible, and this technique does not allow for determination of a design CBR at which swell is not excessive.

Expansive soils can be chemically stabilized to allow building. Adding small amounts of lime considerably reduces the potential for shrinkage and swell. Soils stabilization is further discussed in Chapter 5 of this manual as well as TM 5-822-14 and FMs 5-430-00-1 and 5-430-00-2.

TEST PROGRAM FOR FREE-DRAINING SOILS

Determining a design CBR for free-draining soils requires the least testing of the three remolded laboratory test programs. *Table 2-14, page 2-137*, gives the USCS classification and the uses of the soils in this group.

The ease in testing is due to the free-draining characteristics or lack of fines in the soil. The CBR analysis sheet for borrow pit A shows that the density-versus-moisture-content curves have a concave, upward shape and show maximum densities between 7 and 9 percent moisture, depending on the level of compactive effort (see *Figure 2-72*). For each curve, there is a limiting or minimum moisture content (MMC) at which moisture above that required to fill the voids after compaction is squeezed or drained from the soil (shown by the triangle, square, or circle in *Figure 2-72*). The dashed lines to the right of the MMC represent attempts to compact the soil in a saturated condition, but the results after compaction are densities and moisture contents at the limiting condition. This means that field placement is relatively easy for such soils. To ensure the MDD for any level of compactive effort, the only control measure necessary is to have more water available than that required for the MDD at the appropriate level of compactive effort. The corrected CBR-versus-molding-water-content curves show the same pattern in relation to moisture content as the dry density. Soils placed with a moisture content above the minimum moisture content achieve the maximum CBR possible for that level of compactive effort. In other words, moisture contents of loose soils above the limiting values have little bearing on the strength of a soil after compaction. This makes laboratory testing, field placement, and field control relatively easy matters.

Example

To arrive at a design CBR, the steps outlined in the previous examples will again be followed.

Steps

Step 1. Establish the soil's MMC at 56 blows per layer. It is obtained by locating the moisture content at which the MDD is achieved on the compaction curve. For this example, the MMC is 8 percent with a MDD of 120 pcf. To ensure that a free-draining soil is being tested, this curve should display a MDD at a limiting or minimum moisture content.

Step 2. Compact the samples at different levels of compactive effort. Compaction curves must be made for three levels of compactive effort up to the MMC. As free-draining soils are frequently represented by well- and poorly graded sands and gravels with CBR values above 20, *Table 2-15, page 2-138*, indicates a compactive effort in excess of 100 percent of maximum CE 56 dry density. Therefore, 25, 56, and 72 blows per layer are usually used.

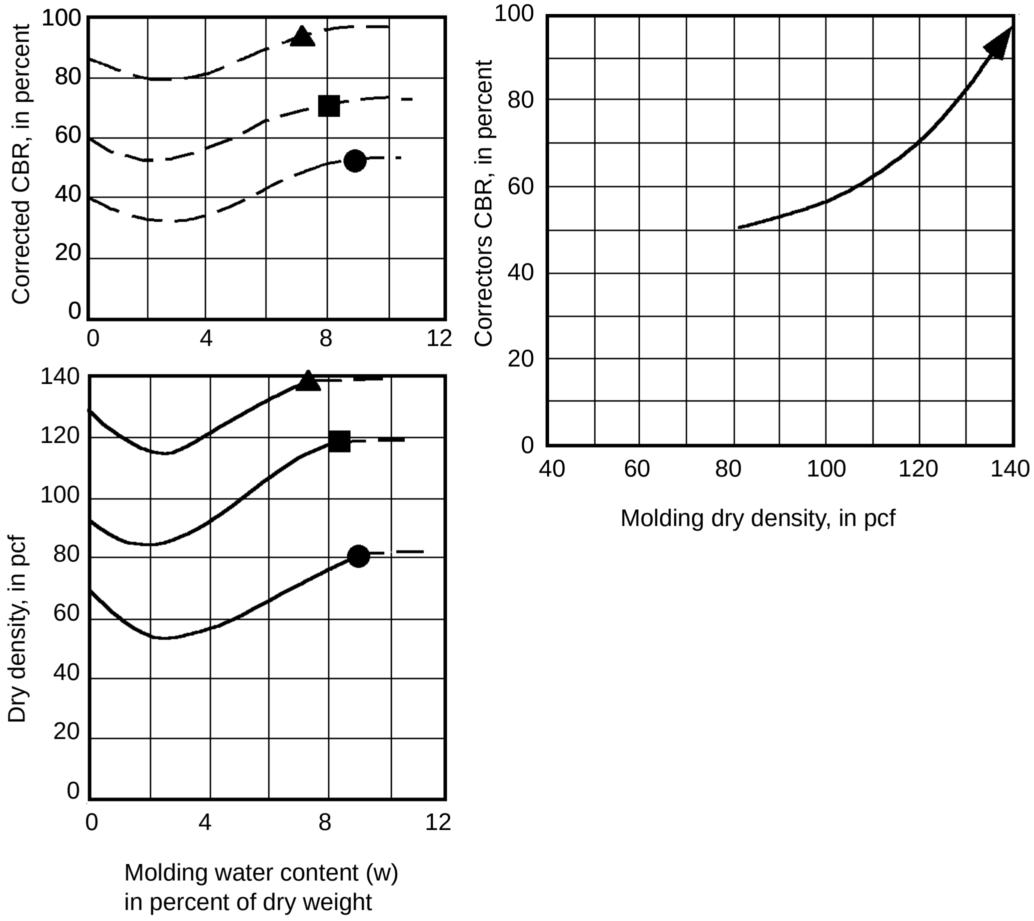


Figure 2-72. Plotted results for a free-draining soil, borrow pit "A"

Step 3. Soak the samples and measure the swell. Swell measurements are not required, and soaking can be eliminated when it is determined that saturation does not affect the strength.

Step 4. Perform CBR penetration tests. Only the samples at the limiting moisture contents for each level of compactive effort need to be tested. Normally, more than one sample at the limiting conditions will be made for each level of compactive effort, and all should be tested.

Step 5. Plot the data on graphs of dry density versus molding moisture content and corrected CBR versus molding moisture content. Only the plot of dry density versus molding moisture content is required. The corrected CBR-versus-molding-moisture-content graph is presented only for discussion.

Step 6. Plot the CBR family of curves. This graph can be condensed into a single line. The three data points are obtained by plotting the corrected CBR against the associated dry density at the limiting moisture content.

Step 7. Establish a density range at which the soil will be placed in the field. Using the criteria from *Table 2-15, page 2-138,* and noting that the CBR value for this soil is usually always greater than 20, the minimum level of compaction allowed is 100 percent of maximum CE 56 dry density. Because no additional information is provided, specify 100 to 105 percent of maximum CE 56 dry density. The actual density range is as follows:

$$1.00 \times 120.0 = 120.0 \text{ pcf}$$

$$1.05 \times 120.0 = 126.0 \text{ pcf}$$

Step 8. Determine the design CBR and placement moisture content. The CBR family of curves shows that the minimum CBR value achieved between the density range is 70, which is achieved at 120 pcf. The placement moisture content necessary to ensure that this strength is obtained is 8 percent or greater.

Considerably greater CBR values can be achieved if more field compaction is applied to the soil. If this is not too costly, it may be advantageous to specify greater densities.

SECTION X. TECHNICAL SOILS REPORT

A good program for soils testing not only requires that careful and complete tests be performed, but also that the tests be completed as quickly as possible and that the data be presented in a clear, logical consistent manner. Therefore, you must be familiar with the tests, the sequence of testing, and the presentation of results.

SOILS TESTS REQUIRED

The tests required by any program depend on the type of construction being planned. However, there are a number of tests run consistently on road and airfield construction programs. A complete testing program should include the following tests:

- Soils exploration (see Sections I and II).
- Compaction (see Section VII).
- Plasticity index (see Section VI).
- Particle-size analysis (see Section V).
- CBR (see Section IX).
- Trafficability (see FMs 5-430-00-1 and 5-430-00-2).

PURPOSE OF THE REPORT

The soil tests listed above are specific tests used to gain knowledge about the control of soils during construction, including the—

- Suitability of subgrade and borrow materials.
- Degree to which soil can be compacted.

- Bearing value of the subgrade and borrow material under projected future conditions.
- Location of ledge rock and groundwater table.
- Susceptibility to detrimental frost action.

Each test supplies the necessary data to answer questions based on engineering evaluation of scientific data rather than a meaningless guess.

ORGANIZATION AND SCOPE OF THE TESTS

Because of the number of tests to be performed for a particular project, careful planning may avert considerable delays in the presentation of the results. List the tests required and their sequence in a manner that will permit running the tests continuously, without delays due to time needed for soaking or drying samples.

An example of the daily activities in a complete soil-testing program are listed in *Figure 2-73*. Some testing programs may not include all the tests listed in the example and soil types or test results may change the activity list. An activity schedule for each unit may be different due to equipment and mission priority.

The following paragraphs cite some considerations that may be helpful in setting up a continuous soil testing program.

Sample Soils Laboratory Testing Schedule

Test/activity	Day 1	Day 2	Day 3	Day 4	Day 5	Day 6	Day 7	Day 8
Grain-size analysis (sieve method)	━	━	━	━	━			
Specific gravity	━	━	━					
Plastic and liquid limits	━	━	━					
Compaction	━	━	━					
Laboratory CBR				━	━	━	━	━
Grain-size analysis (hydrometer method)						━	━	━

Figure 2-73. Sample schedule for soils laboratory testing

Exploration

Before any tests can be performed, representative samples of the soil involved in a given project must be obtained. This, however, is only one objective of soil exploration. Other considerations include plotting a profile of boring results, locating ledge rock, determining the depth to the groundwater table, determining field moisture content, and field-identifying the soils sampled.

Particle/Grain-Size Analysis

The mechanical analysis is another test that should be used to occupy periods between other tests. The test is an evaluation of the grain-size distribution used to establish the gradation of the soil sample. The sieve analysis and hydrometer analysis may be used in obtaining the required information.

LL and PL Tests

One use of LL and PL test results is to predict how the fine-grained portion (No. 200 fraction) will affect the engineering value of a particular soil sample. This evaluation, obtained through soil classification, is to ensure that the sample being tested meets specifications set on the LL and PL for the particular project for which it will be used. These tests should be performed while waiting for compaction-test moisture-content samples to dry and CBR samples to soak or any time between other tests.

Compaction Test

The compaction test indicates the MDD that can be obtained practically in the field. It also indicates the OMC at which this dry density can be obtained. Since this will be a test used for control purposes, it should be run as soon as possible. While moisture-content samples from this test are drying, tests for plasticity and particle-size analysis can be started.

Laboratory CBR Test

The CBR test is determined by an arbitrary penetration procedure to obtain a modulus of shearing resistance of a subgrade or base-course soil. This value is used to determine the required thicknesses of the various base courses through its application to empirically derived design curves. Because the procedure for this test may involve a four-day soaking requirement, CBR samples should be prepared as soon as the necessary information from the results of the compaction test is available. This information may be obtained from approximated values of the OMC.

Field In-Place CBR Test

The latest addition to equipment used for determining the CBR value of a soil is the dual-mass DCP. The procedures for testing with the DCP and the correlation of CBR values can be found in the user’s manual for the equipment or in Annex J of FM 5-430-00-2.

SOILS TECHNICAL REPORT

In writing technical reports, one of the most important aids is a clear, logical outline of the subject. Outlines will vary according to the program conducted and the data required, but the suggested format that follows should help organize a report.

Frequently, portions of the information shown in the outline will be required at different times. For this reason, a preliminary report and several supplementary reports may actually be made before the project is completed. However, if all of the information provided follows the same basic outline, filing the data and assembling the final report will be simplified.

PURPOSE

Use the following information to write a soils technical report that includes all pertinent data given in the conditions and follow the recommended outline.

EQUIPMENT

Use the following items and information to write a soils technical report:

- Soil boring logs.
- The soil's profile.
- The soil's field identification.
- In-place moisture data.
- Particle/grain-size analysis data.
- LL and PL data.
- The USCS laboratory soils classification.
- CBR test data.
- In-place density data (as required locally).
- In-place CBR data.
- A topographic map of the site.
- Aerial photographs (if available).
- Pencils.
- Pens.
- Paper.
- The project directive.
- FM 5-430-00-1.

OUTLINE

Use the following recommended outline to organize a technical soils report:

1. Project.

   A. Description. What type of construction is being performed (for example, Class D road, base camp, and airstrip)?

   B. Purpose and scope. Who is requesting construction? What is the extent and why is it being performed?

2. Site description.

   A. Location. All map references and directions including county or province (if available).

B. Existing facilities. This could include many things such as old logging roads, dirt paths, patrol roads, buildings, overhead power lines, and so forth (see sketches).

C. Topography, cultivation, and drainage. Description of terrain features. (forest, farmland, well drained, swampy, hills, and so forth).

D. Climate. Temperature extremes, seasonal precipitation, average wind speed, and so forth.

3. Geology.

A. Overburden. Main depositing force (river, glacial, and so forth), rock classes (sedimentary, igneous, metamorphic), and depth of overburden.

B. Bedrock. Average depth to bedrock and state of bedrock, such as faulted, fractured, or folded.

4. Site conditions.

A. Field explorations. Details of performed tests, location of test sites, and explanation of sample tags.

B. Field tests. Type of field tests performed and field sample results (enclose copies in Annex G).

C. Laboratory tests. Type of laboratory tests performed. Do not include MDD or laboratory CBR results.

D. Test results. Synopsis of test results and classifications. Refer to forms in Annex F.

5. Fill and borrow materials.

A. Field explorations. Details of performed tests, location of test sites, and explanation of sample tags.

B. Field tests. Type of field tests performed and field sample results (enclose copies in Annex G).

C. Laboratory tests. Type of laboratory tests performed. Do not include MDD or laboratory CBR results.

D. Test results. Synopsis of test results and classifications. Refer to forms in Annex F.

6. Conclusions and recommendations.

A. Final site selection (if applicable). Exact location of finished project. Refer to plans enclosed in Annex C.

B. Economical design. Low-cost or resource alternatives to current suggestions (soil-cement, asphalt binder, chemical stabilization methods, mats or fabrics, and so forth).

C. Minimum specifications. Design CBR, MDD, and OMC specifications. Refer to laboratory forms in Annex F.

7. Annexes.

A. Project directive and all directives involving this project.

B. General plan drawings (geology should be indicated).

C. Location plan drawings (existing and proposed features).

D. Profiles and cross sections.

E. Boring logs.

F. Laboratory testing data.

G. Field testing data.

NOTE: Not every subject will apply to every report. In many reports, some of the items may be covered in several sentences, while others may require a page of discussion. Some items (for example 5, 6, and 7) may have to be repeated for each runway in a major airfield. All laboratory test forms will be included in Annex C. If a specific test form is not necessary or not conducted, write the words "Not Applicable" across the form.

Chapter 3

Bituminous Mixtures

This chapter provides information on the materials used in constructing bituminous surfaces, the methods of testing these materials, and the mixes prepared from them. The design considerations—such as bearing capacity and thickness of pavements—are described in FM 5-430-00-1. Mixing and placing operations, as well as the considerations for preparing the mixes, are described in TM 5-337.

SECTION I. BITUMINOUS PAVEMENTS/SURFACES

Bituminous pavements/surfaces are a mixture of mineral aggregates, mineral filler, and a bituminous material or binder. This mixture is used as the top portion of a flexible pavement structure to provide a resilient, waterproof, load-distributing medium that protects the base course from the detrimental effects of water and the abrasive action of traffic.

AGGREGATES

Mineral aggregates may consist of crushed rock, crushed or uncrushed soils (including gravels and sands), slag, mineral filler, or a combination of some of these materials. Other materials that may be used as aggregate in certain geographical areas include vesicular lava and coral. Aggregates normally constitute 90 percent or more by weight of bituminous mixtures, and their properties greatly affect the finished product. The aggregate provides three basic functions when used in bituminous surfaces:

- It transmits the load from the surface down to the base course. In pavement, this is accomplished through the mechanical interlock of the aggregate particles.
- It withstands the traffic's abrasive action. If a wearing surface were laid consisting of binder alone, it soon would be worn away by the abrasive action of tires.
- It provides a nonskid surface. A portion of the aggregate extends slightly above the normal surface of the wearing mat, thereby providing a roughened surface for tires to grip.

BITUMINOUS MATERIALS

A bituminous material is the adhesive agent or binder in a bituminous mixture. This material or binder provides two functions:

- It binds the aggregate together, holds it in place, and prevents displacement.

- It provides a waterproof cover for the base and keeps surface water from seeping into and weakening the base material.

The binder's functions require it to be a waterproof substance having the ability to bind aggregate particles together. All bituminous materials possess these qualities due to being mainly composed of bitumen—a black solid that provides the black color, cementing ability, and waterproofing properties. Bituminous materials are classified into two main groups—asphalts and tars. They are available in several forms suitable for different procedures of mixing or application under wide variations in temperature. Some bituminous materials are solid or semisolid at room temperature. Other grades are a relatively viscous (thick) liquid at room temperature. Mixing bituminous materials with solvents or water produces cutbacks or emulsions that are liquid at atmospheric temperatures. Such liquid asphalts and tars are used for cold mixes or are applied as sprays in building pavements.

ASPHALTS

Asphalt is obtained only from crude petroleum and has two general classes—natural and manufactured. Natural asphalts occur in lakes (as lake asphalt), pits, or rock structures (as rock asphalt). Manufactured asphalt is produced by distilling crude petroleum (see *Figure 3-1*). A military engineer is seldom concerned with natural asphalts because they are not usually available in those areas of interest. Therefore, this chapter discusses the uses and testing of manufactured asphalts.

All asphalt cements are solid or semisolid at room temperature (77°F) and must be converted to a fluid state by heating, emulsifying, or dissolving in a petroleum solvent.

Grading

There are two grade scales used for identifying asphalt cement—penetration grade and viscosity grade. The penetration grade is determined by the distance a standard needle under a standard load will penetrate a sample in a given time under a given temperature condition. A correlating asphalt-petroleum number from 00 to 7 is assigned to these penetration ranges. The viscosity grade is determined using a standard viscometer under standard conditions. *Table 3-1* lists the penetration ranges and correlating asphalt-petroleum numbers presently recognized along with the relative consistencies.

Table 3-1. Penetration grades and asphalt petroleum numbers of asphalt cement

Penetration Grade	Asphalt-Petroleum Number	Relative Consistency
40 to 50	7	Hard
60 to 70	5	
85 to 100	3	Medium
120 to 150	1	Soft
200 to 300	00	

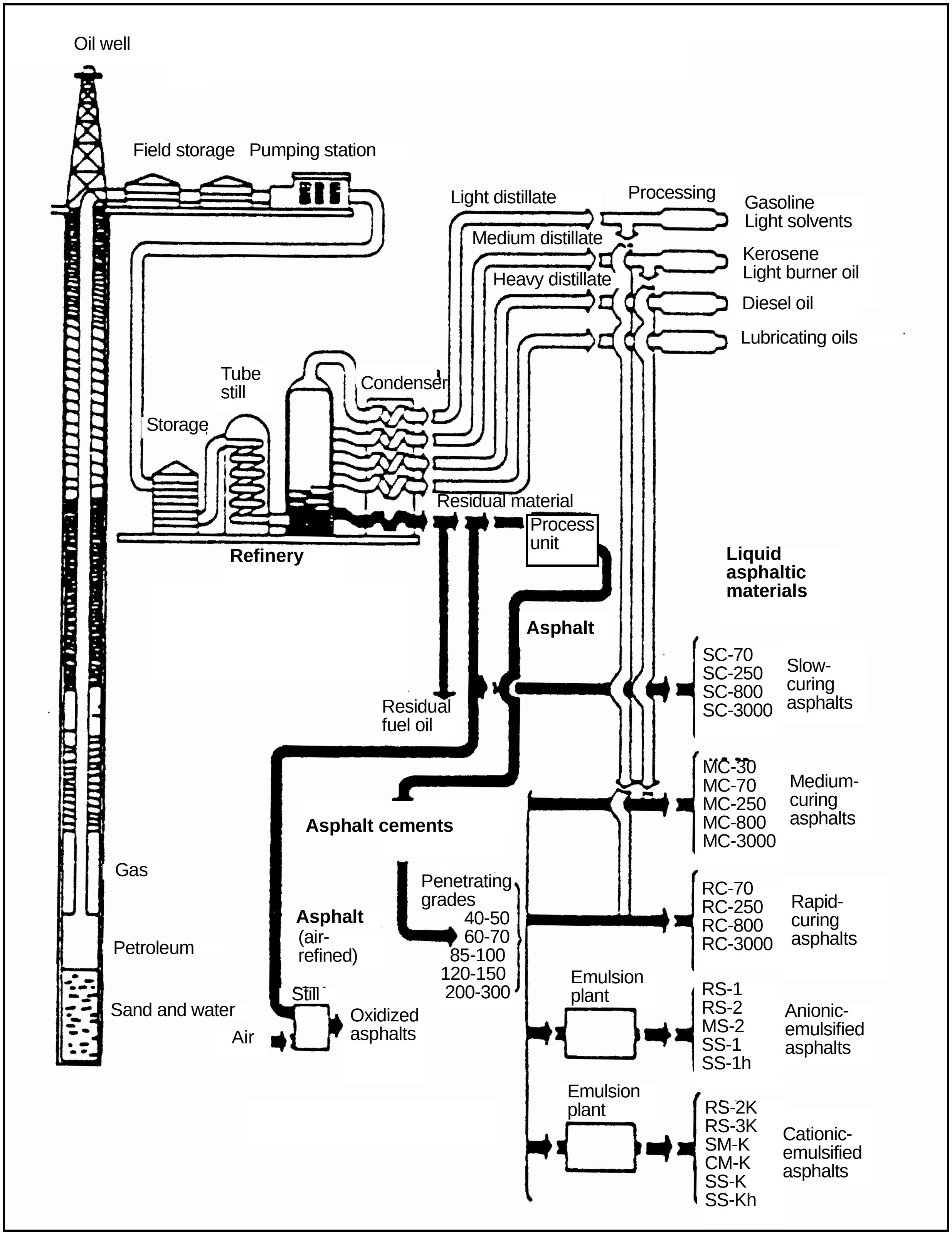


Figure 3-1. Simplified flow chart showing recovery and refinement of petroleum asphaltic materials

Cutback Asphalts

When heating equipment is not available or is impractical to use, asphalt cement can be made fluid by adding solvents (called cutter stock or flux oils). Cutter stock may be any one of the more volatile petroleum-distillate products. The resulting combination is called asphalt cutback. Exposure to air causes the solvents to evaporate and leave the asphalt cement to perform its functions.

The classification of the cutback is based on the evaporation rate of the distillate in the mixture. Gasoline or naphtha (high volatility) produces a rapid-curing (RC) cutback; kerosene (medium volatility) produces a medium-curing (MC) cutback; and fuel oil (low volatility) produces a slow-curing (SC) cutback. Road oils, referred to occasionally, are a heavy petroleum oil in the SC grade of liquid asphalt. *Table 3-2* shows the percentage of components by grade for the three types of asphalt cutbacks.

Table 3-2. Asphalt-cutback composition (expressed in percent of total volume)

Type	Components	Grades				
		30	70	250	800	3,000
RC	Asphalt cement		65	75	83	87
	Gasoline or naphtha		35	25	17	13
MC	Asphalt cement	54	64	74	82	86
	Kerosene	46	36	26	18	14
SC	Asphalt cement		50	60	70	80
	Fuel oil		50	40	30	20

As more cutter stock is mixed with a given amount of asphalt cement, a thinner liquid results. In practice, different amounts of cutter stock are added to a given amount of asphalt cement to obtain various viscosities, or grades, of cutbacks. The Corps of Engineers has approved a set of specifications for cutbacks based on kinematic viscosity. The number assigned to each grade corresponds to the lower limit of kinematic viscosity as determined by a standard test. The upper limit of each grade is equal to twice the lower limit or grade number. The units used in the test are centistokes.

Thus, a number 70 cutback refers to a viscosity range of 70 to 140 centistokes. The other grades and their limits are 250 (250 to 500), 800 (800 to 1,600), and 3,000 (3,000 to 6,000). In addition, the MC has a 30 grade (30 to 60). *Figure 3-2* shows the scale of viscosity grades. The higher the viscosity, the thicker the liquid.

Asphaltic penetrative soil binder is a special cutback asphalt composed of low-penetration-grade asphalt and a solvent blend of kerosene and naphtha. It is similar in character to standard low-viscosity, MC cutback asphalt but differs in many specific properties. It is used as a soil binder and dust palliative.

Asphalt Emulsions

It is often advantageous to use an asphalt material that is liquid at room temperature and yet will not burn. Asphalt emulsions possess these properties. Emulsified asphalt is a liquid material made up of a mixture of asphalt, water, and emulsifier. Asphalt and water will not mix alone so a chemical agent (an emulsifying agent) must be added. Common emulsifying agents are soaps, colloidal clays, and numerous other organic agents. Emulsified asphalt is a heterogeneous system in which water forms the continuous phase of the emulsion and the minute globules of asphalt for the discontinuous phase. There is also an inverted asphalt emulsion in which the continuous phase is asphalt (generally liquid asphalt) and the discontinuous phase is minute globules of water in relatively small quantities. Emulsified asphalts may be of either the anionic (electronegatively charged asphalt globules) or cationic (electropositively charged asphalt globules) types, depending on the emulsifying agent.

Emulsions are classified according to the setting or breaking rate which is the speed at which the emulsion breaks or the asphalt and water separate. This rate usually depends on the emulsifier used and the proportion of water to asphalt. Emulsions are described as rapid-setting (RS), medium-setting (MS), and slow-setting (SS) and also by viscosity numbers (see *Figure 3-2*). Because of this breaking rate, emulsions can also be grouped according to their ability to mix with damp aggregate. The RS emulsion breaks so fast that it cannot be mixed; therefore, it is called a nonmixing type. The MS and SS emulsions break slowly enough to permit good mixing until each particle of the aggregate is uniformly coated. Emulsions may also be satisfactorily used as a tack coat for bituminous pavements.

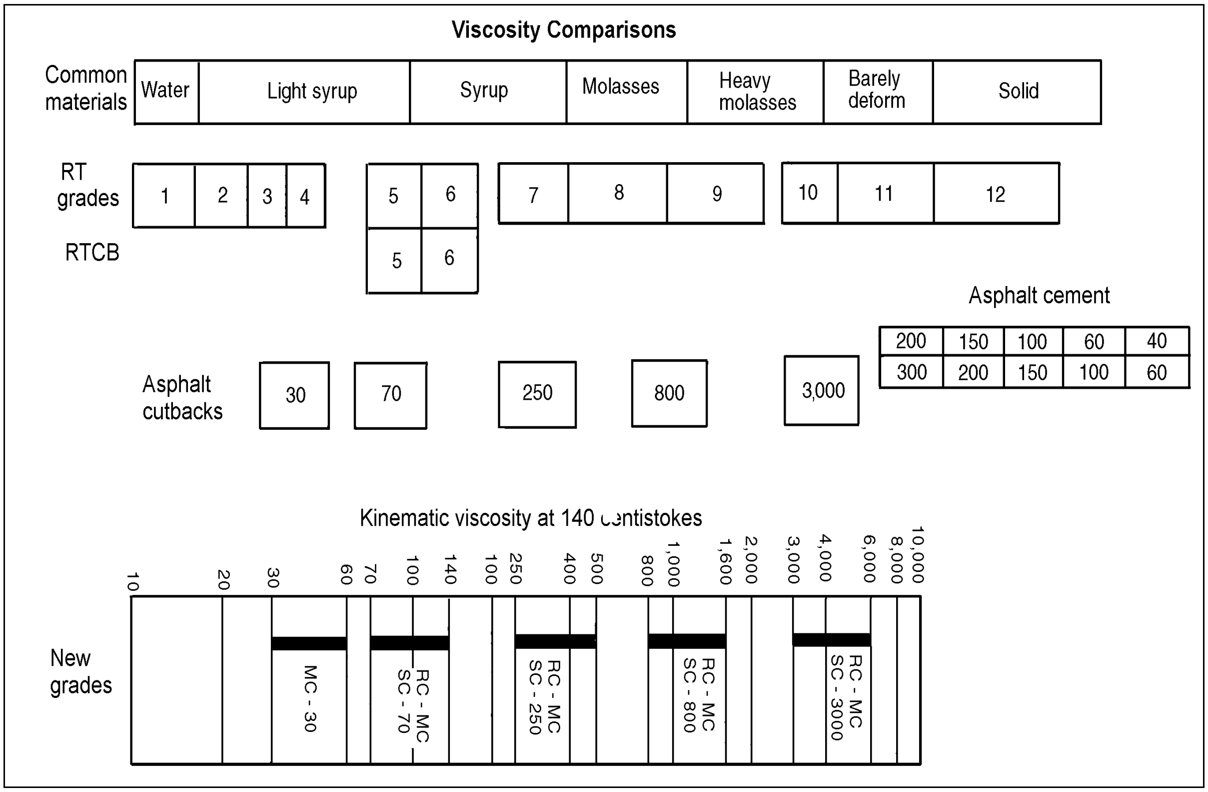


Figure 3-2. Viscosity grades at room temperature

TARS

Tars are products of coal distillation (see *Figure 3-3*). No natural source of tars exists. Coal tar is a general term applied to all varieties of tar obtained from coal. It is produced by one of several methods, depending on the desired end product.

When bituminous coal is destructively distilled, coke and gas are formed. Tar, ammonia, light oils, sulfur, and phenol may be recovered. Coke-oven tar is

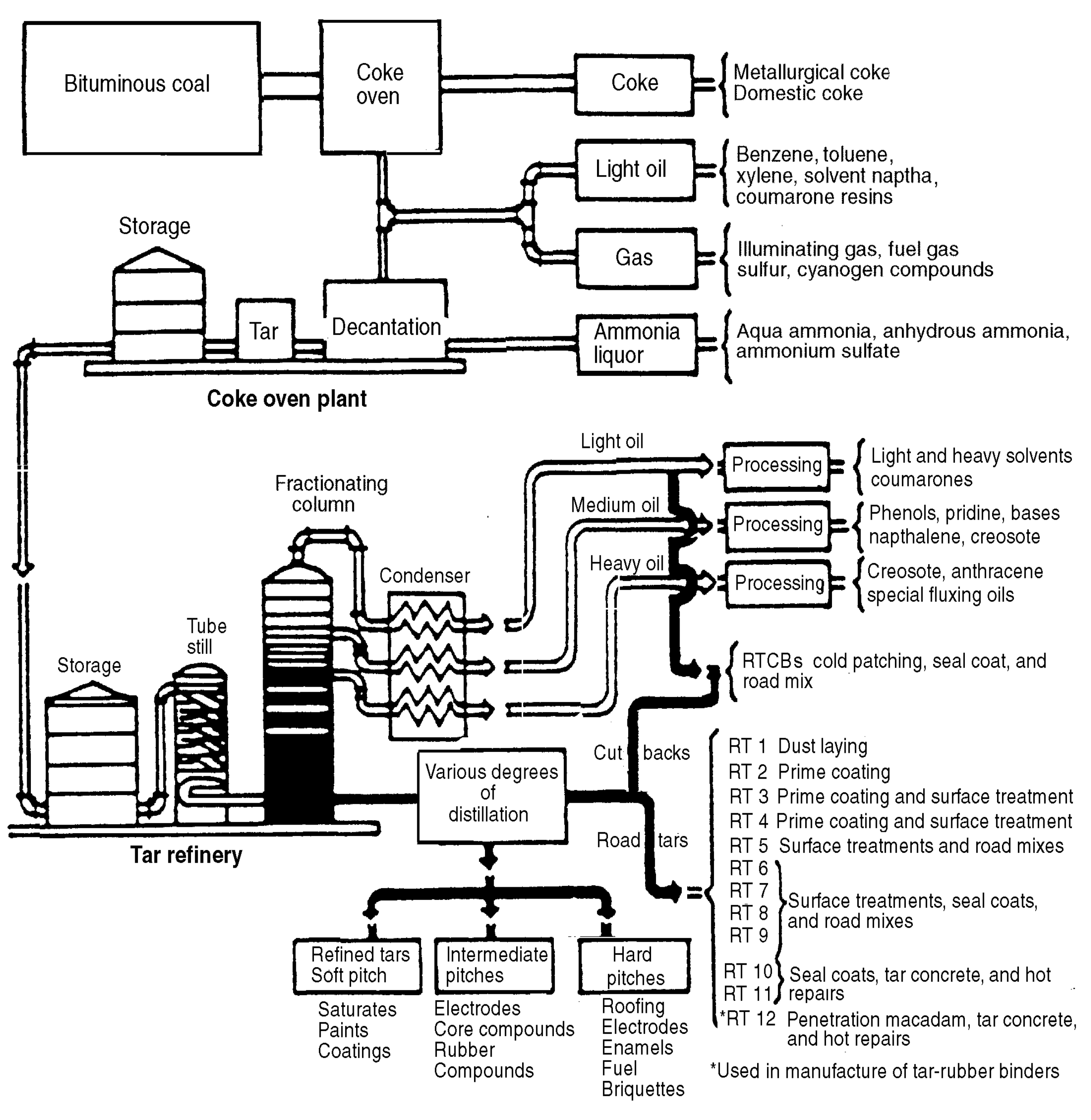


Figure 3-3. Simplified flow chart showing production of road tars from bituminous coals

produced in the greatest amount. Its chemical, physical, and adhesive characteristics make it most suitable for road-tar purposes. Water-gas tar is obtained in the manufacture of carbureted water gas. The nature of the carbureting oil largely determines the character of the water-gas tar produced and may vary widely in specific gravities, viscosities, and other physical and chemical properties.

Road tars are manufactured in 12 grades of viscosity (see *Figure 3-2*, *page 3-5*). There are also some special grades for use in rubberized-tar binders. Grades 1 through 7 are liquid at room temperature, while grades 8 through 12 are semisolid or solid. The difference is due to the liquid coal distillates in the tar; the more distillate, the more liquid (or less solid) the tar. The road-tar cutbacks (RTCBs) are products of cutting back the heavier or harder grades with coal-tar distillates. RTCBs are manufactured in two viscosity grades (5 and 6) only.

Tar, which is insoluble in petroleum distillates, is sometimes mixed with oil-resistant, unvulcanized rubber to form a rubberized-tar binder material.

CHARACTERISTICS AND USES OF BITUMENS

Tables 3-3 and *3-4, pages 3-8 through 3-11,* list the bituminous materials, sources, curing, temperatures, and grades associated with bituminous operations.

SAFETY PRECAUTIONS

Be extremely cautious when handling bituminous materials. Asphalt cement, which is solid at room temperature, is heated to high temperatures to make it workable as a binder material. Heated asphalt can cause severe burns if allowed to come in contact with the skin. The vapors emitted by heating bituminous materials may be harmful if inhaled. Use care during heating to ensure proper ventilation. Asphalt cutbacks contain highly flammable volatiles. The vapors will ignite at relatively low temperatures. The lowest temperature at which they will ignite is called the flash point. The minimum flash point for RC-250, RC-800, and RC-3,000 is 80°F; for MC-30 and MC-70, about 110°F; for MC-250 to MC-3,000 and SC-70, about 150°F; and for the remaining SC grades, about 25° higher per grade up to 225°F for SC-3,000.

NOTES:

1. The spraying and mixing temperatures in many cases are above the flash point (see *Table 3-3*). Use extreme caution when handling these mixtures. Do not expose their vapors to an open flame.

2. Cutbacks may also be classified as an environmentally hazardous material. Check with unit and installation environmental representatives for further guidance as to proper storage, use, and disposal of these substances. If your unit's environmental representative is not known, consult your commander for assistance.

Table 3-3. Characteristics of bituminous materials

Material	Form	Grade Designa-tion	Temperature of Application Ranges				Flash Point (Min)		Remarks
			Spraying **		Mixing				
			°F	°C	°F	°C	°F	°C	
Penetrative soil binder	Liquid		130-150	55 - 65			80	27	Contains naphtha Caution: Highly flammable
Cutback (RC)	Liquids—asphalt residues fluxed with more volatile petroleum distillate	RC-70 RC-250 RC-800 RC-3,000	*105-175 145-220 180-255 *215-290	*41-79 63-104 *82-124 *102-143	95-135 135-175 170-210 200-240	35-57 57-79 77-99 93-116	 80 80 80	 27 27 27	RC cutbacks contain highly-volatile naphtha cutter stock. Naphtha evaporates quickly, leaving an asphalt-cement binder, permitting early use of the surface. Caution: Highly flammable
Cutback (MC)	Liquids—asphalt residues fluxed with more volatile petroleum distillate	MC-30 MC-70 MC-250 MC-800 MC-3,000	70-140 105-175 145-220 180-255 215-290	21-60 41-79 63-104 82-124 102-143	55-95 95-135 135-175 170-210 200-240	13-35 35-57 57-79 77-99 93-116	100 100 150 150 150	37 37 65 65 65	MC cutbacks contain less volatile kerosene cutter stock. Kerosene evaporates less rapidly than naphtha. Caution: Flammable.
Cutback (SC)	Liquids—asphalt residues fluxed with more volatile petroleum distillate	SC-70 SC-250 SC-800 SC-3,000	105-175 145-220 180-225 215-290	41-79 63-104 82-124 102-143	95-135 135-175 170-210 200-240	35-57 57-79 71-99 93-116	150 175+ 200+ 225+	65 79+ 93+ 107+	SC cutbacks contain slightly-volatile diesel-fuel cutter stock. Diesel fuel evaporates slowly. Caution: Flammable.
Asphalt cements	Solids	40-50 60-70 85-100 120-150 200-300	 285-350 285-350 285-350 260-325	 141-177 141-177 141-177 127-163	300-350 275-325 275-325 275-325 200-275	149-177 135-163 135-163 135-163 93-135			Penetrations 40 to 100 used for crack and joint fillers. Penetrations 70 to 300 used for plant mixes, penetration macadam, and surface treatment. Use test to determine flash point.
Powdered asphalt	Hard and solid asphalts ground to powder								Used with SC to produce extra tough road surfaces.

*** RC cutbacks are seldom used for spraying.**
**** Low temperature is based on a viscosity of 200 centistokes kinematic viscosity and the higher temperature is based on a 50-œntistoke viscosity.**

Table 3-3. Characteristics of bituminous materials (continued)

Material	Form	Grade Designa-tion	Temperature of Application Ranges				Flash Point (Min)		Remarks
			Spraying **		Mixing				
			°F	°C	°F	°C	°F	°C	
Asphalt emulsions (RS)	Liquids—asphalt particles held in an aqueous suspension by an emulsifying agent	RS-1	50-140	10-60	Nonmixing	10-60			Freezing destroys emulsion. Use for road and plant mixes with coarse aggregates (SS). All emulsions with “K” suffix are cationic.
		RS-2	50-140	10-60	50-140	10-60			
		RS-2K	50-140	10-60	50-140	10-60			
		RS-3K	50-140	10-60	50-140	10-60			
Asphalt emulsions (MS)	Liquids—asphalt particles held in an aqueous suspension by an emulsifying agent	MS-2	50-140	10-60	50-140	10-60			
		SM-K	50-140	10-60	50-140	10-60			
		CM-K	50-140	10-60	50-140	10-60			
Asphalt emulsions (SS)	Liquids—asphalt particles held in an aqueous suspension by an emulsifying agent	SS-1	50-140	10-60	50-140	10-60			
		SS-1h	50-140	10-60	50-140	10-60			
		SS-K	50-140	10-60	50-140	10-60			
		SS-Kh	50-140	10-60	50-140	10-60			
Road tars	Liquids	RT-1	60-125	15-52					Priming oils. RT-4 through RT-12 not generally used.
		RT-2	60-125	15-52					
		RT-3	60-125	27-66					
RTCBs	Liquids	RTCB-5	60-120	16-49					Patching mixtures. Caution: Flammable.
		RTCB-6	60-120	16-49					
Rock asphalt	Solids								Mixed and used locally where found. Cutback may be added if necessary.

*** RC cutbacks are seldom used for spraying.**

**** Low temperature is based on a viscosity of 200 centistokes kinematic viscosity and the higher temperature is based on a 50-centistokes viscosity.**

Table 3-4. Typical uses of bituminous materials

Purpose or Use	Grade or Designation[1]		
	CB - Asphalt Cutback[2]		
	RC	MC	SC
Dust palliative	DCA-70[3]	MC-30, -70, -250 APSB[4]	SC-70, -250
Prime coat:			
Tightly bonded surfaces		MC-30	
Loosely bonded, fine-grained surfaces		MC-70	SC-70
Loosely bonded, coarse-grained surfaces		MC-250	SC-250
Tack coat	RC-250, -800	MC-250, -800	
Surface treatment and seal coat:			
Coarse sand cover	RC-70, -250	MC-250, -800	
Clean coarse aggregate cover	RC-250, -800, -3,000	MC-800	
Graded gravel aggregate cover		MC-250-, -800	SC-800
Gravel mulch		MC-250	SC-250
Mixed in place road mix:			
Open-graded aggregate:			
Sand	RC-70, -250	MC-800	
Maximum diameter 1 inch, high percentage passing No. 10		MC-800	
Macadam aggregate	RC-250, -800		
Dense-graded aggregate:			
High percentage passing No. 200		MC-250	SC-250
Maximum diameter 1 inch, medium percentage passing No. 200		MC-250, -800	SC-250, -800
Premix or cold patch:			
Open-graded aggregate	RC-250	MC-800	SC-800
Dense-graded aggregate		MC-250	SC-250
Cold-laid plant mix:			
Open-graded aggregate:			
Sand	RC-250, -800		
Maximum diameter 1 inch, high percentage passing No. 10	RC-800		SC-800
Macadam aggregate	RC-800, -3,000		
Dense-graded aggregate:			
High percentage passing No. 200			
Maximum diameter 1 inch, medium percentage passing No. 200			
Aggregate precoating followed with asphalt			

[1] Prevailing temperature during construction also affects selection of bitumen and may be the determining factor rather than size and gradation of aggregate.

[2] Caution: Do not overheat aggregate when cutbacks are used to produce hot mixes.

[3] DCA-70 is a water emulsion of a polyvinyl acetate containing chemical modifiers (formerly UCAR-131). Proprietary product of Union Carbide Corporation, New York, NY.

[4] Asphaltic penetrative soil binder (APSB)

Table 3-4. Typical uses of bituminous materials (continued)

Purpose or Use	Grade or Designation[1]		
	CB - Asphalt Cutback[2]		
	RC	MC	SC
Dust palliative	DCA-70[3]	MC-30, -70, -250 APSB[4]	SC-70, -250
Hot-laid plant mix	RC-3,000	MC-3,000	SC-3,000
Penetration macadam: Cold weather Hot weather	RC-800, -3,000		SC-3,000

[1] Prevailing temperature during construction also affects selection of bitumen and may be the determining factor rather than size and gradation of aggregate.

[2] Caution: Do not overheat aggregate when cutbacks are used to produce hot mixes.

[3] DCA-70 is a water emulsion of a polyvinyl acetate containing chemical modifiers (formerly UCAR-131). Proprietary product of Union Carbide Corporation, New York, NY.

[4] Asphaltic penetrative soil binder (APSB).

ADVANTAGES AND DISADVANTAGES

Advantages and disadvantages of the bituminous materials used in construction are as follows:

Asphalt-cement cutbacks are flammable. Asphalt pavements are susceptible to damage by the blast from jet planes, and they can be dissolved by petroleum products that may be spilled on them such as during refueling at an airfield. Tars, on the other hand, are not affected by petroleum derivatives since they are products of coal. Tars, when used as a prime for base courses, seem to possess better penetration qualities than asphalts and are less susceptible to stripping (loss of bond to aggregate) in the presence of water. Tars are affected by temperatures and have a wide range in viscosity with normal surrounding temperature changes. Tar can become so soft during warmer weather that the pavement will rut under traffic. In colder weather, it can become so brittle that the pavement will crack. The temperature susceptibility of tar binders is improved by blending with oil-resistant rubber. Asphalt pavements and tar pavements are generally ready for traffic within a few hours after placement since they can be used as soon as they reach normal temperature.

Asphalt emulsions are not flammable and are liquid at normal temperatures. Since they are mixed with water, they can be used with more damp aggregate than required for the cutbacks. Additional water may be added to the emulsion up to proportions of 1:3 for use in slurry seal coats. Because emulsions contain water, they have certain disadvantages. During freezing weather, the emulsions can freeze and the components separate. Emulsions are difficult to store for extended periods because they tend to break even in unopened drums. When shipped, the water in the emulsion takes up valuable space which could be used to transport hard-to-obtain materials.

SECTION II. SAMPLING MATERIALS

When conducting tests, use the materials that represent those that will be used in construction; otherwise, test results will be misleading. Sampling of materials for testing should receive close attention. When large samples must be subdivided into small units for the actual tests, take care to keep the sample representative of the original mass. Reduce aggregate samples to the proper size for testing by means of quartering. Methods for sampling natural deposits of sands and gravels are discussed in soil surveys and are applicable here.

Take samples of bituminous materials at the place of manufacture or at the delivery point. (This manual assumes that sampling is done at the point of delivery.) Samples may be taken for either of two purposes:

- To obtain an average of the delivered material.
- To find the maximum variation in the material's characteristics.

Take samples for analysis to identify bituminous materials if records are not available. Obtain sufficient quantities of materials at the time of sampling to meet specification requirements and to provide for laboratory pavement-design tests. Normally, aggregates that will produce 150 pounds of the desired gradation and 2 gallons of bituminous material will produce sufficient data.

BITUMINOUS MATERIALS SAMPLING (ASTM D 140-88)

Use clean, dry containers for sampling. Keep the containers tightly closed and properly marked. A sample for a routine laboratory examination should not be less than one quart.

LIQUID MATERIALS

When sampling liquid bituminous materials from nonagitated vertical tanks, take samples from near the top, middle, and bottom. Test the samples from the three levels separately to detect stratification. Materials shipped in tank cars may be sampled from valves and taps. Take samples from drain cocks on the side of the tank or car. If cocks are not present, take the samples by lowering weighted bottles or cans into the material (see *Figure 3-4*). Fit the bottle or can with a stopper that can be removed by a string or wire after it has been lowered to the proper depth.

SOLID AND SEMISOLID MATERIALS

When sampling solid or semisolid materials in drums, barrels, cartons, and bags, take samples at least 3 inches below the surface and 3 inches from the side of the container or cake. Use a clean hatchet on hard material and a stiff putty knife on soft material.

AGGREGATE SAMPLING (ASTM D 75-87)

Aggregate varies in size from the larger stones or rocks to the gravels and sands. These materials for paving may still be in their natural deposits or may be in stockpiles previously gathered.

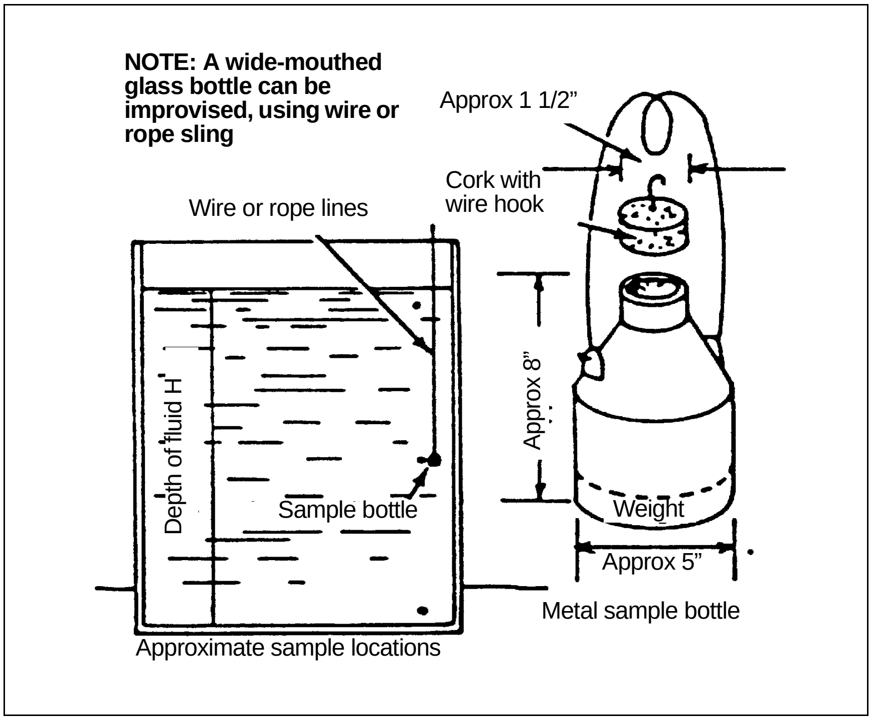


Figure 3-4. Sampling liquid bituminous materials from tank car or storage tank

STONE FROM LEDGES AND QUARRIES

Obtain separate samples of unweathered stone weighing at least 50 pounds each from all strata that appear to vary in color and structure. Prepare a sketch plan with elevation, showing the thickness, length, width, and location of the different layers so that the quantity available can be estimated.

NATURAL DEPOSITS OF SAND AND GRAVEL

Select samples that represent the different materials available in the deposit. Sketch the area and indicate the approximate quantities of different materials. If the deposit is an open-face bank or pit, take the sample by channeling the face so that it will represent material that visual inspection indicates may be used. Cut the face immediately before sampling, and discard any material that has fallen from the surface along the face. Do not include in the sample any overlying material (overburden) that is not suitable for use as an aggregate, since this material would be stripped away when the aggregate is removed from the pit. It may be necessary to make test borings or dig test pits to determine the approximate extent of the material. If test pits are dug, they must be adequately shored to prevent material from caving in on personnel working in the pit. Obtain from the pit representative samples for each change in strata. If the material being sampled is all sand, about 25 pounds is sufficient for tests. If it consists of sand and gravel, a somewhat larger sample (about 75 pounds) is required for preliminary tests. The coarser the gravel portion, the larger the sample required.

STOCKPILES

If the material has been stockpiled previously, take care in obtaining samples. There is a natural tendency toward separation of similar size fractions into groups in the stockpile. The material near the outer edges and near the base of the pile is likely to be coarser than the average. Cut a face into the stockpile near the base, the center, and the top on at least two opposite sides. Combine samples from at least three different sections of the pile to give a representative sample.

COMMERCIAL AGGREGATES

It is preferable to obtain samples of commercial aggregates at the plant, during loading, from stockpiles or bins. Obtain separate samples at different times while the material is being loaded, to determine variations in the grading of the material. Take bin samples from the entire cross section of the flow of material as it is being discharged. Testing separate samples gives a better idea of variations that occur, but samples should be mixed and reduced by quartering when the average condition is desired. When it is not practicable to visit the plant to obtain samples, the next preferred method is to sample the material in cars or trucks or while it is being unloaded. Take railroad-car samples from three or more trenches dug across the car at points that appear on the surface to be representative of the material. When obtaining the sample, remember that segregation of the different sizes has probably taken place and choose samples that are representative.

SECTION III. FIELD IDENTIFICATION

Laboratory tests conducted on bituminous materials to check compliance with specifications are not considered field tests. They are described in this section for information purposes only. The field tests discussed in this section are limited to the bitumen identification procedures, flash-point tests, and penetration tests. These tests are applicable to both tars and asphalts and are conducted to determine safe uses for a material.

Field identification enables the military engineer to determine the type of surface that can be constructed with the type and grade of material available. With the type of surface known, the construction procedure may be outlined and scheduled. This procedure will then determine the proper equipment and the necessary safety procedures.

The aggregate materials must also be tested for acceptable bituminous construction use.

BITUMEN FIELD-IDENTIFICATION TESTS

Perform field tests to identify the bituminous paving materials as asphalt cement, asphalt cutback, asphalt emulsion, road tar, or RTCB. In addition, identify the viscosity grade of the bitumen. To distinguish among the several asphaltic and tar products, it is necessary to know something of their origin, physical properties, and the manner in which they are normally used. Some of this information is contained in *Tables 3-3* and *3-4, pages 3-8 through 3-11*.

The identification procedure outlined in *Figure 3-5* is based on a consideration of the physical properties of these materials.

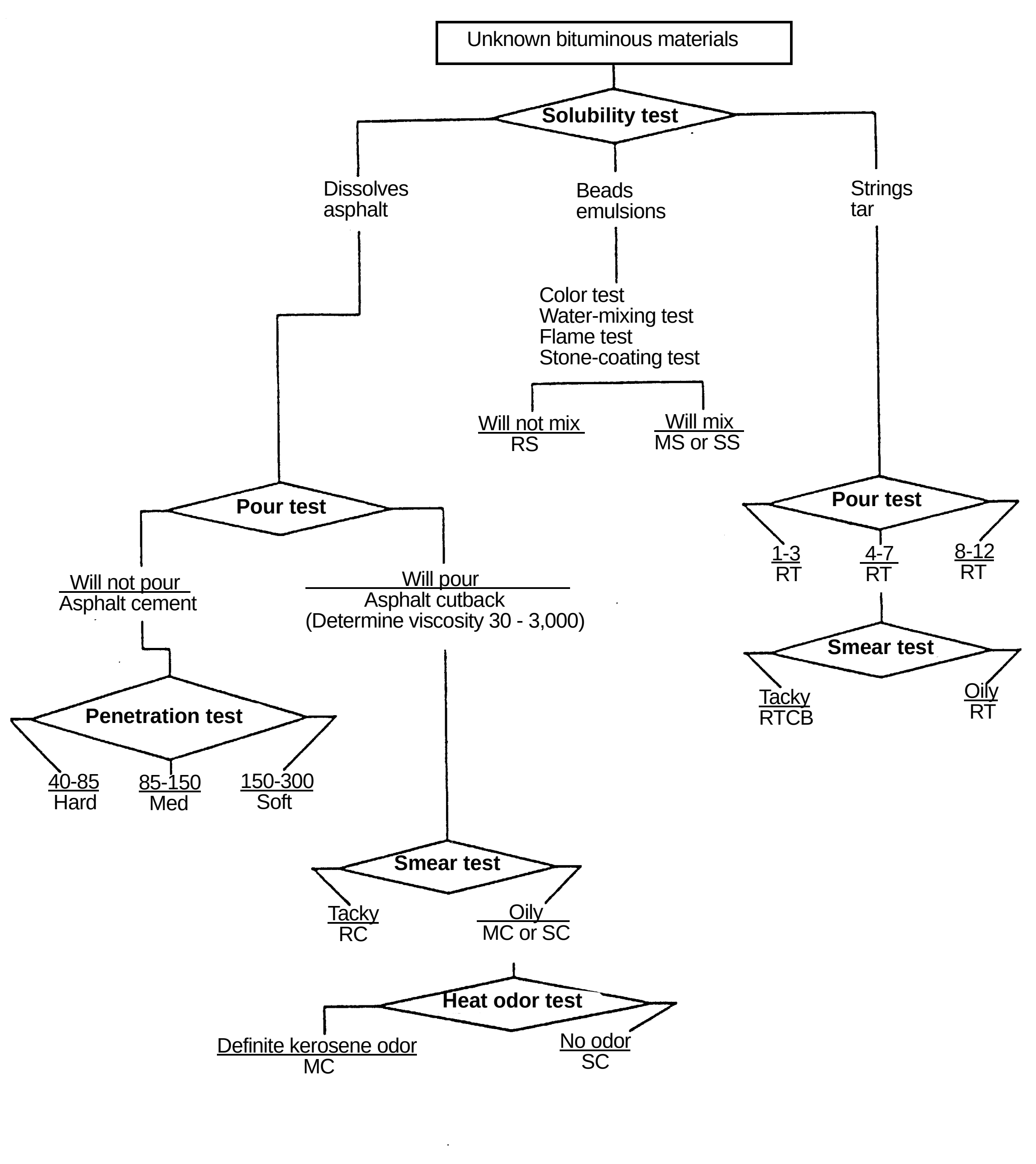


Figure 3-5. Identification of unknown bituminous materials

Ensure that all tests are performed away from open flames and in well-ventilated areas. Also ensure that all materials are properly disposed of according to local environmental policy.

ASPHALTS AND TARS

The first procedure in identifying an unknown bituminous material is to determine, by a solubility test, whether it is an asphalt or a tar. Attempt to dissolve an unknown sample (a few drops, if liquid, or enough to cover the head of a nail, if solid) by stirring it in any petroleum distillate. Kerosene, gasoline, diesel oil, or jet fuel is suitable for this test. Since asphalt is derived from petroleum, it will dissolve in the petroleum distillate. Road tar will not dissolve. If the sample is an asphalt, the sample distillate mix will consist of a dark, uniform liquid. Asphalt cements or cutbacks dissolve completely.

Asphalt in emulsions is also distinguishable as it dissolves and forms black beads or globules in the bottom of the container of distillate. A road-tar sample will be a dark, stringy, undissolved mass in the distillate. A check can be made by spotting a piece of paper or cloth with the mix. Asphalt dissolved in distillate will produce a brown to black stain. The clear distillate above the settled tar will not cause a stain. The solubility test provides a positive method of identification.

ASPHALT CEMENT AND CUTBACKS

Perform the following procedure to determine if the specimen is an asphalt cement or a cutback.

The various grades of asphalt cement are solid at room temperature while cutbacks are liquid, and a pour test will distinguish between them. Place a sample of the material in a small container and attempt to pour it. If the material does not pour, it is an asphalt cement. If it pours, it is a cutback or an emulsion. Note that at 77°F even the softest asphalt cement will not pour or deform if the container is tilted.

The various grades of asphalt cement are distinguished principally by their hardness, as measured by a field penetration test. The information obtained may be sufficient for planning for or starting emergency construction. The exact penetration grade is not determined, but the field test will distinguish between hard, medium, or soft groups of asphalt cement. Perform the test by pushing a sharpened pencil or nail into the container of asphalt (at about 77°F) using about 10 pounds of force. If only a slight penetration is made with considerable difficulty, a hard asphalt cement is present. If the penetration is made with some difficulty, a medium asphalt cement is present. If the penetration is made with ease, the asphalt cement is a soft asphalt cement in the high-penetration scale. Even the highest penetration will not pour or deform at 77°F if its container is tilted.

ASPHALTIC-CUTBACK TESTS

There are three tests used to determine the grade of an asphaltic cutback:

Pour Test

As stated previously, an asphalt cement will not pour at 77°F, but a cutback will. The pour test can be used to determine whether the unknown material is an asphalt cutback. If the material pours, it is an asphalt cutback. The approximate viscosity grade number of the cutback is found by comparing the flow to well-known materials such as water, syrup, and others. If this test is made at a temperature below 77°F, the materials will appear more viscous (stiff) than at 77°F and the opposite if tested when warmer than 77°F. The cutbacks of a given viscosity grade will pour in a manner similar to the following:

- 30—water.
- 70—light syrup.
- 250—syrup.
- 800—molasses.
- 3,000—barely deform.

After the pour test, the approximate viscosity grade of the cutback is known, but the type (RC, MC, or SC) is not.

Smear Test

Perform the smear test to determine if a cutback is an RC. This is done by making a uniform smear of the substance on a piece of glazed paper or other nonabsorbent surface. Volatile materials, if present, will evaporate. Since RC materials are cut back with a very volatile substance, most of the volatiles will evaporate within 10 minutes. The surface of the smear then becomes extremely tacky. This is not true of the lighter grades (MC and SC), which remain fluid and smooth for some time. An MC will not result in a tacky surface for a matter of hours. SC materials may require several days.

Perform a prolonged smear test to identify an 800- or 3,000-grade MC or SC cutback. This is necessary because these grades contain such small quantities of cutter stock that they may become tacky in the 10-minute period specified above. Place a thin smear of the material on a nonabsorbent surface and let it cure for at least 2 hours. By the end of that time, if the material being tested is not an MC or SC, the smear will be hard or just slightly sticky. However, if the material being tested is an MC or SC, the smear will be uncured and still quite sticky. If the material is an RC-3,000, it will cure completely in 3 hours, whereas an RC-800 will take about 6 hours to cure. Even after 24 hours, an MC or SC will still be sticky.

Heat-Odor Test

The main difference between MC material and SC material is that the MC material is cut back with kerosene and the SC with diesel or a low-volatility oil class. In this test, apply heat to the sample to drive off the kerosene, if it is present, and make it show up in the form of an odor. Heat the unknown sample in a closed container to capture the escaping vapors, using minimal heat. An MC sample will have a strong petroleum or kerosene odor. An SC sample will have no kerosene or petroleum odor but may have a faint odor of hot motor oil. The ability to differentiate between the RC, the MC, and the SC is an essential part of field identification.

ASPHALT-EMULSION (ANIONIC) TESTS

Another asphaltic material used in paving is asphalt emulsion, which is a mixture of asphalt, water, and an emulsifying agent. The anionic emulsions specifications cover three types of asphalt emulsion—RS grades 1 and 2, MS grade 2, and SS grades 1 and LH.

Solubility Test

The solubility test will make an emulsion's identity known by forming into globules or beads that fall to the bottom of the container of petroleum distillate. During this test, the emulsion will present a distinctive dark brown color while all other bituminous materials are black.

Water-Mixing Test

If mixed with water, an emulsion will accept the extra water and still remain a uniform liquid. The sample and water will mix uniformly if the material is an emulsion. This test is positive since no other bituminous material will mix with water.

Flame Test

Since an emulsion contains water, a small piece of cloth saturated with it will not burn if a flame is applied. The other bitumens will burn or flame.

Stone-Coating Test

After establishing that the material is an emulsion, determine whether the emulsion is a mixing grade (MS or SS) or a nonmixing grade (RS). Mix a small amount (6 to 8 percent by weight) with damp sand using a metal spoon. Exercise care not to add so much emulsion to the sand as to saturate it. An RS emulsion will break so quickly it will not be possible to mix it with sand. It breaks immediately, gumming up the spoon with the relatively hard original asphalt cement. On the other hand, if the sample is a MS or SS emulsion, the material will mix easily and coat all the particles completely (as well as the mixing spoon) with a uniform coating of asphalt.

ROAD-TAR TESTS

There are three tests for road tars—the solubility test, the pour test, and the smear test.

Solubility Test

As determined earlier, if the unknown bituminous material does not dissolve during the solubility test but forms a stringy mass, the material is a tar (see *Figure 3-5, page 3-15*). The next step is to determine its viscosity grade.

Pour Test

By comparing the flow of the material to that of common materials (see *Figure 3-3, page 3-6*), the viscosity of the tar may be closely estimated. The grades run from RT-1 to RT-12 and vary in consistency from very fluid to solid.

Smear Test

If, during the pour test, the identified tar seems to be in the range of an RT-4 to RT-7 material, perform a smear test to determine whether it is a road tar or an RTCB. Perform the smear test in the manner previously described for cutback asphalt. The material is a road tar if the material remains with the same amount of stickiness. If it shows a great increase in stickiness in 10 minutes, it is an RTCB. If field identification yields a grade of about 5 or 6, it is not of particular importance specifically which grade of cutback it is since both are used under approximately the same conditions.

AGGREGATE IDENTIFICATION AND SELECTION

Identify the aggregate by shape or roughness, hardness, cleanliness, hydrophobicity, gradation, and particle size. Select aggregate with the best combination of these characteristics. Also consider the availability, length of haul, and overburden.

SHAPE AND ROUGHNESS

The aggregate in a pavement must transmit the traffic load to the base, usually by the interlocking of the particles. This interlocking is much more pronounced when the particles are angular in shape and rough in surface texture. If angular pieces of aggregate are in a pavement, the individual particles will not slip or slide over one another, but will lock together. However, more binder may be required since the angular shape has a greater surface area per unit volume than a round particle. Although angular particles are desired, the aggregate should not contain an excessive number of flat or elongated particles, as these particles cause bridging, thereby making compaction difficult.

Aggregates very seldom occur in nature as angular, so it is necessary in most cases to crush the aggregate to obtain the desired angular particles.

HARDNESS AND DURABILITY

The aggregate must be able to withstand the applied loads without cracking or being crushed. Resistance to weathering is also a function of the durability. An aggregate's resistance to wear can be determined by the Los Angeles abrasion test. The Mohs hardness scale may be used to determine the hardness of the aggregate. This scale is fully explained in FM 5-410. It ranges from 1 for talc or mica to 10 for diamond. By trying to scratch the aggregate or the common material, it is possible to establish which is harder; this determines the hardness of the aggregate. If both are scratched, the hardness of both is the same. Rub the scratch mark to see that it is really a scratch and not a powdering of the softer material. Some common materials and their approximate level of hardness are as follows:

- Fingernail—2.0.
- Copper coin—3.5.
- Knife blade—5. 0.
- Window glass—5.5.
- Steel file—6.5.

CLEANLINESS

The bituminous binder must penetrate into the pores of the aggregate and also adhere to the surface of the particles. Aggregates coated with clay or dust or having water-filled pores prevent the penetration or the adherence of bitumen and result in stripping of the binder. If the aggregate is not clean, it should be washed, either as part of the crushing operation or by spreading it on a hard surface and hosing it with water. When washing is impractical, dry screening may remove a great deal of dust and clay. Handpicking may have to be done if no other method can be used. The aggregate should be made as clean as possible with the equipment and manpower available.

HYDROPHOBICITY

Affinity for water can make an aggregate undesirable. If the aggregate is porous and absorbs water easily, the binder can be forced out of the pores, the bond between the aggregate and binder can weaken and break, and stripping can occur. Stripping is the loss of bituminous coating from the aggregate particles due to the action of water, leaving exposed aggregate surfaces. One of the following tests can be used to determine the detrimental effect of water on a bituminous mix:

- The stripping test.
- The swell test.
- The immersion-compression test.

Stripping Test (ASTM D 1664-80)

Prepare a test sample by coating a 100-gram aggregate sample with bituminous material at the right temperature for the grade of bitumen to be used. Spread the mixture in a loose, thin layer and air-cure it for 24 hours. Place a representative sample in a jar (no more than half full) and cover it with water. Close the jar tightly and allow it to stand 24 hours. At the end of 24 hours, vigorously shake the jar with the sample for 15 minutes. Make a visual examination to determine the percentage of exposed aggregate surface and report it as the percent of stripping.

Swell Test

Asphaltic mixtures containing fines of doubtful quality are sometimes measured for swell as a basis for judging the possible effects on a pavement. This test is more frequently used with dense-graded mixtures using emulsified and cutback asphalts. Compact a sample of the mix in a metal cylinder (usually 100 millimeters in diameter), and cool it to room temperature. Obtain a height measurement for the specimen. Place the specimen and mold in a pan of water, and mount a dial gauge above the sample in contact with the surface. Take an initial reading. Allow the specimen to soak for a specified period (usually 24 hours) or until there is no further swelling. Take another dial reading. The difference in readings, divided by the original height and expressed in percent, is the swell of the mixture. Experience has shown that bituminous pavement made with clear, sound stone; slag; or gravel aggregate and mineral filler produced from limestone will show test values of swell of less than 1.5 percent of the original specimen thickness.

Immersion-Compression Test (ASTM D 1075-88)

This test is intended to measure the loss of Marshall stability resulting from the action of water on compacted bituminous mixtures containing penetration-grade asphalt. The result is a numerical index of reduced stability obtained by comparing the Marshall stability with the stability of specimens that have been immersed in water for a prescribed period. Prepare six standard Marshall specimens (4 inches in diameter and 2 1/2 ± 1/16 inches high) for each test. Determine the specific gravity of each specimen. Separate the set of eight into two sets of four so that the average specific gravity of one set is essentially the same as the other. Test one set using the Marshall method. Immerse the other set in water (at 140°F ± 1°) for 24 hours and then test it. Compute the result as a ratio of soaked stability to unsoaked stability, and express it as a percentage as follows:

$$\text{index or reduced stability} = \frac{S_2}{S_1} \times 100$$

where—

S_1 = *average stability of unsoaked specimens*

S_2 = *average stability of soaked specimens*

Mixes with an index of less than 75 percent are rejected or an approved method of processing aggregate and treating asphalt is required to increase the index to a minimum of 75 percent.

GRADATION

The following designations help identify aggregates:

- Uniform gradation occurs when all particles are about the same size, normally less than 1 inch.
- Macadam gradation consists of uniformly sized particles except that they are in excess of 1 inch.
- Open gradation involves a considerable range of particle size, from large to small, usually containing little or no mineral filler. The void spaces in the compacted aggregate are relatively large.
- Dense gradation occurs when there is a good representation of all particle sizes and coarse, fine, and mineral fillers.

PARTICLE SIZE

In bituminous construction, it is common practice to designate aggregates according to particle size. There are three types of designations under this system, based on two sieve sizes—No. 4 and No. 200.

- Coarse aggregate is all material too large to pass the No. 4 sieve (see ASTM D 692-88).

- Fine aggregate passes the No. 4 sieve but is retained on the No. 200 sieve. In bituminous paving, the fine aggregate is usually a sand, but small pieces of crushed rock may be used (see ASTM D 1073-88).
- Mineral filler or mineral dust refers to all nonplastic materials which pass the No. 200 sieve. Most clays are too plastic and are not used. Generally, crushed rock dust, agricultural mineral filler, lime, or portland cement may be used as the mineral filler (see ASTM D 546-88).

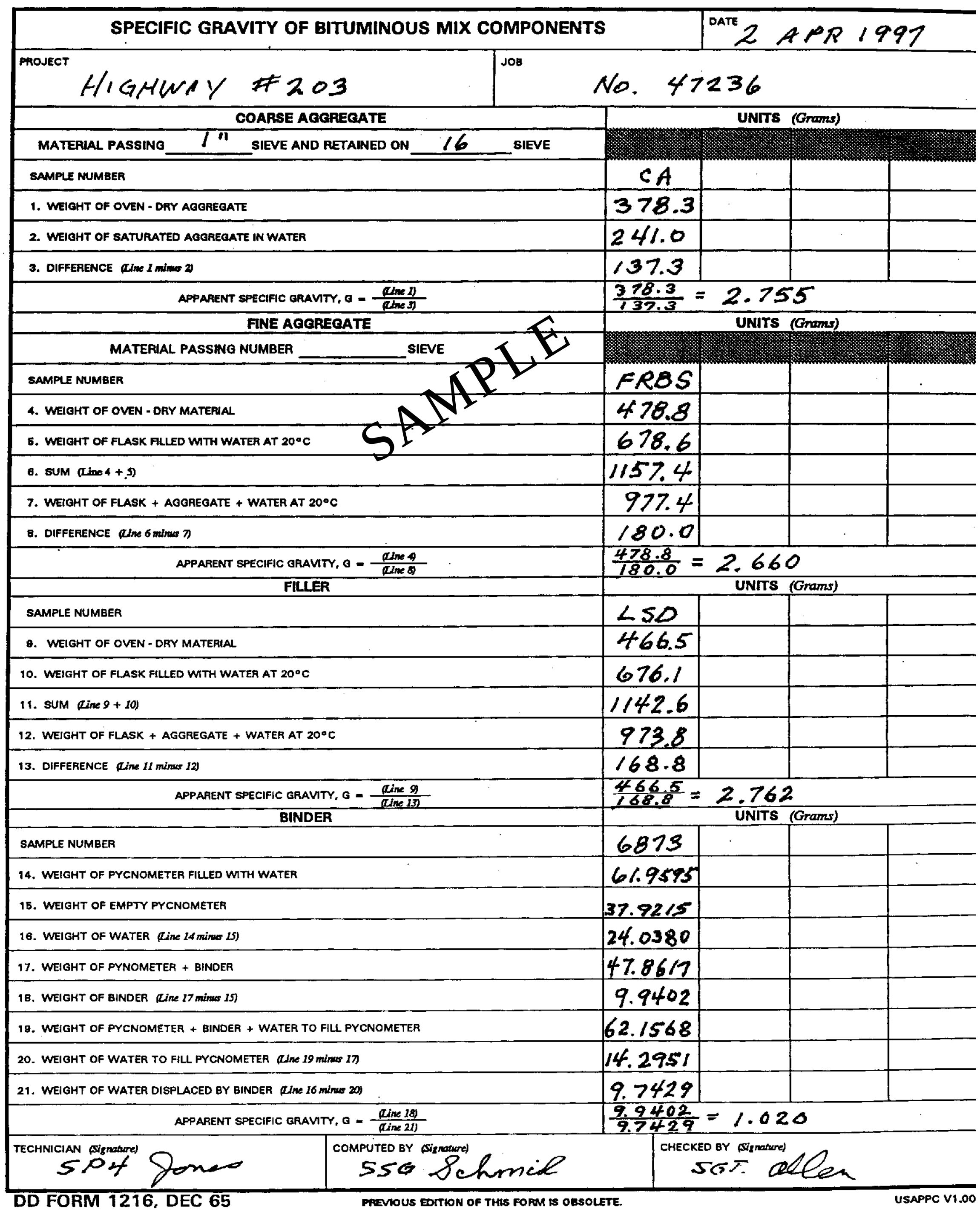
SECTION IV. BITUMEN TESTING

The field-identification tests on bitumens identify the material during expedient conditions or until more detailed tests can be performed. The identification determines whether the material is an asphalt or a tar and whether it is a cutback or an emulsion.

Bituminous materials are manufactured to meet specifications established by the federal government, the AASHTO, and the ASTM. These specifications define the extreme limits permitted in the manufacture of the material and assure the user that the material will possess definite characteristics and fulfill the project requirements. Conforming to specifications tests includes determining the material's specific gravity, solubility, analysis by distillation, and softening point. The equipment for performing these tests is not included in the asphalt test set and is not normally available to the materials technician. However, these tests are described for information and, when the equipment is available, to identify the material beyond field identification, to furnish information for mix design, or to establish safe-handling procedures.

SPECIFIC-GRAVITY TEST (ASTMS C 127-88 AND C 128-93)

Specific gravity of a bituminous material is defined as the ratio of the weight of a given volume of the material at 77°F to the weight of an equal volume of distilled water at the same temperature. The results of a bitumen's specific-gravity test are used in the selection of the temperature-volume-weight correction factor to convert volumes to volume at 77°F. Space is provided on DD Form 1216 to make this determination (see *Figure 3-6*). Weigh an empty pycnometer (specific-gravity bottle), fill it with water, then reweigh. Empty the water from the bottle and add the bitumen. Weigh the pycnometer and bitumen. Add water to the same level as the start of the test and weigh the entire combination. Follow the procedure outlined on the form to compute the weight of water displaced by the bitumen and the bitumen's apparent specific gravity. The specific gravity of asphalt cements will usually be in the range of 1.00 to 1.06, with the higher values being characteristic of the harder materials. The specific gravity of an asphalt has little bearing on quality or other properties of the asphalt. However, the specific gravity is needed for other tests and computations. It is needed to adjust the specific gravity of the water bath in the ductility test. In acceptance and control testing on a job, it is used as a check on the uniformity of succeeding shipments of asphalt.



SPECIFIC GRAVITY OF BITUMINOUS MIX COMPONENTS

DATE 2 APR 1997

PROJECT HIGHWAY #203

JOB No. 47236

COARSE AGGREGATE	UNITS *(Grams)*			
MATERIAL PASSING 1" SIEVE AND RETAINED ON 16 SIEVE				
SAMPLE NUMBER	CA			
1. WEIGHT OF OVEN - DRY AGGREGATE	378.3			
2. WEIGHT OF SATURATED AGGREGATE IN WATER	241.0			
3. DIFFERENCE *(Line 1 minus 2)*	137.3			
APPARENT SPECIFIC GRAVITY, $G = \frac{\text{(Line 1)}}{\text{(Line 3)}}$	$\frac{378.3}{137.3}$ = 2.755			

FINE AGGREGATE	UNITS *(Grams)*			
MATERIAL PASSING NUMBER ____ SIEVE				
SAMPLE NUMBER	FRBS			
4. WEIGHT OF OVEN - DRY MATERIAL	478.8			
5. WEIGHT OF FLASK FILLED WITH WATER AT 20°C	678.6			
6. SUM *(Line 4 + 5)*	1157.4			
7. WEIGHT OF FLASK + AGGREGATE + WATER AT 20°C	977.4			
8. DIFFERENCE *(Line 6 minus 7)*	180.0			
APPARENT SPECIFIC GRAVITY, $G = \frac{\text{(Line 4)}}{\text{(Line 8)}}$	$\frac{478.8}{180.0}$ = 2.660			

FILLER	UNITS *(Grams)*			
SAMPLE NUMBER	LSD			
9. WEIGHT OF OVEN - DRY MATERIAL	466.5			
10. WEIGHT OF FLASK FILLED WITH WATER AT 20°C	676.1			
11. SUM *(Line 9 + 10)*	1142.6			
12. WEIGHT OF FLASK + AGGREGATE + WATER AT 20°C	973.8			
13. DIFFERENCE *(Line 11 minus 12)*	168.8			
APPARENT SPECIFIC GRAVITY, $G = \frac{\text{(Line 9)}}{\text{(Line 13)}}$	$\frac{466.5}{168.8}$ = 2.762			

BINDER	UNITS *(Grams)*			
SAMPLE NUMBER	6873			
14. WEIGHT OF PYCNOMETER FILLED WITH WATER	61.9595			
15. WEIGHT OF EMPTY PYCNOMETER	37.9215			
16. WEIGHT OF WATER *(Line 14 minus 15)*	24.0380			
17. WEIGHT OF PYNOMETER + BINDER	47.8617			
18. WEIGHT OF BINDER *(Line 17 minus 15)*	9.9402			
19. WEIGHT OF PYCNOMETER + BINDER + WATER TO FILL PYCNOMETER	62.1568			
20. WEIGHT OF WATER TO FILL PYCNOMETER *(Line 19 minus 17)*	14.2951			
21. WEIGHT OF WATER DISPLACED BY BINDER *(Line 16 minus 20)*	9.7429			
APPARENT SPECIFIC GRAVITY, $G = \frac{\text{(Line 18)}}{\text{(Line 21)}}$	$\frac{9.9402}{9.7429}$ = 1.020			

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*
SP4 Jones	SSG Schmidt	SGT. Allen

DD FORM 1216, DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE. USAPPC V1.00

Figure 3-6. Sample DD Form 1216

FLASH-POINT AND FIRE-POINT TESTS

These tests are applicable to asphaltic materials and are of some use in identifying these materials. Their greatest usefulness, however, is in determining safe heating temperatures. Material heated above its flash point presents a real danger, particularly if it is exposed to an open flame.

FLASH POINT AND FIRE POINT BY CLEVELAND OPEN CUP

Perform this test on all petroleum products except fuel oils and those having open-cup flash points above 175°F.

Equipment

Use the following equipment for this test:

- The flash-point apparatus (see *Figure 3-7*).
- A knife.
- A frying pan or copper beaker.
- A hot plate.
- A stopwatch.

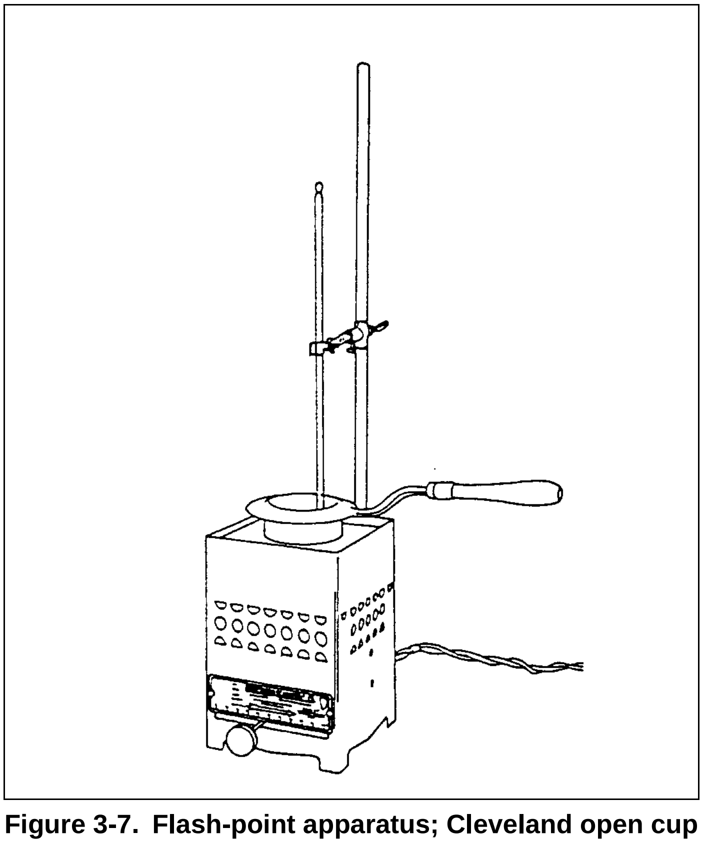


Figure 3-7. Flash-point apparatus; Cleveland open cup

Steps

Perform this test away from any bright light or shield the apparatus, if necessary. Clean the cup thoroughly before starting. Heat the bituminous material until it is fluid enough to pour into the cup. Do not move or disturb the cup and its contents within the last 30° before the expected flash point is reached. Prevent air movement or drafts across the specimen's surface.

Perform the following steps:

Step 1. Set up the open-cup apparatus.

Step 2. Adjust the thermometer in a vertical position, 1/4 inch above the bottom of the cup and about midway between the center and back of the cup.

Step 3. Fill the cup with the heated material until the top of the meniscus is exactly at the fill line. Let the material cool.

Step 4. Apply heat to the cup so the specimen's temperature is raised at a rate of 25° to 30° per minute until a temperature of about 100° below the probable flash point is reached.

Step 5. Reduce the heat and adjust it so that for the last 50° before the expected flash point the temperature rise will be not less than 9° and not more than 11° per minute. Use the stopwatch to regulate this rate. Failure to set the rate of rise between these limits will result in inaccurate readings.

Step 6. Adjust a test flame to 1/8 to 3/16 inch in diameter, the size of the comparison bead if one is mounted on the apparatus.

Step 7. Start at least 50° below the expected flash point and pass the test flame in a straight line across the center of the cup at right angles to the thermometer and level to the upper edge of the cup. The time for each pass should be no more than 1 second. Repeat the test-flame pass for each successive 5°. The flash point is reached when a flash (distinct flicker) appears at any point on the surface of the material. Read the thermometer at this time and record the temperature as the flash point.

Step 8. Continue heating at the same rate and applying the test flame at the same interval until the oil ignites and continues to burn for at least 5 seconds. Record the temperature at this point as the fire point.

Results

Duplicate tests on the same material by the same operator should not differ by more than 15°. Results by different laboratories should be considered suspect if the flash points differ by more than 30° and the fire points differ by more than 25°.

FLASH POINT BY TAG OPEN CUP (ASTM D 4552-87)

Perform this test on RC- and MC-asphalt cutbacks having a flash point below 200°F.

Equipment

Use the following equipment for this test:

- A flash-point tester.
- A tag open cup (see *Figure 3-8).*
- A hot plate.
- A thermometer (20° to 230°F in 1° divisions).
- A torch or test flame.

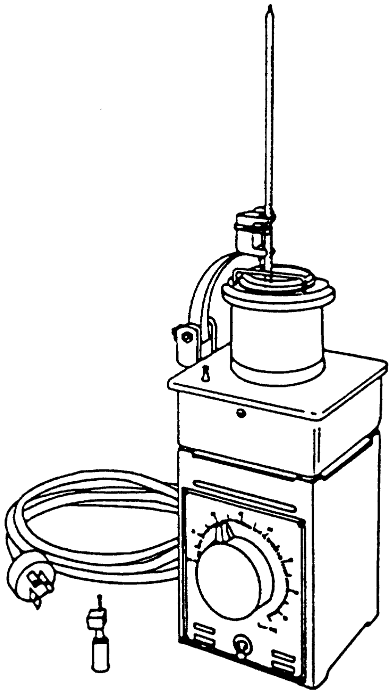


Figure 3-8. Flash-point apparatus; tag open cup

Steps

Set up the tester in a draft-free and dimly-lit location. Fill the copper water bath to 1/8 inch below the top of the glass cup (when the top is in place). The bath may have an overflow to control the water level. Clean and dry the glass cup and assemble the water bath. Perform the following steps:

Step 1. Place the thermometer vertically midway between the center and the outer edge of the cup and diametrically across from the guide wire. Set the bottom of the bulb about 1/4 inch above the bottom of the glass cup.

Step 2. Fill the glass cup with the sample to 5/16 inch below the edge.

Step 3. Place the guide wire in position, touching the rim of the glass cup.

Step 4. Adjust the heat for the sample temperature to rise at 2° ± 1/2° per minute. Stir thicker material occasionally.

Step 5. Adjust the test flame to not greater than 5/32 inch in diameter, the size of the comparison bead if one is mounted on the apparatus. (Some instruments have a 5/32-inch hole in comparison instead of the bead.)

Step 6. Remove any bubbles that may have formed on the surface before starting the flame test.

Step 7. Pass the flame at successive 2° intervals; pass the flame across the sample in a continuous motion, making each pass last 1 second.

Step 8. Record as the flash point the temperature at the time the test-flame application causes a distinct flash in the interior of the cup.

Step 9. Repeat the test using a fresh sample and starting at least 20° below the previously determined flash point.

Results

The results of two properly conducted tests by the same operator on the same asphalt should not differ by more than 18°F. The results of two properly conducted tests from two different samples of the same asphalt should not differ by more than 27°F.

PENETRATION TEST (ASTM D 5-86)

The penetration test determines the grade of an asphalt cement. Penetration is defined as the distance that a standard needle vertically penetrates a sample of the material under standard conditions of time (5 seconds), temperature (77°F), and loading (100 grams). The units of penetration are hundredths of a centimeter. Other conditions of temperature, load, and time that are used for special testing are given.

EQUIPMENT

Use the following equipment for this test:

- A hot-water bath.
- A copper beaker or frying pan.
- A stainless-steel box.
- An electric hot plate.
- A sieve pan, 8-inch diameter.
- An asphalt-testing penetrometer (see *Figure 3-9, page 3-28*)**.**
- A thermometer (66° to 80°F).
- A stopwatch.

STEPS

The described test depends on the water bath being maintained as closely as possible to the standard temperature of 77°F. Since the penetration of an asphalt cement varies with temperature, maintain the bath at 77°F. If this is

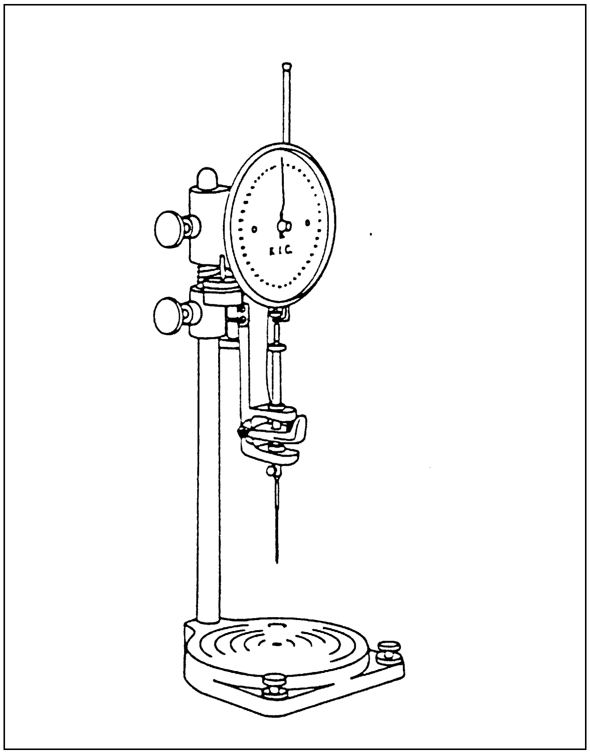


Figure 3-9. Asphalt penetrometer

impractical or less accuracy is acceptable, perform the test with the sample at room temperature. Perform the following steps:

Step 1. Melt the sample at the lowest possible temperature, using the electric hot plate and frying pan or copper beaker. Stir thoroughly until the sample is homogeneous and free of air bubbles.

Step 2. Pour the sample into the sample container (moisture-content box) to a depth of not less than 1.38 inch for the harder grades and 1.77 inch for the softer grades. Protect the sample from dust and allow it to cool in an atmosphere not lower than 65°F for 1 hour.

Step 3. Place the sample in its container in the sieve pan (or any other small, flat-bottomed pan or container that is 2 inches deep). Immerse the pan and sample for 1 hour in the water bath maintained at 77°F.

Step 4. Keep the sample in the pan filled with water from the water bath. The water should completely cover the sample container to maintain the temperature during the test. Place the transfer dish containing the sample in its container and the water on the stand of the penetrometer.

Step 5. Adjust the needle to make contact with the sample's surface of the sample. Place a light in a position so that the actual needle point and its image reflected on the specimen's surface are clearly defined. Contact may be judged with the point and its image touch on the surface.

Step 6. Note the reading of the dial or bring the indicator on the dial to zero.

Step 7. Release the needle for a period of 5 seconds. Read the distance penetrated on the thermometer. The needle and plunger are designed to have a standard weight of 100 grams. The dial is divided into 38 major divisions, each marked in 10 smaller subdivisions. Each of the smaller subdivisions represents a penetration by the needle of 1/100 centimeter. Thus, readings on the dial give penetration values without conversion.

Step 8. Test at least three points on the surface not less than 3/8 inch from the side of the container and not less than 3/8 inch apart. After each test, return the sample and pan to the water bath and carefully wipe the needle toward its point with a clean, dry cloth to remove all adhering asphalt.

RESULTS

The reported penetration is the average of at least three tests whose values do not differ by more than amounts shown in *Table 3-5*.

The penetration test, as pointed out previously, is used to classify asphalt cements for purchasing and identifying purposes, but it has other uses as well. It can be used to detect overheating or prolonged heating of asphalts in storage tanks. Also, when an asphalt is extracted from a pavement, the penetration test affords a means of estimating how the asphalt has changed with time and weathering.

Table 3-5. Penetration results

Penetration	0 to 49	50 to 149	150 to 249	250
Maximum difference between highest and lowest determination	2	4	6	8

DUCTILITY TEST

In the ductility test, dumbbell-shaped specimens of asphalt are molded under standard conditions. The dumbbell-shaped specimens are conditioned in a water bath to standard temperature (usually 77°F) then extended at the rate of 5 centimeters per minute until the threads connecting the two ends break. The difference in centimeters between the final length at the break and the original length is the ductility.

The ductility test is helpful in estimating an asphalt's ability to resist cracking and raveling. High-ductility asphalts have greater flexibility and tenacity. Conversely, low-ductility asphalts are considered more likely to crack under heavy loads or severe changes in temperature. Ductility is affected by various factors, such as method of refining and consistency. Blown asphalts (asphalts that have been hardened by blowing air through them, which causes oxidation) have low ductility. This is one reason why they are not used as paving asphalts. Within the group of asphalts produced by steam and vacuum distillation, ductility will vary according to the consistency at a given temperature. Note that the ductility test is also sensitive to other factors, such as imperfections in the specimens or impurities such as mineral filler in a sample of asphalt recovered from a pavement.

SOFTENING-POINT TEST

For the softening-point test, position a 3/8-inch-diameter steel ball on a brass ring filled with asphalt. Place the assembly in the beaker containing freshly boiled, distilled water and heat it slowly. As the asphalt becomes warmer, it begins to soften, and the weight of the ball forces the asphalt out of the ring. Record the temperature at which the asphalt touches the bottom of the beaker as the softening point.

The softening point is another consistency test and varies inversely with the penetration test. Like the penetration test, the softening-point test can be used to determine changes in an asphalt due to excessively high or prolonged heating. An abnormal increase in the softening point is an indication of excessive heating. The softening-point test is used in studies on asphalts recovered from pavements after extended service to determine effects of aging. If an asphalt shows an unusual increase in softening point, considerable aging and hardening have occurred.

VISCOSITY TESTS

There are two viscosity tests for identifying the qualities of bitumen—the Saybolt-Furor test and the kinematic-viscosity test.

SAYBOLT-FUROR TEST (ASTM D 244-89)

This test measures the time, in seconds, required to pass 60 cubic centimeters of asphalt-emulsion material, at a given temperature, through a tube of standard dimensions. Its purpose is to determine the viscosity of the material from which the spraying temperatures are established for field application of the bitumen.

KINEMATIC-VISCOSITY TEST (ASTM D 2170-85)

This test, like the Saybolt-Furor test, measures the time that a given amount of liquid-asphalt material will flow through a tube of standard dimensions under rigidly controlled conditions of temperature and pressure (or vacuum). The test establishes the viscosity of the liquid and, when correlated with the specific gravity of the material at the same temperature, results in a numerical designation called kinematic viscosity. The units used for kinematic viscosity are stokes (square centimeters per second) or centistokes (1/100 stoke). The kinematic-viscosity test requires special laboratory equipment that is not available in the field.

SOLUBILITY TEST (ASTM D 2042-81)

The solubility of asphalt cement can be determined using trichloroethylene. Trichloroethylene is toxic; therefore, protective equipment (including a ventilator, protective goggles, and protective gloves) must be worn while using it. Trichloroethylene waste is a hazardous waste and must be disposed of as such. Consult your installation environmental office for further guidance as to safe handling and disposal of trichloroethylene.

Dissolve 2 grams of the sample in 100 milliliters of solvent. Pour the mixture into a tared Gooch crucible and wash it through. Dry and weigh the crucible. The increase in the crucible's weight is the portion of the sample that is insoluble in the solvent.

The solubility test is a quality-control test used in specifications to ensure getting an asphalt cement that is not contaminated with mineral matter or is not improperly refined.

SPOT TEST

The spot test (often called the Oliensis spot test) is a solubility test that takes advantage of the selective solvent action of certain hydrocarbons; usually standard naphtha is specified. The test is applicable only to petroleum asphalts and should not be applied to natural asphalts containing nonbituminous matter insoluble in xylene.

Dissolve a 2-gram sample of asphalt in 10 milliliters of naphtha. Thoroughly stir the mixture with a stirring rod. Place a drop of the asphalt and solvent on a piece of Whatman No. 50 filter paper. Examine the filter paper after 5 minutes. If the drop forms a yellowish-brown stain with a darker nucleus, the test is positive. If the stain is uniformly brown, the test is negative. In the latter case, stopper the sample and set it aside for 24 hours, then repeat the test. If the stain with the darker nucleus again develops, the test is positive and is so reported. A negative result is an indication that the asphalt sample is a homogeneous material. A positive result may indicate that the sample is not a homogeneous material. A negative result is regarded as favorable to the sample; a positive result is unfavorable and may be used to reject the asphalt.

Considerable importance is attached to the spot test by some asphalt technologists and paving engineers. It is relied on principally as a means of detecting a cracked asphalt, which is a nonhomogeneous material not regarded as a good paving asphalt. The test can also be used to detect an asphalt that has been overheated or coked. Overheating or coking can occur in storage tanks or when the asphalt is added to aggregate that is too hot. In such cases, the asphalt is no longer a homogeneous substance, and the spot test will often show a nonuniform (positive) stain.

THIN-FILM OVEN TEST (ASTM D 1754-87)

The thin-film oven test was developed to overcome the deficiencies of the standard loss-on-heating test. The test uses the same oven as the loss-on-heating test except for a modification of the rotating shelf. The test has the same period and temperature of heating (5 hours at 325°F). The significant difference is in the sample. Instead of the 50-gram sample in a 3-ounce ointment can, a 1/8-inch-thick layer of asphalt is poured into a wide, shallow, circular aluminum dish. The dish has a flat bottom and is 5 1/2 inches in diameter and 3/8 inch deep. Weigh the sample before and after the heating period and compute the loss in weight. A penetration or viscosity test may also be conducted on the sample after the heating period to evaluate changes in the asphalt.

SECTION V. AGGREGATE AND FILLER TESTING

The aggregate transmits the load, takes the abrasive wear of traffic, and provides a nonskid surface. Desired aggregate characteristics include angular shape, rough surface, hardness, and gradation. Some of these characteristics (such as shape, surface, and cleanliness) are determined visually. Durability and hardness cannot be seen but require knowledge based either on experience or some form of abrasion testing. Rocks that soak up water will eventually reach the condition where the binder is forced from the surface pores and the cementing action breaks down. Gradation may be established to some extent by observation. However, the grain-size distribution (sieve) tests will define the particle sizes and amounts much more accurately.

In bituminous paving, the aggregate constitutes the bulk of the pavement. Common practice subdivides the bituminous aggregates into a coarse aggregate, a fine aggregate, and a mineral filler. The No. 10 sieve separates coarse from fine aggregate, and the No. 200 sieve size is the lower limit for fine aggregate. Usually 65 percent or more of the mineral filler will pass the No. 200 sieve. The distribution of the different sizes determines how many voids will remain and helps determine how much bitumen will be needed.

Bituminous pavement specifications define acceptable gradation limits. The bitumen content for the mix is then determined from the trial-mix properties that are defined in the specifications.

SIEVE ANALYSIS

A sieve analysis of the aggregates to be used in a paving mixture is required to determine the particle-size distribution.

MINERAL FILLER (ASTM D 242-85)

In bituminous paving, particles finer than the No. 200 sieve are referred to as a mineral filler. To measure the amount of filler in a selected sample, perform a washed sieve analysis using the No. 40 and No. 200 sieves. Discard the material that has passed the No. 200 sieve, then return the material on the sieves to the original washed sample, oven-dry it, and weigh it. The amount of mineral filler is computed as—

$$\text{percent finer than No. 200} = \frac{\text{original dry weight} - \text{washed dry weight}}{\text{original dry weight}} \times 100$$

FINE AND COARSE AGGREGATE (WASHED) (ASTMS D 1073-88, 448-86, AND 692-88)

When definite amounts or limits of coarse and fine aggregates are specified, the sieve analysis with prewashing must be made using suitable sieves. If no limits have been designated, select a range of sieves to give adequate information about gradation. Record the results on DD Form 1206 (see *Figure 2-39*, *page 2-74*) and plot them as a gradation curve on DD Form 1207 (see *Figure 2-42*, *page 2-78*). When testing aggregates, obtain a representative sample by quartering, if necessary. The minimum size of the sample depends on the maximum size of particles in the material (see *Table 3-6*).

SPECIFIC GRAVITY

The specific gravities of aggregates and mineral filler used in bituminous paving mixtures are required to compute the percent of air voids and percent of voids filled with bitumens. Apparent specific gravity used with aggregate blends showing water absorption of less than 2 1/2 percent is based on the apparent volume of the material, which does not include those port spaces in

Table 3-6. Aggregate sizes and weights

Approximate Aggregate Size	Approximate Dry Weight of Sample (in Grams)
Fine Aggregate	
At least 95 percent finer than No. 10*	100
At least 90 percent finer than No. 4 and more than 5 percent coarser than No. 10*	500
Coarse Aggregate	
3/8-inch maximum size	1,000
1/2-inch maximum size	2,000
3/4-inch maximum size	3,000
1-inch maximum size	4,000
1 1/2-inch maximum size	5,000
2-inch maximum size	8,000
2 1/2-inch maximum size	12,000
3-inch maximum size	18,000
3 1/2-inch maximum size	25,000
*** ASTM specifies Numbers 8, 16, 30, and 50 instead of Numbers 10, 40, and 60. Tests based on ASTM standards identify the appropriate sieve sizes.**	

the aggregate which are permeable to water. Bulk-impregnated specific gravity is used for aggregate blends with 2 1/2 percent or greater water absorption. The methods for determining absorption of aggregates are described in Chapter 4 of this manual.

APPARENT SPECIFIC GRAVITY OF COARSE AGGREGATE

Apparent specific gravity can be determined using the method described for apparent and bulk specific gravity, or it may be determined using the Dunagan apparatus furnished with the concrete test set. Additional information for course-aggregate testing can be found in ASTM C 127-88.

Equipment

Use the concrete test set to determine the apparent specific gravity. The test set includes—

- The Dunagan apparatus (see *Figure 3-10, page 3-34*).
- Sieves (2-, 1 1/2-, 1/2-, and 3/8-inch and Numbers 4, 10, 40, 60, 80, 100, and 200).
- Evaporating dishes.
- An electric oven.
- Pans.

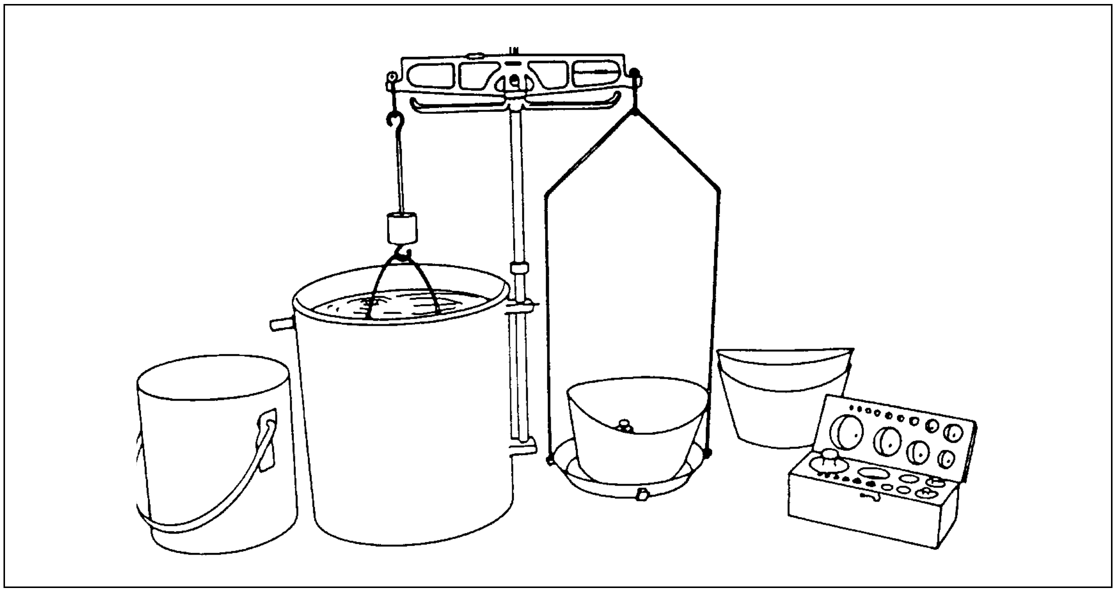


Figure 3-10. Specific-gravity test; Dunagan apparatus

Steps

Perform the following steps to determine the apparent specific gravity:

Step 1. Select about 5,000 grams of aggregate from the sample, not including particles smaller than the 3/8-inch sieve.

Step 2. Wash the aggregate to remove any dust or other coating and dry it to constant weight in the oven. Record the total weight of the oven-dry aggregate on DD Form 1216 (see *Figure 3-6, page 3-23*).

Step 3. Immerse the aggregate in water at 59° to 77°F for a period of 24 hours.

Step 4. Soak the sample and place it in a copper bucket filled with water. Turn the bucket and aggregate sharply back and forth to remove any air.

Step 5. Suspend the bucket from the brass hanger and bring the water level to the overflow pipe.

Step 6. Determine the submerged weight using weights placed in the scoop on the right-hand pan. Record the weight.

Calculations

The calculations required to determine the apparent specific gravity of coarse aggregate are shown on DD Form 1216 and are self-explanatory (see *Figure 3-6*).

APPARENT SPECIFIC GRAVITY OF FINE AGGREGATE (CALIBRATED FLASK)

Perform the procedure below to determine the apparent specific gravity when a calibrated flask is available.

Equipment

Use the following items (see *Figure 3-11*) to determine the apparent specific gravity of fine aggregate (calibrated flask):

- A balance; 2,000 grams, sensitive to 0.1 gram.
- An evaporating dish.
- A battery filler.
- A volumetric flask; 500-milliliter.
- An electric oven.
- A pan.
- A thermometer (0° to 300°F, in 1° gradations).
- An absorbent paper or cloth.

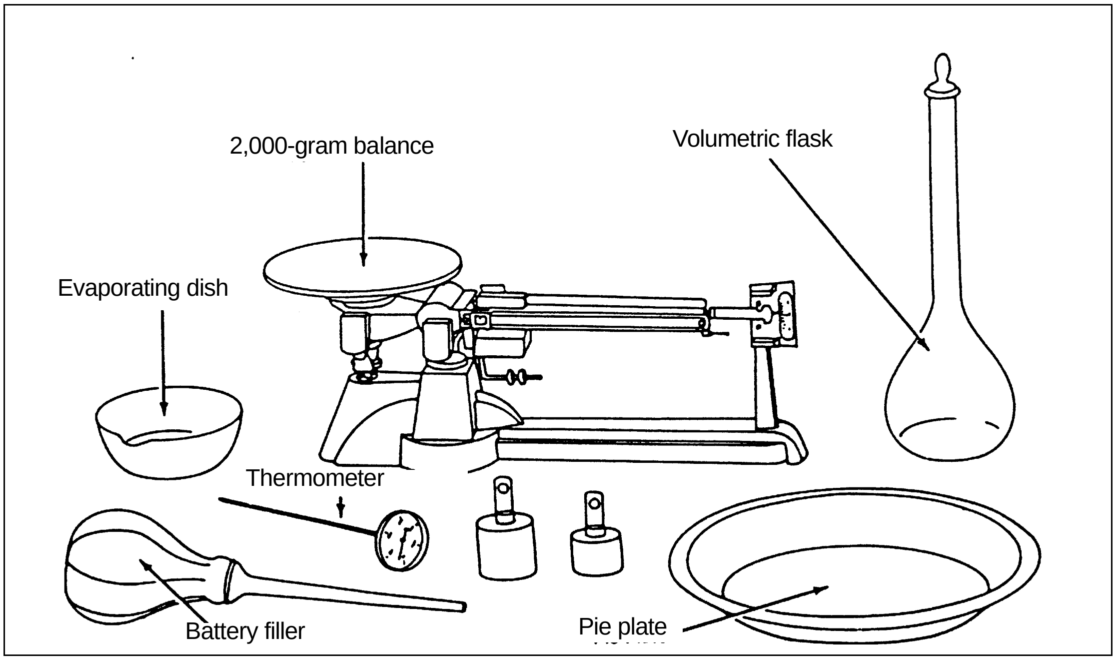


Figure 3-11. Calibrated-flask apparatus for determining apparent specific gravity of fine aggregate

Steps

Perform the following steps for particles finer than the No. 4 sieve (the data is recorded on DD Form 1208—see *Figure 2-36, page 2-65*):

Step 1. Calibrate a 500-milliliter volumetric flask.

Step 2. Dry a representative sample weighing about 500 grams to constant weight in the electric oven.

Step 3. Determine the oven-dry weight of the cooled sample and record in on DD Form 1208 (see *Figure 2-36).*

Step 4. Transfer the sample to the 500-milliliter flask, being careful not to lose any of the material.

Step 5. Add clean water until the level just reaches the neck of the flask. Allow the sample to soak for 24 hours.

Step 6. Hold the flask containing the soaked sample by the neck, and roll it back and forth on a smooth surface until air bubbles stop coming from the sample (see *Figure 3-12*).

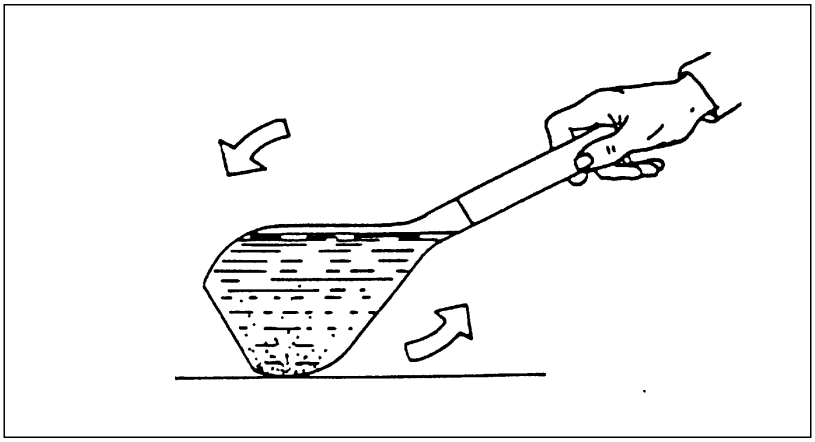


Figure 3-12. Manipulation of calibrated flask to remove air

Step 7. Use the battery filler to bring the water level up in the neck of the flask until the bottom of the meniscus coincides with the calibration mark on the flask. Use absorbent paper or cloth to remove any drops of water from the inside of the neck and on the outside of the flask.

Step 8. Determine the weight of the flask, aggregate, and water. Record the weight on DD Form 1208 (see *Figure 2-36, page 2-65*).

Step 9. Measure and record the temperature.

Calculations

The calculations for apparent specific gravity are the same as those indicated for soil in Figure 2-36.

APPARENT SPECIFIC GRAVITY OF FINE AGGREGATE (UNCALIBRATED FLASK)

Perform the procedure below to determine the apparent specific gravity when a calibrated flask is not available.

Equipment

Use the same equipment as for the calibrated-flask test, with the addition of a water bath maintained at 68°F. Do not use this procedure unless the temperature can be maintained.

Steps

Perform the following steps to determine the apparent specific gravity:

Step 1. Obtain a representative sample of aggregate passing the No. 4 sieve weighing about 500 grams. Dry it to constant weight in the electric oven maintained at a temperature of 230°F ± 9°.

Step 2. Obtain and record the dry weight after the sample has cooled in air.

Step 3. Transfer the sample to the flask, being careful not to lose any of the material.

Step 4. Add clean water until its level just reaches the neck of the flask. Allow the sample to soak for 24 hours.

Step 5. Roll the flask back and forth on a smooth surface until air bubbles stop coming from the sample (see *Figure 3-12*).

Step 6. Use the battery filler to bring the water level up in the neck of the flask to slightly above the calibrated mark.

Step 7. Place the flask with water and aggregate in the water bath maintained at a temperature of 68°F. Use the glass thermometer to check the temperature of the water in the flask from time to time.

Step 8. Bring the water in the flask to a uniform temperature of 68°F. Use the battery filler and absorbent paper to adjust the bottom of the meniscus to coincide with the calibration mark. Remove any drops of water inside the neck of the flask.

Step 9. Remove the flask from the bath and dry the outside thoroughly. Determine and record the weight of the flask plus the aggregate, plus the water at 68°F. It does not matter if the level of the water in the neck of the flask changes after removal from the bath. The proper adjustment was made at 68°F, and the total weight is not affected by the subsequent change in volume.

Step 10. Repeat the procedure in steps 6 through 9 above using water only. Enter the weight of the flask filled with water at 68°F on DD Form 1216 (see *Figure 3-6*, *page 3-23*). This needs to be done only once for a given flask; this value can be tabulated and used in subsequent tests. The weight of the flask filled only with water at 68°F must be known.

Calculations

Indicate the calculations necessary to determine the apparent specific gravity of fine aggregate using an uncalibrated flask on DD Form 1216 (see *Figure 3-6*).

SPECIFIC GRAVITY OF BULK-IMPREGNATED AGGREGATE

This test is used for determining the specific gravity of the blended aggregates (including filler) used in hot asphaltic mixtures. This method is to be used only when the water absorption for the entire blend of aggregate selected for the job-mix formula exceeds 2 1/2 percent. The method is not applicable to determine specific gravity of mineral filler except when included in the blended aggregate. See Military Standard (MIL-STD) 620A, method 105, for additional testing details.

Equipment

Use the following items to perform this test:

- The Dunagan apparatus.
- An electric oven (sensitive to ± 5° in the range of 275° to 325°F).
- Gallon-capacity pails.
- A balance; 5-kilogram capacity sensitive to 0.1 gram.
- Baking pans.
- A heavy sheet-metal strip for stirring the contents of the pail.
- A wire handle. **NOTE: A wire handle is convenient for handling the pail, but is not essential, since the container will be placed in the copper bucket of the Dunagan apparatus to determine the weight submerged in water. A No. 10 can (an empty fruit or vegetable can) with the top smoothly cut out is satisfactory, but care must be taken to eliminate air trapped under the bottom when the can is submerged.**

Samples should consist of 1,500 grams of blended aggregate (ensuring that the sample represents prototype grading) and 85 to 100 penetration-grade asphalt cement.

Steps

Perform the following steps to determine the specific gravity of bulk-impregnated aggregate:

Step 1. Dry the aggregate sample to constant weight at a temperature not less than 230°F nor greater than 290°F. After cooling the sample in air, weigh it to the nearest 0.1 gram.

Step 2. Heat the asphalt to 280°F ± 5°, using care to ensure that the temperature never exceeds 285°F. Add a sufficient amount to the 1-gallon pail to fill it about 1/3 full.

Step 3. Insert the sheet-metal stirrer and allow the pail and its contents to cool to room temperature. Allow 8 hours for cooling (preferably overnight).

Step 4. Weigh the pail plus the asphalt and stirrer in air at room temperature and submerged in water at 72°F ± 2°.

Step 5. Place the pail of asphalt with stirrer and also the sample of aggregate in an oven at 280°F ± 5° until temperatures of both are equalized. (A minimum of 4 hours is usually required.)

Step 6. Remove the aggregate and asphalt from the oven and gradually add aggregate to the asphalt, stirring thoroughly. After all of the aggregate is added, continue stirring until the total elapsed time from the start of mixing to the end of stirring is 2 minutes. During the cooling period, apply a flame to the surface to remove air bubbles. Cool the sample to room temperature (preferably overnight).

Step 7. Weigh the sample in air and in water at 72°F ± 2°.

Calculations

Calculate the bulk-impregnated specific gravity as follows:

$$\text{bulk-impregnated specific gravity} = \frac{A}{(D - E) - (B - C)}$$

where—

A = *weight of oven-dry aggregate, in grams*

B = *weight of pail + stirrer + asphalt in air, in grams*

C = *weight of pail + stirrer + asphalt in water, in grams*

D = *weight of pail + stirrer + asphalt + aggregate in air, in grams*

E = *weight of soil + stirrer + asphalt + aggregate in water, in grams*

Duplicate determinations should check within 0.04 gram. If the values are within the 0.04 tolerance, use an average value. If the initial duplicate tests are not within the 0.04 tolerance, repeat the tests in duplicate. If the second set of test values is within the tolerance, discard the first two test values and use an average value of the second two tests.

SPECIFIC GRAVITY OF MINERAL FILLER

The specific gravity of mineral filler used in bituminous mixes is required for void computation. The methods described in the specific-gravity test apply (including procedures, calibration, testing, and calculations). Note that when the bulk-impregnated specific gravity is used, the mineral filler is included in the blended aggregate. Details can be found in ASTM D 854-92.

LOS ANGELES ABRASION TEST

The Los Angeles abrasion test requires a special machine consisting of a revolving drum rotated at the rate of 30 to 33 revolutions per minute (rpm) by an electric motor. Inside the drum is a shelf that picks up the aggregate sample along with a charge of steel balls and drops them together on the opposite side of the drum. Details can be found in ASTM C 131-89.

Conduct the test using various numbers of drum revolutions, sizes of samples, and numbers of steel balls, depending on the grading of the sample. For a sample that passes the 3/4-inch sieve and is retained on the 3/8-inch sieve, use 5,000 grams of material with 11 balls and 500 revolutions. Wash and dry the sample to constant weight before placing it in the machine. After the 500 revolutions are completed, remove the sample from the drum and sieve it over a No. 12 sieve. Wash, dry to constant weight, and weigh the portion retained on the No. 12 sieve. The difference between the original weight and the final weight of the sample is expressed as a percentage of the original weight of the sample

Job specifications usually require that the loss in weight as determined in the Los Angeles abrasion test shall not be greater than 40 percent for pavement aggregates and 50 percent for base and subbase aggregates.

SECTION VI. BITUMINOUS-MIX DESIGN

Hot-mix bituminous concrete for pavements is a mixture of blended aggregate filled with bituminous cement binder. The materials are heated when mixed so that the bitumen becomes fluid and thoroughly covers the aggregate particles. The design of a bituminous concrete mix is an economical blend and gradation of aggregates with bituminous cement. This produces a mixture that is durable, has the stability to withstand traffic loads, and is workable for placement and compaction with the construction equipment available.

The procedures described in this section are performed during the design of a hot-mix bituminous concrete. They include testing, plotting the results on graphs, and checking the readings against values from the design tables. Testing of the ingredients and the mix is started before and continued throughout the paving operations. FM 5-430-00-1 covers the design considerations in more detail. The paving operations and the blending and bitumen-content criteria are explained in TM 5-337. The testing phases are described in this manual.

The selection of the mix ratios of materials is tentative. The bitumen should be the same as the one used in the construction. The aggregates and fillers must meet definite requirements. In general, several blends should be considered for laboratory mix-design tests.

At times it will be necessary to shorten the design procedures to expedite military construction. Suggestions for expediting design mix are given at the end of this chapter. The final step is the preparation of a job-mix formula to be furnished to the construction unit.

HOT-MIX DESIGN CONSIDERATIONS

The objective of designing a hot mix is to determine the most economical blend of components that will produce a final product that meets specifications.

STEPS

Perform the following steps for determining the most economical blend of components:

Step 1. Prepare a sieve analysis of the aggregate available.

Step 2. Determine the aggregate blend that will achieve the specified gradation (see TM 5-337). Plot the selected blend proportions on a graph with the allowable limits to see that the blend conforms.

Step 3. Determine the specific gravity of the components

Step 4. Use selected percentages of bitumen (see TM 5-337), make trial mixes, and determine the mix's design test properties.

Step 5. Plot the test properties on individual graphs using the selected bitumen percentages. Draw smooth curves through the plotted points.

Step 6. Select the optimum bitumen content for each test property from the curves (as explained in the criteria tables).

Step 7. Average the bitumen content values from step 6 and from the graphs. Read the test-property value corresponding to this average.

Step 8. Check these values read in step 7 with the satisfactoriness-of-mix criteria.

VARIABLES

Gradation specifications are based on limits established as satisfactory by the Corps of Engineers. Within these limits, the following variables will affect the final mix design:

- The use of the mix (surface course, binder course, or road mix).
- The binder (asphalt, cement, or tar).
- Loading (low tire pressure [100 psi and under] or high tire pressure [over 100 psi]).
- The maximum size of the aggregate (in a stockpile or based on the thickness of the pavement course).

BLENDS

Once the gradation specifications have been selected, check the available materials to determine how to proportion the blend to meet these specifications. Study the sieve analysis of the available aggregates, and compute a series of trial blends. Make any necessary adjustments of the blend after testing the design and prepared mix. The considerations for establishing and adjusting the blend are explained in TM 5-337.

OPTIMUM BITUMEN CONTENT

The determination of optimum bitumen content is based on the gyratory test method or the Marshall test method.

GYRATORY TEST METHOD

The purposes of the gyratory test are to—

- Prepare specimens by kneading compaction at a pressure equal to the tire pressure for which the pavement is designed.
- Indicate optimum bitumen content directly by plasticity indicators called gyragraph recordings and by direct readings of shear resistance. The gyragraph recordings begin to widen, and the shearing resistance begins to decrease when the maximum permissible bitumen content is exceeded.
- Measure shear at the applied tire pressure which is used to calculate a shear-strength factor. This factor is used to predict whether the paving mixture will withstand the proposed tire-contact pressure.
- Obtain (by direct measurement) unit weight values required to minimize settlement under the design loads. Unit weight calculations are based on direct measurement of the sample height and the known sample diameter.

Definitions

The following terms are used for the gyratory test:

- Gyragraph. A recording of shear strain experienced by the bituminous mixture during the compaction test.
- Gyratory angle. A measure of the magnitude of the gyratory strain. Three pertinent angles are defined as follows:
  - Initial gyratory angle or shear strain (machine setting) 0_o.
  - Minimum gyratory angle or shear strain (minimum gyragraph band width) 0_1.
  - Maximum gyratory angle or shear strain (maximum gyragraph band width) 0_{max}.
- Gyratory stability index (GSI). The ratio of the maximum gyratory angle to the minimum gyratory angle.
- Gyratory compactibility index (GCI). The ratio of the unit mass (total mix) at 30 revolutions of the gyratory testing machine (GTM) to the unit mass (total mix) at 60 revolutions of the GTM.
- Gyratory shear strength (S_G). The shear resistance of the specimen under the imposed loading conditions.
- Gyratory shear factor (GSF). The ratio of the measured gyratory shear strength to the approximate theoretical maximum induced shear stress.

Equipment

Use the following items to perform the gyratory test:

- A GTM and appurtenances (the primary equipment for this test).
- Spacer blocks. Two metal spacer blocks used to zero the equipment that measures the specimen height. They are 2 inches in diameter with one each of the following lengths: 2.50 ± 0.005 inches and 3.75 ± 0.005 inches.
- An oven, thermostatically controlled to maintain the required temperature within 5°.
- An electric hot plate.
- An electric mixer. A heavy-duty commercial food mixer complete with mixing bowl and beaters.
- Balances. Two balances are required, one having a capacity of 5 kilograms or more, sensitive to 1.0 gram; and one having a capacity of 2 kilograms or more, sensitive to 0.1 gram.
- Thermometers, armored glass or dial-type with metal stems are recommended. A range from 50° to 400°F with sensitivity to 5° is required.
- A metal beaker, about 1,000-milliliter capacity.

- Tongs for the beaker.
- A metal cooking pan, 12 inches in diameter.
- A kitchen scoop.
- Paper disks, 4 inches and 6 inches in diameter.
- Work gloves.
- Rags or paper towels.
- Kerosene (asphalt solvent).
- Creosote (tar solvent).

Steps

The gyratory method is applicable to mixtures containing asphalt cement, asphalt cutback, asphalt emulsion, tar or rubberized tar, and aggregate up to 1-inch maximum size in the 4-inch-diameter specimen and 1.5-inch maximum size in the 6-inch-diameter specimen. Perform the following procedures for the gyratory test:

Select the Bitumen Content

The bitumen content is expressed as a percent of the mixture's total weight. Using the procedures outlined in the following paragraphs, conduct preliminary tests with one specimen each at a minimum of three bitumen contents: one above, one below, and one at the estimated optimum. Once the range of bitumen contents for the design test has been selected, test at least four specimens at each of the selected bitumen contents. The formulas listed in the surface-area method may be used to make a rough estimate of optimum bitumen content. The GTM indicates excessive bitumen by the widening of the gyragraph and the reduction in upper-roller pressure during the compaction test. In these preliminary tests, bracket the optimum bitumen content by tests in which these phenomena occur at the higher bitumen contents.

The incremental change of the bitumen content should be generally 0.5 percent. For extremely critical mixes, lower the incremental change of bitumen content to 0.3 percent. For highly absorptive aggregate, increase the incremental change of bitumen content to 1.0 percent. The gyratory method does not use voids criteria to select the optimum bitumen content. However, the mix must be sufficiently dense (low in voids) to widen the gyragraph and reduce the roller pressure since these indicate overfilled voids. For this reason, the gyratory method selects mixtures with the most desirable durability properties, the maximum permissible bitumen content, and the minimum acceptable voids content.

Prepare the Aggregate

Procedures for determining particle-size distribution and blending to meet specification requirements have been discussed. The amount of aggregate required is discussed below.

Prepare the Mixture

For mixes employing penetration grades of bitumen, the temperature of the aggregate and asphalt at the time of mixing should correspond to the temperatures anticipated at the plant while manufacturing the paving mix. These temperatures will be in the range of 200°F for rubberized tar mixes.

For tar and rubberized tar, the temperature of the aggregate and the binder at the time of mixing should correspond to the temperature to be used at the plant during manufacture of the paving mix. This temperature will usually not exceed 225°F for tar mixes and 250°F for rubberized tar mixes.

For mixtures employing liquid asphalts (cutbacks or emulsions), the aggregate should be dried to the moisture content expected during construction (up to a maximum of 2 percent by dry weight). Combine the liquid asphalt with the aggregate at the temperature recommended for field application. Following mixing, cure the loose mixture in a ventilated oven maintained at 140°F ± 5° for at least 12 hours before compaction at this temperature. Occasionally stir the mix during curing to accelerate the loss of volatiles.

Combine the aggregates into batches large enough to make specimens about 2.50 inches long in the 4-inch-diameter mold and 3.75 inches long in the 6-inch-diameter mold. For normal aggregates, this will require about 1,200 grams for the 4-inch-diameter specimen and about 4,050 grams for the 6-inch-diameter specimen. Heat the aggregate to the proper mixing temperature, then weigh the required amount of bitumen at the proper temperature into the aggregate mixture. Mix the aggregate and bitumen as thoroughly and rapidly as possible. Mechanical mixing is recommended.

Perform the Compaction and Shear Test

For this test, set the initial gyratory angle, 0_o, at 1°. Roller positions 2 and 4 are used to set the initial gyratory angle, T. Use a trial batch of mix in making the 0_o adjustment. Ensure that the specimen molds are thoroughly clean and free of defects. Excessive wear or grooving in the molds in the area of contact with the upper and lower plates will have an adverse effect on the compaction as well as the gyragraph (shear strain) recording. (Instructions for the compaction temperatures for the laboratory specimens are presented above.) Set the GTM at 140°F at least 15 minutes before starting the compaction test. Preheat the mold and baseplate at 140°F. Place paper disks in the bottom of the mold and on top of the loose mix to prevent the bitumen from adhering to the end plates. Place the entire batch in the mold. Avoid hand troweling or tamping so that the compaction process will be completely controlled mechanically and will be the most precise and reproducible. Place the wall-friction yoke in position, then use the mold-carrying tray to load the mold containing the mixture into the machine. Raise the ram and use just enough pressure to retain the specimen while tightening the front of the mold chuck securely in position. When the mold chuck is securely tightened, increase the vertical pressure to the full compaction test pressure. Bring the gyragraph recorder pin into contact, actuate the roller carriage, and continue until 29 revolutions have been applied. After 29 revolutions, stop the carriage and record the specimen height and roller-pressure readings at three positions: 1, 3, and 4 (29 to 30 revolutions), thus completing 30 revolutions. Continue to apply additional revolutions until a total of 59 are reached. Again, record the height and roller pressure readings at three positions: 1, 3, and 4 (59 to 60 revolutions), thus completing 60 revolutions.

Perform the Wall-Friction Test

Immediately following the compaction and shear test, lower the vertical ram slightly to relieve the pressure on the bottom roller. Lower the bottom roller enough turns to ensure that it will be out of contact with the mold chuck. (Keep account of the exact number of turns so that the roller can be reset to exactly the same position.) Reapply the compaction pressure to the ram, and cycle the roller carriage several times to level the specimen. With the compaction pressure still acting on the specimen, loosen the mold-chuck bolts and remove the front of the chuck so that the specimen mold is no longer restrained by the chuck. Install the two wall-friction apparatus jacks beneath the wall-friction yoke.

With the vertical load acting on the specimen, determine the force required to overcome wall friction and move the mold by observing the pressure gauge of the jack while actuating the jack. The pressure reading will increase with each thrust of the jack until there is enough force to move the mold. The pressure reading will then stabilize to about the same minimum value after each thrust of the jack. Record the low reading of the wall-friction gauge in the space provided. Remove the test specimen from the GTM immediately after the wall-friction test is completed, and bring the lower roller back to the 1° setting so that the machine is ready for the next test specimen.

Calibrate the Machine

When conducting shear tests with the GTM, it is necessary to make machine corrections for the gyratory shear value S_G. For this correction, shift the Mohr's diagram for test results on a cohesionless material enough to cause the envelope to pass through the origin of the Mohr's diagram. The cohesionless material used for this test is standard dry ottawa sand, all passing a No. 20 sieve and retained on a No. 40 sieve. A correction is needed for each combination of compaction pressure and gyratory angle used in the GTM compaction and shear tests. This correction is determined only once for any combination of vertical pressure and gyratory angle. The dry ottawa sand is first compacted under the same pressure, gyratory angle, and number of revolutions as scheduled for the compaction and shear tests on a given bituminous mixture. The shear test on the dry sand is then conducted for at least three different magnitudes of vertical pressure: starting at some lower value; including an intermediate value; and finally, using the same value that was used for compaction. The roller carriage is cycled once after each incremental adjustment in vertical pressure and before reading the upper roller values under that pressure.

Calculations and Presentation of Results

Perform the following calculations for the gyratory method:

Compaction Calculations

Calculate the following compaction properties for each specimen:

- Unit mass, total mix.

- Unit mass, aggregate only.
- GCI.

Shear Calculation

Calculate the following gyratory shear properties:

- GSI.
- Gyratory shear strength (S_G).
- GSF.

Graphical Presentation

For convenience of analysis, the calculations above are plotted against the bitumen content. The graphs may be to any convenient arithmetic scales.

Application of the Gyratory Method

The bitumen content must be as high as possible when using the gyratory method to select the optimum bitumen content and judge the satisfactoriness of the mix. The GSI must not be significantly greater than 1, and the GSF at this bitumen content must exceed 1.

MARSHALL TEST METHOD (ASTM D 1559-89)

The purposes of the Marshall test method are to—

- Prepare specimens by drop-hammer compaction. The number of drop-hammer blows used on the specimens is based on empirical correlations with two different traffic conditions: 50 blows on each end of the specimen for tires with less than 100-psi pressure and 75 blows on each end of the specimen for tires with greater than 100-psi pressure.
- Find the optimum bitumen content by averaging four measured properties: the peak of the compaction curve, the peak of the stability curve, the percent of the voids of the total mix at a specified amount, and the percent of the voids filled with bitumen at a specified amount. These values for total mix and bitumen vary with the aggregate's gradation, absorption properties, and compaction effort. There are ten separate sets of criteria to cover these variations. A special exception is made to use only voids total mix when the overall average falls outside the voids total-mix limits. This occurs for open-graded mixes or highly porous aggregates. There are no standards for sand-asphalt mixes at 75-blow compaction.
- Measure maximum breaking load (stability) and corresponding deformation (flow) for specimens prepared according to the compaction procedure. These values of stability and flow are empirically correlated for conditions outlined in the paragraph above.
- Obtain the unit weights of specimens from calculations based on weighing the specimens in air and in water. Porous specimens are coated with paraffin before weighing them in water.

Equipment

Use the following items to perform the Marshall test (see *Figure 3-13*):

- A mixing pan.
- A hot plate.

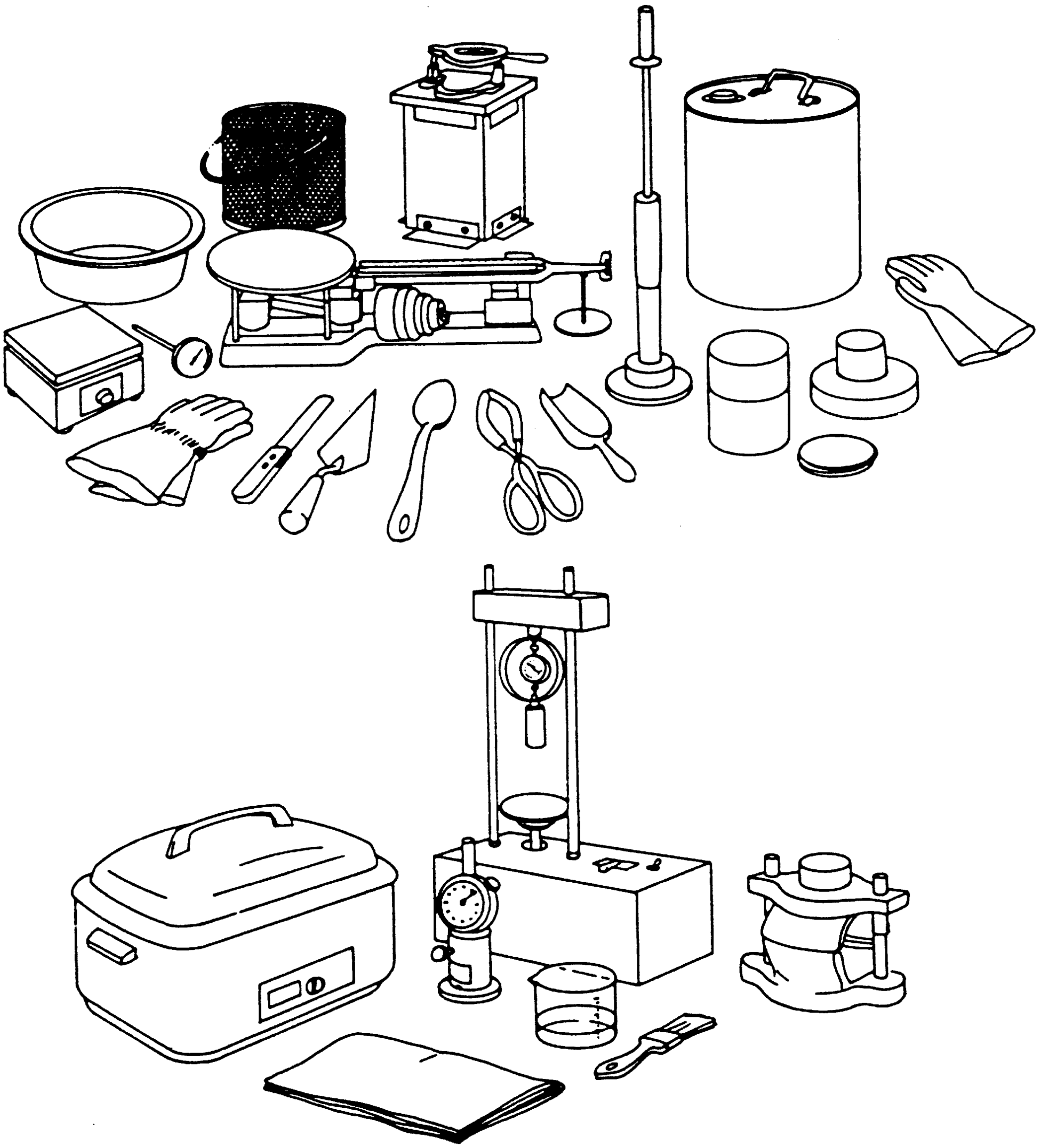


Figure 3-13. Apparatus for bituminous mix design—Marshall method

- Insulated gloves.
- A laboratory spatula.
- A trowel.
- A spoon.
- Insulated tongs.
- A scoop.
- A compaction hammer.
- A compaction mold.
- An extractor.
- Chemical-resistant gloves.
- Trichloroethane technical.
- A holder, compaction mold.
- A compaction pedestal.
- A triple-beam scale.
- A wire basket.
- A thermometer.
- A hot-water bath.
- A Marshall stability-testing machine.
- A flow indicator.
- Lab towels.
- A beaker with oil.
- A brush.
- A stability mold.

Steps

The Marshall method is applicable to hot-mix mixtures using penetration grades of asphalt cement and containing aggregate with not more than 10 percent of the aggregate larger than the 1-inch sieve. The procedure for handling large aggregate as well as for cold mix is described later. Use the following procedure and example to determine the optimum asphalt content for one particular blend of aggregates:

Select the Bitumen Content

Start the laboratory tests by estimating the optimum amount of bitumen for the aggregate to be tested. Continue the tests until results show at least two bitumen contents above and two below what would be the optimum content. Since the optimum is not determined until after the results are plotted, specimens are usually prepared for each of six different contents. Prepare at least three specimens at each bitumen content. You may use 1 percent incremental changes of bitumen content for preliminary work. However, use increments of 1/2 or 1 percent for final tests. Express the percent of bitumen as a percent of the total weight of the batch of paving mix. The procedure for establishing the estimated optimum content is explained in TM 5-337.

Prepare the Aggregate

Procedures for determining the particle-size distribution and blending to meet specification requirements have already been presented. Dry 50 pounds of the selected blend to constant weight at 221° to 230°F. This amount of material provides for 1,220 grams per specimen for 18 specimens (three in each of six bitumen contents) with allowance for some loss. The Marshall method uses a 4-inch-diameter mold and is not applicable, without special handling, when more than 10 percent of the aggregate is larger than 1 inch. The previously given total amounts of aggregate required assume less than 10 percent of the particle will exceed 1 inch. (Special handling of oversize aggregates is covered later.)

Prepare the Hot Mixture

Heat the bitumen and the aggregates to specified temperatures for mixing. These temperatures are based on the bitumen that will be used (see *Table 3-7*). Bitumen should not be held at the mixing temperature for more than 1 hour before using. Therefore, plan the preparation so that the mixing will be done within this time limit. Preheat the mixing pans to a temperature about 50° above the mixing temperature. Pour the heated dry aggregate fractions into the pans and mix thoroughly. Form a crater in the mixed aggregate, and pour the required amount of bitumen at the proper temperature into the crater. At this point, the temperature of the ingredients should be within the limits specified above. Mix the aggregates and bitumen as rapid and as thorough as possible to ensure that the bitumen is uniformly distributed throughout the aggregate.

Table 3-7. Temperatures for mixing bitumens and aggregates

Bitumen Type	Mixing Temperature (°F)	
	Aggregate	Bitumen
Asphalt cement	300 ± 5	270 ± 5
Tar (RT-10, -11, or -12)	225 ± 5	200 ± 5
Rubberized tar	250 ± 5	225 ± 5

Compact the Hot Mixture

Prepare three specimens at each bitumen content, and prepare the molds to receive the specimens as soon as they are mixed. Thoroughly clean and heat the striking face of the compaction hammers and the compaction molds to 200° to 300°F. Oil the mold and other metal in contact with the mix before the mixture is introduced to facilitate removing the specimen after compaction. A silicone spray is convenient for this use. Wipe the parts with a rag or paper towel before using. Place the mix in the mold (rodding the material as it is added). Remove the collar and, with a trowel, smooth the top surface of the mix to a slightly rounded shape. The thickness of the compacted specimen should be 2.5 ± .05 inches. One or two trials will indicate the quantity of mix required to produce such a specimen. Replace the collar and place the mold assembly on the compaction pedestal. The temperature at this point must be as specified in *Table 3-8* for compaction.

Table 3-8. Temperatures for compacting bitumens and aggregates

Bitumen Type	Compaction Temperature (°F)
Asphalt cement	300 ± 5
Tar (RT-10, -11, or -12)	225 ± 5
Rubberized tar	250 ± 5

Apply the required number of blows with the compaction hammer (see *Figure 3-14*). Remove the baseplate and collar, and reverse and reassemble the mold. Apply the required number of blows to the other side of the specimen. For example, roads, streets, and facilities for an airfield designed for aircraft whose tires carry 100 psi or less should be compacted by 50 blows on each end of the specimen. If the pavement is being designed for aircraft which carry tires with pressure greater than 100 psi, the compactive effort should be 75 blows per side.

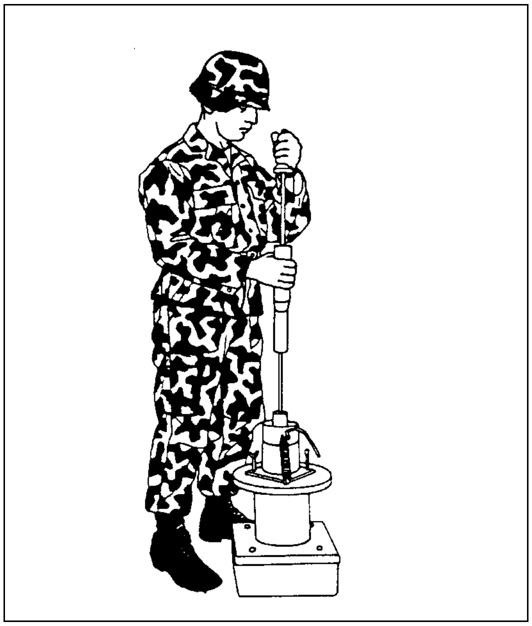


Figure 3-14. Compaction of bituminous trial-mix specimens

Cool the Hot Specimen

After compacting, remove the baseplate and collar, and either air cool the mold and specimen (normally overnight) or place them in cold water for a minimum of 2 minutes for fast cooling. Remove the cooled specimen from the mold with an extension jack or by placing the collar on the floor (with the mold and specimen on top) and forcing the specimen out with blows from the compaction hammer. The specimen is easier to remove if the mold is placed in a 140°F oven for a few minutes just before ejecting the specimen. Place the specimen (carefully handled and suitably identified) on a smooth surface until it is ready for testing as described below.

Weigh the Specimen in Air and in Water

Weigh each specimen in air and in water to obtain the weight and volume measurements used in calculating the unit weight of the compacted mix (see *Figure 3-15*).

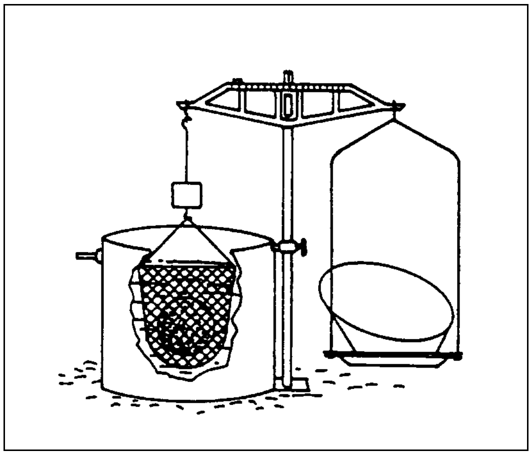


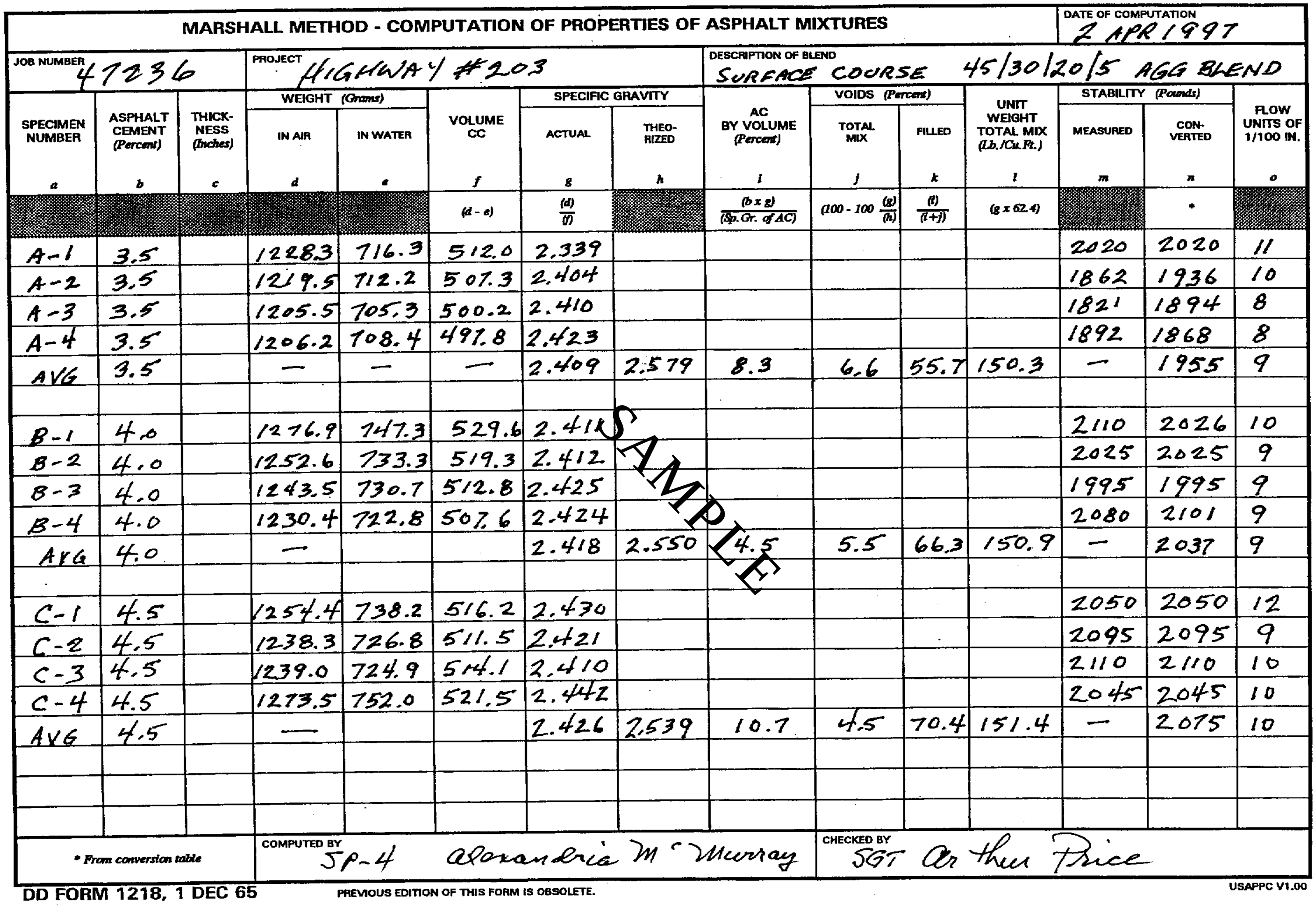
Figure 3-15. Weighing compacted bituminous specimen in water

DD Form 1218 provides space for recording those measurements made at room temperature (see *Figure 3-16, pages 3-52 and 3-53*). A direct weight in water of open-textured or porous specimens will give erroneous results because of water penetration and absorption. For such specimens, you must use other means to determine the volume. One means of measuring the volume of the porous specimens is to coat it with paraffin to seal all the voids and then weigh the coated specimens in air and in water.

Measure Stability and Flow

Bring the test specimens to the desired temperature for the test by immersing them in an oven for at least 2 hours. The bath temperature for asphalt samples is 140°F ± 1.8° and 100°F ± 1.8° for tar samples (RT-10 to RT-12). Record test measurements on DD Form 1218 (see *Figure 3-16*).

Clean the inside surfaces of the test heads and the guide rods thoroughly before performing the stability test, and lubricate the guide rods so the upper test head will slide easily over the guide rods on the lower test head. Remove the specimen from the water bath and place it on its side in the lower section of the breaking head. Position the upper section of the breaking head on the guide rods and on the specimen, then place the complete assembly in position in the testing machine (see *Figure 3-17, page 3-54*). To prevent excessive cooling of this specimen with a resulting increase in stability value, perform the entire procedure as quickly as possible (within 30 seconds) from the time the specimen is removed from the water bath.



MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

DATE OF COMPUTATION: 2 APR 1997

JOB NUMBER: 47236

PROJECT: HIGHWAY #203

DESCRIPTION OF BLEND: SURFACE COURSE 45/30/20/5 AGG BLEND

SPECIMEN NUMBER	ASPHALT CEMENT (Percent)	THICK-NESS (Inches)	WEIGHT (Grams)		VOLUME CC	SPECIFIC GRAVITY		AC BY VOLUME (Percent)	VOIDS (Percent)		UNIT WEIGHT TOTAL MIX (Lb./Cu.Ft.)	STABILITY (Pounds)		FLOW UNITS OF 1/100 IN.
			IN AIR	IN WATER		ACTUAL	THEO-RIZED		TOTAL MIX	FILLED		MEASURED	CON-VERTED	
a	b	c	d	e	f	g	h	i	j	k	l	m	n	o
▨	▨	▨	▨	▨	(d - e)	(d)/(f)	▨	(b x g)/(Sp.Gr. of AC)	(100 - 100 (g)/(h))	(i)/(i+j)	(g x 62.4)	▨	*	▨
A-1	3.5		1228.3	716.3	512.0	2.339						2020	2020	11
A-2	3.5		1219.5	712.2	507.3	2.404						1862	1936	10
A-3	3.5		1205.5	705.3	500.2	2.410						1821	1894	8
A-4	3.5		1206.2	708.4	497.8	2.423						1892	1868	8
AVG	3.5		—	—	—	2.409	2.579	8.3	6.6	55.7	150.3	—	1955	9
B-1	4.0		1276.9	747.3	529.6	2.411						2110	2026	10
B-2	4.0		1252.6	733.3	519.3	2.412						2025	2025	9
B-3	4.0		1243.5	730.7	512.8	2.425						1995	1995	9
B-4	4.0		1230.4	722.8	507.6	2.424						2080	2101	9
AVG	4.0		—			2.418	2.550	4.5	5.5	66.3	150.9	—	2037	9
C-1	4.5		1254.4	738.2	516.2	2.430						2050	2050	12
C-2	4.5		1238.3	726.8	511.5	2.421						2095	2095	9
C-3	4.5		1239.0	724.9	514.1	2.410						2110	2110	10
C-4	4.5		1273.5	752.0	521.5	2.442						2045	2045	10
AVG	4.5		—			2.426	2.539	10.7	4.5	70.4	151.4	—	2075	10

SAMPLE

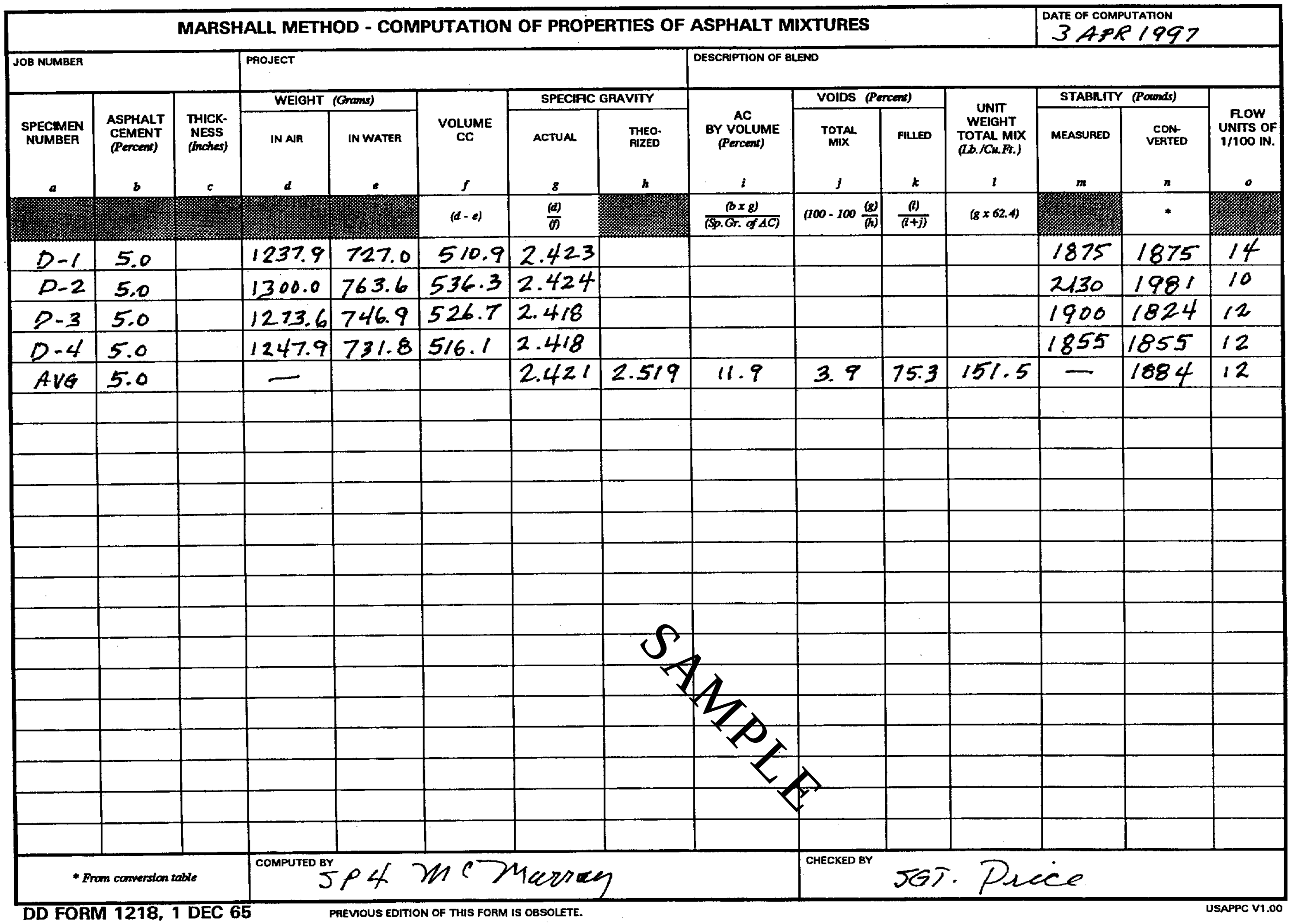
* From conversion table

COMPUTED BY: SP-4 Alexandria M^c Murray

CHECKED BY: SGT Arthur Price

DD FORM 1218, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE. USAPPC V1.00

Figure 3-16. Sample DD Form 1218



MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

DATE OF COMPUTATION 3 APR 1997

JOB NUMBER

PROJECT

DESCRIPTION OF BLEND

SPECIMEN NUMBER	ASPHALT CEMENT *(Percent)*	THICKNESS *(Inches)*	WEIGHT *(Grams)*		VOLUME CC	SPECIFIC GRAVITY		AC BY VOLUME *(Percent)*	VOIDS *(Percent)*		UNIT WEIGHT TOTAL MIX *(Lb./Cu.Ft.)*	STABILITY *(Pounds)*		FLOW UNITS OF 1/100 IN.
			IN AIR	IN WATER		ACTUAL	THEORIZED		TOTAL MIX	FILLED		MEASURED	CONVERTED	
a	b	c	d	e	f	g	h	i	j	k	l	m	n	o
▨	▨	▨	▨	▨	(d - e)	$\frac{(d)}{(f)}$	▨	$\frac{(b \times g)}{(Sp. Gr. of AC)}$	$(100 - 100 \frac{(g)}{(h)})$	$\frac{(i)}{(i+j)}$	(g x 62.4)	▨	*	▨
D-1	5.0		1237.9	727.0	510.9	2.423						1875	1875	14
D-2	5.0		1300.0	763.6	536.3	2.424						2130	1981	10
D-3	5.0		1273.6	746.9	526.7	2.418						1900	1824	12
D-4	5.0		1247.9	731.8	516.1	2.418						1855	1855	12
AVG	5.0		—			2.421	2.519	11.9	3.9	75.3	151.5	—	1884	12

* *From conversion table*

COMPUTED BY SP4 McMurray

CHECKED BY SGT. Price

DD FORM 1218, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE. USAPPC V1.00

Figure 3-16. Sample DD Form 1218 (continued)

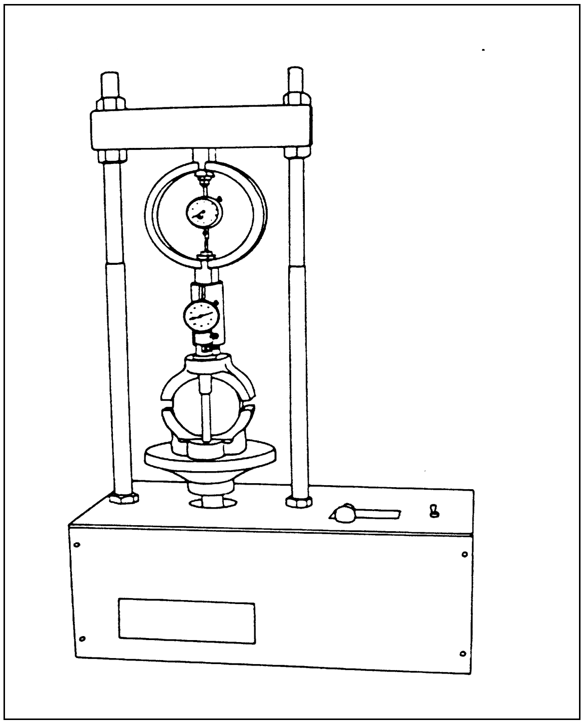


Figure 3-74. Stability-test assembly

Place the flowmeter over one of the guide rods and take an initial reading, estimated to 0.01 inch. Hold the flowmeter firmly over the guide rod while loading the specimen. Read or remove the flowmeter from its position over the guide rod just when the load first begins to decrease, as indicated by the dial gauge in the proving ring. DD Form 1218 provides space for recording the flow valve, which is the difference between the initial reading and the final reading (see *Figure 3-16, pages 3-52 and 3-53*).

Apply load to the specimen at a constant rate of strain of 2 inches per minute until specimen failure occurs. The load builds up on the typical test as movement occurs, until it reaches a maximum and falls off. The maximum reading of the dial, converted to pounds, is the stability value for the specimen. Record this reading on the form (see *Figure 3-16*).

Calculation and Presentation of Results

DD Form 1218 is used to summarize the measured and calculated Marshall test properties (see *Figure 3-16*). The specimen numbers—placed on each specimen with a marking crayon—are given for identification. Note that there are four duplicate tests for each bitumen content and that these four test values are averaged in each instance. Note also that the specimen's thickness is not indicated since the volume can be used to find the stability correlation ratio from *Table 3-9*. The theoretical specific gravity is transferred from DD Form 1218 to calculate the voids (see *Figure 3-16, pages 3-52 and 3-53*). Note that the stability value is shown directly in pounds. Unless the testing machine provides a load-measuring device that reads directly in pounds, it will be necessary to convert this value. Use the calibration factor furnished with the ring dynamometer on the testing machine. The stability value varies directly with the specimen's thickness. Therefore, it is necessary to correct the stability values for specimens of a thickness greater or less than the standard 2 1/2 inches. *Table 3-9* shows the necessary conversion factors for specimens varying in thickness from 1 to 3 inches. *Table 3-9* also contains data whereby the stability conversion factor can be determined on the basis of the volume of the specimen, since the volume is a direct function of height for

Table 3-9. Stability correlation ratios, Marshall stability test

Volume of Specimen (in Cubic Centimeters)	Approximate Thickness of Specimen (in Inches)	Correlation Ratio
200 to 213	1	5.56
214 to 225	1 1/16	5.00
226 to 237	1 1/8	4.55
238 to 250	1 3/16	4.17
251 to 264	1 1/4	3.85
265 to 276	1 5/16	3.57
277 to 289	1 3/8	3.33
290 to 301	1 7/16	3.03
302 to 316	1 1/2	2.78
317 to 328	1 9/16	2.50
329 to 340	1 5/8	2.27
341 to 353	1 11/16	2.08
354 to 367	1 3/4	1.92
368 to 379	1 13/16	1.79
380 to 392	1 7/8	1.67
393 to 405	1 15/16	1.56
406 to 420	2	1.47
421 to 431	2 1/16	1.39
432 to 443	2 1/8	1.32
444 to 456	2 3/16	1.25
457 to 470	2 1/4	1.19
471 to 482	2 5/16	1.14
483 to 495	2 3/8	1.09
496 to 508	2 7/16	1.04
509 to 522	2 1/2	1.00
523 to 535	2 9/16	0.96
536 to 546	2 5/8	0.93
547 to 559	2 11/16	0.89
560 to 573	2 3/4	0.86
574 to 585	2 13/16	0.83
586 to 598	2 7/8	0.81
599 to 610	2 15/16	0.78
611 to 625	3	0.76

a constant 4-inch-diameter specimen. All other calculations are indicated directly on the form (see *Figure 3-16*, *pages 3-52 and 3-53*).

Graphical Presentation

The average Marshall test properties from the tabulation (see *Figure 3-16*) for each bitumen content are shown graphically on DD Form 1219 (see *Figure 3-18*). The average values for each property are plotted on their respective graphs using the bitumen content as ordinates. A smooth curve is drawn through the plotted points in each instance.

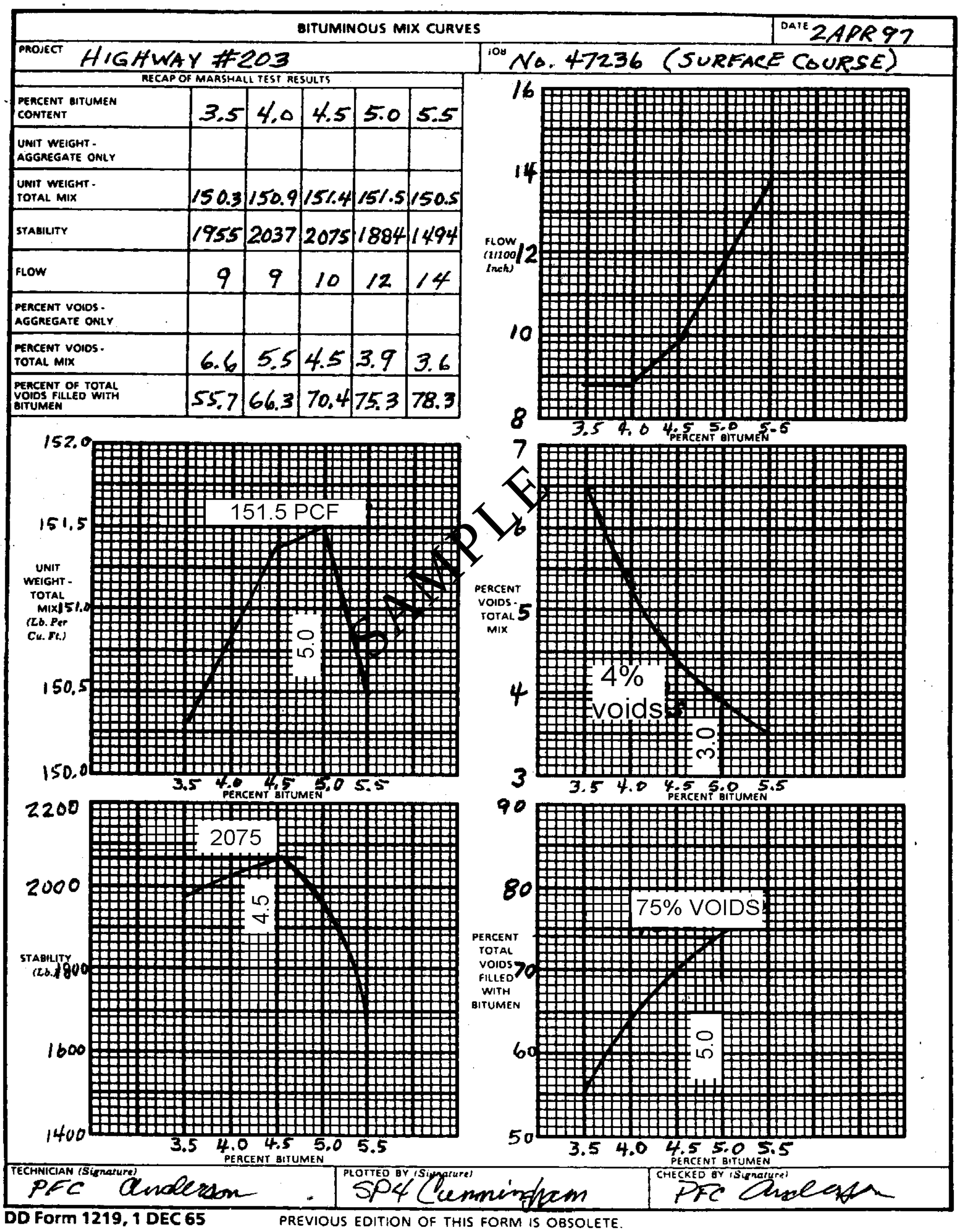
Application of the Marshall Test

Table 3-10, *page 3-58*, lists the criteria for determining the optimum bitumen content along with the Marshall specifications for a satisfactory mix. The optimum bitumen content is determined by averaging the bitumen content read from the curves in *Figure 3-18* at the four points indicated for determination of the optimum bitumen content in *Table 3-10*. Once this average bitumen content is obtained, the Marshall properties at this average are read from curves representing the mix, as in *Figure 3-18*. These values are compared with the specification limits in *Table 3-10* to evaluate the satisfactoriness of the mix. The exceptions allowed when the values obtained fail to conform with the specification limits given in the table are shown in notes at the bottom of *Table 3-10*.

Modified Marshall Test for Cold-Mix Pavement

This method is used as an aid in determining the asphalt content for cold-mix design of light-duty pavement. It can be used where asphalt cutbacks will be the binder. The procedure follows those used for hot-mix design in general, with the following modifications:

- Aggregates. These are dried to a moisture content expected during construction (up to a maximum of 2 percent by weight).
- Asphalt. The selected bitumen is mixed with the aggregates, but at the temperature recommended for field application. The aggregates remain at room temperature.
- Curing. Before compaction, the mixture is cured at least 12 hours in an oven set at 140°F ± 5°.
- Compaction. After curing, the mixture is compacted at 140°F using 50 blows of the hammer at each end of the specimen.
- Cooling. After molding, the specimens are cooled to room temperature in the molds. Care must be taken to remove the specimens, undisturbed and undamaged, from the molds.
- Testing. The specimens are heated in an oven to 100°F ± 2° and tested in the Marshall machine. Heating will normally take about 2 hours.
- Design amount of asphalt. The asphalt contents at maximum density and maximum stability, after averaging, are used as the design amount.



BITUMINOUS MIX CURVES

DATE 2 APR 97

PROJECT HIGHWAY #203

JOB No. 47236 (SURFACE COURSE)

RECAP OF MARSHALL TEST RESULTS

PERCENT BITUMEN CONTENT	3.5	4.0	4.5	5.0	5.5
UNIT WEIGHT - AGGREGATE ONLY					
UNIT WEIGHT - TOTAL MIX	150.3	150.9	151.4	151.5	150.5
STABILITY	1955	2037	2075	1884	1494
FLOW	9	9	10	12	14
PERCENT VOIDS - AGGREGATE ONLY					
PERCENT VOIDS - TOTAL MIX	6.6	5.5	4.5	3.9	3.6
PERCENT OF TOTAL VOIDS FILLED WITH BITUMEN	55.7	66.3	70.4	75.3	78.3

TECHNICIAN (Signature) PFC Anderson

PLOTTED BY (Signature) SP4 Cunningham

CHECKED BY (Signature) PFC Anderson

DD Form 1219, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

Figure 3-75. Asphalt mix curves, Marshall test properties

Table 3-19. Marshall test specifications and determination of optimum asphalt content

Property	Course	Criteria		Determination of OAC	
		75 Blows	50 Blows	High Press	Low Press
		**High Press	Low Press		
Aggregate blends showing water absorption up to 2 1/2 percent (used with ASTM apparent specific gravity)					
Stability	Surface	1,800 or higher	500 or higher	Peak of curve	Peak of curve
Unit weight	Surface	—	—	Peak of curve	Peak of curve
Flow	Surface	16 or less	20 or less	Not used	Not used
Percent voids total mix	Surface	3 to 5 percent	3 to 5 percent	4 percent	4 percent
Percent voids filled w/AC	Surface	70 to 80 percent	75 to 85 percent	75 percent	80 percent
Stability	Binder	1,800 or higher	500 or higher	Peak of curve*	Peak of curve*
Unit weight	Binder	—	—	Peak of curve*	Peak of curve*
Flow	Binder	16 or less	20 or less	Not used	Not used
Percent voids total mix	Binder	5 to 7 percent	4 to 6 percent	6 percent	5 percent
Percent voids filled w/AC	Binder	50 to 70 percent	65 to 75 percent	60 percent	70 percent*
Stability	Sand asphalt	**	500 or higher	**	Peak of curve
Unit weight	Sand asphalt	—	—	**	Peak of curve
Flow	Sand asphalt	**	20 or less	Not used	Not used
Percent voids total mix	Sand asphalt	**	5 to 7 percent	**	6 percent
Percent voids filled w/AC	Sand asphalt	**	65 to 75 percent	**	70 percent
Aggregate blends showing water absorption greater than 2 1/2 percent (used with bulk-impregnated specific gravity)					
Stability	Surface	1,800 or higher	500 or higher	Peak of curve	Peak of curve
Unit weight	Surface	—	—	Peak of curve	Peak of curve
Flow	Surface	16 or less	20 or less	Not used	Not used
Percent voids total mix	Surface	2 to 4 percent	2 to 4 percent	3 percent	3 percent
Percent voids filled w/AC	Surface	75 to 85 percent	80 to 90 percent	80 percent	85 percent
Stability	Binder	1,800 or higher	500 or higher	Peak of curve*	Peak of curve*
Unit weight	Binder	—	—	Peak of curve*	Peak of curve*
Flow	Binder	16 or less	20 or less	Not used	Not used
Percent voids total mix	Binder	4 to 6 percent	3 to 5 percent	5 percent	4 percent
Percent voids filled w/AC	Binder	55 to 75 percent	70 to 80 percent	65 percent	75 percent
Stability	Sand asphalt	**	500 or higher	**	Peak of curve
Unit weight	Sand asphalt	—	—	**	Peak of curve
Flow	Sand asphalt	**	20 or less	Not used	Not used
Percent voids total mix	Sand asphalt	**	4 to 6 percent	**	5 percent
Percent voids filled w/AC	Sand asphalt	**	70 to 80 percent	**	75 percent

***If the inclusion of bitumen contents at these points in the average causes the voids total mix to fall outside the limits, then the optimum bitumen should be adjusted so that the voids total mix are within the limits.**

****Criteria for sand asphalt to be used in designing pavement for high-pressure tires have not been established.**

Test Variations

These variations apply to aggregates with 10 percent or more larger than 1-inch maximum size. The procedure previously described is applicable where the amount of aggregate larger than the 1-inch sieve is less than 10 percent of the total. When the +1-inch material exceeds 10 percent of the total, the following variations are made in the procedure:

- Mix bitumen at the selected content with the entire aggregate, including the +1-inch portion.
- Pass the mixed hot batch through a 1-inch sieve. Discard the +1-inch portion.
- Make compacted specimens from the portion that passes the 1-inch sieve and perform the Marshall test. Do not calculate the voids of the compacted specimens at this time.
- Determine the bulk specific gravity of the +1-inch aggregate and, with the specific gravity of the compacted specimens, compute the adjusted specific gravity (G_A) as follows:

$$G_A = \frac{100}{\left(\frac{A}{C} + \frac{B}{D}\right) \times f}$$

where—

A = *weight of dry, 1-inch material expressed as a percentage of the total batch weight (bitumen plus aggregate)*

B = *portion of the total batch remaining after the dry, +1-inch portion is removed (100 percent – A percent)*

C = *bulk specific gravity of the +1-inch aggregate*

D = *actual specific gravity of the compacted specimen*

f = *empirical factor = 0.995*

- Calculate the voids by using the adjusted specific gravity and apply the design criteria for this value.
- Use stability and flow values as measured on the compacted specimens.

SURFACE-AREA METHOD

The following approximation formulas may be used for estimating the optimum bitumen content when the gradation of the aggregate blend is known. These estimates must be considered rough approximations since the optimum bitumen content is a function of the compaction effort as well as the gradation and surface area. The greater the anticipated pavement loading, the greater the compaction effort that must be used. The greater the compaction effort for a given aggregate, the lower the optimum bitumen content.

Asphalt Cement

Use the following formula for asphalt cement, based on the surface area of the aggregate:

$$P = 0.02a + 0.07b + 0.15c + 0.20d$$

where—

P = *percent of asphalt material by weight of dry aggregate*

a = *percent of mineral aggregate retained on the No. 50 sieve*

b = percent of mineral aggregate passing the No. 50 and retained on the No. 100 sieve

c = percent of mineral aggregate passing the No. 100 and retained on the No. 200 sieve

d = percent of mineral aggregate passing the No. 200 sieve

NOTE: Express all percentages as whole numbers.

Asphalt Emulsion

Use the following formula for asphalt emulsion, based on the surface area of the aggregate:

$$P = 0.05A + 0.1B + 0.5C$$

where—

P = percent by weight of asphalt emulsion, based on weight of graded mineral aggregate

A = percent of mineral aggregate retained on the No. 8 sieve

B = percent of mineral aggregate passing the No. 8 sieve and retained on the No. 200 sieve

C = percent of mineral aggregate passing the No. 200 sieve

NOTES:

1. Express all percentages as whole numbers.

2. Absorptive aggregate, such as slag, lime rock, vesicular lava, and coral, will require additional asphalt.

JOB-MIX FORMULA

When the necessary laboratory tests have been completed and the optimum bitumen content has been determined, the job-mix formula must be established for use by plant personnel producing the paving mix. Setting up the job-mix formula involves the relative percentages of the available aggregate and the bitumen. In the mix-design test already illustrated, the optimum content was found to be 4.7 percent of the total mix. Accordingly, the aggregate portion of the mix will be 95.3 percent of the total mix. Referring to the aggregate-blend calculations on DD Forms 1217 (see *Figure 3-19*) and 1207 *(see Figure 3-20, pages 3-62 and 3-63),* the portions were 45 percent coarse aggregate, 30 percent fine aggregate, 20 percent fine river-bar sand (FRBS), and 5 percent limestone dust or mineral filler.

The job-mix formula is then computed as follows:

Coarse aggregate = 95.3 x 45 = 42.9 percent

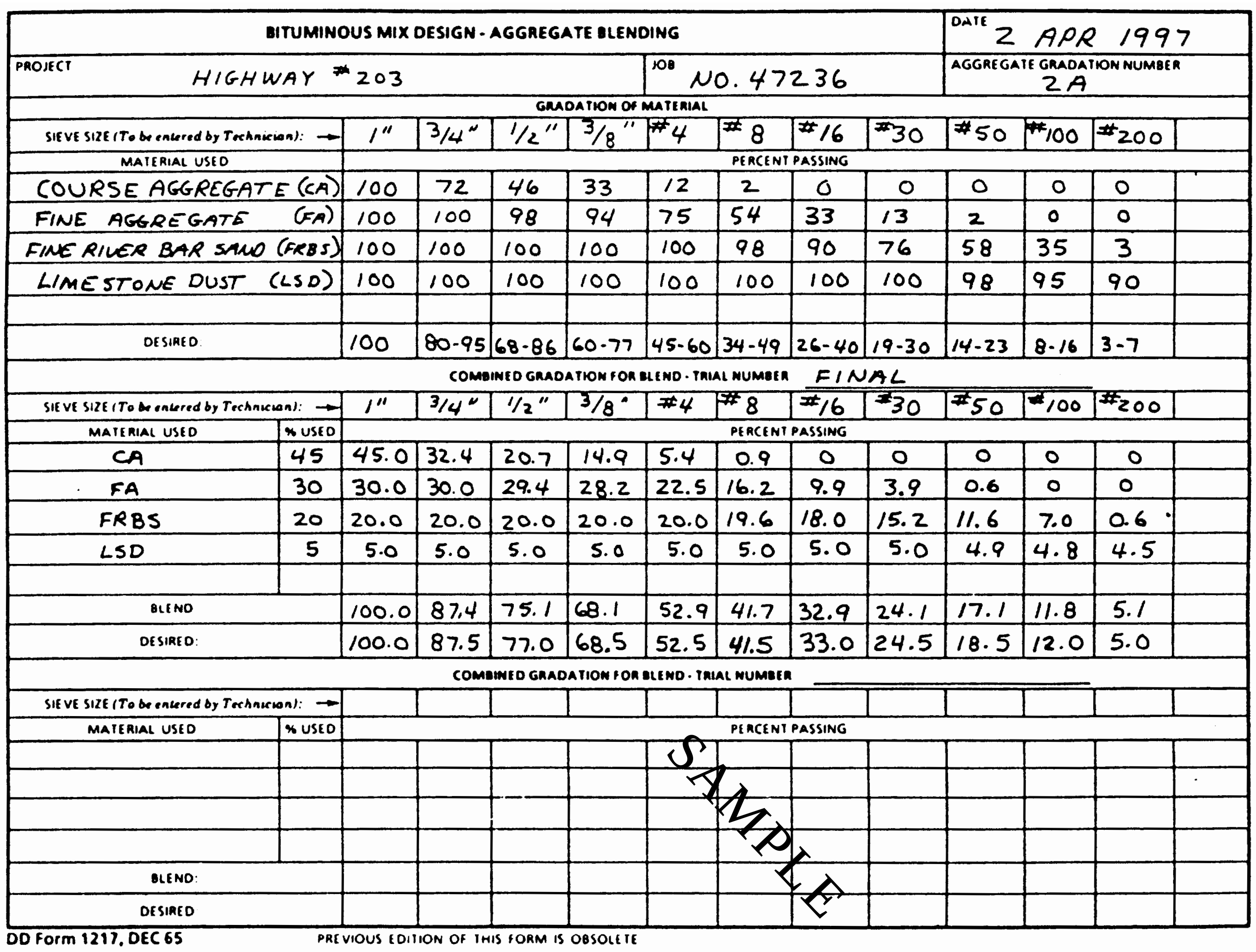
Fine aggregate = 95.3 x 30 = 28.6 percent

FRBS = 5.3 x 20 = 19.0 percent

Mineral filler = 95.3 x 5 = 4.8 percent

Bitumen = 4.7 percent

Total = 100.0 percent



BITUMINOUS MIX DESIGN - AGGREGATE BLENDING

DATE 2 APR 1997

PROJECT HIGHWAY #203

JOB NO. 47236

AGGREGATE GRADATION NUMBER 2A

GRADATION OF MATERIAL

SIEVE SIZE (To be entered by Technician): →	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	
MATERIAL USED	PERCENT PASSING											
COURSE AGGREGATE (CA)	100	72	46	33	12	2	0	0	0	0	0	
FINE AGGREGATE (FA)	100	100	98	94	75	54	33	13	2	0	0	
FINE RIVER BAR SAND (FRBS)	100	100	100	100	100	98	90	76	58	35	3	
LIMESTONE DUST (LSD)	100	100	100	100	100	100	100	100	98	95	90	
DESIRED	100	80-95	68-86	60-77	45-60	34-49	26-40	19-30	14-23	8-16	3-7	

COMBINED GRADATION FOR BLEND - TRIAL NUMBER FINAL

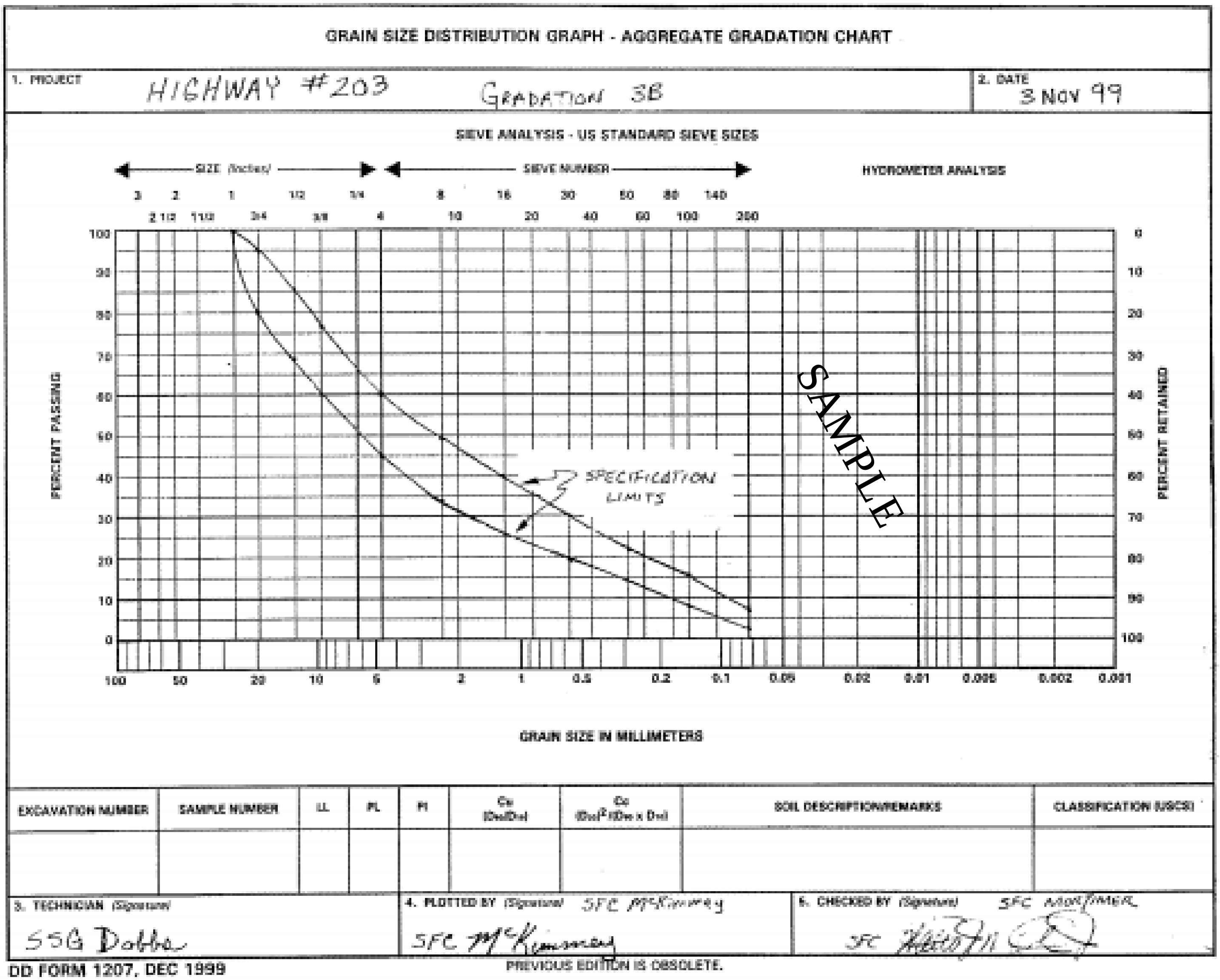
SIEVE SIZE (To be entered by Technician): →		1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200	
MATERIAL USED	% USED	PERCENT PASSING											
CA	45	45.0	32.4	20.7	14.9	5.4	0.9	0	0	0	0	0	
FA	30	30.0	30.0	29.4	28.2	22.5	16.2	9.9	3.9	0.6	0	0	
FRBS	20	20.0	20.0	20.0	20.0	20.0	19.6	18.0	15.2	11.6	7.0	0.6	
LSD	5	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	4.9	4.8	4.5	
BLEND		100.0	87.4	75.1	68.1	52.9	41.7	32.9	24.1	17.1	11.8	5.1	
DESIRED:		100.0	87.5	77.0	68.5	52.5	41.5	33.0	24.5	18.5	12.0	5.0	

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE (To be entered by Technician): →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED													

DD Form 1217, DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE

Figure 3-19. Sample DD Form 1217



GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADATION CHART

1. PROJECT HIGHWAY #203 GRADATION 3B

2. DATE 3 NOV 99

EXCAVATION NUMBER	SAMPLE NUMBER	LL	PL	PI	Cu (D_{60}/D_{10})	Cc $(D_{30})^2/(D_{60} \times D_{10})$	SOIL DESCRIPTION/REMARKS	CLASSIFICATION (USCS)

3. TECHNICIAN (Signature) SSG Dobbs

4. PLOTTED BY (Signature) SFC McKinney SFC McKinney

5. CHECKED BY (Signature) SFC MORTIMER SFC

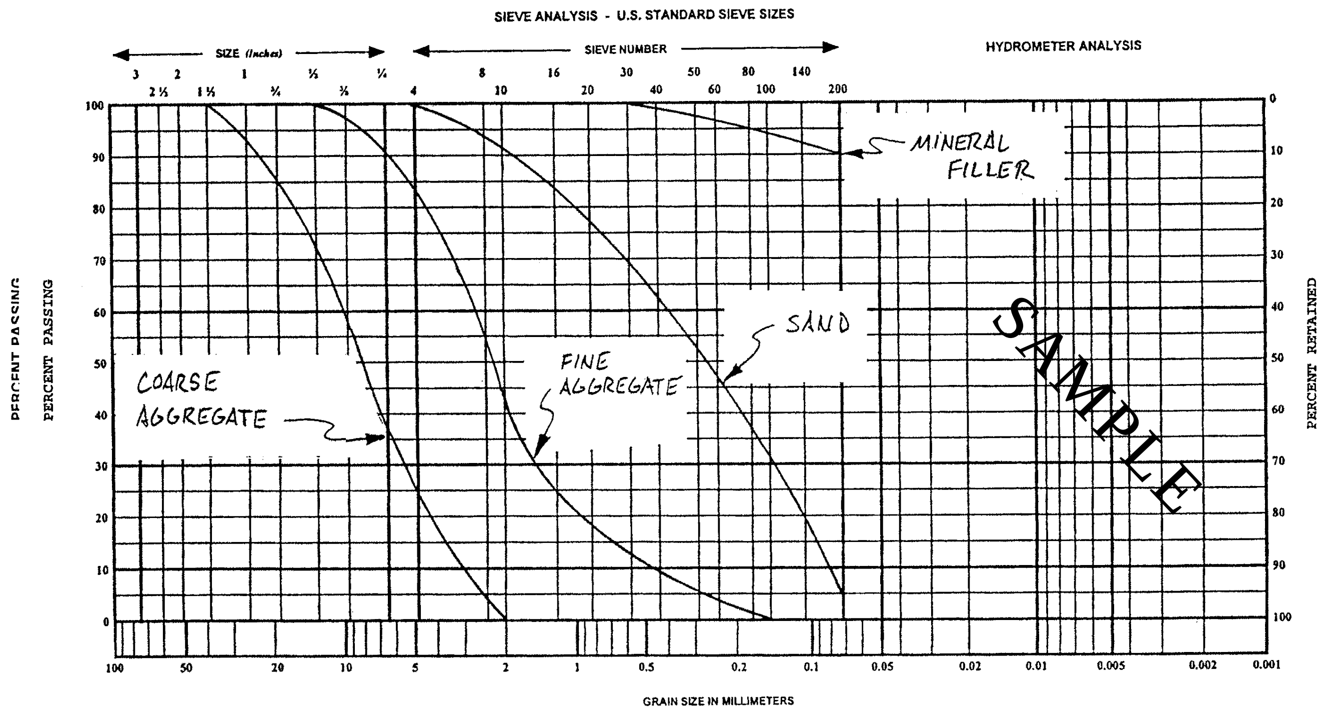
DD FORM 1207, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

Figure 3-20. Sample DD Form 1207

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADATION CHART

1. PROJECT HIGHWAY #203	2. DATE 3 NOV 99



EXCAVATION NUMBER	SAMPLE NUMBER	LL	PL	PI	Cu (D60/D10)	Cc (D30)2/(D60 x D10)	SOIL DESCRIPTION/REMARKS	CLASSIFICATION (USCS)

3. TECHNICIAN (Signature) SSG Dobbs	4. PLOTTED BY (Signature) SFC McKimmey SFC McK	5. CHECKED BY (Signature) SFC MORTIMER SFC

DD FORM 1207, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

Figure 3-20. Sample DD Form 1207 (continued)

SECTION VII. PLANT CONTROL

Aggregate is mixed in correct proportions for specified construction in high-type bituminous paving plants and intermediate-type plants. The type of plant used depends on proximity and the requirements for the aggregate. Laboratory procedures must be done to obtain correct mixes and must be correlated with production procedures. Proper control over all procedures and equipment must be exercised to ensure quality for each aggregate specified.

PLANT TYPES

Figures 3-21 and 3-22 are schematic drawings of a batch plant and a drum-mix plant.

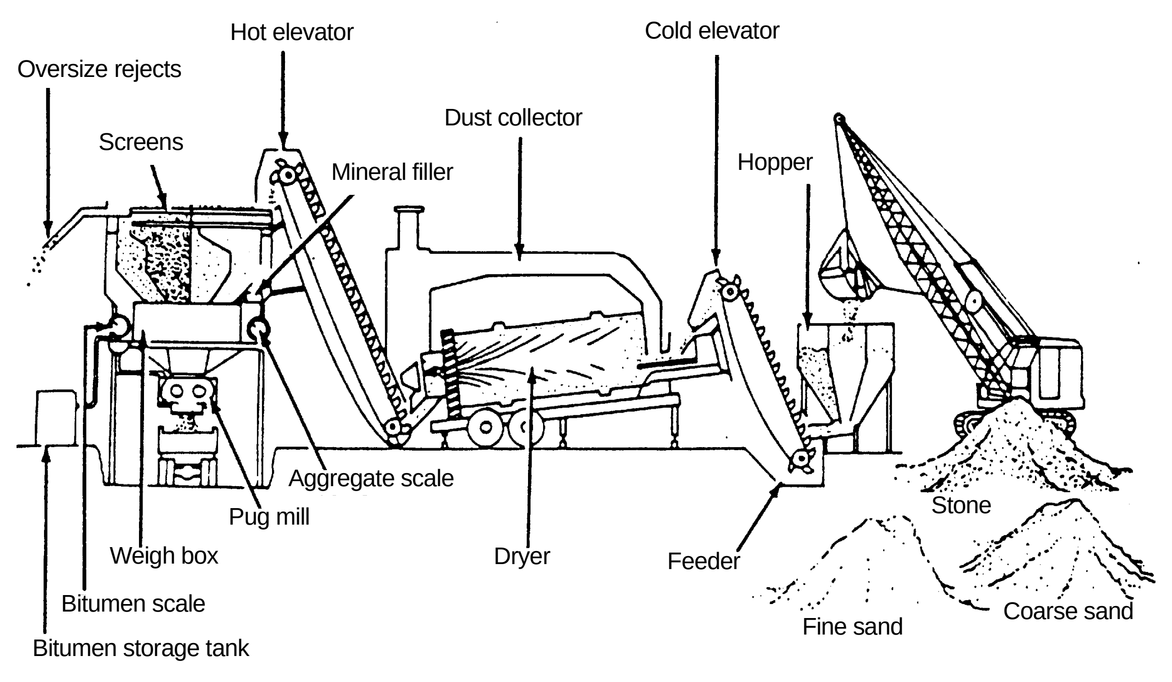


Figure 3-21. Bituminous hot-mix batch plant

HIGH-TYPE BITUMINOUS PAVING PLANT

In the operation of a high-type bituminous paving plant, aggregates from two or more sources are fed into the aggregate dryer in the approximate proportions required to produce the desired gradation. This initial proportioning usually is accomplished by means of a hopper-type feeder, operating from one or more bins, which feeds the aggregates into a cold elevator that delivers them to the dryer. The mechanical feeder is loaded by a clamshell or other suitable means.

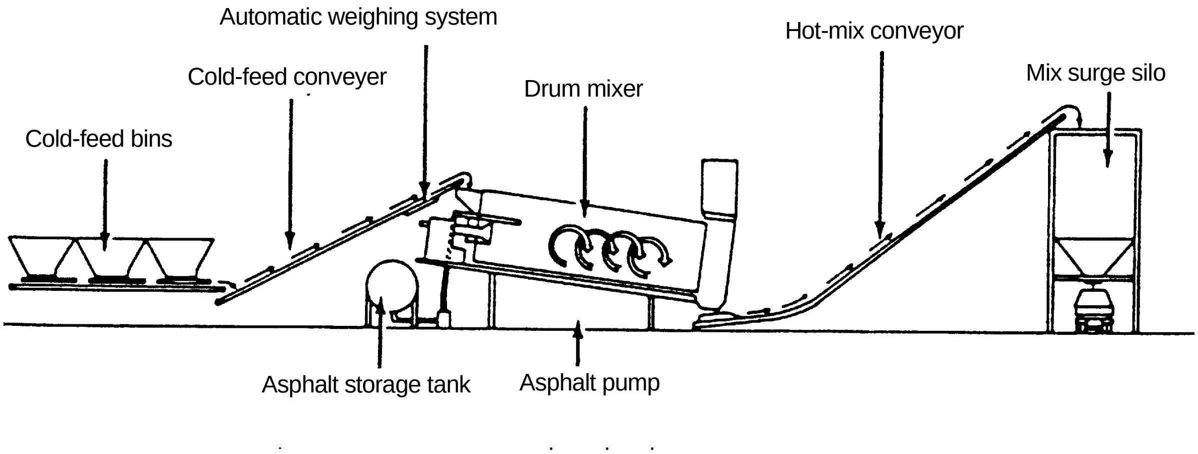


Figure 3-22. Bituminous hot-mix continuous-mix plant

They are heated to the desired temperature. Upon leaving the dryer, they pass over vibrating screens where they are separated according to size. The usual screening equipment for a three-bin plant consists of a rejection (scalping) screen for eliminating oversized material and screens for dividing the coarse aggregate into two separate sizes (bins). Fine aggregate goes into the third bin. An additional screen is provided for separating the coarse aggregate in a four-bin plant. Additional mineral filler, if required, usually is stored and weighed or proportioned into the mix separately. It may be obtained from the plant's dust collector or from an external source. Plant screens may be changed to provide a variation in the size of openings. The sizes used depend largely on the type of mixture being produced. In some cases, it may be necessary to change the size of the screens to obtain a proper balance of aggregate sizes in each bin.

The aggregates must be fed through the plant uniformly, preferably by a mechanical feeder, to obtain efficient plant operation and to produce a desired mixture. It is usually necessary to make some slight adjustments in the plant-bin proportions, since a screen analysis of the hot storage bins will not entirely duplicate the screen analysis used in the laboratory design. This may result from—

- Fines lost while passing through the dryer (unless the equipment includes an effective dust collector and the fines are returned to the mix).
- Aggregate degradation in the dryer.
- Plant screens that are not completely efficient in the separation of the aggregate, with the result that some fines are carried over into the coarser bins.
- Separation of material at hot bins into more or fewer fractions than represented at stockpiles.

INTERMEDIATE-TYPE PLANTS

An intermediate-type plant does not have the refinements of the high-type plant described above. Such items as the gradation control unit, the dryer unit, and even the storage hoppers or bins may not be components of this type of plant. The aggregates from stockpiles or trucks are added directly (in correct proportion) into the elevator and then fed into the mixer. Corrections or changes in the aggregate proportions must be made during the feed. If a dryer is not part of the plant, adjustment to compensate for moisture must be anticipated and made externally.

INITIATING PLANT PRODUCTION

The heaviest demands on laboratory facilities arise at the beginning of plant production. Preliminary computations, which provide the gradation for the mixture design, may be made to determine the weight of material from each bin. The gradation of aggregate supplied by the plant according to computed bin weights may not reproduce precisely the desired gradation. The gradation of the plant-produced aggregates approximates the one used in design, within reasonable tolerances, if initial sampling has been done properly and if the plant is operated efficiently. Certain steps should be taken, however, to ensure that satisfactory mixtures are reproduced from the beginning and throughout the period of plant production. Procedures outlined in this section will ensure satisfactory paving mixtures.

SIEVE ANALYSIS

A sieve analysis is made on material from each plant bin. Samples for these sieve analyses are obtained after a few tons of aggregate have been processed through the dryer and screens so that the sample will be representative. Final bin proportions may be based on these sieve analyses.

MIX REDESIGN

The aggregates from the bins sometimes cannot be proportioned to satisfactorily reproduce the gradation of the aggregate used in the laboratory design. It is then necessary to redesign the mix, using plant-produced aggregates. Specimens are prepared and tested for the new design in the same manner as for the original. This gives optimum asphalt content and a satisfactory mix produced by the plant. Occasions may arise in which the gradation of the plant-produced aggregate will differ from the laboratory design so that part of the aggregates may be wasted. The mix should be redesigned to use all of the available aggregate. Sufficient additional tests should be performed to establish optimum asphalt requirements and ensure that the mix meets applicable criteria.

CONTROLLING PLANT PRODUCTION

Obtain from each of the first four truckloads enough paving mix for the preparation of four test specimens. Prepare the four specimens from each of these samples and compact and test them according to standard procedures described previously. Conduct the tests as rapidly as possible, and delay plant production until data from these tests are available. The data must conform to final design data at the same asphalt content, within reasonable tolerances, before plant production is resumed. If necessary, make adjustments to secure a conforming mix. Such procedures will delay plant production generally less than 2 hours and assure production of satisfactory mixes. As soon as the data from the testing of the plant-produced mix are obtained, compare it with corresponding design data for further adjustments of the mix, if necessary. Probable causes of paving-mixture deficiencies for both batch and continuous mixing plants are shown in *Figure 3-23*. These deficiencies are observed at the plant. Other imperfections and their causes that may be encountered in placing the mix in the pavement are given in *Figure 3-24, page 3-68*.

Probable causes of deficiencies in hot plant-mix paving mixtures																												
1 Aggregate scales out of adjustment	2 Bridging of hot aggregate in bin	3 Bitumen scales out of adjustment	4 Lack of proper weighing by operator	5 Faulty pug mill dump	6 Bitumen temperature too high	7 Drum feed not uniform	8 Improper dryer operation	9 Too little bitumen	10 Too much bitumen	11 Sampling method not uniform	12 Excessive aggregate moisture	13 Bin overflow pipes not functioning	14 Leaky bins	15 Separation of aggregates in bins	16 Mixing time not uniform	17 Mineral filler feed not uniform	18 Aggregate temperature too high	19 Temp mechanism out of adjustment	20 Improper feed of aggregates to dryer	21 Overrated dryer capacity	22 Faulty screen operation	23 Overloaded pug mill	24 Improper set on pug mill	25 Bitumen & aggregate feed adjusting	26 Bitumen pump not functioning right	27 Not enough hot aggregate in bins	28 Aggregate gates not set properly	**Types of deficiencies that may be encountered in producing hot plant-mix paving mixtures**
☒		☒	☒	☒				☒	☒	☒		☒	☒		☒	☒			☒				☒	☒	☒	☒	☒	Bitumen content formula error
☒	☒		☒	☒						☒		☒	☒	☒		☒			☒		☒		☒	☒		☒	☒	Gradation formula error
				☒	☒										☒							☒	☒					Poorly mixed loads
☒		☒	☒	☒	☒				☒	☒			☒			☒			☒					☒	☒		☒	Fat, rich mixtures
☒		☒	☒	☒	☒		☒	☒		☒			☒				☒	☒	☒					☒	☒		☒	Lean or burned mixtures
						☒	☒										☒	☒		☒								Mixture temperature error
						☒	☒										☒	☒	☒									Smoking loads
						☒	☒				☒							☒	☒	☒								Steaming loads
	☒	☒	☒	☒														☒										Overweight or underweight loads
	☒	☒	☒	☒	☒			☒	☒		☒	☒	☒	☒	☒	☒		☒	☒	☒	☒	☒	☒	☒		☒	☒	Lack of mixture uniformity

Items 6 to 23 inclusive are applicable to all types of plants. Items 1 to 5 inclusive and items 24 to 28 inclusive are applicable to batch plants and volumetric plants respectively.

Figure 3-74. Probable causes of paving-mixture deficiencies detected at plant

GYRATORY TEST CONTROL

The GSI of the individual test specimens should never exceed 1.05. The GSF of the individual test specimens should never be less than 1.0. The average value of the other test properties for the four test specimens from any given truckload should not deviate from the final design values by more than the following amounts:

- Unit weight total mix ± 1.5 pcf.
- Gyratory shear (S_G) ± 15 percent.

Probable causes of imperfections in finished pavements																					
Excessive prime coat	Improper proportioning	Unsatisfactory batches in load	Poor handwork behind spreader	Excessive segregation in laying	Inadequate rolling	Poor spreader operation	Mixture too hot or burned	Mixture too cold	Rolling mixture when too hot	Rolling mixture when too cold	Poorly graded mixture	Unstable base course	Faulty allowance for compaction	Roller standing on hot pavement	Mixture too coarse	Lack of bitumen in mixture	Excess of bitumen in mixture	Inadequate cross rolling	Not cut back to uniform thickness	Excessive moisture in mixture	**Types of pavement imperfections that may be encountered in laying hot plant-mix paving mixtures**
X		X															X				Bleeding
							X									X				X	Brown, dead appearance
	X	X		X							X				X						Poor surface texture
	X	X	X	X	X	X		X	X	X		X		X				X			Rough uneven surface
			X					X					X						X		Uneven lateral joints
			X			X		X		X			X						X		Uneven longitudinal joints
					X					X								X			Roller marks
	X	X					X	X	X		X						X			X	Pushing
			X			X		X	X					X							Waves
	X	X		X				X		X	X				X	X					Cracking
	X	X	X	X		X		X							X						Honeycomb
			X		X	X			X			X						X			Distortion
		X				X		X								X					Tearing of surface during laying
	X	X															X				Rich or fat spots

Figure 3-74. Probable causes of imperfections encountered in laying pavement

MARSHALL TEST CONTROL

The flow value of the individual test specimens should never exceed the specification limits and should not vary more than two points between specimens. The average value of the other test properties for the four test specimens from any given truckload should be within the specification limits and not vary from the final design values by more than the following amounts:

- Unit weight total mix ± 1.5 pcf
- Marshall stability ± 5 percent.
- Voids total mix ± 0.5 percent.

No limit is placed on voids filled since control is ensured by the limitations on voids total mix.

PLANT-CONTROL EXECUTION

Stop plant production if the measured test properties fail to comply with the limitations cited above. Resume production on a trial basis until the problem has been isolated and corrective measures have been taken. Small variations in aggregate quantities usually do not change test properties significantly, while small changes in bitumen content (a few tenths of a percent) can have a very significant effect on test properties. If trouble is indicated, check all calculations as a first step. Check the total of the batch weights dumped into the truck against the total load on the truck. These total weights should not vary from each other by more than 2 percent.

Improper weighing or faulty scales may be detected readily. Take corrective measures by maintaining a close check on load weights. Check plant scales and gate openings and recalibrate them as necessary. Adjust scales and proportioning devices found to be inaccurate. After checking the plant components and making the necessary adjustments, obtain, sample, and test four additional truckloads of plant mix. Do not place the plant in continuous operation until the test properties conform to the specifications and allowable tolerance. Once the plant has been placed in continuous operation, prepare test specimens from approximately each 400 tons of mix produced.

CENTRIFUGAL-EXTRACTION METHOD (ASTM D 2172-88)

The procedure for control of plant production (based on the test properties obtained by using the Marshall apparatus or the GTM) may be supplemented by the determination of asphalt content using a centrifugal-extraction apparatus. The measured asphalt content is compared with the optimum asphalt content previously established. Discrepancies may be corrected by plant adjustments, if necessary. After the asphalt has been extracted, the remaining aggregates may be subjected to a sieve analysis for comparison with the previously selected aggregate gradation. Corrections again may be made in the operation of the plant, as necessary. The method is also used to test tar or tar-rubber mixes.

PURPOSE

Use the centrifugal-extraction-device method to determine the asphalt content of a bituminous plant mixture.

EQUIPMENT

Use the following items for this method:

- A balance; 2,000-gram capacity.
- A beaker; 600-milliliter capacity.
- A varnish brush.
- A graduate; 1,000-milliliter glass.
- Evaporating dishes.
- A centrifugal extractor, hand-driven with pad.
- A filter ring (paper-basket type).
- An electric hot plate.

- An electric oven.
- A baking pan.
- Solvent (trichloroethane); 5-gallon can.
- A spatula.
- A cooking spoon.
- Crucible tongs.

The Dulin-Rotarex centrifugal-extraction device and related equipment are shown in *Figure 3-25*.

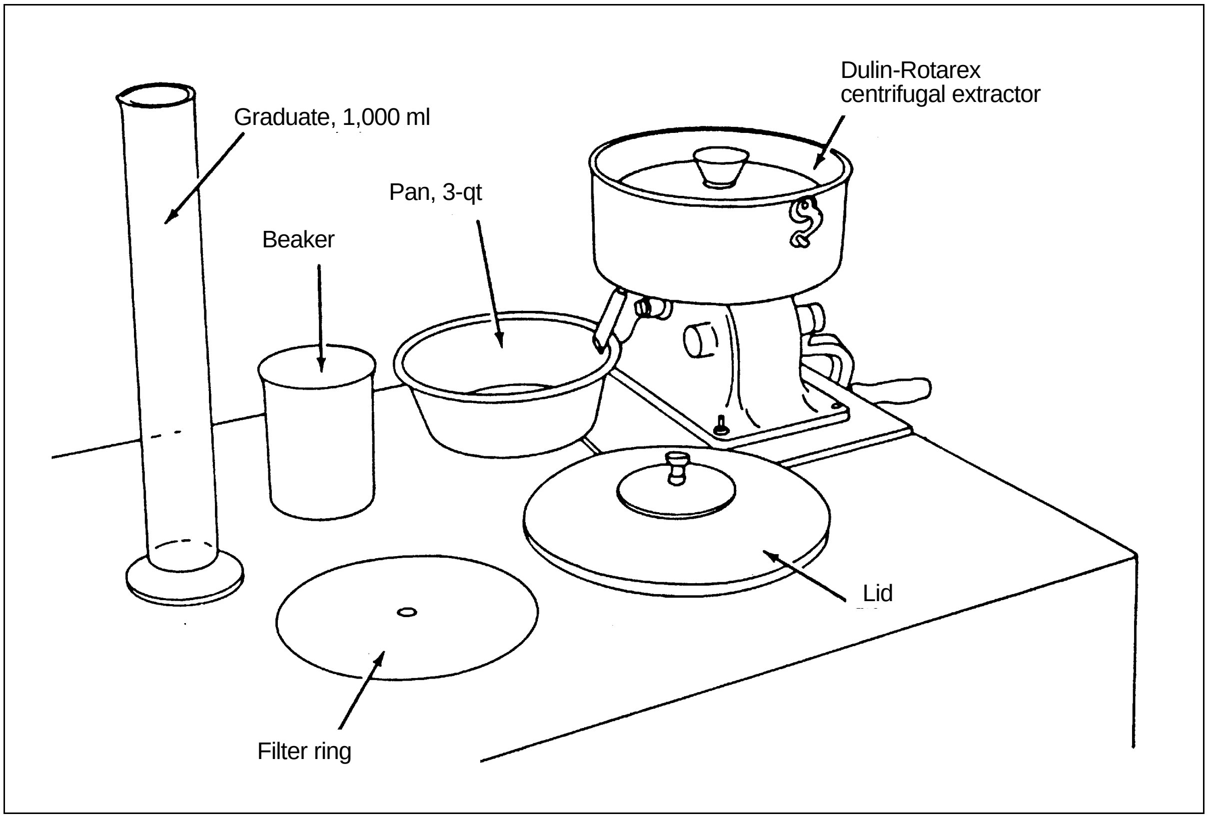


Figure 3-75. Dulin-Rotarex extraction equipment

STEPS

The centrifugal-extraction test may be performed on a representative sample taken from an individual batch or on a composite sample representing several batches. Perform the following steps:

Step 1. Take four samples upon initiation of the asphalt plant operation, one from each of the first four trucks to leave the plant. Delay plant production until the test is complete. After the plant is in normal operations, obtain a suitable composite sample by combining samples taken every 4 hours or every 400 tons of production, whichever comes first. Do this by using a shovel and cutting completely across the stream of hot mixture as it is discharged from the mixer. Place the hot samples in a tin pail or other container with a tight-fitting lid until the last sample for the day is taken.

Step 2. Heat the sample in a pan on the hot plate or in the oven until it is soft enough to be easily disintegrated with a large spoon. Ensure that the individual particles of aggregate are not crushed.

Step 3. Prepare a representative sample by the quartering method and allow it to cool. Weigh not less than 650 grams of a sample containing particles smaller than 1/2 inch and place it in the bowl of the extraction device. Record the initial weight of the sample.

Step 4. Weigh an oven-dried, gasket-type, filter-paper ring and record the weight. Fit the filter paper on the rim of the bowl. Position the cover plate on the bowl and securely lock it in place. Place the bowl on the shaft of the extractor and fasten it securely.

Step 5. Place the empty 1,000-milliliter glass graduate under the spout of the extraction apparatus. Pour about 200 milliliters of solvent into the bowl through the solvent funnel. Solvent flowing through the weep holes indicates a full bowl. Allow the solvent to set in the bowl for 10 to 15 minutes before operating the equipment.

Step 6. Put the lid on and rotate the bowl by turning the hand crank until the solvent discharges from the spout in a thin stream. After the first charge is drained, remove the lid and add more solvent. Replace the lid and rotate the bowl again. Repeat this operation several times until the discharged solvent is clean. With a little experience, the operator can soon judge exactly what treatment is necessary for any given material.

Step 7. Remove the bowl when the last addition of solvent has drained off. Carefully disassemble the bowl and allow the bowl, the cover plate, and the filter ring to air-dry. After air-drying, carefully brush the sample out of the bowl and off the cover plate into the preweighed tare. Oven-dry the sample and the filter ring at 230°F ± 9° to a constant weight. Allow the sample and filter ring to cool in a desiccator. Weigh the sample and record the weights.

Step 8. Subtract the weight of the clean aggregate, plus the weight of any mineral filler retained in the filter paper, from the weight of the original sample to determine the amount of asphalt extracted. This value for the amount of asphalt extracted is subject to correction, depending on the amount of mineral filler contained in the solvent, as indicated below.

Step 9. Determine the amount of mineral filler that passed through the filter paper and is contained in the solvent extract.

a. Measure and record the total amount of solvent extract (volume, in milliliters).

b. Agitate the solvent thoroughly, and measure 100 milliliters into a preweighed evaporating dish.

c. Place the evaporating dish on a hot plate and evaporate over low heat until all that remains in the dish is a black residue.

a. Heat the residue to a dull-red ash (932° to 1,112°F). A laboratory furnace or blow torch may be used to heat the residue. The heating process will burn off any organic matter (asphalt binder) remaining in the residue.

b. Allow the residue to cool to room temperature once it has become a dull-red glowing ash.

c. Prepare a saturated solution of ammonium carbonate and add it to the ash in the proportions of 5 milliliters of solution to 1 gram of ash remaining in the dish. (A saturated solution of ammonium carbonate is prepared by dissolving as much ammonium carbonate into water as the water can retain. At 60°F about 100 grams of ammonium carbonate will dissolve in 100 milliliters of water.)

NOTE: Prepare the solution at room temperature since ammonium carbonate decomposes at 135°F. Allow the ammonium-carbonate solution and ash mixture to stand for 1 hour. Oven-dry the mixture at 230°F to a constant weight. After oven-drying, allow the mixture to cool in a desiccator. Weigh the mixture (in the evaporating dish), and the record the weight.

CALCULATIONS

Perform calculations on DD Form 1793 as shown in *Figure 3-26*.

SIEVE ANALYSIS OF AGGREGATE

The clean aggregate may be subjected to sieve analysis. The amount of material passing the No. 200 sieve (mineral filler) in the sieve analysis must be increased by the weight of filler retained in the filter-paper gasket and in the solvent as determined in the calculations.

TESTING TAR OR TAR-RUBBER MIXES

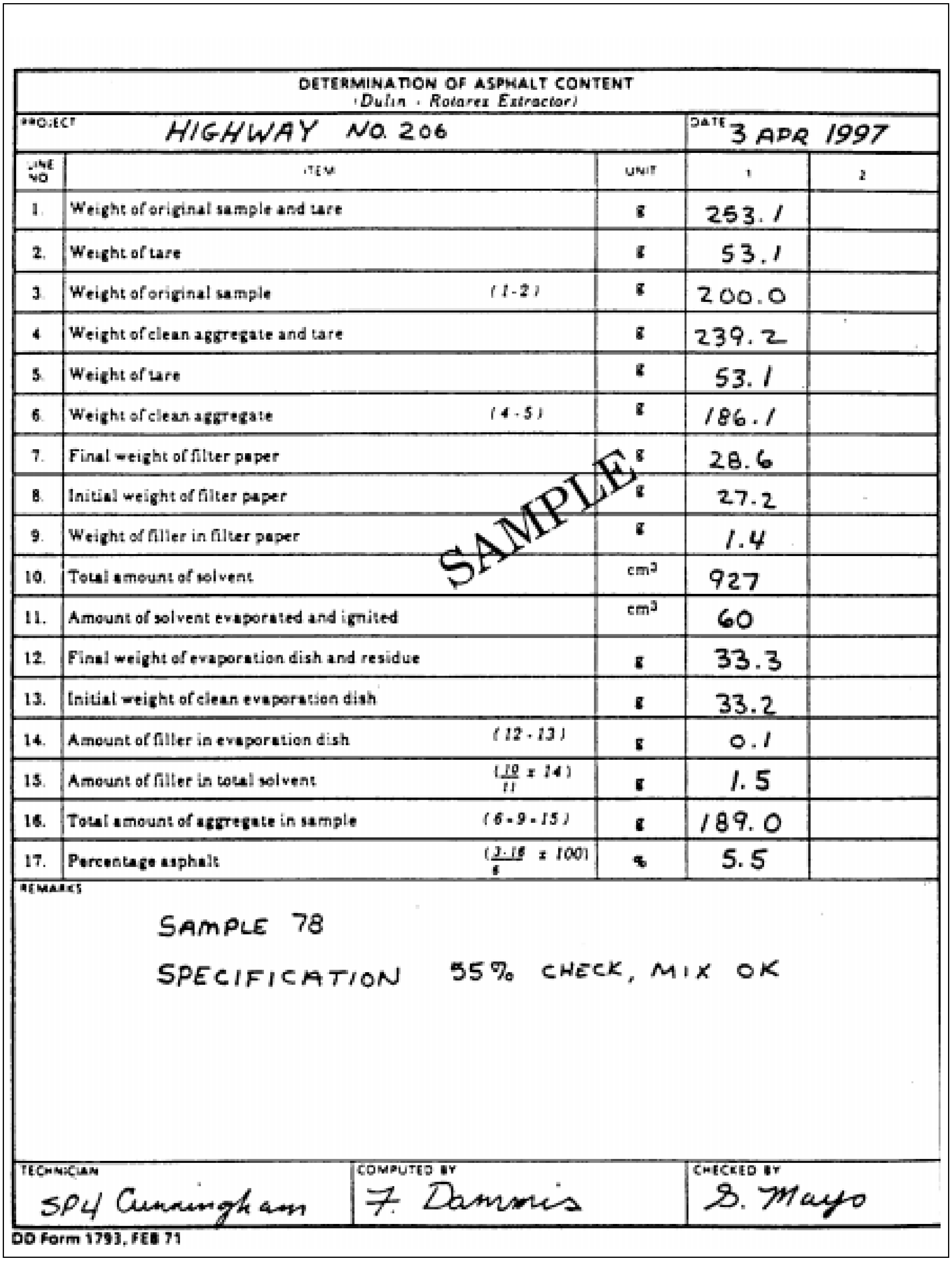
For tar or tar-rubber mixes, the above procedure should be modified as follows:

- Soak the sample in crystal-free creosote overnight.
- Transfer the soaked sample and creosote to the centrifugal apparatus and centrifuge to remove the creosote.
- Wash the sample with benzene until the solvent is a light straw color.

DENSITY TESTS

Density test samples should be taken and tests performed as often as conditions require, but at least once for every 400 tons of mix placed. To obtain a satisfactory specimen, take the samples early in the morning when the pavement is cool. Perform any additional rolling required as a result of the tests during the heat of the day. Take the sample for testing from any portion of the bituminous pavement, provided the area is typical of placing and rolling conditions.

A coring machine or concrete saw may be used for cutting out the samples. Avoid chopped or jack-hammered samples, if possible, as these are likely to develop cracks or other disturbances which would lead to erroneous results. Cut samples completely through the thickness of the pavement and remove



DETERMINATION OF ASPHALT CONTENT
(Dulin - Rotarex Extractor)

PROJECT HIGHWAY NO. 206

DATE 3 APR 1997

LINE NO	ITEM		UNIT	1	2
1.	Weight of original sample and tare		g	253.1	
2.	Weight of tare		g	53.1	
3.	Weight of original sample	(1-2)	g	200.0	
4.	Weight of clean aggregate and tare		g	239.2	
5.	Weight of tare		g	53.1	
6.	Weight of clean aggregate	(4-5)	g	186.1	
7.	Final weight of filter paper		g	28.6	
8.	Initial weight of filter paper		g	27.2	
9.	Weight of filler in filter paper		g	1.4	
10.	Total amount of solvent		cm^3	927	
11.	Amount of solvent evaporated and ignited		cm^3	60	
12.	Final weight of evaporation dish and residue		g	33.3	
13.	Initial weight of clean evaporation dish		g	33.2	
14.	Amount of filler in evaporation dish	(12-13)	g	0.1	
15.	Amount of filler in total solvent	$\left(\frac{10}{11} \times 14\right)$	g	1.5	
16.	Total amount of aggregate in sample	(6+9+15)	g	189.0	
17.	Percentage asphalt	$\left(\frac{3-16}{3} \times 100\right)$	%	5.5	

REMARKS

SAMPLE 78

SPECIFICATION 5.5% CHECK, MIX OK

TECHNICIAN	COMPUTED BY	CHECKED BY
SP4 Cunningham	F. Dammis	D. Mayo

DD Form 1793, FEB 71

Figure 3-74. Sample DD Form 1793

them carefully to avoid damage. In hot weather, it may be necessary to chill the area with ice for 15 to 30 minutes before cutting out the sample.

When density samples are taken from a surface course placed on a binder course, it is practically impossible to remove the specimen from the tacked binder-course surface. To assist in removing samples of the surface course from the binder course, select the spot for the test before laying the surface course. Place a piece of wrapping paper about 18 inches square on the spot selected. On the side of the project least subject to construction traffic, drive two nails into the base or pavement a convenient distance apart and an equal distance from the center of the paper so that you may readily locate the center of the paper after the surface course is laid. The roller should not treat the mix placed over the paper any differently than the remainder of the pavement. The area of pavement over the paper is small, and practice has demonstrated that specimen density secured by this method is about the same as the density in the surrounding pavement. You should not use frames or separators around the proposed sample.

Determine the bulk specific gravity for each specimen and use the following formula to calculate the specimen's density:

$$D = G_m \times 62.4$$

where—

D = density of specimen, in pcf

G_m = *bulk specific gravity of specimen*

62.4 = density of water, in pcf

Compare the calculated density of the bituminous pavement with the densities obtained during the bituminous-mix design and Marshall stability tests. Normally, the field density must be at least 95 percent of the maximum density calculated in the bituminous-mix design. However, individual project specifications do vary, and the acceptable density ranges must be verified for each project. When the density test on samples from the bituminous pavement show that the minimum specified field density has not been obtained, correct the deficiency by additional rolling or remove and replace the pavement. Where constant difficulty is experienced in meeting the specified density, check the job-mix formula thoroughly.

EXPEDITING THE DESIGN

When military expediency demands it, the preliminary laboratory mix designs are eliminated, and the mix is designed directly from plant-produced aggregates. In such cases, the engineering officer will, on the basis of sieve analysis or other information or judgment, select the most promising of the available aggregates and start the plant using this aggregate. As pointed out previously, in nearly all cases the aggregate will show some breakdown of the particles because of heating and screening. Consequently, the grading curve for the material that has passed through the plant will differ from the grading curve of the material before entering the plant. Laboratory tests using aggregate from the plant bins combined in the most desirable proportions must be conducted to determine the optimum asphalt content (OAC).

Manufacture of the paving mix then can be initiated. If the available aggregate must be used, regardless of whether it produces a mix that meets all the design criteria at OAC, the asphalt content should be selected to give a mix which meets the criteria for flow and percent of voids in the total mix. The criteria for stability and percent of voids filled with asphalt is given less consideration.

Chapter 4

Concrete

This chapter discusses the testing of both fresh and hardened concrete and the construction materials used to mix it. These tests are performed to ensure that the concrete meets design requirements before it is poured. Since there are many factors that contribute to the success of the finished product, all test methods must adhere to ASTM standards.

Further information on concrete types, components, design, and uses in military construction can be found in FM 5-428.

SECTION I. CHARACTERISTICS AND IDENTIFICATION

Concrete is one of the most economical, versatile, and universally used construction materials. It is one of the few building materials that can be produced by the user directly on the job to meet the specific requirements. Concrete is an artificial stone which, when first mixed, forms a plastic or putty-like mixture. This mixture can then be placed into a form and allowed to harden or cure for a prescribed length of time. When cured, the finished concrete is a hard, stone-like material. It is used for pavements, foundations, dams and retaining walls, bridges, and buildings of all types.

DESCRIPTION AND COMPONENTS

Concrete is a mixture of portland cement, fine and coarse aggregates, entrapped or entrained air, and water. During mixing, the cement, air, and water form a fluid paste that contributes to thorough mixing and effective placement of the concrete. The cement and water, when mixed, combine chemically to bind the aggregate particles together. This combining process—called hydration—results in a rapid development of strength in the first few hours after mixing, followed by less rapid gains in strength during the following weeks.

CEMENT

Cement is a substance that hardens with time and holds or entraps objects or particles in a definite relation to each other. For concrete, portland cement usually is used.

Portland cement is a substance that when mixed with water, hardens and binds objects or particles together to form concrete. This process begins immediately and continues as long as moisture and temperature conditions are favorable. As hydration continues, concrete becomes harder and stronger. Most of the hydration takes place during the first 30 days. Hydration can continue well over 50 years but at a much slower rate. References to cement in this manual mean portland cement. The ASTM specifies eight common types of portland cement (ASTM C 150-97). These are adequate for most purposes. The various types of portland cement are known as hydraulic cements because they are capable of hardening and developing strength in the presence of water. These cements include—

- Type I. Cement used for general construction when special properties for any other type are not required.
- Type IA. Air-entraining cement used for the same purposes as Type I, except that air entrainment is desired. Entrained air improves workability and provides resistance to frost action, freezing, and thawing.
- Type II. Cement used for general purposes, especially when moderate sulfate resistance or moderate heat of hydration is desired. It has a lower heat of hydration than the normal Type I, generates heat at a slower rate, and has improved resistance to sulfate attack. Type II cement is used in locations where a high temperature rise in the concrete is objectionable, as in structures of considerable mass such as large piers, heavy abutments, and heavy retaining walls.
- Type IIA. Air-entraining cement used for the same purposes as Type II, except that air entrainment is desired.
- Type III. Cement used when a high strength is needed quickly. This may be due to a demand for early use or in cold-weather construction to reduce the period of protection against low or freezing temperatures.
- Type IIIA. Air-entraining cement used for the same purposes as Type III, except that air entrainment is desired.
- Type IV. Cement used when a low heat of hydration is desired to keep the amount and rate of heat generated to a minimum. Type IV cement develops strength at a slower rate than Type I cement but helps prevent the development of high temperatures in the structure with the attendant danger of thermal cracking later when it cools.
- Type V. Cement used when high sulfate resistance is desired. Sulfates react chemically with the cement compounds, causing undesirable expansion of the mixture. The sulfates may be present in the water used to mix the concrete or may be created by sulfurous gases from nearby industrial areas. The principal source of sulfate attack, however, occurs on foundations and other concrete in contact with the earth in certain regions and is caused by a reaction between the groundwater (containing dissolved reactive minerals or acid) and the hardened cement. Type V cement is low in calcium aluminate and is highly resistant to sulfate attack.

AIR-ENTRAINED CEMENT

Concrete made with air-entrained cement is resistant to severe frost action and to salts used for ice and snow removal. In general, air entrainment may be controlled to a much greater extent by using admixtures with normal cements during mixing. This combination results in concrete with tiny, distributed and separated air bubbles (up to millions per cubic foot). The entrained air bubbles improve the workability of fresh concrete. These bubbles reduce the capillary and water-channel structure within water, which prevents the buildup of damaging water. Air-entrained concrete has greatly increased durability in outdoor locations exposed to freezing weather. Each of the first three Types (I, II, and III) are available as air-entrained. To signify this characteristic, a letter A is added after the type. For example, a Type II cement with an air-entrained admixture is identified as Type IIA.

WATER

Water plays an important part in the concrete mix. Its principal uses are to make the mix workable and to start the chemical reaction. Any material in the water that retards or changes the reaction is detrimental. A good rule of thumb is, "If it's good enough to drink, it may be used for concrete."

Ordinary Water

The materials found in some types of water include organic compounds, oil, alkali, and acid. Each has an effect on the hydration process.

- Organic material and oil. These compounds tend to coat the aggregate and cement particles and prevent the full chemical action and adherence. The organic material may also react with the cement and create a weakened cementing action, thus contributing to deterioration and structural failure of the concrete.
- Alkalies, acids, and sulfates. Certain limiting amounts of these chemical impurities in the water tend to react adversely with the cement. The result is inadequate cementing and weakened concrete. Water must be substantially free of these chemicals for use in concrete mixing.

Sea Water

The salts in sea water are normally thought of as being corrosive. However, sea water is sometimes used for concrete mixing with satisfactory results. A 10 to 20 percent loss in compressive strength can be expected when using the same amount of sea water as fresh water. This can be compensated for somewhat by reducing the water-cement ratio.

AGGREGATES

The aggregates commonly used for concrete are natural deposits of sand and gravel, where available, or crushed stone. Crushed aggregate may cost more to produce; however, this may be the only way to obtain substantial quantities of large-sized stone. Artificial aggregates such as a blast-furnace slag or specially burned shales and clays are used.

Aggregates are divided into the following types:

- Fine aggregate.
- Coarse aggregate.

When properly proportioned and mixed with cement, these two groups will yield an almost voidless stone that is strong and durable. Aggregate should be equal to or better in strength and durability than the hardened cement paste if it is to withstand the design loads and effects of severe weather.

Fine Aggregates

Fine aggregates are the material that will pass a No. 4 sieve and will be predominantly retained on a No. 200 sieve. To increase workability and for economy as reflected by using less cement, the fine aggregates should have a rounded shape. Their purpose is to fill the voids between coarse-aggregate particles and to modify the concrete's workability. This workability characteristic is discussed more in the description of finished concrete.

Coarse Aggregates

Coarse aggregates are the material that will be retained on a No. 4 sieve. In determining the maximum size of coarse aggregate, other factors must also be considered. The coarser the aggregate used, the more economical the mix, as aggregate costs less than cement. Larger pieces offer less surface area of the particles than an equivalent volume of small pieces. Using the largest permissible maximum size of coarse aggregate permits a reduction in cement and water requirements. One restriction usually assigned to coarse aggregate is its maximum size. Large pieces can interlock and form arches or obstructions within a concrete form. This restricts the area below to a void or at best, fills the area below with the finer particles of sand and cement. This is either a weakened area or a cement-sand concentration that does not leave enough mortar to coat the rest of the aggregate. The capacity of mixing equipment, the spacing of reinforcement, or the minimum width of forms limits the maximum aggregate size. A listing of maximum sizes of coarse aggregate is indicated in Section II of this chapter.

PROPERTIES OF CONCRETE

To combine the ingredients correctly and to form the required concrete, it is essential to know the required physical properties of both the plastic and the hardened concrete. The hardened concrete must have the following properties:

- Strength.
- Durability.
- Watertightness.
- Workability.
- Consistency.
- Uniformity.

The quality and character of the hardened concrete is greatly influenced by the properties of the mix when it is plastic. To attain optimum quality, the plastic mix must be uniform, consistent, and workable. This permits placing the concrete without developing segregation, honeycombing, or other defects in filling the forms or in producing the desired smooth, hard, and resilient surface.

STRENGTH

Strength is the concrete's ability to resist a load in compression, bending, or shear (see Sections IV and V of this chapter). The desired design strength is obtained by proportioning the mixture with correctly graded aggregates, an adequate amount of cement to coat the surface area of the particles, and the proper amount of mixing water. The most important influencing factor on strength is the ratio of water to cement (W/C ratio). For plastic and workable mixes, lower values of the W/C ratio give higher strengths. Two and one-half gallons of water is the minimum amount necessary to hydrate a sack of cement adequately.

This minimal amount of water is not sufficient to economically provide the needed plasticity and workability for freshly mixed concrete. Additional water must be added to the mixture to improve workability but must be minimized to obtain the desired strength with an economical cement content. Additional water thins the paste content and therefore coats more particles. This increases the yield from each sack of cement and produces a more economical mix. Excessive amounts of water (too high a W/C ratio) weakens the paste by allowing the cement particles to hydrate while suspended in water without being in contact with the aggregate or other cement particles. This water eventually evaporates, leaving holes or voids in the hardened concrete that cause additional losses in strength. Minimum and maximum amounts of water are specified to assure an economical mix with no loss in strength. This ranges from 4 to 8 gallons per sack of cement (94 pounds).

DURABILITY

Durability is the concrete’s ability to resist the elements of weathering and loading. The primary elements affecting concrete are wind, abrasion, freezing and thawing, wetting and drying, and the chemical action of salts. As the W/C ratio is increased (4 gallons per sack), more voids develop in the hardened concrete. Therefore, more surface area is available for the detrimental elements to attack, resulting in a less-durable structure. Weak or easily crushed rock or other mineral particles that break down under applied loads introduce internal stresses that cause a breakdown of the concrete. Rocks or mineral particles that are absorptive or susceptible to swelling when saturated will deteriorate when subjected to severe weather conditions. Freezing moisture causes expansion stresses that can easily rupture absorptive rocks. Rocks swollen from the sun's radiant heat and then subjected to shrinkage from sudden cooling by rain or temperature drop may break down from the severe weathering. The concrete aggregate must withstand all these forces of nature.

WATERTIGHTNESS

A well-mixed, well-proportioned concrete presents a solid surface to prevent water penetration. Superficial voids permit some water to enter below the concrete’s surface but the water soon meets a dense, solid mass that prevents further penetration. As the W/C ratio is increased, the excess water forms more holes or voids that eventually interconnect to form channels into and throughout the concrete. The end result is a more porous concrete that permits water to pass. For watertightness, 6 gallons of water or less per sack of cement will meet the requirement.

WORKABILITY

Workability is the relative ease of difficulty of placing and consolidating concrete. It is controlled primarily by the amount of each aggregate in proportion to a given quantity of cement paste. As more aggregate is added to a given amount of paste, the mixture becomes harsh and stiff. The increased stiffness makes it more difficult to work the concrete into the forms and around the reinforcing bars. The consistency needed depends on the conditions under which the concrete must be placed and finished. Very dry and stiff mixtures may be placed in most situations where high-frequency vibrations are used to assist in consolidating and compacting fresh concrete. In other situations, difficult placing conditions may require a more fluid concrete mixture to fill narrow forms and to flow around reinforcement.

CONSISTENCY

Concrete is a fluid mixture containing particles of different size, shape, and mass. Heavier particles have a tendency to settle out through the mixture faster than lighter particles. Often the result is a segregated mixture of a very poor quality. When concrete is properly proportioned and mixed and carefully handled, segregation is held to a minimum. The mixture must have the proper proportion of cement/sand mortar to prevent the larger coarse-aggregate particles from separating from the batch during mixing, transporting, and placing. When cement is allowed to drop (free fall) over a considerable distance, it can cause segregation of the mixture. To minimize segregation for drops in excess of 3 to 5 feet, bottom dump buckets should be used to place concrete as close to the final location as possible. See TM 5-742 for construction procedures.

UNIFORMITY

Uniformity refers to a single batch of concrete and to all batches for an entire project. The same amount of each ingredient should be mixed into each batch or a nonuniform structure will result. Design would not be met in all sections of the structure and possible failure of these sections could result. Proper supervision in mixing and handling of the concrete ensures uniformity.

CONCRETE CURING

Concrete does not develop its full strength until the chemical process of curing (hydration) is complete. Cement must have sufficient water to continue its hydration. Curing is the means of keeping water available so the hydration can continue. The curing process takes place over an extended period. The most critical time is the first 7 days. The extent and rate of curing depends on the—

- Temperature within the concrete.
- Presence of moisture.

TEMPERATURE

The ideal temperature for concrete work is between 55° and 70°F. Above this temperature, rapid evaporation of moisture creates serious problems such as—

- Increased water demand.
- Slump loss.
- Decreased setting time.
- Increased tendency for plastic shrinkage cracking.

The hydration process is delayed at lower temperatures. Temperatures below 32°F completely stop the hydration process. Since the chemical reaction gives off some heat, proper methods must be used to keep the heat within the structure during times of low temperatures. Cold-weather construction may require heating the individual ingredients or the concrete and covering the emplaced concrete or providing a heated enclosure. In hot weather, extra care is required to prevent a high temperature rise and rapid drying of the fresh concrete. Spraying the aggregate stockpiles with cool water helps lower the concrete temperature. To keep the water as cool as possible, reflective white or aluminum paint is applied to the water supply lines and storage tanks.

On massive construction projects, such as dams and heavy retaining walls, the mixing water is often kept cool by substituting ice for part of the mixing water. The ice must be melted by the time the concrete is fully mixed and is ready to leave the mixer. Large voids result from unmelted ice in the concrete. Cement replacement materials (such as pozzolans and diatomaceous earth, pumicites, or fly ash) may be used to depress concrete temperature by reducing the heat of hydration in a structure. However, pozzolans vary widely and may have adverse effects on strength, air content, and durability if used in excessive amounts.

MOISTURE

Concrete curing depends on a chemical reaction in the presence of water. Moisture lost during the curing process—by seepage or evaporation—delays or prevents a complete hydration of the cement and ultimately prevents the development of optimum strength and watertightness. Saturating the subgrade on which the concrete will be placed will delay, if not prevent, seepage from occurring. Impervious membranes (plastic or polyethylene sheets) can also be used to prevent seepage through the subgrade. Wood forms should be thoroughly wetted if they have not been otherwise treated with a moisture sealer.

One method of reducing evaporation is to cover the concrete with a material such as straw, burlap, plastic, or a sprayed-on chemical curing compound as soon after finishing as possible. The preferred method of curing is by using continuous sprays and flowing or ponded water after the concrete has set initially so it does not damage the finish. This water application can also be part of the temperature control during cold- and hot-weather concreting. The increase of the concrete’s compressive strength with age is shown by the curves in *Figure 4-1, page 4-8*. Note the long-time gain in strength that occurs when proper temperature and moisture conditions are maintained.

CONCRETE ADMIXTURES

Chemical agents or admixtures are available for almost any purpose such as increasing workability, durability, and strength or compensating for inadequate curing conditions.

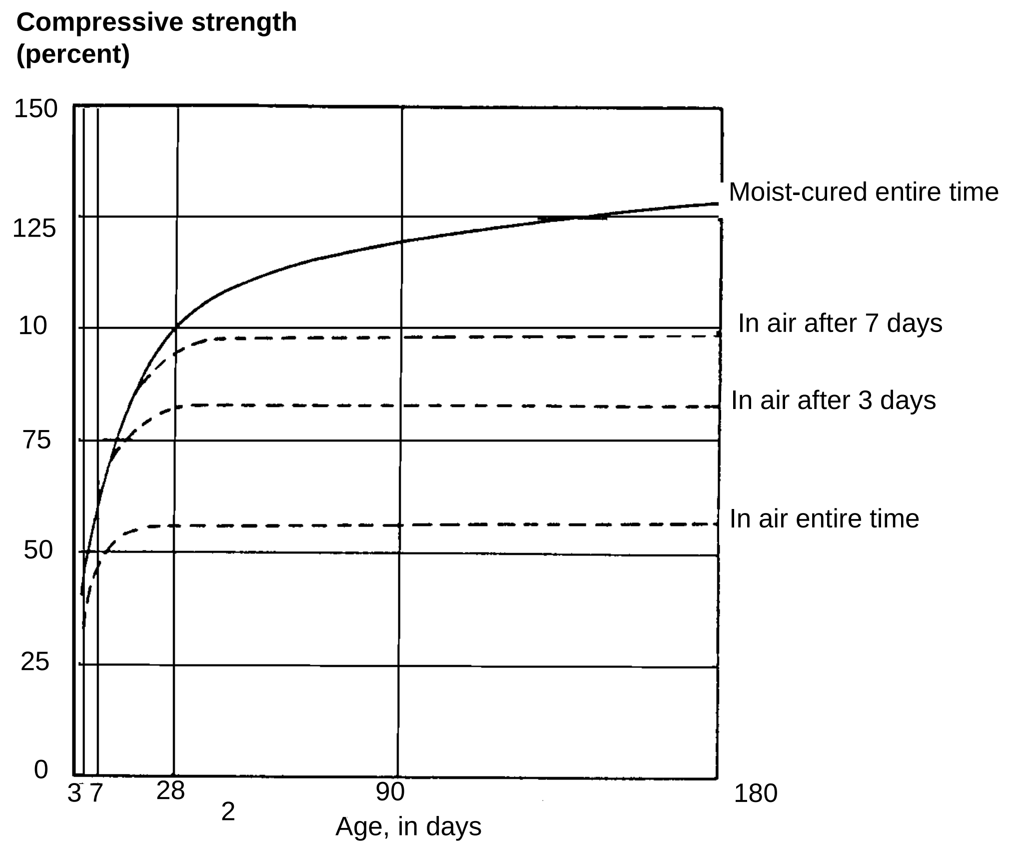


Figure 4-1. Increase of compressive strength with curing age

ACCELERATORS

Sometimes it is desirable to accelerate the hydration process to obtain a high-early strength and a high rate of heat production. This combination is useful for cold-weather concreting operations. The addition of a chemical accelerating admixture (generally calcium chloride) to the concrete mixture produces the desired reactions. The recommended maximum dosage for calcium chloride is 2 percent by weight of cement. The ultimate strength of concrete will be slightly lower with the use of an accelerator.

RETARDERS

Retarders are used when excessively high heat or too-rapid setting of concrete will prevent full hydration of the cement. Many materials retard the setting of concrete, but the most common is hydroxylated carboxylic acid salts. Sugar has also been used quite successfully.

AIR-ENTRAINING AGENTS

The greatest improvement in watertightness and resistance to the disruptive action of freezing and thawing is obtained by incorporating 4 to 7.5 percent by volume of entrained air into the concrete. Workability of fresh concrete is also enhanced by entraining air. Soaps, oils, acids, wood resins, alkali salts, fine pozzolans, and several proprietary compounds are available for use as air-entraining admixtures with hydraulic cement. These agents form very small, uniformly spaced, discrete air voids that relieve the buildup of damaging pressures from the expansion of freezing water into ice.

WATER REDUCERS (PLASTICIZERS)

The concrete's workability is governed by the proportions of cement, water, and aggregate in a concrete mixture. When a reduction of aggregate or an increase in cement is impractical, the concrete's workability can be increased by adding a water-reducing admixture or plasticizer. Another primary characteristic is the strength gained from a decreased water demand. Less water is required for the same workability, which leads to a lower W/C ratio, and therefore higher strength. Water requirements may be reduced as much as 10 percent for most water-reducing admixtures. Air-entraining agents are also considered as plasticizers because the void system reacts as a lubricant in concrete.

SECTION II. AGGREGATE TESTING

Aggregate used in mixing concrete is a mixture of fine and coarse material, usually sand with either natural gravel or crushed rock. It serves as an inert filler to provide the bulk material required. Well-graded aggregates contain particles of all sizes, from the largest permitted by the dimension of the member to be formed to sand fines. The smaller particles fill the spaces between the larger particles, thus providing a dense material that requires a minimum of cement paste for binder. The aggregate materials must be clean and hard, resist weathering, and have no unfavorable reaction with the cement.

An aggregate must provide maximum strength and durability in a concrete mixture. Fineness, coarseness, and aggregate gradation are factors considered when deriving the correct concrete mix for a specific construction purpose. Specific gravity, absorption, and moisture also affect the aggregate's ability to bind well with cement and water in a concrete mix. The components of the final mix (cement, water, and aggregate) must bond adequately for structural strength and must resist weather and loads. Correct aggregate selection also reduces the project's cost. An engineering analysis determines the aggregate best suited for a particular purpose. Testing allows the best selection.

For the aggregate tests to be worthwhile, the samples for testing must be representative of the aggregates to be used. Take aggregate samples as close as possible to the finished product to give the best representative sample of the aggregate. Take a sufficient size and number of samples from the processing-plant's discharge point to represent the material in the stockpile. The sample should consist of at least four times as much material as is needed for the tests and should be reduced to the desired size through splitting and or quartering the sample. Minimum sample sizes can be found in *Table 4-1, page 4-10*.

Table 4-1. Minimum sample sizes

Nominal Maximum Size		Minimum Weight of Test Sample	
mm	**in**	**kg**	**lb**
12.5 or less	1/2 or less	2	4.4
19.0	3/4	3	6.6
25.0	1	4	8.8
37.5	1 1/2	5	11.0
50.0	2	8	18.0
63.0	2 1/2	12	26.0
75.0	3	18	40.0
90.0	3 1/2	25	55.0
100.0	4	40	88.0
112.0	4 1/2	50	110.0
125.0	5	75	165.0
150.0	6	125	276.0

STOCKPILE SAMPLING (ASTM D 75-87)

It is difficult to ensure that unbiased samples are obtained from stockpiles. This is due to the segregation that often occurs when material is stockpiled, with coarser particles rolling to the outside base of the pile. For coarse or mixed coarse and fine aggregates, every effort should be made to enlist the services of power equipment to develop a separate, small sampling pile composed of materials drawn from several increments.

When power equipment is not available, take samples from at least three increments—from the top third, the midpoint, and the bottom third of the pile. Shove a board vertically into the pile just above the sampling point to prevent further segregation. When sampling fine-aggregate stockpiles, remove the outer surface before taking the sample.

Take samples from near the top, middle, and bottom of the stockpile and recombine them to represent their particular stockpile. Push a board into the stockpile just above the points of sampling to prevent the material above the sampling points from falling into the sample and causing size contamination.

Pit samples are sources of sand and gravel. Sample them by channeling exposed faces or channeling in pits if exposures are not available. Take care to ensure that the samples include only materials that are below the overburden or strip zone.

GRADATION DETERMINATION

Gradation of aggregate refers to the distribution of particles of aggregate among various sizes. Aggregates having a smooth grading curve and neither a deficiency nor an excess of any one particle size usually produces mixtures with fewer voids between particles. A too-large proportion of coarse aggregate leaves voids that require more cement paste to fill. This affects the economy of the mix. Too much fine aggregate increases the amount of surface area that must be coated with cement paste. This may weaken the concrete and is uneconomical. Good gradation results in—

- A dense mass of concrete with a minimum volume of voids.
- An economical mix.
- A strong structure.

Optimum strength, water tightness, and durability in the hardened concrete require careful control of aggregate gradation.

A gradation or sieve analysis indicates whether an aggregate's particle-size distribution meets the project's requirements. Dense aggregates can result in a concrete that is denser and stronger and more economical, watertight, and resistant. See ASTM C 136-90 for analysis methods and *Table 4-2 and Tables 4-3* and *4-4, page 4-12*, for recommended size and gradation limits.

Table 4-2. Maximum recommended size of coarse aggregate

Structure	**Minimum Dimension (Inches)**			
	2 1/2 to 5	**6 to 11**	**12 to 29**	**30 or More**
Reinforced walls, beams, and columns	1/2 to 3/4	3/4 to 1 1/2	1 1/2 to 3	1 1/2 to 3
Unreinforced walls	3/4	1 1/2	3	6
Slabs, heavily reinforced	3/4 to 1	1 1/2	1 1/2 to 3	1 1/2 to 3
Slabs, lightly reinforced	3/4 to 1 1/2	1 1/2 to 3	3	3 to 6
NOTE: Maximum size not to exceed 1/5 of minimum dimension of a wall or similar structure, 1/3 of slab thickness for horizontal slab, or 3/4 of minimum clear spacing between reinforcing bars				

APPARATUS, TEST PROCEDURES, AND CALCULATIONS

The apparatus, test procedures, and calculations required to determine the gradation of aggregate for portland-cement concrete are the same as explained for sieve analysis, except that the No. 4 sieve is taken as the dividing line between fine and course aggregates. The minimum sample size required in the sieve analysis of fine aggregate is 500 grams. The result of this test is a gradation curve for the aggregate concerned.

MATERIAL FINER THAN .075 MILLIMETERS (NO. 200 SIEVE)

The extremely fine mineral material (clay, silt, dust, or loam) occurring in most aggregates requires relatively large increases in the amounts of mixing water. Fines tend to work to the surface of concrete and cause cracking upon drying, due to shrinkage. If the fines adhere to the larger aggregate particles, they also tend to interfere with the bond between the aggregate particles and cement-water paste. Specifications limit the amount of such material to a small percentage. ASTM C 117-95 gives the standard test method for fine materials. The apparatus, test procedures, and calculations to determine this percentage are described in the test for impurities. Fine material, not to exceed 3 to 5 percent of the total aggregate weight, is generally not harmful to concrete. For some purposes, a small amount of such fines may improve the workability.

Table 4-3. Desirable gradation for coarse aggregate in concrete

Percent Passing Indicated Sieve											
Nominal Sieve Size (Inches)	**4**	**3 1/2**	**3**	**2 1/2**	**2**	**1 1/2**	**1**	**3/4**	**1/2**	**3/8**	**No. 4**
3 1/2 to 1 1/2	100	90 to 100		25 to 60		0 to 15		0 to 5			
2 1/2 to 1 1/2			100	90 to 100	35 to 70	0 to 15		0 to 5			
2 to 1				100	90 to 100	35 to 70	0 to 15		0 to 5		
2 to No. 4				100	95 to 100		35 to 70		10 to 30		0 to 5
1 1/2 to 3/4					100	90 to 100	20 to 55	0 to 15		0 to 5	
1 1/2 to No. 4					100	95 to 100		35 to 70		10 to 30	0 to 5
1 to No. 4						100	95 to 100		25 to 60		0 to 10
3/4 to No. 4							100	90 to 100		20 to 55	0 to 10
1/2 to No. 4								100	90 to 100	40 to 70	0 to 15
3/8 to No. 4									100	85 to 100	10 to 30

Table 4-4. Desirable gradation for fine aggregate in concrete

Sieve Size US Standard	Percent by Weight Passing
4	95 to 100
8	80 to 100
10	75 to 95
16	50 to 85
20	40 to 75
30	25 to 60
40	20 to 50
50	10 to 30
60	10 to 25
100	2 to 10

FINENESS MODULUS

Fineness modulus is an empirical factor that gives a relative measure of the proportional particle-size distribution of an aggregate's fine and coarse particles. The fineness modulus does not represent any gradation of the material although the process is similar. A 500-gram sample of sand is sieved through a series of sieves (No. 4, 8, 16, 30, 50, and 100). The weight retained on each sieve is converted into a cumulative weight and a cumulative percentage retained, starting with the No. 4 sieve. The sum of the 6 percentages divided by 100 is the fineness modulus. Another procedure for determining the fineness modulus is calculated using the cumulative percentage passing, the usual means of expressing aggregate gradation. The total number of sieves involved times 100 minus the sum of the cumulative percentage passing and divided by 100 gives the fineness modulus. The fineness-modulus values range from 2.20 for fine aggregate to 7.50 for coarse aggregate. Typical values are 2.70 for fine aggregate, 7.40 for coarse aggregate, and 5.80 for 35 to 65 fine-coarse combination. Fineness-modulus ranges for fine aggregate are shown in *Table 4-5*.

Table 4-5. Fineness-modulus ranges for fine aggregates

Fineness Modulus	Designation
2.3 to 2.6	Fine sand
2.6 to 2.9	Medium sand
2.9 to 3.1	Coarse sand

TESTS FOR SPECIFIC GRAVITY, ABSORPTION, AND SURFACE MOISTURE

Perform tests for specific gravity, absorption, and surface moisture on the aggregates before making the necessary calculations to design the concrete mixture. For aggregates used in portland-cement concrete, measure to determine the bulk specific gravity of the aggregates in a saturated, surface-dry (SSD) condition. This is the condition in which the pores in each aggregate particle are filled with water and no excess water is on the particle surface. When used in concrete, this moisture condition of an aggregate can be defined as neither absorbing water from nor contributing water to the concrete mixture. Specific gravity is thus based on determining the total volume occupied by the aggregate particles, including the permeable pore space. Absorption and surface-moisture determinations are necessary to calculate the amount of mixing water used in a concrete mixture.

SPECIFIC GRAVITY AND ABSORPTION OF COARSE AGGREGATE (ASTM C 127-88)

This test method covers the specific gravity and absorption of coarse aggregate. The specific gravity may be expressed as bulk specific gravity, bulk specific gravity SSD, or apparent specific gravity.

Equipment

Use the following items to perform tests for bulk specific gravity SSD, percent absorption, and surface moisture:

- A balance, sensitive to 0.5 gram, capable of suspending the sample container in water from the center of the weighing platform or pan of the weighing device.
- A wire sample basket or a bucket with a 4- to 7-liter capacity for 1 1/2 -inch or smaller aggregate and a larger basket or bucket for larger aggregate sizes.
- A water tank large enough to hold the basket.
- A pycnometer, 2 to 3 cubic feet.
- A heat source (oven or hot plate).
- A metal sample container.
- A metal spatula.
- An absorbent towel.

Steps

Perform the following steps to determine the bulk specific gravity of coarse aggregate in an SSD condition:

Step 1. Wash a representative sample over the No. 4 sieve to obtain a sample size according to *Table 4-1, page 4-10.*

Step 2. Dry the sample to a constant weight at 110°C ± 5°.

Step 3. Allow the sample time to cool to 50°C, immerse it in water, and allow it to soak at room temperature for 24 hours.

Step 4. Remove the sample from the water and roll it in a large, absorbent cloth until all visible films of water are removed. The surfaces of the particles will still appear to be slightly damp. The larger fragments may be wiped individually. The aggregate sample is now in an SSD condition. Weigh the sample in air in its SSD condition. Record this and subsequent weights to the nearest 0.5 gram on DD Form 1208.

Step 5. Place the weighed SSD sample immediately in the wire basket container. Determine its weight in water at 23°C ± 1.7°. Shake the basket or container while it is immersed to remove any entrapped air. This weight is the immersed weight (or weight in water).

Step 6. Calculate the bulk specific gravity in an SSD condition as follows:

$$\frac{B}{B - C}$$

where—

B = weight, in grams, of SSD sample in air

C = weight, in grams, of SSD sample in water

SPECIFIC GRAVITY OF FINE AGGREGATE (ASTM C 128-93)

This test method covers the specific gravity and absorption of fine aggregate. The specific gravity of fine aggregate may be expressed as bulk specific gravity, bulk specific gravity SSD, or apparent specific gravity. For this test method, fine aggregate is defined as material smaller than the No. 4 sieve and larger than the No. 200 sieve.

Equipment

Use the following items to perform this test:

- A pycnometer; 500-milliliter.
- A mold; metal-frustum (half-cone brass mold) water-absorption cone.
- A metal flat-head tamper.

Steps

Perform the following steps to determine the bulk specific gravity of fine aggregate in an SSD condition:

Step 1. Obtain a representative sample weighing about 1,000 grams.

Step 2. Dry the sample to a constant weight at 110°C.

Step 3. Cool the sample to a comfortable handling temperature. Immerse it in water, and allow it to soak for 24 ± 4 hours.

Step 4. Decant the excess water carefully, ensuring that no loss of fines occurs. Spread the sample on a flat, nonabsorbent surface and stir it to obtain uniform drying. Continue drying the sample until it approaches a surface-dry condition.

Step 5. Place the metal frustum water-absorption cone (half-cone brass mold, see *Figure 4-2*) with the large opening down on a smooth surface and fill it loosely with the aggregate. Lightly tamp the surface (raise the metal tamper about 5 millimeters and allow it to fall under its own weight) of the aggregate 25 times with the metal tamper.

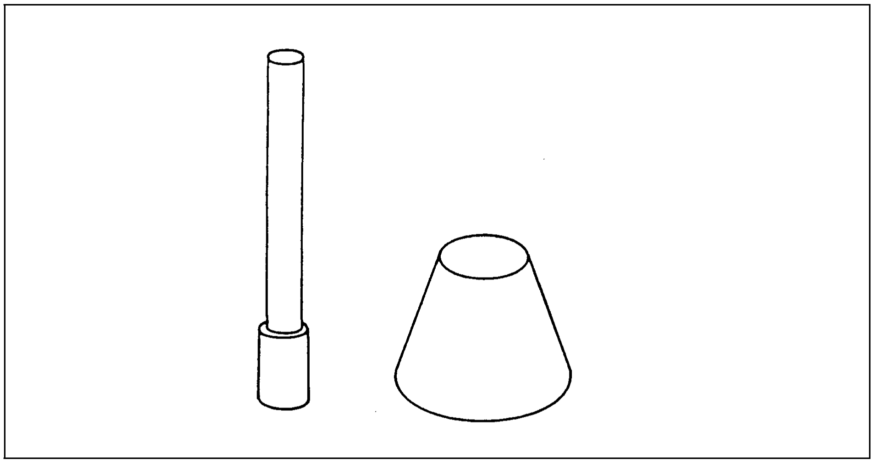


Figure 4-2. Water-absorption cone and tamping rod

Step 6. Remove the loose sand from around the base and lift the mold vertically. The fine aggregate is at the SSD condition when it slightly slumps when you lift the mold. If the material does not slump, continue the drying, accompanied by constant stirring. Repeat the cone tests at frequent intervals until the cone of fine artillery slumps slightly upon removal of the water-absorption cone.

Step 7. Weigh 500 ± 10 grams of the SSD sample, and introduce it into a partially water-filled 500-milliliter pycnometer. Agitate the sample to remove all entrapped air bubbles. Adjust the water temperature to 23°C ± 1.7° and fill the pycnometer to 90 percent of its calibrated capacity. Roll, invert, and agitate the pycnometer 15 to 20 minutes to eliminate the air bubbles. Fill the pycnometer to calibrated capacity, weigh it, and record the weight to the nearest 0.1 gram.

Step 8. Calculate the bulk specific gravity in an SSD condition as follows:

$$\frac{S}{B + S - C}$$

where—

B = weight, in grams, of pycnometer filled with water to calibrated capacity

S = weight, in grams, of SSD specimen

C = weight, in grams, of pycnometer filled with the sample and water to calibrated capacity

COARSE- AND FINE-AGGREGATE ABSORPTION

Absorption in aggregates is the aggregate's ability to steal moisture from the concrete-mix design until its thirst or attraction is satisfied.

Equipment

The following procedure is a continuation of the specific-gravity determinations; therefore, the same equipment shall be used.

Steps

Perform the following steps to determine the percent absorption of coarse and fine aggregates.

Step 1. Weigh the coarse aggregate in water and the fine aggregate in the pycnometer.

Step 2. Remove the aggregates and dry to a constant weight at a temperature of 110°C ± 5°.

Step 3. Weigh and record the oven-dried samples.

Step 4. Calculate the percent of absorption using the following formula:

$$P = \frac{S - A}{A} \times 100$$

where—

P = absorption of the aggregate, in percent

S = weight of SSD specimen, in grams

A = weight of SSD sample in the oven-dried state, in grams

The percent absorption represents the moisture content (oven-dried basis) of the aggregate when it is in an SSD condition.

SURFACE MOISTURE

Surface moisture is the excess moisture remaining after the absorption requirement of the aggregate has been met. This excess moisture determines how much water is added to the concrete mix to meet the required W/C ratio for the proper strength requirements. Perform this test just before mixing the concrete as designed. This allows for adjusting the water, coarse-, and fine-aggregate weights to retain design integrity.

Surface moisture is the water present in both the fine and coarse aggregates, exceeding that which corresponds to an SSD condition. This water will become part of the mixing water when the aggregate is used in making concrete. The amount of mixing water used must be corrected to allow for its presence. See ASTM C 566-89 and ASTM C 70-79.

SECTION III. FRESH-CONCRETE TESTS

After concrete is first mixed, a slump test and an air-content test are performed and used as a control measure to determine the concrete's quality and consistency throughout a project.

Take samples of concrete for test specimens at the mixer by repeatedly passing a receptacle through the entire discharged stream until sufficient concrete is collected into the pan. In the case of ready-mixed concrete, take samples from the transporting vehicle while it's discharging the concrete (see *Figure 4-3*).

Figure 4-3. Sampling concrete from a truck mixer

The contents of a paving mixer should be discharged into a pile and sample material taken by a shovel from at least five different portions of the pile. The sample of concrete from which test specimens are made will be representative of the entire batch. Obtain two or more samples by repeatedly passing a scoop or pail through the discharging stream of concrete from the middle portion of the batch to obtain the amount of material required by the test method. Transport the samples to the testing site. To counteract segregation, mix the concrete with a shovel until the concrete is uniform in appearance. Note the truck, time, and location of the placement of the concrete for future reference. In the case of paving concrete, samples may be taken from the batch immediately after depositing on the subgrade. Take at least five samples from different portions of the pile, and mix these samples thoroughly to form the test specimen.

SLUMP TEST (ASTM C 143-90A)

When the mixture appears to have reached the desired consistency, perform a slump test. This method of testing covers the procedure to be used in the laboratory and in the field for determining the consistency of concrete, which is a characteristic of workability. It is not an exact method, but it gives sufficiently accurate results.

Use this test to measure the consistency of a concrete mix by measuring the vertical distance that the concrete settles to the nearest 1/4 inch.

NOTE: This test is not applicable when there is a considerable amount of aggregate over 1 1/2 inches in the concrete.

EQUIPMENT

Use the following items to perform this test in a field or simulated field environment:

- A ruler.
- A scoop.
- A trowel.
- A water source.
- A flat, smooth surface.
- A slump cone with tamping rod.
- A pencil.
- Paper.

STEPS

Perform the following steps to determine the slump:

Step 7. Moisten the inside of the slump cone and place it on a flat, moist, nonabsorbent (rigid) surface. Hold it in place during filling by standing on the two foot pieces.

Step 8. Fill the slump cone to one third of its volume (2 5/8 inches high) with plastic concrete.

NOTE: From steps 2 to 10, a total time of no more than 2 1/2 minutes should elapse.

Step 9. Rod the concrete by applying 25 evenly distributed strokes, penetrating the full depth of the first layer in the slump cone.

Step 10. Add a second layer of concrete to the slump cone until two thirds of its volume is filled (about 6 1/8 inches high).

Step 11. Rod the second layer in the same manner as the first, with the rod just penetrating the underlying layer.

Step 12. Add the third and last layer of concrete, overfilling if possible.

Step 13. Rod the third layer following the procedure in step 5. If the concrete height subsides below the top of the cone, add additional concrete to keep it above the top of the mold.

Step 14. Strike off the excess concrete with a screeding and rolling motion of the tamping rod so the cone is completely filled.

Step 15. Remove the slump cone from the concrete.

a. Place the hands on the handles and press downward.

b. Step off the footholds.

c. Raise the cone carefully and quickly in a vertical direction. Raise the cone a distance of 12 inches within 5 to 7 seconds by a steady upward lift with no lateral or twisting motion.

d. Place the cone directly beside the slumped concrete. At this point about 2 1/2 minutes should have elapsed since the start of filling in step 2.

Step 10. Measure and record the slump immediately (see *Figure 4-4).*

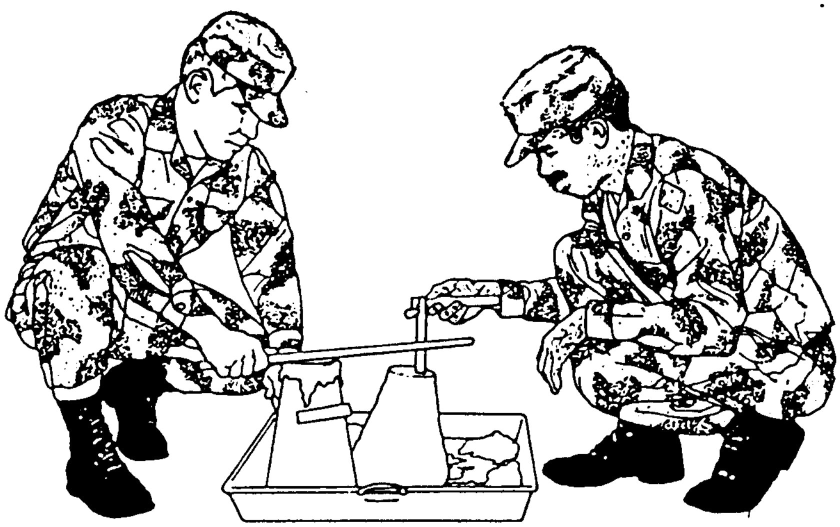


Figure 4-4. Measuring the slump of fresh concrete

a. Place the tamping rod along the top of the cone so it projects over the concrete.

b. Measure the slump from the bottom of the rod to the top center of the concrete with a ruler.

c. Record the slump to the nearest 1/4 inch.

SUPPLEMENTARY TEST PROCEDURE

After completing the slump measurement, gently tap the side of the specimen with the tamping rod. The behavior of the concrete under this treatment is a valuable indication of the cohesiveness, workability, and placeability of the mix. A well-proportioned, workable mixture will gradually slump to lower elevations and retain its original identity. A poor mix will crumble, segregate, and fall apart. Slump is usually indicated in the project specifications as a range, such as 2 to 4 inches, or as a maximum value not to be exceeded. When it is not specified, an approximate value can be selected from the list in *Table 4-6*.

Table 4-6. Recommended slumps for various types of construction

Types of Construction	Slump, in Inches	
	Maximum *	Minimum
Reinforced foundation walls and footings	3	1
Plain footings, caissons, and substructure walls	3	1
Beams and reinforced walls	4	1
Pavements and slabs	4	1
Mass concrete	2	1
*** May be increased 1 inch for consolidation by hand methods such as rodding and spading.**		

AIR-CONTENT TEST (ASTM C 231-97)

Add an air-entraining admixture to the concrete mix so that enough air will be entrained to improve the mixture's workability, durability, watertightness, and freeze-thaw resistance but not enough to substantially reduce the strength. Air-entrained cements may also be available for use in some military situations. The desired amount of air is generally from 4.0 to 7.5 percent of the total mix.

Perform this test to determine the percentage (± 0.5 percent) of entrained air in a plastic (fresh) concrete sample.

EQUIPMENT

Use the following items to perform this test in a field or simulated field environment:

- An air-entrainment meter with 5 percent calibration cup and instructions.
- A trowel.

- A tamping rod (5/8 inch in diameter and 24 inches long with a rounded end).
- A sample of plastic (fresh) concrete.
- Water.
- Oil.
- Rags.
- A pail.
- A mixing pan (from the concrete test set).
- A kitchen scoop.
- Paper.
- A pencil.
- A rubber mallet.

STEPS

There are many different air-entrainment meters currently fielded and replacements of old equipment may not be the same. For this reason, it is recommended that the steps outlined in the manufacturer's user's manual be followed.

SECTION IV. FLEXURAL-STRENGTH TEST (MODULUS OF RUPTURE)

The flexural strength of hardened concrete is measured by using a simple concrete beam and third-point loading mechanism. The flexural strength is determined by calculating measured breaks of the beam and is expressed as a modulus of rupture in psi.

TEST BEAMS

Beam forms for casting test beams from fresh concrete are available in many sizes. The most commonly used size is 6 x 6 x 21 inches. Although equipment for obtaining sawed specimens may not be available, the test may be performed on beams sawed from existing concrete structures for evaluation purposes.

FORMING THE BEAMS (ASTM C 192-90A)

Assemble a standard 6- x 6- x 21-inch concrete-beam mold and lightly oil the inside. Fill the mold with two layers of concrete from the production batches, each about 3 inches deep. Consolidate each layer by rodding, using one stroke per 2 square inches (63 per layer), evenly distributed over the layer's surface. Tap the sides lightly 10 to 15 times with a rubber mallet to close the voids left by rodding. Lightly spade the concrete along the mold's sides with a trowel to help remove surface voids. When rodding the second layer, penetrate the first layer about 1/2 inch. Strike off the top surface with a straightedge, and finish it with a wood or magnesium float.

TAKING THE SPECIMENS

Take test specimens at least once for each 100 cubic yards or fraction thereof, for each class of concrete placed in any one day, or as directed in the project specifications. Make at least three specimens for each test age and mixture design being evaluated in the lab. Additional specimens may be made for future testing. Test ages are normally 14 and 28 days for flexural-strength tests. For testing field-placed concrete, a minimum of two specimens for each test age is required.

CURING THE BEAMS

Place a suitable identifying label on the finished surface of the specimens. Cover the entire specimens—still in the mold—with a double thickness of wet burlap. Ensure that the specimens remain on site and are undisturbed for an initial curing period (the first 16 to 48 hours after molding). After this curing period, move them to the testing laboratory and remove them from the molds for further curing. The most satisfactory curing range for concrete is 68° to 86°F, with 73.4°F being the most favorable temperature. Moist-cure the beams in saturated lime water, totally submerged in a wet-tank humidity room, or keep them wet until they are tested.

FLEXURAL-STRENGTH TEST (ASTM C 78-94)

Perform this test to determine the flexural strength (modulus of rupture) of the test specimen to ± 10 psi. Record the specimen identification, modulus of rupture, any defects noted, and specimen's age.

EQUIPMENT

Use the following items to perform this test in a laboratory environment:

- The flexural-strength test apparatus.
- A concrete beam, 6 x 6 x 21 inches.
- A measuring tape.
- A stopwatch.
- Pens.
- Pencils.
- Paper.
- Safety goggles.
- A proving-ring with proving-ring calibration and constant.
- Specimen identification.
- A calculator.

STEPS

Perform the following steps to determine the flexural strength. Wear safety goggles throughout this test.

Step 1. Assemble the test apparatus and check for functional operation (see *Figure 4-5*).

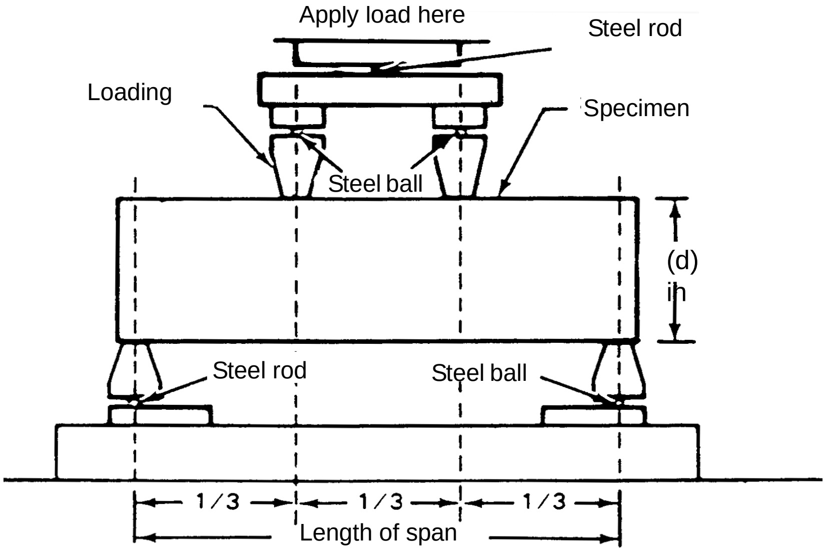


Figure 4-5. Apparatus for flexural-strength test

Step 2. Measure the length of span and record the measurement on a piece of paper. The length of span is determined by measuring the distance from center to center of the two loading points (or supports) on the base of the apparatus (see *Figure 4-5*). The normal length of specimen is 21 inches and the normal length of span is 18 inches.

Step 3. Place the specimen in the tester and bring the loading surface into contact with the test specimen (see *Figure 4-5).*

Step 4. Zero the gauge. Some apparatus are equipped with a hydraulic pump and corresponding gauge while others are equipped with a loading jack and proving ring.

Step 5. Apply a load at a continuous rate that constantly increases the extreme fiber stress between 125 and 175 psi per minute. This is an approximate load of 1,500 to 2,100 pounds per minute.

Step 6. Obtain the total load, in pounds, at the time of specimen failure, and record the weight on the paper provided. On machines equipped with hydraulics, take the reading directly from the gauge. For machines equipped with a proving ring, this reading is the product of the dial-gauge reading and the proving-ring constant.

Step 7. Determine and record the width and depth of the specimen, in inches, at the point of failure (normally 6 x 6 inches).

Step 8. Determine the point of failure in the specimen, and calculate the modulus of rupture. If the specimen fails outside the middle third of the span length by more than 5 percent of the total span length, then the specimen is considered unusable and should be discarded (not more than 0.9 inches for an 18-inch span (18 x 0.05 = 0.9).

a. Use the following formula to calculate the modulus of rupture if the specimen fails within the middle third of the span length:

$$R = \frac{P \times L}{b \times d^2}$$

where—

R = modulus of rupture, in psi

P = applied load, in pounds

L = length of span, in inches

b = width of specimen at failure point, in inches

d = depth of specimen at failure point, in inches

b. Use the following formula to calculate the modulus of rupture if the specimen fails outside the middle third of the length of span by not more than 5 percent of the span length:

$$R = \frac{3P \times a}{b \times d^2}$$

where—

R = modulus of rupture, in psi

P = applied load, in pounds

b = width of specimen at failure point, in inches

d = depth of specimen at failure point, in inches

a = distance between the failure point and the nearest support, measured along the centerline of the bottom of the specimen, in inches

Step 9. Record the following information about the test (some information may be unavailable at the time of the test):

- Specimen’s identification number.
- Average width to the nearest 0.05 inch.
- Average depth to the nearest 0.05 inch.
- Span length, in inches.
- Maximum applied load, in pounds.
- Modulus of rupture, to the nearest 10 psi.
- Curing history (how the specimen was cured) and apparent moisture content of the specimen at the time of the test.
- Any defects noted in the specimen.
- The age of the specimen.

SECTION V. COMPRESSIVE-STRENGTH TEST

The compressive strength of hardened concrete, as measured by compression tests on standard forms of cylindrical specimens, is used in the design of structures. Compressive-strength tests are made on concrete trial mixtures to evaluate the performance of available materials and to establish mixture proportions that give the required strength. Strength tests are used also to control the quality of concrete being manufactured in the field. Compressive strength is defined as the average of the strengths of all cylinders of the same age made from a sample taken from a single batch of concrete. At least two cylinders are required to constitute a test. Therefore, a minimum of four specimens are required if tests are to be made at 7 and 28 days. The test results will be the average of the strengths of the two specimens tested at 28 days.

CASTING A CONCRETE CYLINDER

The standard test specimen is a cylinder 6 inches in diameter by 12 inches long, capped with a suitable material to provide smooth, bearing surfaces on each end. Load is applied to the end surfaces through metal platens on the testing machine (cylinder breaker), causing compressive stress in the longitudinal direction of the cylinder.

Make the cylinders as near as possible to the place where they will be stored for the first 16 to 48 hours. Sufficient concrete (about 1 cubic foot) for the desired number of cylinders must be available in the trial mixture or field sample. Material from the air-content test must not be reused, since this may be contaminated with excess water. Use appropriate sampling procedures for procuring your sample as stated in Section III.

EQUIPMENT

Use the following items and information to perform this test in a field or simulated field environment:

- A tamping rod (5/8 inch in diameter and 24 inches long with a rounded end).
- A sample of fresh concrete.
- A trowel.
- Oil.
- Rags.
- A disassembled cylinder mold.
- A sheet of plastic or burlap.
- A kitchen scoop.
- A pan (24 inches wide x 24 inches long x 3 inches deep).

- A grease pencil.
- Waterproof paper tags.
- Gummed labels.
- An ink pen.
- Paper.
- Water.
- The test-specimen number.
- The origin of a concrete sample.

STEPS

Perform the following steps to produce and label a concrete cylinder for testing:

Step 1. Prepare the mold.

a. Clean and dry the mold.

b. Oil the mold lightly.

c. Assemble the mold.

Step 2. Make the cylinder.

a. Fill the mold one-third full with fresh concrete.

b. Consolidate the concrete by applying 25 evenly distributed strokes over the mold's surface area with the tamping rod. The tamping rod must totally penetrate the layer of concrete.

c. Tap the side of the mold 8 to 10 times with the tamping rod.

d. Add concrete to the mold so as to fill it two-thirds full.

e. Apply 25 evenly distributed strokes to the mold's surface area using the rounded end of the tamping rod, which must pass entirely through the second layer of concrete and 1 inch into the preceding layer.

f. Tap the side of the mold 8 to 10 times with the tamping rod.

g. Add concrete to the mold to slightly overfill it.

h. Repeat step 2e. The tamping rod must pass entirely through the top layer and 1 inch into the preceding layer.

i. Tap the side of the mold 8 to 10 times with the tamping rod.

j. Trowel off the concrete so that it is flush with the top of the mold and smoothly finished.

Step 3. Label the mold. The label should include, as a minimum, all of the following information:

- The specimen number.
- The date the cylinder was made.
- The project or placement that the concrete came from.

The system of labeling is optional. The information should be recorded on a paper tag or gummed label and attached to the mold.

Step 4. Cover the cylinder with plastic or wet burlap to maintain moisture in the sample. The covering should be tight around the cylinder but should not make contact with the fresh concrete.

Step 5. Allow the cylinder to cure undisturbed for 24 hours.

Step 6. Remove the covering and the mold from the cylinder after 24 (± 8) hours.

Step 7. Transfer the label from the mold to the concrete cylinder. The label itself may be transferred or the information may be recorded directly on the cylinder with a grease pencil.

Step 8. Cure the cylinder.

NUMBER OF SPECIMENS

The number of specimens tested depends on the job specifications. If no requirement is listed in the specifications, a minimum of 2 will be molded for each test age for each 100 cubic yards, or fraction thereof, of each class of concrete placed in any one day. A third specimen may be taken to assist in determining when forms may be removed. The test specimens must remain on site and undisturbed for an initial curing period (the first 16 to 48 hours after molding). Normally the test ages are 7 and 28 days for compressive strength tests.

CURING AND STORING CYLINDERS

After an initial curing period for 16 to 48 hours, remove (from the jobsite) specimens that are intended for checking the strength of laboratory trial mixtures or to serve as the basis for acceptance or quality control of field concrete. Take them to the testing laboratory and moist-cure them at 73.4°F. Store them in moist rooms, in damp sand or sawdust, or in limewater to maintain free water on all surfaces of the specimen at all times.

Occasionally, test specimens are made in the field to determine when forms may be removed. Form these in addition to the specimens formed for strength determination. Give these specimens (as much as possible) the same protection from the elements on all surfaces as is given to the portions of the structure that they represent. Store them in or on the structure as near as possible to the point of use. Test them in the moist condition resulting from the specified curing treatment. Specimens intended for testing to determine when a structure may be put into use are removed from the molds at the same time the forms are removed from the structure.

When shipping specimens from the field to the laboratory for testing, pack them in a sturdy wooden box or other suitable container surrounded by wet sawdust or wet sand. Provide protection from freezing during storage or shipment. Moist curing is continued when the specimens are received in the laboratory.

CAPPING CYLINDERS

Plane the ends of compression-test specimens within 0.002 inch and within 0.5 degree of being perpendicular to the axis of the cylinder.

Cap (with neat cement) specimens formed in strong metal molds having accurately flat baseplates 2 to 4 hours after molding. Make a stiff paste of portland cement and water at the time the cylinder is molded so that the capping mixture will shrink before application. Remove any free water or laitance a (layer of fine particles on the surface) from the end of the specimen. Apply the paste to the top of the concrete and work it with a flat plate until it is smooth and level with the top of the mold.

Grind hardened concrete specimens to smooth the ends or cap them with a material having greater compressive strength than the concrete. Prepared mixtures of sulfur and granular materials, special high-strength gypsum plasters, and neat high-early strength cement are satisfactory capping materials (ordinary low-strength plaster of paris, compressible rubber, or fibrous materials are not suitable for caps). Apply the selected material in a plastic state and finish it to the desired plane surface by applying glass or metal plates and squeezing out excess material to provide a cap that is as thin as possible.

Apply sulfur caps in time to harden at least 2 hours before testing. Plaster caps cannot be stored over 4 hours in the moist room. Age neat cement caps 6 days or more in the moist room (2 days when Type II cement is used). During capping, protect moist-cured specimens against drying by covering them with wet burlap. There are numerous alternatives to sulfur caps listed in ASTM C 617-94. The metal cap with a rubber membrane is not an ASTM-approved method; however, specific guidelines for their use are under review by the ASTM. The test procedures used in this manual refer to the metal caps due to their availability within the supply system. If you must use sulfur caps, ensure that sulfur vapors are not inhaled while heating the capping compound. Ensure that there is adequate ventilation and that respiratory protection is used. Used sulfur capping compound is a hazardous material and must be properly disposed of.

DETERMINING COMPRESSIVE STRENGTH OF A CYLINDRICAL SPECIMEN (ASTM C 39-96)

Perform this test to determine the compressive strength of the concrete cylinder to within breakage and to determine anything unusual about the break.

EQUIPMENT

Use the following items to perform this test in a well-ventilated laboratory:

- A concrete cylinder (6 inches in diameter and 12 inches in height).
- A concrete capping set.
- Heat-resistant gloves.
- Capping compound.

- A ruler accurate to 0.01 inch.
- Calipers with at least a 6-inch opening.
- Paper.
- Pencils.
- Safety goggles and protective apron.
- A face shield.
- Oil.
- Rags.
- A calculator.
- A concrete compression tester with a 250,000-pound capacity.
- A hammer (ball peen or carpenter's).

STEPS

Perform the following steps to determine the concrete cylinder's compressive strength:

Step 1. Prepare the concrete cylinder.

NOTE: If rubber-filled metal is used, go to step 1j.

a. Melt the capping compound in the melting pot. Ensure that you melt enough compound to make several caps.

b. Clean and lightly oil the baseplate of the capping apparatus.

c. Set the baseplate into the capping-apparatus stand.

d. Pour a small amount of the heated (liquid) capping compound into the baseplate.

e. Position the cylinder at midheight against the backrest of the capping apparatus. Slowly lower the cylinder into the baseplate while keeping the cylinder flush against the backrest. If the cylinder is not kept flush with the backrest while capping, the caps and the cylinder will not be perpendicular, and a proper break will not occur.

f. Remove the cylinder from the capping apparatus once the capping compound has solidified.

g. Inspect the cap for uniformity and defects. If you see any defects, remove the cap and recap the cylinder; then return to step 1a.

h. Repeat steps 1a through 1g for the uncapped end of the cylinder.

i. Determine and record the average diameter of the concrete cylinder. The average diameter is the average of two diameters taken perpendicular to each other at midheight of the cylinder.

j. Clean and examine the bearing surface of the steel cap (if used) for nicks, gouges, and warping. Check the rubber inserts for tears, rips, cuts, and gouges. Replace them if they are in poor condition or if the maximum number of serviceable uses has been exceeded. Place the steel cap firmly on the cylinder's ends.

Step 2. Prepare the compression tester (see *Figure 4-6*).

a. Clean the tester's bearing plates (loading surfaces).

b. Check the tester for proper operation.

c. Set the gauge to zero.

Step 3. Place the capped cylinder into the compression tester and center it on the bearing plates. Secure the protective cage around the cylinder.

Step 4. Apply the test load at a rate of 20 to 50 psi per second, not to exceed 50 psi per second (50 psi per second is about equivalent to a load [gauge reading] of 1,400 pounds per second).

Step 5. Read the gauge and record the load applied at the time of failure.

Step 6. Inspect the broken cylinder and record the following information:

- Identification number.
- Diameter.
- Cross-sectional area, in square inches.
- Maximum load applied, in pounds.
- Compressive strength, calculated to the nearest 10 psi.
- Type of break (see *Figure 4-7*).
- Defects in either specimen or caps.
- Age of specimen.

Step 7. Calculate and record the compressive strength of the cylinder using the following formula:

$$\textit{Compressive strength} = \frac{P}{A}$$

where—

P = *load at time of failure, in pounds*

$A = \pi r^2$

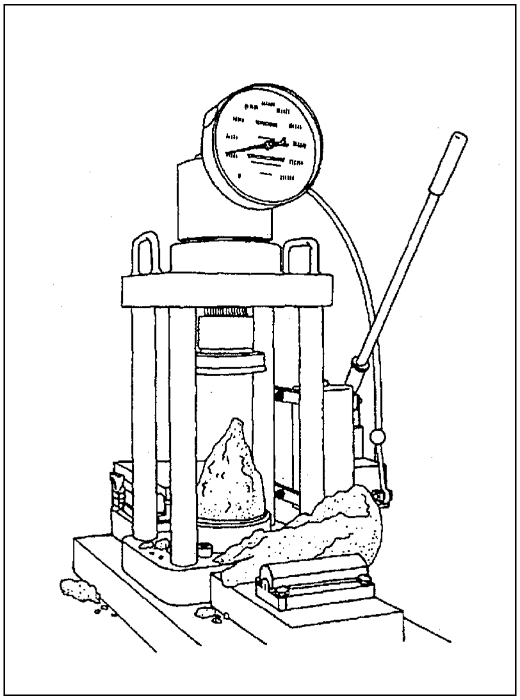


Figure 4-6. Compression tester

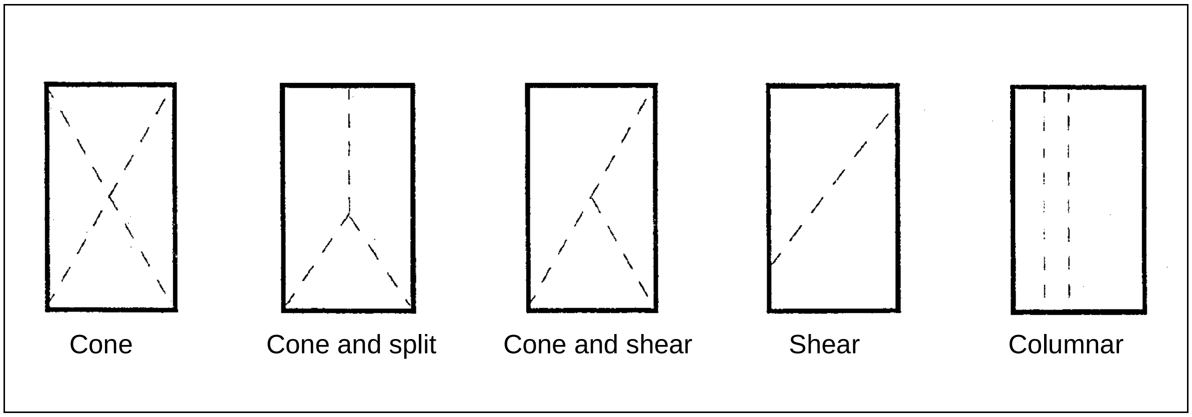


Figure 4-7. Types of fractures

Chapter 5

Soil Stabilization

This chapter presents criteria and tests for improving the structural quality and workability of soils used for base courses, subbase courses, select materials, and subgrades for pavements. It is applicable to all elements responsible for Army, Air Force, or Navy pavement construction.

SECTION I. MECHANICS OF SOIL STABILIZATION AND MODIFICATION

Stabilized soils can often be adequate for airfields, traffic pavements, and parking and storage areas where an all-weather surface is required, yet traffic does not justify a higher-strength pavement. Surface treatments are also used to provide dust control. The most widely recognized form of stabilization is compaction, which improves the mechanical stability of virtually any soil. However, compaction alone is often not enough.

STABILIZATION

Stabilization is the process of blending and mixing materials with a soil to improve the soil's strength and durability. The process may include blending soils to achieve a desired gradation or mixing commercially available additives that may alter the gradation, change the strength and durability, or act as a binder to cement the soil.

USES OF STABILIZATION

Pavement design is based on the premise that specified levels of quality will be achieved for each soil layer in the pavement system. Each layer must—

- Resist shearing within the layer.
- Avoid excessive elastic deflections that would result in fatigue cracking within the layer or in overlying layers.
- Prevent excessive permanent deformation through densification.

As the quality of a soil layer is increased, the ability of that layer to distribute the load over a greater area is generally increased enough to permit a reduction in the required thickness of the soil and surface layers.

Improve Quality

Stabilization is commonly used for better soil gradation, reduction of the PI or swelling potential, and increased durability and strength. Soils stabilized by additives often provide an all-weather working platform for construction operations. These types of soil-quality improvements are referred to as soil modifications.

Reduce Thickness

A soil layer's tensile strength and stiffness can be improved by using additives and can thereby reduce the thickness of the stabilized layer and overlying layers within the pavement system. Procedures for designing pavements that use stabilized soils are presented in TM 5-822-5, Chapter 3, and TM 5-825-2, Chapter 2. Before a stabilized layer can be used to reduce the required thickness in the design of a pavement system, the stabilized material must meet the durability requirements of various types of additive stabilization and the minimum strength requirements. Generally, as the percent of fines and the PI increase, pulverization becomes more difficult and it is harder to obtain uniform distribution of the stabilizing additive. For these types of soils, preprocessing or pretreatment with other additives may be necessary. For example, fine-grained soils may be pretreated with lime to aid in their pulverization, making the mixing of a bitumen or cement additive more successful.

METHODS OF STABILIZATION

The two general stabilization methods are mechanical and additive. The effectiveness of stabilization depends on the ability to obtain uniformity in blending the various materials. Mixing in a stationary or traveling plant is preferred. However, other means of mixing (such as scarifiers, plows, disks, graders, and rotary mixers) have been satisfactory.

The soil-stabilization method is determined by the amount of stabilizing required and the conditions encountered on the project. An accurate soil description and classification are essential for selecting the correct materials and procedure. FM 5-410, Chapter 9, lists the most suitable treatments for various soil types to stabilize these soils for different objectives.

Mechanical

Mechanical stabilization is accomplished by mixing or blending two or more gradations of material to obtain a mixture meeting the required specifications. The blending of these materials may take place at the construction site, at a central plant, or at a borrow area. The blended material is then spread and compacted to the required densities by conventional means. If, after blending these materials, the mixture does not meet the specifications, then stabilization with an additive may be necessary.

Additive

Additive refers to a manufactured commercial product that, when added to the soil in the proper quantities, will improve the quality of the soil layer. The two types of additive stabilization discussed mainly in this chapter are chemical and bituminous. Chemical stabilization is achieved by the addition of proper percentages of portland cement, lime, lime-cement-fly ash (LCF), or combinations of these materials to the soil. Bituminous stabilization is achieved by the addition of proper percentages of bituminous material to the soil. Selecting and determining the percentage of additives depend on the soil classification and the degree of improvement in the soil quality desired. Smaller amounts of additives are usually required to alter soil properties (such as gradation, workability, and plasticity) than to improve the strength and durability sufficiently to permit a thickness-reduction design. After the additive has been mixed with the soil, spreading and compacting are achieved by conventional means.

MODIFICATION

Modification refers to the stabilization process that results in the improvement in some property of the soil but does not, by design, result in a significant increase in the soil's strength and durability.

Soil modification usually results in something less than a thoroughly cemented, hardened, or semihardened material. This type of stabilization may be accomplished by—

- Compacting.
- Blending mechanically.
- Adding cementing material in small amounts.
- Adding chemical modifiers.

Cement and lime modifiers (cement-modified and lime-modified soils) are used in quantities too small to provide high-strength cementing action. They reduce the plasticity of clay soils. Calcium chloride or sodium chloride are added to the soil to retain moisture (and also control dust), to hold fine material for better compaction, and to reduce frost heave by lowering the freezing point of water in the soil. Bituminous materials (such as cutback asphalts or APSB) and certain chemicals (such as polyvinylacetate emulsion [DCA-1295]) are used to waterproof the soil's surface and to control dust

SECTION II. STABILIZING AGENTS

This section provides a method for determining the type or types of stabilizers and the amount of stabilizer to be used with a particular soil. It also considers the stabilization of soils with lime, cement, fly ash, and bituminous materials.

TYPES OF STABILIZERS

To select the proper stabilizer type for a particular soil, perform a sieve-analysis test and an Atterberg-limits test according to the procedures given in this manual.

CEMENT

Portland cement can be used either to modify and improve the quality of the soil or to transform the soil into a cemented mass with increased strength and durability.

Cement can be used effectively as a stabilizer for a wide range of materials; however, the soil should have a PI less than 30. For coarse-grained soils, the amount passing the No. 4 sieve should be greater than 45 percent. The amount of cement used depends on whether the soil is to be modified or stabilized.

LIME

Experience shows that lime will react with many medium-, moderately fine-, and fine-grained soils to produce decreased plasticity, increased workability, reduced swell, and increased strength. Soils classified according to the USCS as CH, CL, MH, ML, OH, OL, SC, SM, GC, GM, SW-SC, SP-SC, SM-SC, GW-GC, GP-GC, ML-CL, and GM-GC should be considered as potentially capable of being stabilized with lime. Lime should be considered with all soils having a PI greater than 10 and more than 25 percent of the soil passing the No. 200 sieve.

FLY ASH

Fly ash, when mixed with lime, can be used effectively to stabilize most coarse- and medium-grained soils; however, the PI should not be greater than 25. Soils classified by the USCS as SW, SP, SP-SC, SW-SC, SW-SM, GW, GP, GP-GC, GW-GC, GP-GM, GW-GM, GC-GM, and SC-SM can be stabilized with fly ash.

BITUMINOUS

Most bituminous soil stabilization has been performed with asphalt cement, cutback asphalt, and asphalt emulsions. Soils that can be stabilized effectively with bituminous materials usually contain less than 30 percent passing the No. 200 sieve and have a PI less than 10. Soils classified by the USCS as SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, SC, SM-SC, GW, GP, SW-GM, SP-GM, SW-GC, GP-GC, GM, GC, and GM-GC can be effectively stabilized with bituminous materials, provided the above-mentioned gradation and plasticity requirements are met.

COMBINATION

Combination stabilization is specifically defined as lime-cement, lime-asphalt, and LCF stabilization. Combinations of lime and cement are often acceptable expedient stabilizers. Lime can be added to the soil to increase the soil's workability and mixing characteristics as well as to reduce its plasticity. Cement can then be mixed into the soil to provide rapid strength gain. Combinations of lime and asphalt are often acceptable stabilizers. The lime addition may prevent stripping at the asphalt-aggregate interface and increase the mixture's stability.

TIME REQUIREMENTS FOR TESTING

The more thorough a testing program, the more assurance there is for the long-term success of the project. Time is often of primary concern to a military engineer—particularly in a tactical situation—and the rapid completion of a project may override the requirement for a complete series of laboratory tests (see *Table 5-1*). Because of this, the method presented allows for a rapid or expedient approximation along with a more precise laboratory determination of the type and quantity of stabilizer. An estimate for testing time is presented in *Table 5-1*.

Table 5-7. Estimated time required for test procedures

Construction Type	Stabilizing Agent	Time Required*
Expedient	Lime LCF Cement Bitumen	None None None None
Nonexpedient	Lime LCF Cement Bitumen	30 days 30 days 6 to 9 days 1 day
***These criteria do not include time required for gradation.**		

STABILIZER SELECTION

When selecting a stabilizer additive, many factors must be considered. These factors, design criteria, and the selection and mixing of stabilizers can be found in FM 5-410, Chapter 9; TM 5-822-14; and FM 5-430-00-2.

If lime is used as a preliminary additive to reduce the PI or to alter the soil gradation before adding the primary stabilizing agent (such as bitumen or cement), then the design’s lime content is the minimum treatment level that will achieve the desired results. For nonplastic and low PI materials in which lime alone generally is not satisfactory for stabilization, adding fly ash may produce the necessary reaction.

The lime used for soil stabilization is also used to determine lime requirements in the pH test.

EQUIPMENT

Use the following items for the pH test:

- A pH meter (the meter must be equipped with an electrode having a pH range of 14).
- 150-milliliter (or larger) plastic bottles with screw-top lids.
- Distilled water, free of carbon dioxide.
- A balance.
- An oven.

STEPS

Perform the following steps to determine the pH:

Step 1. Standardize the pH meter with a buffer solution having a pH of 12.45.

Step 2. Weigh, to the nearest 0.01 gram, representative samples of air-dried soil passing the No. 40 sieve and equal to 20.0 grams of oven-dried soil.

Step 3. Pour the soil samples into the 150-milliliter plastic bottles with screw-top lids.

Step 4. Add varying percentages of lime, weighed to the nearest 0.01 gram, to the soils. (Lime percentages of 0, 2, 3, 4, 5, 6, 8, and 10—based on the dry soil weight—may be used.)

Step 5. Mix the soil thoroughly and dry the lime.

Step 6. Add 100 milliliters of the distilled water to the soil-lime mixtures.

Step 7. Shake the soil-lime-water mixture for a minimum of 30 seconds or until there is no evidence of dry material on the bottom of the bottle.

Step 8. Shake the bottles for 30 seconds every 10 minutes.

Step 9. Transfer part of the slurry, after 1 hour, to a plastic beaker and measure the pH.

Step 10. Record the pH for each of the soil-lime mixtures. The lowest percent of lime giving a pH of 12.40 is the percent required to stabilize the soil. If the pH does not reach 12.40, the minimum lime content giving the highest pH is required to stabilize the soil.

SOIL STABILIZATION IN FROST AREAS

While bituminous, portland-cement, lime, and combinations of LCF stabilization are the most common additives, other stabilizers may be used for pavement construction in areas of frost design, but only with approval from—

- Headquarters, Department of the Army (DAEN-MPE-D), Washington, DC 20314 (for Army projects).
- Headquarters, Air Force Engineering and Services Center (AFESC/DEM), Tyndall AFB, FL 32401 (for Air Force projects).
- Headquarters, Naval Facilities Engineering Command, Alexandria, VA 22332 (for Navy or Marine Corps projects).

LIMITATIONS

In frost areas, stabilized soil should be used only in a layer or layers comprising one of the upper elements of a pavement system and directly beneath the pavement's surfacing layer. The structural advantage in reducing the required thickness of the pavement system compensates for the added cost of stabilization. Treatment with a lower degree of chemical stabilization should be used in frost areas only with caution and after intensive tests, because weakly cemented material usually has less capacity to endure repeated freezing and thawing than firmly cemented material. A possible exception is using a low level of stabilization to improve a soil that will be encapsulated within an impervious envelope as part of a membrane-encapsulated soil-layer pavement system. A soil that is unsuitable for encapsulation due to excessive moisture migration and thaw weakening may be made suitable for such use by a moderate amount of a stabilizing additive. Materials that are modified by a small amount of a chemical additive to improve certain properties of the soil without significant cementation also should be tested to determine that the desired improvement is durable through repeated freeze-thaw cycles. The improvement should not be achieved at the expense of making the soil more susceptible to ice segregation.

CONSTRUCTION CUTOFF

For materials stabilized with cement, lime, or LCF whose strength increases with curing time, it is essential that the stabilized layer be constructed sufficiently early in the season to allow the adequate strength to develop before the first freezing cycle begins. The rate of strength gain is substantially lower at 50°F than at 70° or 80°F. Chemical reactions will not occur rapidly for lime-stabilized soils when the soil temperature is less than 60°F and is not expected to increase for one month or for cement-stabilized soils when the soil temperature is less than 40°F and is not expected to increase for one month. In frost areas, it is not always sufficient to protect the mixture from freezing during a 7-day curing period as required by the applicable guide specifications. A construction cutoff date well in advance of the onset of freezing conditions may be essential.

WEATHER

Hot, dry weather is preferred for all types of bituminous stabilization. When asphalt cements are used for stabilization, proper compaction must be obtained. If thin lifts of asphalt-stabilized material are being placed, the air temperature should be 40°F and rising and the compaction equipment should be used immediately after lay-down operations. Adequate compaction can be obtained at freezing temperatures if thick lifts are used. When cutbacks and emulsions are used, the air and soil temperatures should be above freezing. Heavy rains on mixed, uncompacted material may be detrimental.

PICK-AND-CLICK TESTS

Specimens covering a wide range of cement contents (for example: 10, 14, and 18 percents) are molded at optimum moisture and maximum density. After at least 36 hours of hardening while kept moist and after a 3-hour soaking period, the specimens are inspected by picking with a pointed instrument (such as a dull ice pick or bayonet) and by sharply clicking each specimen against a hard object (such as concrete or another sound specimen) to determine their relative hardness when set. If the specimen cannot be penetrated more than 1/8 to 1/4 inch by picking, and if it produces a clear or solid tone upon clicking, an adequate cement factor (CF) is indicated. When a dull thud or plunky sound is obtained, there is inadequate cement even though the specimen may resist picking.

The specimen's age is a factor, and a specimen that may not test properly at first may harden properly a few days later. Some satisfactory specimens require 7 days or longer to produce adequate hardening. The test results will indicate the proper content. If the results show that some intermediate content may be satisfactory, new test specimens (at the suggested content) should be prepared and tested. It is important to remember that too much cement is not harmful (although more expensive), but too little cement will not produce a satisfactory stabilization.

WET-DRY AND FREEZE-THAW TESTS

After determining the maximum density and OMC, mold the specimens for the wet-dry and freeze-thaw tests.

PREPARATION

Prepare the specimens using the computed OMC and the cement contents previously described for the different soil classifications. Select the cement contents in 2 percent increments on either side of the median value. Mold two specimens for each of the three cement contents—one for the wet-dry test and one for the freeze-thaw test. Use the same procedure to mold the specimens as used for the OMC determination. Take special care to scarify the surfaces between layers to ensure a good bond. When the second layer is being placed, take a 750-gram sample for a moisture determination. Place the molded specimens in a moisture cabinet in an atmosphere of high humidity for 7 days to permit cement hydration before testing.

WET-DRY TEST PROCEDURE

After the 7-day curing period, submerge the specimens in tap water at room temperature for a period of 5 hours and then remove them. Dry the specimens in an oven at 160°F for 42 hours and then remove them. Wire brush the entire surface area to remove all material loosened during wetting and drying. Use two firm strokes on each portion of the surface. Apply these strokes the full height and width with a 3-pound force. One cycle consists of 5 hours of water immersion, 42 hours of drying, and 1 hour of handling. Repeat the operation for a total of 12 cycles. After 12 cycles of the test, dry the specimens to a constant weight at 230°F, and weigh them to determine the oven-dry weights.

FREEZE-THAW TEST PROCEDURE

After the curing period, place water-saturated felt pads about 1/4 inch thick, blotters, or similar absorptive materials between the specimens and specimen carriers. Place the assembly in a freezing cabinet with a constant temperature not warmer than -10°F for 24 hours and then remove them. Allow the assembly to thaw in a moist room or in suitable covered containers with a temperature of 70°F and a relative humidity of 100 percent for 23 hours. Make free water available to the absorbent pads to permit the specimens to absorb water by capillarity during the thawing period. Give the specimens two firm strokes on all areas with the wire brush to remove material loosened during freezing and thawing. If necessary, use a sharp-pointed instrument to remove any scale that has formed. One cycle consists of 24 hours of freezing, 23 hours of thawing, and 1 hour of handling (total 48 hours). After being brushed at the end of each thawing period, turn the specimens over, end for end, before replacing them on the water-saturated pads. Continue the test for a total of 12 cycles, dry the specimens to a constant weight at 230°F, and weigh them to determine their oven-dry weights.

CALCULATIONS AND CRITERIA

The results of the wet-dry and freeze-thaw cycles are indicated as soil-cement losses. These losses are computed by using the original dry weights and final corrected dry weights.

Water-of-Hydration Correction

The final oven-dry weight of the specimen includes some water used for cement hydration that cannot be driven off at 230°F. The average amount of this water retained in the specimen is based on the type of soil—gravels,

$\pm$ 1 1/2; sand, $\pm$ 2 1/2 percent; silt, $\pm$ 3 percent; and clays, $\pm$ 3 1/2 percent. This correction is computed by the following formula:

$$\text{corrected oven-dry weight} = \left(\frac{\text{measured oven-dry specimen wt}}{\text{percent water retained}} + 100\right) \times 100$$

Example: A sample composed mostly of sand weighs 3.77 pounds at the end of the test. Water of hydration is 2.5 percent.

$$\text{corrected oven-dry weight} = \left(\frac{3.77}{2.5} + 100\right) \times 100$$

Soil-Cement Loss

The soil-cement loss can now be calculated as a percentage of the original dry weight, or—

$$\text{soil-cement loss} =$$

$$\left(\frac{\text{original oven-dry weight} - \text{final corrected oven-dry weight}}{\text{original oven-dry weight}}\right) \times 100$$

Example: A sample of soil has an original weight of 3.99 pounds.

$$\text{soil-cement loss} = \left(\frac{3.99 - 3.68}{3.99}\right) \times 100$$

This value would be reported to the nearest whole number or as 8 percent.

Weight-Loss Criteria

The minimum cement content recommended for use is the one for which losses of specimen weight during 12 cycles of the wet-dry test or freeze-thaw test conform to the following standards:

- GW, GM, GC, SW, SM, SC, and SP soils—not over 14 percent.
- ML and MH soils—not over 10 percent.
- GL, CH, OH, and OL soils—not over 7 percent.

Strength Criteria

The strength of soil-cement specimens tested in compression at various ages should increase with age and with increases in cement. The ranges of cement contents should produce results meeting the requirements above. A sample that has an unconfined compression strength of about 300 psi after curing 7 days and shows increasing strength with age can be considered adequately stabilized.

Cement Weight-to-Volume Conversion

The required cement content by weight must be converted to the equivalent cement content by volume for control during construction since this is the easier quantity to use in the field. The following formula illustrates the calculation:

$$volume\ of\ cement\ (percent) = \frac{D - \left(\frac{D}{C}\right)}{94} \times 100$$

where—

D = oven-dry density of soil-cement, in pcf

$$C = \frac{100 + percent\ cement\ by\ weight}{100}$$

94 = weight of 1 cubic foot of cement

The nomograph in *Figure 5-1* makes the conversion without computation. Use a straightedge placed at the soil-cement density and at the percent by weight of cement. Read the percent of cement by volume on the right-hand scale.

MODIFIED MIX DESIGN FOR SANDY SOILS

Sandy soils are usually the most readily and economically stabilized because they require the least amount of cement for adequate hardening and they contain a minimum amount of material that prevents intimate mixing of soil and cement. The following shortcut testing procedures for sandy soils will not always indicate the minimum cement contents required, but the results will be close enough to be on the safe side and economical. If time permits, the procedures for the freeze-thaw test are followed for greater design economy.

The two procedures used are for—

- Soils with no material retained on the No. 4 sieve.
- Soils with material retained on the No. 4 sieve.

The procedures can be used only with soils containing less than 50 percent of material smaller than 0.05 millimeter (silt and clay) and less than 20 percent smaller than 0.005 millimeter (clay). Dark gray to black sandy soils obviously containing appreciable organic impurities together with miscellaneous granular materials (such as cinders, caliche, chat, chart, marl, red dog, scoria, shale, and slag) should be tested using the full procedures and not tested by the modified methods for sandy soils. When coarse-grained or sandy soils (usually of groups GW, GP, GM, SW, or SM) are encountered, they may be classified for testing purposes using either the first or the second procedure. There is one other exception. Granular soils with materials retained on the No. 4 sieve whose bulk specific gravity is less than 2.45 cannot be tested.

Perform the following steps for modifying the mix design for sandy soils:

Step 1. Determine the soil gradation.

Step 2. Determine the bulk specific gravity of the material retained on the No. 4 sieve.

Step 3. Perform the moisture-density test of an estimated soil-cement mixture.

Step 4. Locate the indicated cement requirements from the charts.

Step 5. Perform compressive-strength tests to verify the cement requirement.

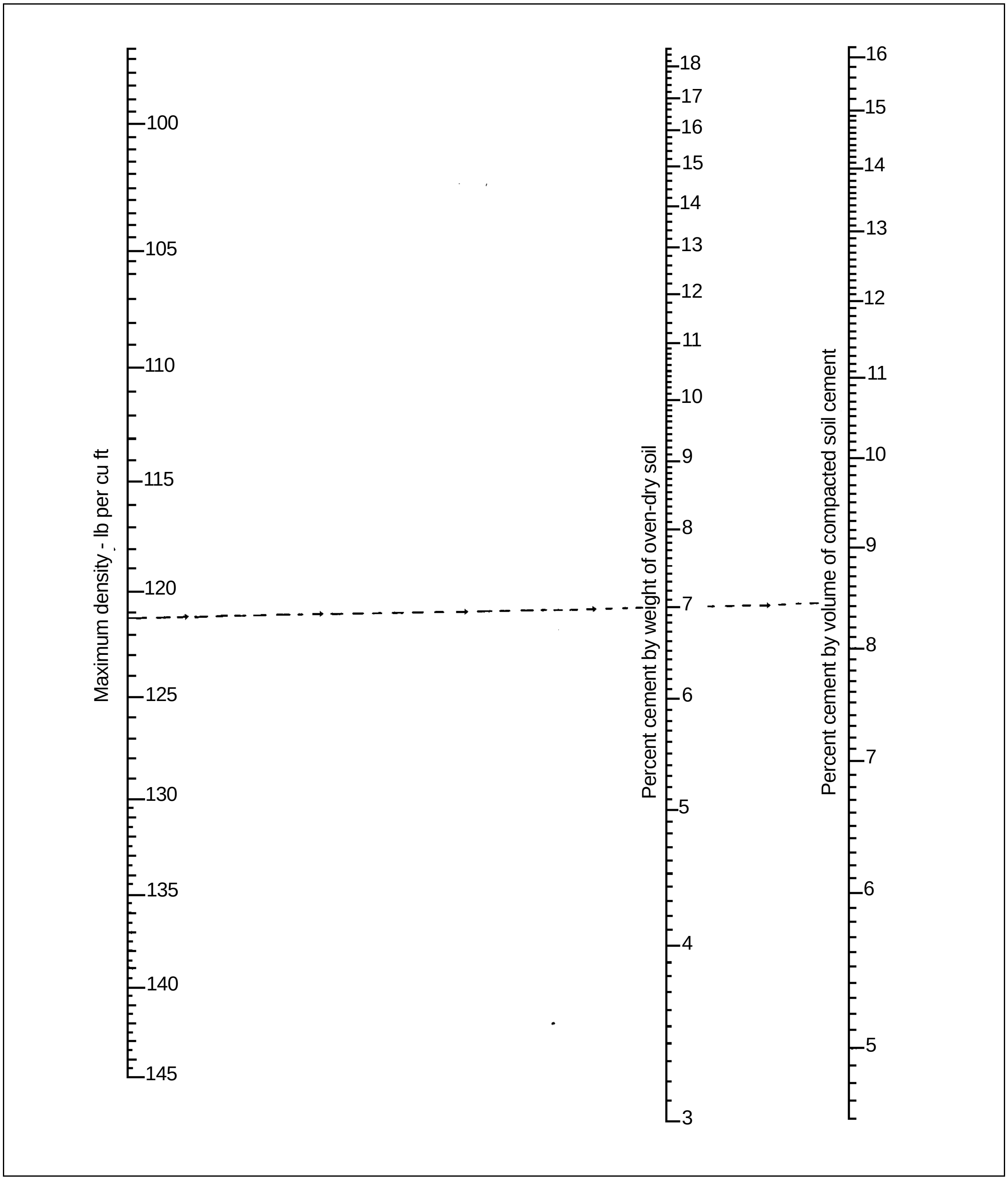


Figure 5-33. Relation of cement content by weight to cement content by volume

SOILS WITH NO MATERIAL RETAINED ON THE NO. 4 SIEVE

Perform the following steps for soils with no material retained on the No. 4 sieve:

Step 1. Determine the maximum density and OMC for a mixture of soil and cement. (*Figure 5-2* will give an estimated density. This value and the percentage of material smaller than 0.05 millimeter are used with *Figure 5-3* to determine an indicated cement content.)

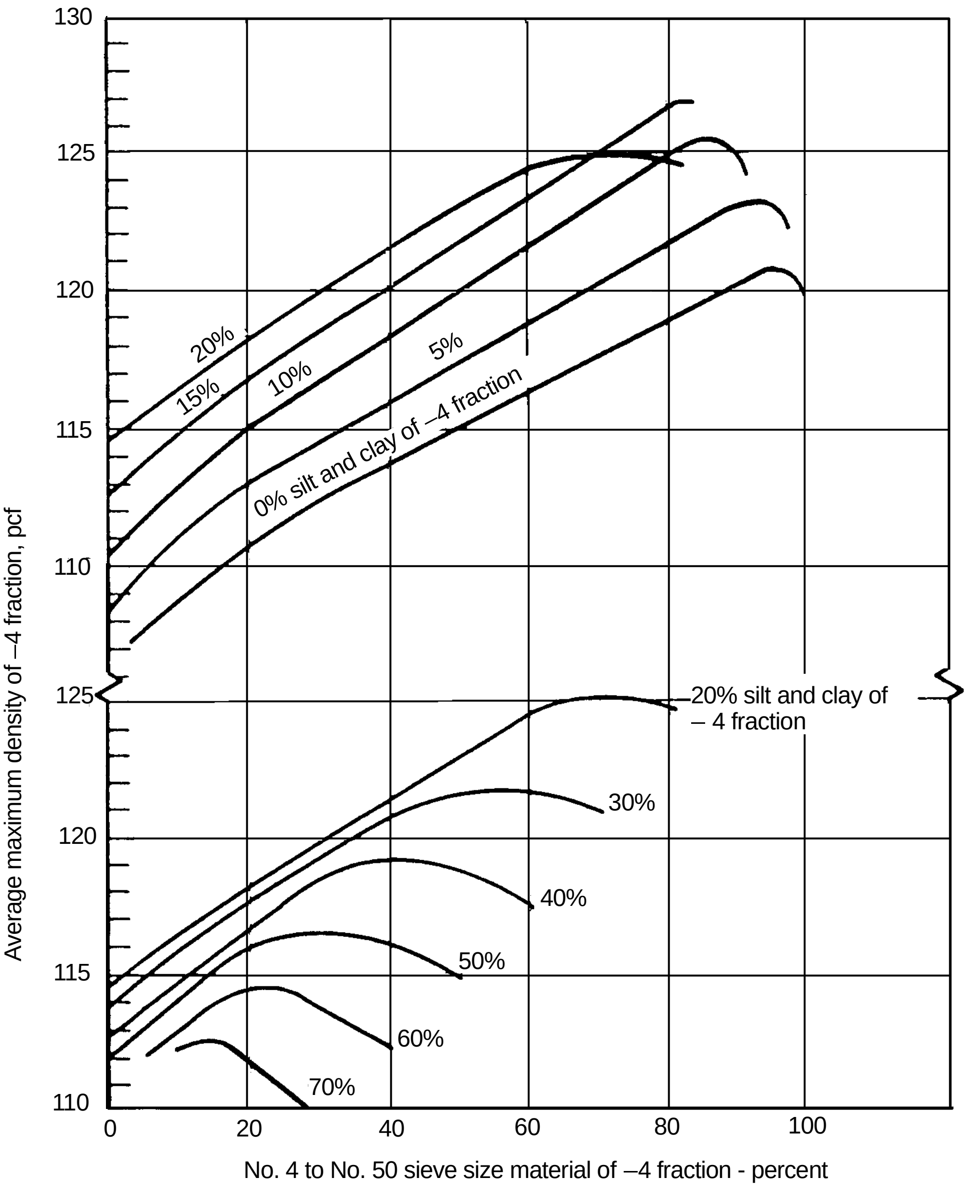


Figure 5-34. Average maximum densities of the –4 fraction of soil-cement mixtures

Step 2. Use the maximum density value and *Figure 5-3* to determine an indicated cement requirement.

Step 3. Mold three compressive-strength specimens at maximum density and OMC.

Step 4. Moist-cure the specimens for 7 days and test for strength.

Step 5. Plot the value of the averaged compressive strength of *Figure 5-4, page 5-14*. If this plot is above the curve, the CF is probably too low and needs adjusting. Prepare two new test specimens: one at the cement content as computed above, and the second with a 2 percent higher cement content. Perform the full freeze-thaw test on these two specimens.

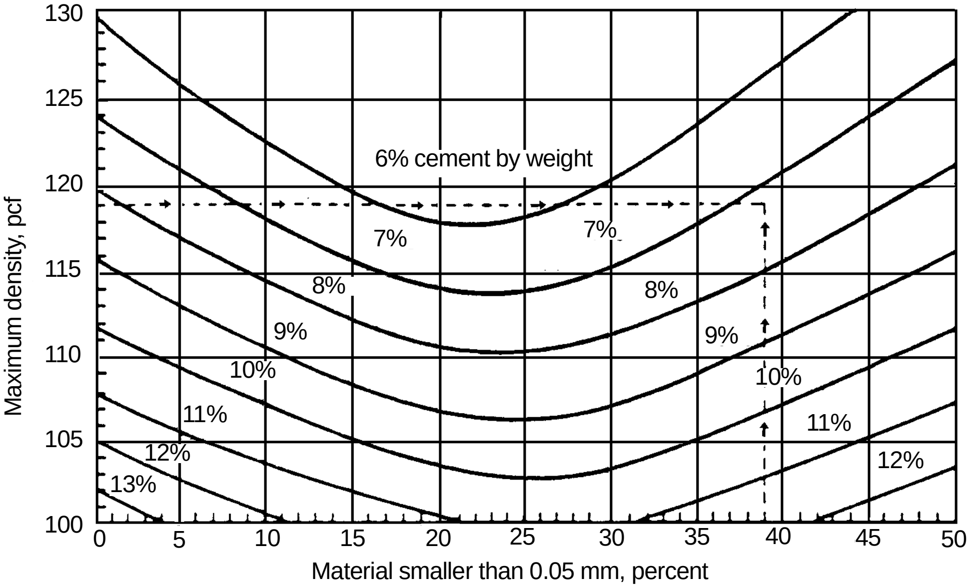


Figure 5-35. Indicated cement contents of soil-cement mixtures not containing material retained on the No. 4 sieve

SOILS WITH MATERIAL RETAINED ON THE NO. 4 SIEVE

Perform the following steps for soils with material retained on the No. 4 sieve:

Step 1. Determine the maximum density and OMC for a mixture of soil and cement. Use *Figure 5-5, page 5-14*, for an estimated maximum density. Using this density, the percentage of material retained on the No. 4 sieve, and the percentage smaller than 0.05 millimeter, determine the moisture content (see *Figure 5-6, page 5-15*). The 45 percent maximum retained on the No. 4 sieve still applies. Also, replace any material larger than 1/4 inch with an equivalent weight of the material passing the 1/4-inch sieve and retained on the No. 4 sieve.

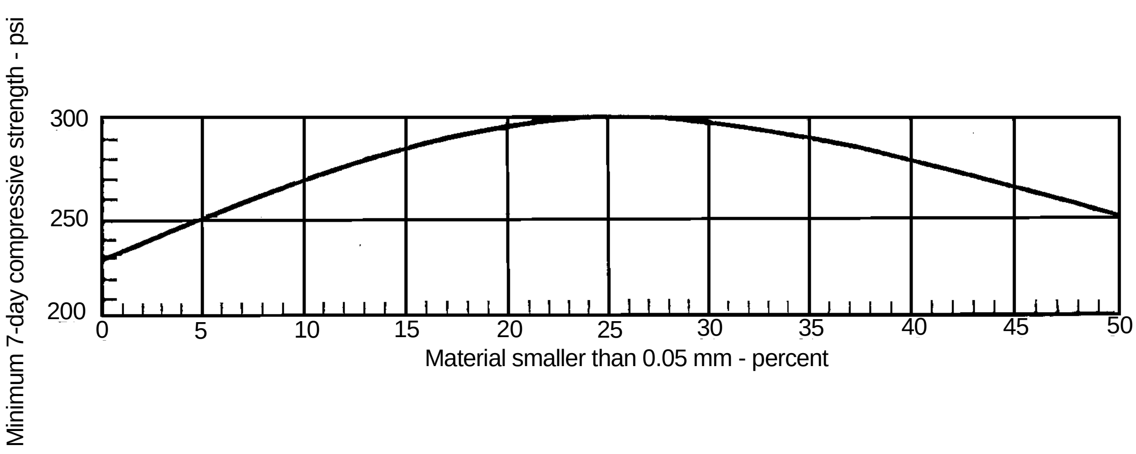


Figure 5-36. Minimum 7-day compressive strengths required for soil-cement not containing material retained on the No. 4 sieve

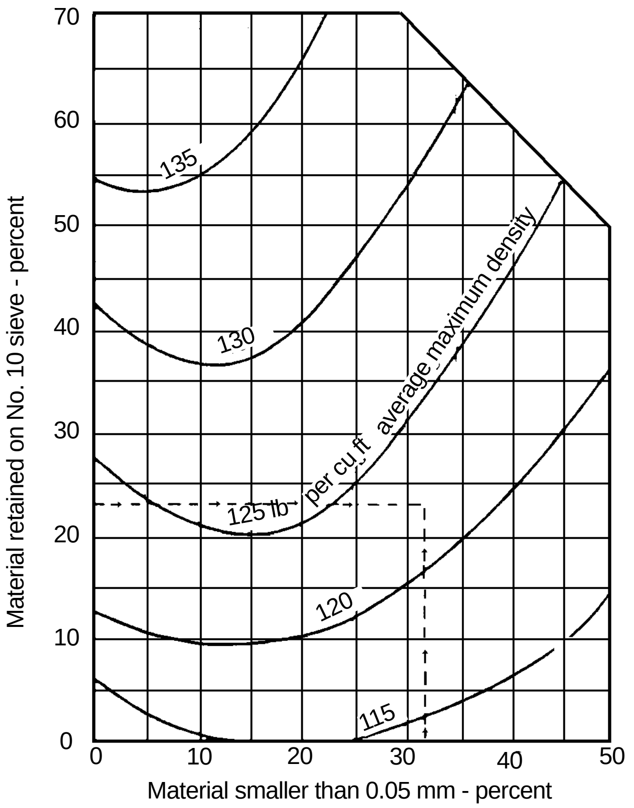


Figure 5-37. Average maximum densities of soil-cement mixtures containing material retained on the No. 4 sieve

Step 2. Determine the indicated cement requirement using the maximum density from above and *Figure 5-6.*

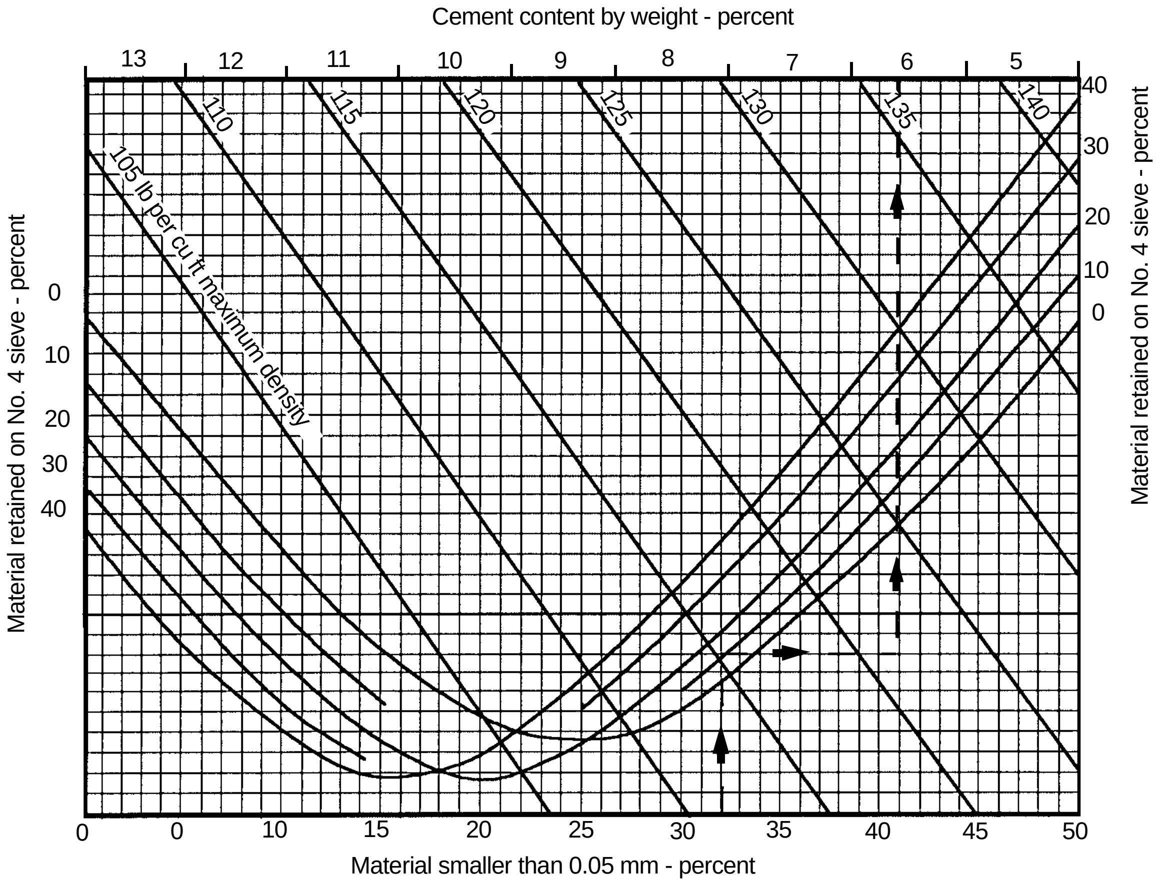


Figure 5-38. Indicated cement contents of soil-cement mixtures containing material retained on the No. 4 sieve

Step 3. Mold-test specimens at maximum density and OMC.

Step 4. Moist-cure for 7 days and test for compressive strength and average.

Step 5. Use *Figure 5-7, page 5-16,* to determine the allowable compressive strength for the soil-cement mixture. Connect the points on the right- and left-hand scales of the nomograph, and read the minimum required compressive strength from the inclined center scale. If the strength is equal to or greater than the allowable strength, the cement content is adequate. If the strength is too low, the CF is also too low and a full test should be performed.

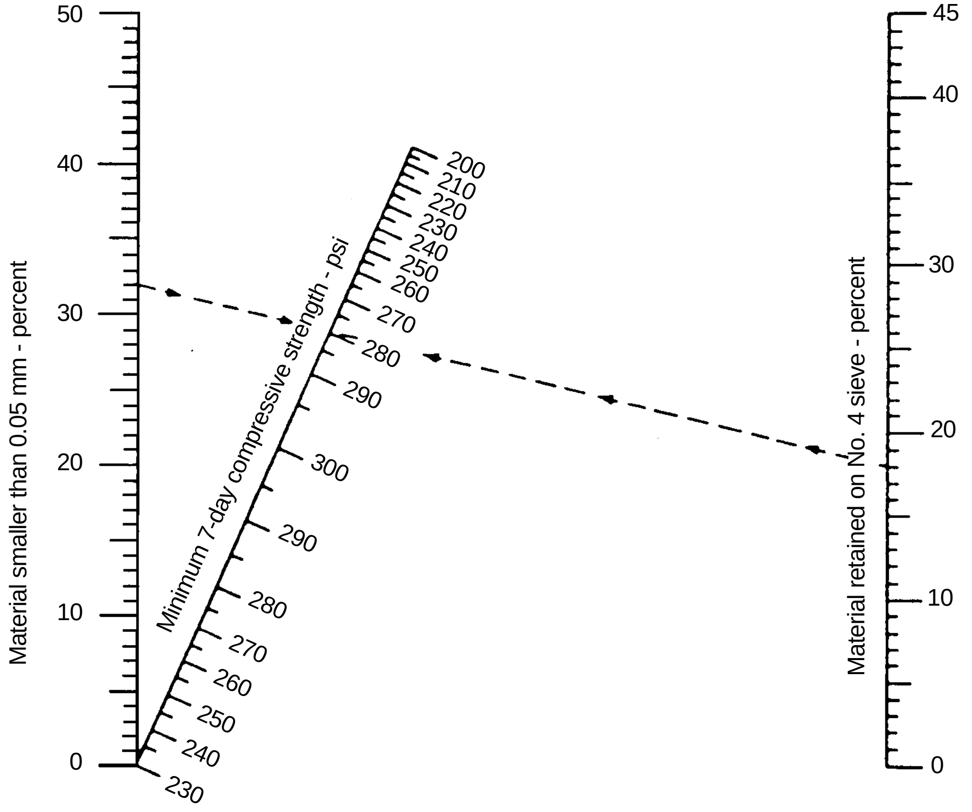


Figure 5-39. Minimum 7-day compressive strengths required for soil-cement mixtures containing material retained on the No. 4 sieve

Appendix A

Metric Conversion Chart

This appendix complies with current Army directives which state that the metric system will be incorporated into all new publications. Table A-1 is a conversion chart.

Table A-1. Metric conversion chart

Metric to English			English to Metric		
Multiply	**By**	**To Obtain**	**Multiply**	**By**	**To Obtain**
Length					
Centimeters	0.0394	Inches	Inches	2.54	Centimeters
Meters	3.28	Feet	Feet	0.0305	Meters
Meters	1.094	Yards	Yards	0.9144	Meters
Kilometers	0.621	Miles (stat)	Miles (stat)	1.5609	Kilometers
Kilometers	0.540	Miles (naut)	Miles (naut)	1.853	Kilometers
Millimeters	0.039	Inches	Inches	25.40	Millimeters
Area					
Square centimeters	0.1550	Square inches	Square inches	6.45	Square centimeters
Square meters	10.76	Square feet	Square feet	0.0929	Square meters
Square meters	1.196	Square yards	Square yards	0.836	Square meters
Volume					
Cubic centimeters	0.610	Cubic inches	Cubic inches	16.39	Cubic centimeters
Cubic meters	35.3	Cubic feet	Cubic feet	0.0283	Cubic meters
Cubic meters	1.308	Cubic yards	Cubic yards	0.765	Cubic meters
Milliliters	0.0338	US liq ounces	US liq ounces	29.6	Milliliters
Liters	1.057	US liq quarts	US liq quarts	0.946	Liters
Liters	0.264	US liq gallons	US liq gallons	3.79	Liters
Weight					
Grams	0.0353	Ounces	Ounces	28.4	Grams
Kilograms	2.20	Pounds	Pounds	0.454	Kilograms
Metric tons	1.102	Short tons	Short tons	0.907	Metric tons
Metric tons	0.984	Long tons	Long tons	1.016	Metric tons
Temperature					
Celsius	Add 17.8 and multiply by 1.8	Fahrenheit	Fahrenheit	Subtract 32 and multiply by 0.5556	Celsius

Appendix B

The Unified Soil Classification System

The adoption of the principles of soil mechanics by the engineering profession has inspired numerous attempts to devise a simple classification system that will tell the engineer the properties of a given soil. As a consequence, many classifications have come into existence based on certain properties of soils such as texture, plasticity, strength, and other characteristics. A few classification systems have gained fairly wide acceptance, but rarely has any system provided the complete information on a soil that the engineer needs. Nearly every engineer who practices soil mechanics will add judgment and personal experience as modifiers to whatever soil classification system he uses. Obviously, within a given agency (where designs and plans are reviewed by persons entirely removed from a project) a common basis of soil classification is necessary so that when an engineer classifies a soil as a certain type, this classification will convey the proper characteristics and behavior of the material. Further than this, the classification should reflect those behavior characteristics of the soil that are pertinent to the project under consideration.

BASIS OF THE USCS

The USCS is based on identifying soils according to their textural and plasticity qualities and on their grouping with respect to behavior. Soils seldom exist in nature separately as sand, gravel, or any other single component. They are usually found as mixtures with varying proportions of particles of different sizes; each component part contributes its characteristics to the soil mixture. The USCS is based on those characteristics of the soil that indicate how it will behave as an engineering construction material. The following properties have been found most useful for this purpose and form the basis of soil identification. They can be determined by simple tests and, with experience, can be estimated with some accuracy.

- Percentages of gravel, sand, and fines (fraction passing the No. 200 sieve).
- Shape of the grain-size-distribution curve.
- Plasticity and compressibility characteristics. In the USCS, the soil is given a descriptive name and a letter symbol indicating its principal characteristics.

PURPOSE AND SCOPE

It is the purpose of this appendix to describe the various soil groups in detail and to discuss the methods of identification so that a uniform classification procedure may be followed by all who use the system. Placement of the soils into their respective groups is accomplished by visual examination and laboratory tests as a means of basic identification. It is recognized that the USCS in its present form may not prove entirely adequate in all cases. However, it is intended that the classification of soils according to this system have some degree of elasticity and that the system not be followed blindly nor regarded as completely rigid.

DEFINITIONS OF SOIL COMPONENTS

Before soils can be classified properly in any system, including the one presented in this manual, it is necessary to establish a basic terminology for the various soil components and to define the terms used. In the USCS, the terms cobbles, gravel, sand, and fines (silt or clay) are used to designate the size ranges of soil particles. The gravel and sand ranges are further subdivided into the groups as presented in *Table B-1*. The limiting boundaries between the various size ranges have been arbitrarily set at certain US standard sieve sizes as listed in *Table B-1*. In the finest soil component (below the No. 200 sieve), the terms silt and clay are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity. The minus No. 200 sieve material is silt if the LL and PI plot below the “A” line on the plasticity chart and is clay if the LL and PI plot above the “A” line on the chart (all LL and PL tests are based on minus No. 40 sieve fraction of a soil). The foregoing definition holds for inorganic silts and clays and for organic silts but is not valid for organic clays since these latter soils plot below the “A” line. The names of the basic soil components can be used as nouns or adjectives when describing or classifying a soil.

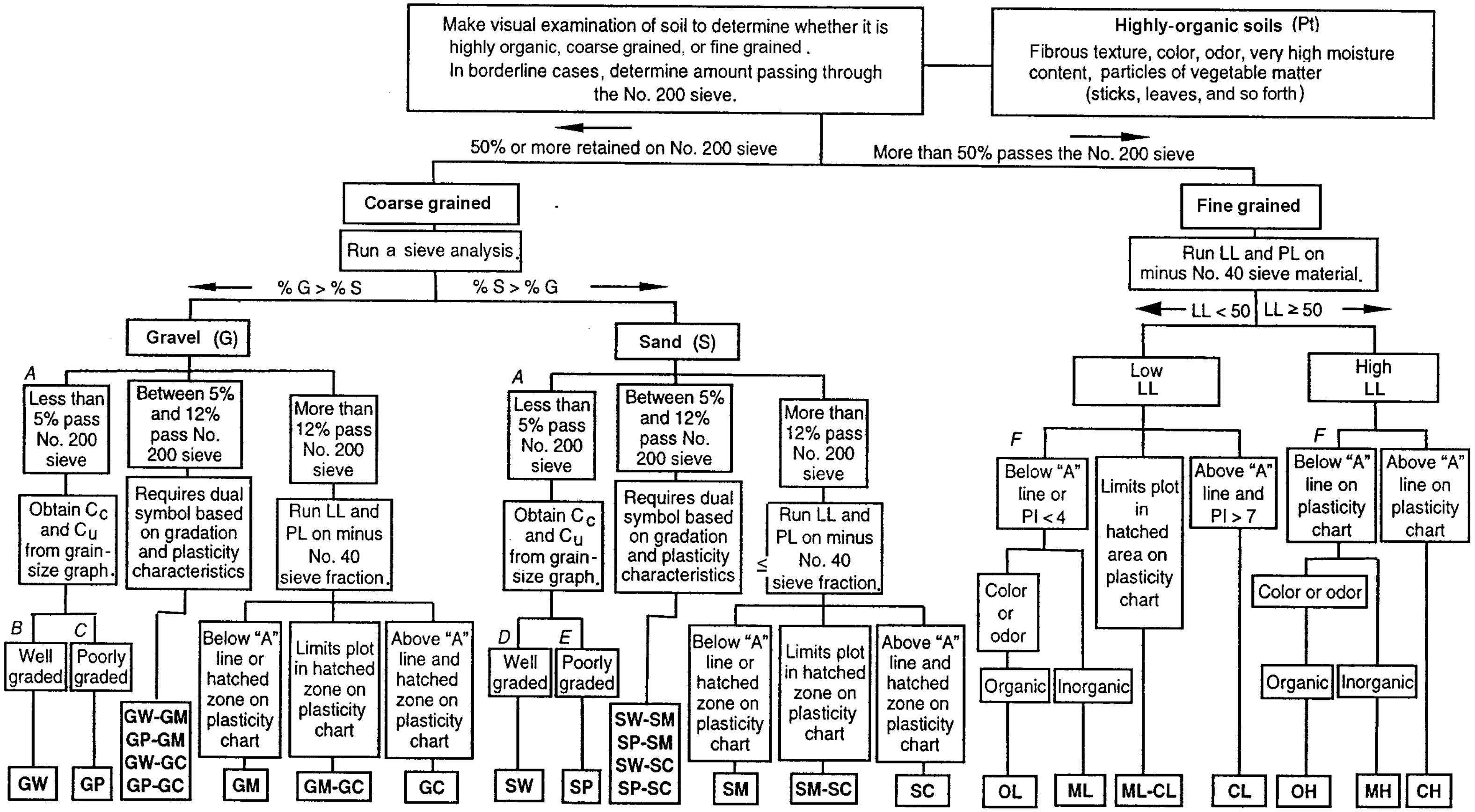
THE CLASSIFICATION SYSTEM

In its simplest form, *Figure B-1* illustrates the process of the classification system. The following paragraphs provide detailed information on the soil properties and groups as they pertain to the system.

Table B-1. Soil particle-size ranges

Component	Size Range
Cobbles	Above 3 inches
Gravel Coarse Fine	3 inches to No. 4 sieve 3 inches to 3/4 inch 3/4 inch to No. 4 sieve
Sand Coarse Medium Fine	No. 4 to No. 200 sieves No. 4 to No. 10 sieves No. 10 to No. 40 sieves No. 40 to No. 200 sieves
Fines (clay or silt)	Below No. 200 sieve (no minimum size)

A short discussion of the USCS procedures (see *Figure B-1, page B-3*) is presented so that the succeeding detailed description may be better understood. The procedures are designed to apply generally to the



A If fines interfere with free-draining properties, use a double symbol such as GW-GM.

B For well-graded gravel, the C_u must be > 4 and the C_c must be ≥ 1 and ≤ 3.

C For poorly graded gravel, the C_u must be ≤ 4 and/or the C_c is < 1 or > 3.

D For well graded sand, the C_u must be > 6 and the C_c must be ≥ 1 and ≤ 3.

E For poorly graded sand, the C_u must be ≤ 6 and/or the C_c is < 1 or > 3.

F In cases where organic material can't be determined by color or odor, a LL and PL test must be conducted on a sample of natural moisture content and a sample that has been oven-dried. Organic soils will show a radical drop in plasticity for the oven-dried sample compared to the retained-moisture sample. Inorganic soils generally fall within ± 1 or 2 percent of each other.

Figure B-1. USCS procedures

identification of soils regardless of the intended engineering uses. *Table B-2, pages B-6 and B-7,* also assists in identifying the symbols and soil descriptions within this system. *Figure B-1* shows the schematic method of classifying soils from the results of laboratory tests. Columns 1 through 5 of *Table B-2, pages B-6 and B-7* identify the three major divisions of the classification system and the group symbols that distinguish the individual soil types. Names of typical and representative soil types found in each group are shown in column 6.

SOIL GROUPS AND GROUP SYMBOLS

Soils are primarily identified as coarse grained, fine grained, and highly organic. On a textural basis, coarse-grained soils are those that have 50 percent or more by weight of the overall soil sample retained on the No. 200 sieve; fine-grained soils are those that have more than 50 percent by weight passing the No. 200 sieve. Highly-organic soils are, in general, readily identified by visual examination. The coarse-grained soils are subdivided into gravel and gravelly soils (G) and sands and sandy soils (S). Fine-grained soils are subdivided on the basis of their LL and plasticity properties; the symbol L is used for soils with LLs of 50 and less and the symbol H for soils with LLs in excess of 50. Peat and other highly organic soils are designated by the symbol Pt and are not subdivided.

In general practice there is no clear-cut boundary between gravelly soils and sandy soils and, as far as behavior is concerned, the exact point of division is relatively unimportant. For identification purposes, coarse-grained soils are classified as G if the greater percentage of the coarse fraction (that which is retained on the No. 200 sieve) is larger than the No. 4 sieve. They are classed as S if the greater portion of the coarse fraction is finer than the No. 4 sieve. Borderline cases may be classified as belonging to both groups. The G and S groups are each divided into four secondary groups as follows:

- Well-graded material with little or no fines—symbol W, groups GW and SW.
- Poorly graded material with little or no fines—symbol P, groups GP and SP.
- Coarse material with nonplastic fines or fines with low plasticity—symbol M, groups GM and SM.
- Coarse material with plastic fines—symbol C, groups GC and SC.

The fine-grained soils are subdivided into groups based on whether they have a relatively low (L) or high (H) LL. These two groups are further subdivided as follows:

- Inorganic silts and very fine sandy soils, silty or clayey fine sands, micaceous and diatomaceous soils, and elastic silts—symbol M, groups ML and MH.
- Inorganic clays—symbol C, groups CL and CH.
- Organic silts and clays—symbol O, groups OL and OH.

Coarse-Grained Soils

In the following paragraphs, soils of the GW, GP, SW, and SP groups are defined as having less than 5 percent passing the No. 200 sieve. Soils which have between 5 and 12 percent passing the No. 200 sieve are classed as borderline and will be discussed later in this appendix.

GW and SW Groups

These groups comprise well-graded gravelly and sandy soils having little or no nonplastic fines (less than 5 percent passing the No. 200 sieve). The presence of the fines must not noticeably change the strength characteristics of the coarse-grained fraction and must not interfere with its free-draining characteristics. If the material contains less than 5 percent fines that exhibit plasticity, this information should be evaluated and the soil classified and discussed subsequently under “Laboratory Identification.” In areas subject to frost action, the material should not contain more than 3 percent of soil grains smaller than 0.02 millimeter in size.

GP and SP Groups

Poorly-graded gravels and sands containing little or no nonplastic fines (less than 5 percent passing the No. 200 sieve) are classed in the GP and SP groups. The materials may be classed as uniform gravels, uniform sands, or nonuniform mixtures of very coarse material and very fine sand, with intermediate sizes lacking (sometimes called skip graded, gap graded, or step graded). The latter group often results from borrow excavation in which gravel and sand layers are mixed. If the fine fraction exhibits plasticity, this information should be evaluated and the soil classified as discussed subsequently under “Laboratory Identification.”

GM and SM Groups

In general, the GM and SM groups comprise gravels or sands with fines (more than 12 percent passing the No. 200 sieve) having low or no plasticity. The PI and LL of soils in the group should plot below the “A” line on the plasticity chart. The gradation of the materials is not considered significant and both well- and poorly graded materials are included. Some of the sands and gravels in this group will have a binder composed of natural cementing agents, so proportioned that the mixture shows negligible swelling or shrinkage. Thus, the dry strength of such materials is provided by a small amount of soil binder or by cementation of calcareous material or iron oxide. The fine fraction of other materials in the GM and SM groups may be composed of silts or rock-flour types having little or no plasticity, and the mixture will exhibit no dry strength.

GC and SC Groups

In general, the GC and SC groups comprise gravelly or sandy soils with fines (more than 12 percent passing the No. 200 sieve) which have either low or high plasticity. The PI and LL of soils in the group should plot above the “A” line on the plasticity chart. The gradation of the materials is not considered significant and both well- and poorly graded materials are included. The plasticity of the binder fraction has more influence on the behavior of the soils than does variation in gradation. The fine fraction is generally composed of clays.

Table B-2. Characteristics of soil groups pertaining to embankments and foundations

Major Divisions (1)	(2)	Letter (3)	Symbols Hatching (4)	Color (5)	Name (6)	Value for Embankments (7)	Permeability cm per sec (8)
Coarse-Grained Soils	Gravel and Gravelly Soils	GW	░	Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Very stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GP	▒		Poorly graded gravels or gravel-sand mixtures, little or no fines	Reasonably stable, pervious shells of dikes and dams	$k > 10^{-2}$
		GM	▥	Yellow	Silty gravels, gravel-sand-silt mixtures	Reasonably stable, not particularly suited to shells, but may be used for impervious cores or blankets	$k = 10^{-3}$ to 10^{-6}
		GC	▨		Clayey gravels, gravel-sand-clay mixtures	Fairly stable, may be used for impervious core	$k = 10^{-6}$ to 10^{-8}
	Sand and Sandy Soils	SW	○	Red	Well-graded sands or gravelly sands, little or no fines	Very stable, pervious sections, slope protection required	$k > 10^{-3}$
		SP	●		Poorly graded sands or gravelly sands, little or no fines	Reasonably stable, may be used in dike section with flat slopes	$k > 10^{-3}$
		SM	▥	Yellow	Silty sands, sand-silt mixtures	Fairly stable, not particularly suited to shells, but may be used for impervious cores or dikes	$k = 10^{-3}$ to 10^{-6}
		SC	▩		Clayey sands, sand-silt mixtures	Fairly stable, use for impervious core or flood-control structures	$k = 10^{-6}$ to 10^{-8}
Fine-Grained Soils	Silts and Clays LL < 50	ML	▥	Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor stability, may be used for embankments with proper control	$k = 10^{-3}$ to 10^{-6}
		CL	▨		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Stable, impervious cores and blankets	$k = 10^{-6}$ to 10^{-8}
		OL	▥		Organic silts and organic silt-clays of low plasticity	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}
	Silts and Clays LL $\geq$ 50	MH	▥	Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor stability, core of hydraulic-fill dam, not desirable in rolled-fill construction	$k = 10^{-4}$ to 10^{-6}
		CH	▨		Inorganic clays of high plasticity, fat clays	Fair stability with flat slopes, thin cores, blankets and dike sections	$k = 10^{-6}$ to 10^{-8}
		OH	▨		Organic clays of medium to high plasticity, organic silts	Not suitable for embankments	$k = 10^{-6}$ to 10^{-8}
Highly Organic Soils		Pt	≋	Orange	Peat and other highly organic soils	Not used for construction	

NOTES: 1. Values in columns 7 and 11 are for guidance only. Design should be based on actual test results.

2. The equipment listed in column 9 will usually produce the desired densities with a reasonable number of passes when moisture conditions and thickness of lift are properly controlled.

3. The range of dry unit weights listed in column 10 are for compacted soil at OMC when using the Standard Proctor Test (ASTM 1557-91).

Table B-2. Characteristics of soil groups pertaining to embankments and foundations (continued)

Compaction Characteristics (9)	Max Dry Unit Weight Std Proctor (pcf) (10)	Value for Foundations (11)	Requirements for Seepage Control (12)
Good; tractor, rubber-tired, or steel-wheeled roller	125 -135	Good bearing value	Positive cutoff
Good; tractor, rubber-tired, or steel-wheeled roller	115 -125	Good bearing value	Positive cutoff
Good; with close control; rubber-tired or sheepsfoot roller	120 -135	Good bearing value	Toe trench to none
Fair; rubber-tired or sheepsfoot roller	115 -130	Good bearing value	None
Good; tractor	110 -130	Good bearing value	Upstream blanket and toe drainage or wells
Good; tractor	100 -120	Good to poor bearing value depending on density	Upstream blanket and toe drainage or wells
Good with close control; rubber-tired or sheepsfoot roller	110 -125	Good to poor bearing value depending on density	Upstream blanket and toe drainage or wells
Fair; sheepsfoot or rubber-tired roller	105 -125	Good to poor bearing value	None
Good to poor; close control essential; rubber-tired or sheepsfoot roller	95 -120	Very poor, susceptible to liquefaction	Toe trench to none
Fair to poor; sheepsfoot or rubber-tired roller	95 -120	Good to poor bearing value	None
Fair to poor; sheepsfoot roller	80 -100	Fair to poor bearing value, may have excessive settlements	None
Poor to very poor; sheepsfoot roller	70 - 95	Poor bearing value	None
Fair to poor; sheepsfoot roller	75 -105	Fair to poor bearing value	None
Poor to very poor; sheepsfoot roller	65 - 100	Very poor bearing value	None
Compaction not practical		Remove from foundations	

Fine-Grained Soils

The following paragraphs discuss fine-grained soils in their subgroupings:

ML and MH Groups

In these groups, the symbol M has been used to designate predominantly silty materials and micaceous or diatomaceous soils. The symbols L and H represent low and high LLs, respectively, and an arbitrary dividing line between the two is set at an LL of 50. The soils in the ML and MH groups are sandy silts, clayey silts, or inorganic silts with relatively low plasticity. Also included are loess-type soils and rock flours. Micaceous and diatomaceous soils generally fall within the MH group but may extend into the ML group when their LL is less than 50. The same is true for certain types of kaolin clays and some elite clays having relatively low plasticity.

CL and CH Groups

In these groups, the symbol C stands for clay, with L and H denoting low or high LL. These soils are primarily inorganic clays. Low-plasticity clays are classified as CL and are usually lean, sandy, or silty clays. The medium and high plasticity clays are classified as CH. These include the fat clays, gumbo clays, certain volcanic clays, and bentonite. The glacial clays of the northern US cover a wide band in the CL and CH groups.

OL and OH Groups

The soils in the OL and OH groups are characterized by the presence of organic matter, hence the symbol O. Organic silts and clays are classified in these groups. The materials have a plasticity range that corresponds with the ML and MH groups.

Highly-Organic Soils

The highly-organic soils usually are very compressible and have undesirable construction characteristics. They are classified into one group, designated by the symbol Pt. Peat, humus, and swamp soils with a highly-organic texture are typical soils of the group. Particles of leaves, grass, branches, or other fibrous vegetable matter are common components of these soils.

IDENTIFICATION OF SOIL GROUPS

The USCS is arranged so that most soils may be classified into at least the three primary groups (coarse grained, fine grained, and highly organic) by means of visual examination and simple field tests. Classification into the subdivisions can also be made by visual examination with some degree of success. More positive identification may be made through laboratory testing. However, in many instances a tentative classification determined in the field is of great benefit and may be all the identification that is necessary, depending on the purposes for which the soils in question are to be used. The general or field-identification methods as well as the individual laboratory test methods are all explained in great detail in Chapter 2. It is emphasized that the two methods of identification are never entirely separated. Certain characteristics can only be estimated by visual examination. In borderline cases, it may be necessary to verify the classification by laboratory tests. Conversely, the field methods are entirely practical for preliminary laboratory identification and may be used to an advantage in grouping soils in such a manner that only a minimum number of laboratory tests need be run.

LABORATORY IDENTIFICATION

Identifying soils in the laboratory is done by determining the gradation and plasticity characteristics of the materials. The gradation is determined by sieve analysis, and a grain-size curve is usually plotted as percent finer (or passing) by weight against a logarithmic scale of grain size in millimeters. DD Form 1207 is typically used for this purpose. Plasticity characteristics are evaluated by means of the LL and PL tests on the soil fraction finer than the No. 40 sieve. The laboratory test procedures for the LL and PL determination can be found in Section IV of Chapter 2.

MAJOR SOIL GROUPS

In the laboratory-identification procedures shown in *Figure B-1, page B-3*, the first step in identifying a soil is to determine whether it is coarse grained, fine grained, or highly organic. This may be done by visual examination in most cases. In some borderline cases, as with very-fine sands or coarse silts, it may be necessary to screen a representative dry sample over a No. 200 sieve and determine the percentage passing. Fifty percent or less passing the No. 200 sieve identifies the soil as coarse grained, and more than 50 percent identifies the soil as fine grained. The percentage limit of 50 has been selected arbitrarily for convenience in identification, as it is obvious that a numerical difference of 1 or 2 in this percentage will make no significant change in the soil's behavior. After the major group is established, the identification procedure is continued according to the proper headings in *Figure B-1*.

Coarse-Grained Soils

A complete sieve analysis must be run on coarse-grained soils and a gradation curve plotted on a grain-size chart. For some soils containing a substantial amount of fines, it may be desirable to supplement the sieve analysis with a hydrometer analysis to define the gradation curve for particle sizes smaller than the No. 200 sieve size. Preliminary identification is made by determining the percentage of material in the gravel (above No. 4 sieve) and sand (No. 4 to No. 200 sieve) sizes. If there is a greater percentage of gravel than sand, the material is classed as G; if there is a greater percentage of sand than gravel, the material is classed as S. Once again, the distinction between these groups is purely arbitrary for convenience in following the system. The next step is to determine the amount of material passing the No. 200 sieve. Since the subgroups are the same for gravels and sands, they will be discussed jointly in the following paragraphs.

GW, SW, GP, and SP Groups

These groups comprise nonplastic soils having less than 5 percent passing the No. 200 sieve and in which the fine fraction does not interfere with the soil's free-draining properties. If the above criteria are met, an examination is made of the shape of the grain-size curve. Materials that are well graded are classified as GW or SW; poorly graded materials are classified as GP or SP.

A soil's gradation curve and curve data should meet the following qualifications to be classed as well graded:

- The grain-size distributions of well-graded materials generally plot as smooth and regular concave curves with no sizes lacking or no excess of material in any size range.
- The coefficient of uniformity (C_u) of well-graded gravels is greater than 4 and of well-graded sands is greater than 6. The C_u is determined by dividing the grain-size diameter passing at 60 percent by the grain-size diameter passing at 10 percent.
- The coefficient of curvature (C_c) must be between 1 and 3. The C_c is determined by the following formula:

$$\frac{(D_{30})^2}{D_{60} \times D_{10}} = \text{between 1 and 3}$$

where—

D_{30} *= grain diameter at 30 percent passing*

D_{60} *= grain diameter at 60 percent passing*

D_{10} *= grain diameter at 10 percent passing*

The C_c ensures that the grading curve will have a concave curvature within relatively narrow limits for a given D_{60} and D_{10} combination. All gradations not meeting the foregoing criteria are classed as poorly graded. Thus, poorly graded soils (GP and SP) are those having nearly straight-line gradations, convex gradations, nearly vertical gradations, and “hump” gradations typical of skip-graded materials.

NOTE: In the preceding paragraph, soils of the GW, GP, SW, and SP groups were defined as having less than a 5 percent fraction passing the No. 200 sieve. Soils having between 5 and 12 percent passing the No. 200 sieve are classed as borderline and are discussed later.

GM, SM, GC and SC Groups

The soils in these groups are composed of those materials having more than a 12 percent fraction passing the No. 200 sieve. They may or may not exhibit plasticity. For identification, the LL and PL tests are required on the fraction finer than the No. 40 sieve. The tests should be run on representative samples of moist material—not on air- or oven-dried soils. This precaution is desirable as drying affects the limits values to some extent, as will be explained further in the discussion of fine-grained soils. Materials in which the LL and PI plot below the “A” line on the plasticity chart (see *Figure 2-54, page 2-100*) are classed as GM or SM. Gravels and sands in which the LL and PI plot above the “A” line on the plasticity chart are classed as GC or SC. It is considered that in the identification of materials in these groups, the plasticity characteristics overshadow the gradation characteristics; therefore, no distinction is made between well- and poorly graded materials.

Borderline Soils

Coarse-grained soils containing between 5 and 12 percent material passing the No. 200 sieve are classed as borderline and carry a dual symbol (for example, GW-GM). Similarly, coarse-grained soils having less than 5 percent passing the No. 200 sieve but which are not free draining, or wherein the fine fraction exhibits plasticity, are also classed as borderline and are given a dual symbol.

Fine-Grained Soils

Once the identity of a fine-grained soil has been established, further identification is accomplished principally by the LL and PL tests in conjunction with the plasticity chart. The plasticity chart is a plot of LL versus PI on which is imposed a diagonal line called the “A” line and a vertical line at a LL of 50. The “A” line is defined by the equation PI = 0.73 (LL-20). The “A” line above a liquid limit of about 29 represents an important empirical boundary between typical inorganic clays (CL and CH), which are generally located above the line and plastic soils containing organic colloids (OL and OH) or inorganic silty soils (ML and MH). The vertical line at an LL of 50 separates silts and clays of low LL (L) from those of high LL (H). In the low part of the chart below an LL of about 29 and in the range of PI from 4 to 7, there is considerable overlapping of the properties of the clayey and silty soil types. Hence, the separation between CL and OL or ML soil types in this region is accomplished by a cross-hatched zone on the plasticity chart between 4 and 7 PI and above the “A” line. The CL soils in this region are those having a PI above 7 while OL or ML soils are those having a PI below 4.

Soils plotting within the cross-hatched zone should be classed as borderline. The various soil groups are shown in their respective positions on the plasticity chart. Experience has shown that compressibility is about proportional to the LL and that soils having the same LL possess about equal compressibility (assuming that other factors are essentially the same). On comparing the physical characteristics of soils having the same LL, you find that with increasing the PI, the cohesive characteristics increase and the permeability decreases. From plots of the results of limits tests on a number of samples from the same fine-grained deposit, it is found that for most soils these points lie on a straight line or in a narrow band that is almost parallel to the “A” line. With this background information in mind, the identification of the various groups of fine-grained soils is discussed in the following paragraphs.

ML, CL, and OL Groups

A soil having an LL of less than 50 falls into the low LL (L) group. A plot of the LL and PI on the plasticity chart will show whether the soil falls above or below the “A” line and cross-hatched zone. Soils plotting above the “A” line and cross-hatched zone are classed as CL and are usually typical inorganic clays. Soils plotting below the “A” line or cross-hatched zone are inorganic silts or very fine sandy silts (ML) or organic silts or organic silt-clays of low plasticity (OL). Since two groups fall below the “A” line or cross-hatched zone, further identification is necessary. The distinguishing factor between the ML and OL groups is the absence or presence of organic matter. This is usually identified by color and odor. However, a comparison may be made between the LL and PL of a moist sample and one that has been oven-dried.

An organic soil will show a radical drop in plasticity after oven- or air-drying. An inorganic soil will generally show a change in the limits values of only 1 or 2 percent, which may be either an increase or a decrease. For the foregoing reasons, the classification should be based on the plot of limits values determined before drying. Soils containing organic matter generally have lower specific gravities and may have decidedly higher water contents than inorganic soils; therefore, these properties may be of assistance in identifying organic soils. In special cases, determining the organic content may be made by chemical methods, but the procedures just described are usually sufficient.

MH, CH, and OH Groups

Soils with an LL greater than 50 are classed in group H. To identify such soils, the LL and PI values are plotted on the plasticity chart. If the points fall above the “A” line, the soil classifies as CH; if they fall below the “A” line, a determination is made as to whether or not organic material is present (as described in the preceding paragraph). Inorganic materials are classed as MH and organic materials are classed as OH.

Highly-Organic Soils

Little more can be said as to the laboratory identification of highly-organic soils (Pt) than has been identified in the field-identification procedures. These soils are usually identified readily on the basis of color, texture, and odor. Moisture determinations usually show a natural water content of several hundred percent, which is far in excess of that found for most soils. Specific gravities of the solids in these soils may be quite low. Some peaty soils can be remolded and tested for the LLs and PLs. Such materials usually have an LL of several hundred percent and fall well below the “A” line on the plasticity chart.

Borderline Classifications

It is inevitable in the use of the classification system that soils will be encountered that fall close to the boundaries established between the various groups. In addition, boundary zones for the amount of material passing the No. 200 sieve and for the lower part of the plasticity chart have been incorporated as a part of the system, as discussed subsequently. The accepted rule in classifying borderline soils is to use a double symbol (for example, GW-GM). It is possible, in rare instances, for a soil to fall into more than one borderline zone and, if appropriate symbols were used for each possible classification, the result should be a multiple designation consisting of three or more symbols. This approach is unnecessarily complicated, and it is considered best to use only a double symbol in these cases, selecting the two that are believed most representative of the probable behavior of the soil. In cases of doubt, the symbols representing the poorer of the possible groupings should be used.

Coarse-Grained Soils

In previous discussions, the coarse-grained soils were classified in the GW, GP, SW, and SP groups if they contained less than 5 percent of material passing the No. 200 sieve. Similarly, soils were classified in the GM, GC, SM, and SC groups if they had more than 12 percent passing the No. 200 sieve. The range between 5 and 12 percent passing the No. 200 sieve is designated as borderline. Soils falling within it are assigned a double symbol depending on both the gradation characteristics of the coarse fraction and the plasticity characteristics of the minus No. 40 sieve fraction. For example, a well-graded sandy soil with 8 percent passing the No. 200 sieve, a LL of 28, and a PI of 9 would be designated as SM-SC. Another type of borderline classification occurs for those soils containing appreciable amounts of fines (groups GM, GC, SM, and SC) and whose LL and PL values plot in the lower portion of the plasticity chart. The method of classifying these soils is the same as for fine-grained soils plotting in the same region, as presented in the following paragraph.

Fine-Grained Soils

Discussion has been presented of a zone on the plasticity chart below a LL of about 29 and ranging between PI values of 4 and 7. Several soil types exhibiting low plasticity plot in this general region on the plasticity chart, and no definite boundary between silty and clayey soils exists. Thus, if a fine-grained soil, groups CL and ML, or the minus No. 40 sieve fraction of a coarse-grained soil (groups GM, GC, SM, and SC) plots within the cross-hatched zone on the plasticity chart, a double symbol (such as ML-CL) is used.

Note that in the descriptive name of the soil type as indicated on *Table B-2, pages B-6 and B-7*, silty and clayey may be used to describe silt or clay soils. Since the definitions of these terms are now somewhat different from those used by many soils engineers, it is considered advisable to discuss their connotation as used in this system. In the USCS, the terms silt and clay are used to describe those soils with LLs and PLs plotting respectively below and above the “A” line and cross-hatched zone on the plasticity chart. As a logical extension of this concept, the terms silty and clayey may be used as adjectives in the soil names when the limits values plot close to the “A” line. For example, a clay soil with an LL of 40 and a PI of 16 may be called a silty clay. In general, the adjective silty is not applied to clay soils having an LL in excess of about 60.

Expansion of Classification

In some cases, it may be necessary to expand the USCS by subdividing existing groups to classify soils for a particular use. The indiscriminate use of subdivisions is discouraged and careful study should be given to any soil group before adopting such a step. In all cases, subdivisions should be designated preferably by a suffix to an existing group symbol. The suffix should be selected carefully so there will be no confusion with existing letters that already have meanings in the classification system. In each case where an existing group is subdivided, the basis and criteria for the subdivision should be explained so that anyone unfamiliar with it may understand the subdivision properly.

Descriptive Soil Classification

At many stages in the soils investigation of a project—from the preliminary boring log to the final report—the engineer finds it convenient to give the soils he is working with a name rather than an impersonal classification symbol (such as GC). This results primarily from the fact that he is accustomed to talking in terms of gravels, sands, silts, and clays and finds it only logical to use these same names in presenting the data. The soil names have been associated with certain grain sizes in the textural classification as shown on the grain-size chart. Such a division is generally feasible for the coarse-grained soils; however, the use of such terms as silt and clay may be entirely misleading on a textural basis. For this reason, the terms silt and clay have been defined on a plasticity basis, as discussed previously. Within a given region of the country, the use of a name classification based on texture is often feasible since the general behavior of similar soils is consistent over the area. However, in another area, the same classification may be entirely inadequate. The descriptive classification, if used intelligently, has a rightful place in soil mechanics, but its use should be carefully evaluated by all concerned.

Description From Classification Sheet

Column 6 of *Table B-2, pages B-6 and B-7*, lists typical names given to the soil types usually found within the various classification groups. By following either the field- or laboratory-investigation procedure and determining the proper classification group in which the soil belongs, it is usually an easy matter to select an appropriate name from the classification sheet. Some soils may be readily identified and properly named by only visual inspection. A word of caution is considered appropriate on the use of the classification system for certain soils (such as marls, calyces, coral, and shale) where the grain size can vary widely depending on the amount of mechanical breakdown of soil particles. For these soils, the group symbol and textural name have little significance and the locally used name may be important.

Other Descriptive Terms

Records of field explorations in the form of boring logs can be of great benefit to the engineer if they include adequate information. In addition to the group symbol and the name of the soil, the general characteristics of the soils as to plasticity, strength, moisture, and so forth provide information essential to a proper analysis of a particular problem. Locally accepted soil names should also be used to clarify the data to local bidders and to protect the government against later legal claims. For coarse-grained soils, the size of particles, mineralogical composition, shape of grains, and character of the binder are relevant features. For fine-grained soils, strength, moisture, and plasticity characteristics are important. When describing undisturbed soils, such characteristics as stratification, structure, consistency in the undisturbed and remolded states, cementation, and drainage are pertinent to the descriptive classification. Pertinent items to be used in describing soils are shown in column 6 of *Table B-3, pages B-16 and B-17*. To achieve uniformity in estimating the consistency of soils, it is recommended that the Terzaghi classification based on unconfined compressive strength be used as a tentative standard. This classification is given in *Table B-4, page B-18*.

Several examples of descriptive classifications are shown below:

- Uniform, fine, clean sand with rounded grains—SP.
- Well-graded gravelly silty sand; angular chert gravel, 1/2 inch maximum size; silty binder with low plasticity, well-compacted and moist—SM.
- Light brown, fine, sandy silt; very low plasticity; saturated and soft in the undisturbed state—ML.
- Dark gray, fat clay; stiff in the undisturbed state; soft and sticky when remolded—CH.

CHARACTERISTICS OF SOIL GROUPS PERTAINING TO EMBANKMENTS AND FOUNDATIONS

The major properties of a soil proposed for use in an embankment or foundation that are of concern to the design or construction engineer are its strength, permeability, and consolidation and compaction characteristics. Other features may be investigated for a specific problem, but in general, some or all of the properties mentioned are of primary importance in an earth-embankment or foundation project of any magnitude. It is common practice to evaluate the properties of the soils in question by means of laboratory or field tests and to use the results of such tests as a basis for design and construction. The factors that influence strength, consolidation, and other characteristics are numerous, and some of them are not completely understood; consequently, it is impractical to evaluate these features by means of a general soils classification. However, the soil groups in a given classification do have reasonably similar behavior characteristics. While such information is not sufficient for design purposes, it will give the engineer an indication of the behavior of a soil when used as a component in construction. This is especially true in the preliminary examination for a project when neither time nor money for a detailed soils-testing program is available.

Keep in mind that only generalized characteristics of the soil groups are included therein, and they should be used primarily as a guide and not as the complete answer to a problem. For example, it is possible to design and construct an earth embankment of almost any type of soil and on practically any foundation. However, when a choice of materials is possible, certain of the available soils may be better-suited to the job than others. It is on this basis that the behavior characteristics of soils are presented in the following paragraphs and on the classification sheet. A structure's use is often the principal deciding factor in selecting soil types as well as the type of protective measures that will be used. Since each structure is a special problem within itself, it is impossible to cover all possible considerations in the brief description of pertinent soil characteristics contained in this appendix.

FEATURES ON THE SOILS-CLASSIFICATION SHEET

General characteristics of the soil groups pertinent to embankments and foundations are presented in *Table B-2, pages B-6 and B-7*. Columns 1 through 5 show major soil divisions, group symbols, and the hatching and color symbols. The names of soil types are given in column 6. The basic features are the same as those presented previously in soils classification. Columns 7 through 12 show the following: the suitability of the materials for use in embankments (strength and permeability characteristics); the minimum or range of permeability values to be expected for the soil groups; general compaction characteristics; the suitability of the soils for foundations (strength and consolidation); and the requirements for seepage control, especially when the soils are encountered in the foundation for earth embankments (permeability). Brief discussions of these features are presented in the following paragraphs.

Table B-3. Characteristics of soil groups pertaining to roads and airfields

Major Divisions (1)	(2)	Letter (3)		Symbols		Name (6)	Value As Subgrade When not Subject to Frost Action (7)	Value As Subbase When not Subject to Frost Action (8)
				Hatching (4)	Color (5)			
Coarse-Grained Soils	Gravel and Gravelly Soils	GW		░	Red	Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Excellent
		GP		░		Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Good
		GM	d	▥	Yellow	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Good
			u				Good	Fair
		GC		▨		Clayey gravels, gravel-sand-clay mixtures	Good	Fair
	Sand and Sandy Soils	SW		○	Red	Well-graded sands or gravelly sands, little or no fines	Good	Fair to good
		SP		●		Poorly graded sands or gravelly sands, little or no fines	Fair to good	Fair
		SM	d	○	Yellow	Silty sands, sand-silt mixtures	Fair to good	Fair to good
			u				Fair	Poor to fair
		SC		▩		Clayey sands, sand-silt mixtures	Poor to fair	Poor
Fine-Grained Soils	Silts and Clays LL < 50	ML		▥	Green	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor to fair	Not suitable
		CL		▨		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable
		OL		▥		Organic silts and organic silt-clays of low plasticity	Poor	Not suitable
	Silts and Clays LL ≥ 50	MH		▥	Blue	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable
		CH		▨		Inorganic clays of high plasticity, fat clays	Poor to fair	Not suitable
		OH		▨		Organic clays of medium to high plasticity, organic silts	Poor to very poor	Not suitable
Highly Organic Soils		Pt		≋	Orange	Peat and other highly-organic soils	Not suitable	Not suitable

NOTES: 1. **Divisions of the GM and SM groups (column 3) into subdivisions of d and u are applicable to roads and airfields only. Subdivision is based on the LL and PI; suffix d (for example, GMd) will be used when the LL is 25 or less and the PI is 5 or less; the suffix u will be used otherwise.**

Table B-3. Characteristics of soil groups pertaining to roads and airfields (continued)

Value As Base When not Subject to Frost Action (9)	Potential Frost Action (10)	Compressibility and Expansion (11)	Drainage Characteristics (12)	Compaction Equipment (13)	Dry Unit Weight (pcf) (14)	Typical Design Values	
						CBR (15)	Subgrade Modulus k (lb per cu in) (16)
Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125 -140	40 - 80	300 - 500
Fair to Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110 -140	30 - 60	300 - 500
Fair to Good	Slight to medium	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	125 -145	40 - 60	300 - 500
Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	115 -135	20 - 30	200 - 500
Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	130 -145	20 - 40	200 - 500
Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110 -130	20 - 40	200 - 400
Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	105 -135	10 - 40	150 - 400
Poor	Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120 -135	15 - 40	150 - 400
Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100 -130	10 - 20	100 - 300
Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100 -135	5 - 20	100 - 300
Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90 -130	15 or less	100 - 200
Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired roller, sheepsfoot roller	90 -130	15 or less	50 - 150
Not suitable	Medium to high	Medium to high	Poor	Rubber-tired roller, sheepsfoot roller	90 -105	5 or less	50 - 100
Not suitable	Medium to very high	High	Fair to poor	Rubber-tired roller, sheepsfoot roller	80 -105	10 or less	50 - 100
Not suitable	Medium	High	Practically impervious	Rubber-tired roller, sheepsfoot roller	90 -115	15 or less	50 - 150
Not suitable	Medium	High	Practically impervious	Rubber-tired roller, sheepsfoot roller	80 -110	5 or less	25 - 100
Not suitable	Slight	Very high	Fair to poor	Compaction not practical	-	-	-

NOTES (continued):

2. The equipment listed in column 13 will usually produce the required densities with a reasonable number of passes when moisture conditions and thickness lift are properly controlled. In some instances, several types of equipment are listed because variable soil characteristics within a given soil group may require different equipment. In some instances, a combination of two types may be necessary.

a. Processed base materials and other angular material. Steel-wheeled and rubber-tired rollers are recommended for hard, angular materials with limited fines or screenings. Rubber-tired equipment is recommended for softer materials subject to degradation.

b. Finishing. Rubber-tired equipment is recommended for rolling during final shaping operations for most soils and processed materials.

c. Equipment Size. The following sizes of equipment are necessary to assure the high densities required for airfield construction:

- **Crawler-type tractor—total weight in excess of 30,000 pounds.**
- **Rubber-tired equipment—wheel load in excess of 15,000 pounds; wheel loads as high as 40,000 pounds may be necessary to obtain the required densities for some materials (based on contact pressure of approximately 65 to 150 psi).**
- **Sheepsfoot roller—unit pressure (on 6- to 12-square-inch foot) to be in excess of 250 psi and unit pressures as high as 650 psi may be necessary to obtain the required densities for some materials. The area of the feet should be at least 5 percent of the total peripheral area of the drum, using the diameter measured to the faces of the feet.**

3. The range of dry unit weights listed in column 14 are for compacted soil at OMC when using the Standard Proctor Test (ASTM 1557-91).

4. The maximum CBR values (column 15) that can be used in design of airfields is, in some cases, limited by gradation and plasticity requirements.

Table B-4. Terzaghi classification

Unconfined Compressive Strength (Tons/Sq Ft)	Consistency
< 0.25	Very soft
0.25 to 0.50	Soft
0.50 to 1.00	Medium
1.00 to 2.00	Stiff
2.00 to 4.00	Very stiff
> 4.00	Hard

Suitability of Soils for Embankments

Three major factors that influence the suitability of soils for use in embankments are permeability, strength, and ease of compaction. The gravelly and sandy soils with little or no fines (groups GW, GP, SW, and SP) are stable, pervious, and able to attain good compaction with crawler-type tractors and rubber-tired rollers. The poorly graded materials may not be quite as desirable as those which are well graded, but all of the materials are suitable for use in the pervious sections of earth embankments. Poorly graded sands (SP) may be more difficult to use and, in general, should have flatter embankment slopes than the SW soils. The gravels and sands with fines (groups GM, GC, SM, and SC) have variable characteristics depending on the nature of the fine fraction and the gradation of the entire sample. These materials are often sufficiently impervious and stable to be used for impervious sections of embankments. The soils in these groups should be carefully examined to ensure that they are properly zoned with relation to other materials in an embankment.

Of the fine-grained soils, the CL group is best adapted for embankment construction; the soils are impervious, fairly stable, and give fair to good compaction with sheepsfoot or rubber-tired rollers. The MH soils, while not desirable for rolled-fill construction, may be used in the core of hydraulic-fill structures. Soils of the ML group may or may not have good compaction characteristics and, in general, must be closely controlled in the field to secure the desired strength. CH soils have fair stability when used on flat slopes but have detrimental shrinkage characteristics which may necessitate blanketing them or incorporating them in thin interior cores of embankments. Soils containing organic matter (groups OL, OH, and Pt) are not commonly used for embankment construction because of the detrimental effects of the organic matter present. Such materials may often be used to advantage in blankets and stability berms where strength is not important.

Permeability and Seepage Control

Since the permeability (column 8) and requirements for seepage control (column 12) are essentially functions of the same property of a soil, they will be discussed jointly. The subject of seepage in relation to embankments and foundations may be roughly divided into three categories:

- Seepage through embankments.
- Seepage through foundations.
- Control of uplift pressures.

These are discussed in relation to the soil groups in the following paragraphs.

Seepage Through Embankments

In the control of seepage through embankments, it is the relative permeability of adjacent materials rather than the actual permeability of such soils that governs their use in a given location. An earth embankment is not watertight, and the allowable quantity of seepage through it is largely governed by the use to which the structure is put. For example, in a flood-control project, considerable seepage may be allowed and the structure will still fulfill the storage requirements; whereas for an irrigation project, much less seepage is allowable because pool levels must be maintained. The more impervious soils (GM, GC, SM, SC, CL, MH, and CH) may be used in core sections or in homogeneous embankments to retard the flow of water. Where it is important that seepage not emerge on the downstream slope or the possibility of drawdown exists on upstream slopes, more pervious materials are usually placed on the outer slopes. The coarse-grained, free-draining soils (GW, GP, SW, SP) are best-suited for this purpose. Where a variety of materials is available, they are usually graded from least pervious to more pervious from the center of the embankment outward. Care should be used in the arrangement of materials in the embankment to prevent piping within the section. The foregoing statements do not preclude the use of other arrangements of materials in embankments. Dams have been constructed successfully entirely of sand (SW, SP, and SM) or of silt (ML) with the section made large enough to reduce seepage to an allowable value without the use of an impervious core. Coarse-grained soils are often used in drains and toe sections to collect seepage water in downstream sections of embankments. The soils used will depend largely on the material that they drain; in general, free-draining sands (SW and SP) or gravels (GW and GP) are preferred, but a silty sand (SM) may effectively drain a clay (CL and CH) and be entirely satisfactory.

Seepage Through Foundations

As in the case of embankments, the use of the structure involved often determines the amount of seepage control necessary in foundations. Cases could be cited where the flow of water through a pervious foundation would not constitute an excessive water loss and no seepage control measures would be necessary if adequate provisions were made against piping in critical areas. If seepage control is desired, then the more pervious soils are the soils in which necessary measures must be taken. Free-draining gravels (GW and GP) are capable of carrying considerable quantities of water, and some means of positive control (such as a cutoff trench) may be necessary. Clean sands (SW and SP) may be controlled by a cutoff or by an upstream impervious blanket. While a drainage trench at the downstream toe or a line of relief wells will not reduce the amount of seepage, either will serve to control seepage and route the flow into collector systems where it can be led away harmlessly. Slightly less pervious material (such as silty gravels [GM], silty sands [SM], or silts [ML]) may require a minor amount of seepage control such as that afforded by a toe trench, or if they are sufficiently impervious, no control may be necessary. The relatively impervious soils (GC, SC, CL, OL, MH, CH, and OH) usually pass such a small volume of water that seepage control measures are not necessary.

Control of Uplift Pressures

The problem of control of uplift pressures is directly associated with pervious foundation soils. Uplift pressures may be reduced by lengthening the path of seepage (by a cutoff or upstream blanket) or by measures for pressure relief in the form of wells, drainage trenches, drainage blankets, or pervious downstream shells. Free-draining gravels (GW and GP) may be treated by any of the aforementioned procedures; however, to obtain the desired pressure relief, the use of a positive cutoff may be preferred, as blanket, well, or trench installations would probably have to be too extensive for economical accomplishment of the desired results. Free-draining sands (SW and SP) are generally less permeable than the gravels and, consequently, the volume of water that must be controlled for pressure relief is usually less. Therefore a positive cutoff may not be required and an upstream blanket, wells, or a toe trench may be entirely effective. In some cases a combination of blanket and trench or wells may be desirable.

Silty soils (silty gravels [GM], silty sands [SM], and silts [ML]) usually do not require extensive treatment; a toe drainage trench or well system may be sufficient to reduce uplift pressures. The more impervious silty materials may not be permeable enough to permit dangerous uplift pressures to develop, and in such cases, no treatment is indicated. In general, the more impervious soils (GC, SC, CL, OL, MH, CH, and OH) require no treatment for control of uplift pressures. However, they do assume importance when they occur as a relatively thin top stratum over more pervious materials. In such cases, uplift pressures in the lower layers acting on the base of the impervious top stratum can cause heaving and formation of boils; treatment of the lower layer by some of the methods mentioned above is usually indicated in these cases. It is emphasized that control of uplift pressures should not be applied indiscriminately just because certain types of soils are encountered. Rather, the use of control measures should be based on a careful evaluation of conditions that do or can exist, and an economical solution should be reached that will accomplish the desired results.

Compaction Characteristics

Column 9 of *Table B-2, pages B-6 and B-7*, shows the general compaction characteristics of the various soil groups. The evaluations given and the equipment listed are based on average field conditions where proper moisture control and thickness of lift are attained and a reasonable number of passes of the compaction equipment are required to secure the desired density. For lift construction of embankments, the sheepsfoot and rubber-tired rollers are commonly used pieces of equipment. Some advantages may be claimed for the sheepsfoot roller in that it leaves a rough surface that affords better bond between lifts and it kneads the soil—affording better moisture distribution. Rubber-tired equipment referred to in the table is considered to be heavily loaded compactors or earthmoving equipment with a minimum wheel load of

15,000 pounds. If ordinary wobble-wheel rollers are used for compaction, the thickness of a compacted lift is usually reduced to about 2 inches.

Granular soils with little or no fines generally show good compaction characteristics, with the well-graded materials (GW and SW) usually furnishing better results than the poorly graded soils (GP and SP). The sandy soils, in most cases, are best compacted by crawler-type tractors; on the gravelly materials, rubber-tired equipment and sometimes steel-wheel rollers are also effective. Coarse-grained soils with fines of low plasticity (groups GM and SM) show good compaction characteristics with either sheepsfoot rollers or rubber-tired equipment; however, the range of moisture contents for effective compaction may be very narrow and close moisture control is desirable. This is also true of the silty soils in the ML group. Soils of the ML group may be compacted with rubber-tired equipment or with sheepsfoot rollers. Gravels and sands with plastic fines (groups GC and SC) show fair compaction characteristics, although this quality may vary somewhat with the character and amount of fines.

Rubber-tired or sheepsfoot rollers may be used. Sheepsfoot rollers are generally used for compacting fine-grained soils. The compaction characteristics of such materials are variable—lean clays and sandy clays (CL) being the best, fat clays and lean organic clays or silts (OL and CH) fair to poor, and organic or micaceous soils (MH and OH) usually poor.

For most construction projects of any magnitude, it is highly desirable to investigate the compaction characteristics of the soil by means of a field test section. Column 10 shows the ranges of unit dry weight for soils compacted according to the compaction test method as described in ASTM 1557-91 and Chapter 2 of this manual. It is emphasized that these values are for guidance only. Design or construction control should be based on laboratory test results.

Suitability of Soils for Foundations

Suitability of soils for foundations of embankments or structures depends primarily on the strength and consolidation characteristics of the subsoils. The type of structure and its use will largely govern the adaptability of a soil as a satisfactory foundation. For embankments, large settlements may be allowed and compensated for by overbuilding; whereas the allowable settlement of structures (such as control towers) may be small to prevent overstressing the concrete or steel of which they are built or because of the necessity for adhering to established grades. Therefore, a soil may be entirely satisfactory for one type of construction but may require special treatment for other types.

Strength and settlement characteristics of soils depend on a number of variables (such as structure, in-place density, moisture content, and cycles of loading in their geologic history) which are not readily evaluated by a classification system such as used here. For these reasons, only very general statements can be made as to the suitability of the various soil types as foundations. This is especially true for fine-grained soils.

In general, the gravels and gravelly soils (GW, GP, GM, and GC) have good bearing capacity and undergo little consolidation under load. Well-graded sands (SW) usually have a good bearing value. Poorly graded sands and silty sands (SP and SM) may exhibit variable bearing capacity depending on their density. This is true to some extent for all coarse-grained soils but is especially critical for uniformly graded soils of the SP and SM groups. Such soils, when saturated, may become “quick” and present an additional construction problem. Soils of the ML group may be subject to liquefaction and may have poor bearing capacities, particularly where heavy structure loads are involved. Of the fine-grained soils, the CL group is probably the best from a foundation standpoint, but in some cases, the soils may be soft and wet and exhibit poor bearing capacity and fairly large settlements under load. Soils of the MH groups and normally consolidated CH soils may show poor bearing capacity and large settlements. Organic soils (OL and OH) have poor bearing capacity and usually exhibit large settlement under load.

For most of the fine-grained soils discussed above, the type of structure foundation selected is governed by such factors as the bearing capacity of the soil and the magnitude of the load. It is possible that simple spread footings might be adequate to carry the load without excessive settlement in many cases. If the soils are poor and structure loads are relatively heavy, then alternate methods are indicated. Pile foundations may be necessary in some cases and in special instances—particularly in the case of some CH and OH soils—it may be desirable and economically feasible to remove such soils from the foundation. Highly-organic soils are generally very poor foundation materials. These may be capable of carrying very light loads but, in general, are unsuited for most construction purposes. If highly-organic soils occur in the foundation, they may be removed (if limited in extent), they may be displaced (by dumping firmer soils on top), or piling may be driven through them to a stronger layer. Proper treatment will depend on the structure involved.

GRAPHICAL PRESENTATION OF SOILS DATA

It is customary to present the results of soils explorations on drawings or plans as schematic representations of the borings or test pits with the soils encountered using various symbols. Commonly used hatching symbols are small, irregular round symbols for gravel; dots for sand; vertical lines for silts; and diagonal lines for clays. Combinations of these symbols represent the various combinations of materials found in the explorations. This system has been adapted to the various soil groups in the USCS and the appropriate symbols are shown in column 4 of *Table B-2, pages B-6 and B-7*. As an alternative to the hatching symbols, they may be omitted and the appropriate group letter symbol written in the boring log. In addition to the symbols on logs of borings, the effective size of coarse-grained soils and the natural water content of fine-grained soils should be shown by the side of the log. Other descriptive abbreviations may be used as deemed appropriate. In certain instances, the use of color to delineate soil types on maps and drawings is desirable. A suggested color scheme to show the major soil groups is described in column 5 of *Table B-2*.

CHARACTERISTICS OF SOIL GROUPS PERTAINING TO ROADS AND AIRFIELDS

The properties desired in soils for foundations under roads and airfields and for base courses under flexible pavements are adequate strength, good compaction characteristics, adequate drainage, resistance to frost action in areas where frost is a factor, and acceptable compression and expansion characteristics. Some of these properties, if inadequate in the soils available, may be supplied by proper construction methods. For instance, materials having good drainage characteristics are desirable, but if such materials are not available locally, adequate drainage may be obtained by installing a properly designed water-collecting system. Strength requirements for base-course materials (to be used immediately under the pavement of a flexible pavement structure) are high and only good-quality materials are acceptable. However, low strengths in subgrade materials may be compensated for in many cases by increasing the thickness of overlying concrete pavement or of base materials in flexible pavement construction. From the foregoing brief discussion, it may be seen that the proper design of roads and airfield pavements requires the evaluation of soil properties in more detail than is possible by using the general soils classification system. However, the grouping of soils in the classification system is such that a general indication of their behavior in road and airfield construction may be obtained.

FEATURES ON THE SOILS-CLASSIFICATION SHEET

General characteristics of the soil groups pertinent to roads and airfields are presented in *Table B-3, pages B-16 and B-17*. Columns 1 through 5 show major soil divisions, group symbols, hatching and color symbols; column 6 gives names of soil types; column 7 evaluates the performance (strength) of the soil groups when used as subgrade materials that will not be subject to frost action; columns 8 and 9 make a similar evaluation for the soils when used as subbase and base materials; column 10 shows potential frost action; column 11 shows compressibility and expansion characteristics; column 12 presents drainage characteristics; column 13 shows types of compaction equipment that perform satisfactorily on the various soil groups; column 14 shows ranges of unit dry weight for compacted soils; column 15 gives ranges of typical CBR values; and column 16 gives ranges of modulus of subgrade reaction (k). The various features presented are discussed in the following paragraphs.

Subdivision of Coarse-Grained Soil Groups

Note that in column 3 the basic soil groups (GM and SM) have each been subdivided into two groups designated by the suffixes d and u which have been chosen to represent desirable and less desirable (undesirable) base materials, respectively. This subdivision applies to roads and airfields only and is based on field observation and laboratory tests on the behavior of the soils in these groups. Basis for the subdivision is the LL and PI of the fraction of the soil passing the No. 40 sieve. The suffix d is used when the LL is 25 or less and the PI is 5 or less; otherwise, the suffix u is used. Typical symbols for soils in these groups are GMd and SMu.

Values of Soils as Subgrade, Subbase, or Base Materials

The descriptions in columns 7 through 9 give a general indication of the suitability of the soil groups for use as subgrades, subbase, or base materials, provided they are not subject to frost action. In areas where frost heaving is a problem, the value of materials as subgrades or subbases will be reduced, depending on the potential frost action of the material as shown in column 10. Proper design procedures should be used in situations where this is a problem. The coarse-grained soils, in general, are the best subgrade, subbase, and base materials. The GW group has excellent qualities as a subgrade and subbase, and is good as base material. Note that the adjective "excellent" is not used for any of the soils for base courses; "excellent" should be used in reference to a high-quality processed crushed stone. Poorly graded gravels and some silty gravels (groups GP and GMd) are usually only slightly less desirable as subgrade or subbase materials and, under favorable conditions, may be used as base materials for certain conditions. However, poor gradation and other factors sometimes reduce the value of such soils to the extent that they offer only moderate strength, and their value as a base material is less. The GMu, GC, and SW groups are reasonably good subgrade materials but are generally poor to not suitable as bases. The SP and SMd soils are usually considered fair to good subgrade and subbase materials but, in general, are poor to not suitable for base materials. The SMu and SC soils are fair to poor subgrade and subbase materials and are not suitable for base materials. The fine-grained soils range from fair to very poor subgrade materials as follows:

- Silts and lean clays (ML and CL)—fair to poor.
- Organic silts, lean organic clays, and micaceous or diatomaceous soils (OL and MH)—poor.
- Fat clays and fat organic clays (CH and OH)—poor to very poor.

These qualities are compensated for in flexible pavement design by increasing the thickness of overlying base material and in rigid pavement design by increasing the pavement thickness or by adding a base-course layer. None of the fine-grained soils are suitable as subbase or base materials. The fibrous organic soils (group Pt) are very poor subgrade materials and should be removed wherever possible; otherwise, special construction measures should be adopted. They are not suitable as subbase and base materials. The CBR values shown in column 15 give a relative indication of the strength of the various soil groups as used in flexible pavement design. Similarly, values of subgrade modulus (k) in column 16 are relative indications of strengths from plate-bearing tests as used in rigid pavement design. As these tests are used for the design of pavements, actual test values should be used for this purpose instead of the approximate values shown in the tabulation.

For wearing surfaces on unsurfaced roads, sand-clay-gravel mixtures (GC) are generally considered the most satisfactory. However, they should not contain too large a percentage of fines and the PI should be in the range of 5 to about 15.

Potential Frost Action

The relative effects of frost action on the various soil groups are shown in column 10. Regardless of the frost susceptibility of the various soil groups, two conditions must be present simultaneously before frost action will be a major consideration—a source of water during the freezing period and a sufficient period for the freezing temperature to penetrate the ground. Water necessary for the formation of ice lenses may become available from a high groundwater table or a capillary supply, within the soil voids, or through infiltration. The degree of ice formation that will occur in any given case is influenced by environmental factors such as topographic position, stratification of the parent soil, transitions into cut sections, lateral flow of water from side cuts, localized pockets of perched groundwater, and drainage conditions. In general, the silts and fine silty sands are the worst offenders as far as frost is concerned. Coarse-grained materials with little or no fines are affected only slightly if at all. Clays (CL and CH) are subject to frost action, but the loss of strength of such materials may not be as great as for silty soils. Inorganic soils containing less than three percent of grains finer than 0.02 millimeter in diameter by weight are generally not frost susceptible. Where frost-susceptible soils are encountered in subgrades and frost is a definite problem, two acceptable methods of design of pavements are available. Either a sufficient depth of acceptable granular material is placed over the soils to prevent freezing in the subgrade and thereby prevent the detrimental effects of frost action or a reduced depth of granular material is used, thereby allowing freezing in the subgrade, and the design is based on the reduced strength of the subgrade during the frost-melting period. In many cases, appropriate drainage measures to prevent the accumulation of water in the soil pores will help to diminish ice segregation in the subgrade and subbase.

Compressibility and Expansion

Two types of soil characteristics are applicable to road and runway design. The first is the relatively long-term compression or consolidation under the dead weight of the structure; the second is the short-term compression and rebound under moving wheel loads. The long-term consolidation of soils becomes a factor in design primarily when heavy fills are made on compressible soils. If adequate provision is made for this type of settlement during construction, it will have little influence on the pavement’s load-carrying capacity. However, when elastic soils subject to compression and rebound under wheel load are encountered, adequate protection must be provided, as even small movements of this soil may be detrimental to the base and wearing course of pavements.

It is fortunate that the free-draining, coarse-grained soils (GW, GP, SW, and SP), which in general make the best subgrade and subbase materials, exhibit almost no tendency toward high compressibility or expansion. In general, the compressibility of soils increases with an increasing LL. The foregoing is not completely true, as compressibility is also influenced by soil structure, grain shape, previous loading history, and other factors that are not evaluated in the classification system. Undesirable compressibility or expansion characteristics may be reduced by distributing the load through a greater thickness of overlying material. This is adequately handled by the CBR method of design for flexible pavements; however, rigid pavements may require the addition of an acceptable base course under the pavement.

Drainage Characteristics

The drainage characteristics of soils are a direct reflection of their permeability. The evaluation of drainage characteristics for use in roads and runways is shown in column 12. The presence of moisture in base, subbase, and subgrade materials—except for free-draining, coarse-grained soils—may cause the development of pore water pressures and loss of strength. The moisture may come from infiltration of rainwater or by capillary rise from an underlying water table. While free-draining materials permit rapid draining of water, they permit rapid ingress of water also. If such materials are adjacent to less-pervious materials and have free access to water they may serve as reservoirs to saturate the less-pervious materials. It is obvious, therefore, that in most instances adequate drainage systems should be provided. The gravelly and sandy soils with little or no fines (groups GW, GP, SW, and SP) have excellent drainage characteristics. The GMd and SMd groups have fair-to-poor drainage characteristics, whereas the GMu, GC, SMu, and SC groups may be practically impervious. Soils of the ML, MH, and Pt groups have fair-to-poor drainage characteristics. All of the other groups have poor drainage characteristics or are practically impervious.

Compaction Equipment

The compaction of soils for roads and runways, especially for the latter, requires that a high degree of density be attained at the time of construction so that detrimental consolidation will not take place under traffic. In addition, the detrimental effects of water are lessened in cases where saturation or near saturation takes place. Processed materials, such as crushed rock, are often used as base course and such materials require special treatment in compaction. Types of compaction equipment that will usually produce the desired densities are shown in column 13. Note that several types of equipment are listed for some of the soil groups; this is because variations in soil type within a given group may require the use of different equipment. In some cases, more than one type of equipment may be necessary to produce the desired densities.

Steel-wheeled rollers are recommended for angular materials with limited amounts of fines, crawler-type tractors or rubber-tired rollers for gravels and sands, and sheepsfoot rollers for coarse-grained or fine-grained soils having some cohesive qualities. Rubber-tired rollers are also recommended for final compaction operations for most soils except those with a high LL. Suggested minimum weights of the various types of equipment are shown in note 2 of *Table B-3, pages B-16 and B-17*. Column 14 shows ranges of unit dry weight for soils compacted according to the compaction test method as described in ASTM 1557-91 and Chapter 2. These values are included primarily for guidance. Design or control of construction should be based on actual test results.

GRAPHICAL PRESENTATION OF SOILS DATA

It is customary to present the results of soils explorations on drawings as schematic representations of the borings or test pits or on soil profiles with the various soils encountered shown by appropriate symbols. As one approach, the group's letter symbol may be written in the appropriate section of the log. As an alternative, the hatching symbols shown in column 4 of *Table B-2, pages*

B-6 and B-7, or *Table B-3, pages B-16 and B-17*, may be used. In addition, the natural-water content of fine-grained soils should be shown along the side of the log. Other descriptive abbreviations may be used as deemed appropriate. In certain instances, the use of color to delineate soil types on maps and drawings is desirable. A suggested color scheme to show the major soil groups is indicated in column 5 of *Tables B-2* or *B-3*.

Glossary

% F	percent of fines
% G	percent of gravel
% S	percent of sand
20d	twentypenny
AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt cement
ACI	American Concrete Institute
AFB	Air Force Base
AFESC	Air Force Engineering and Services Center
AFJMAN	Air Force Joint Manual
AFJP	Air Force Joint Pamphlet
AFM	Air Force Manual
agg	aggregate
AP	asphalt petroleum
approx	approximately
Apr	April
APSB	asphaltic penetrative soil binder
ASTM	American Society for Testing and Materials
attn	attention
Aug	August
C	Celsius
C	clay
CB	asphalt cutback
CBR	California Bearing Ratio
cc	cubic centimeter(s)
CF	cement factor
CH	clay, high plasticity
CL	centerline
CL	clay, low plasticity
cm	centimeter(s)
CM-K	medium-setting cationic asphalt emulsion
Co	company

coeff	coefficient
corr	corrected
cu ft	cubic foot, feet
cu in	cubic inch(es)
cu yd	cubic yard(s)
DA	Department of the Army
DC	District of Columbia
DCP	dynamic cone penetrometer
DD	Department of Defense
Dec	December
DM	design manual
ed	edition
elev	elevation
EM	engineering manual
F	Fahrenheit
FL	Florida
FM	field manual
FRBS	fine river-bar sand
g	gram(s)
G	gravel
gal	gallon(s)
GC	gravel, clayey
GCI	gyratory compactibility index
GM	gravel, silty
gm	gram(s)
GP	gravel, poorly graded
GSF	gyratory shear factor
GSI	gyratory stability index
GTM	gyratory testing machine
GW	gravel, well graded
H	high plasticity
HQ	headquarters
hydro	hydrometer
IL	Illinois
in	inch(es)
kg	kilogram
L	low plasticity
lb	pound(s)

LCF	lime-cement-fly ash
liq	liquid
LL	liquid limit
LRA	local reproduction authorized
M	silt
max	maximum
MC	medium-curing
MDD	maximum dry density
med	medium
MH	silt, high plasticity
MIL-STD	military standard
min	minimum
ml	milliliter(s)
ML	silt, low plasticity
mm	millimeter(s)
MMC	minimum moisture content
MS	medium setting
N/A	not applicable
naut	nautical
NAVFAC	Naval Facilities Engineering Command
NCOIC	noncommissioned officer in charge
NFS	not frost susceptible
NJ	New Jersey
No.	number
NRC	Nuclear Regulatory Commission
NSN	national stock number
NY	New York
O	organic material
OAC	optimum asphalt content
OH	organic material, high plasticity
OL	organic material, low plasticity
OMC	optimum moisture content
oz	ounce(s)
P	poorly graded
PA	Pennsylvania
PCA	Portland Cement Association
pcf	pounds per cubic foot
PFC	private first class

PI	plasticity index
PL	plastic limit
psi	pound(s) per square inch
Pt	peat, highly organic
PV2	private second class
PVT	private
qt	quart(s)
RC	rapid-curing asphalt cutback
RPO	radiation protection officer
RS	rapid setting
RT	road tar
RTCB	road-tar cutback
S	sand
SC	sand, clayey
SC	slow-curing
SFC	sergeant first class
SGT	sergeant
SM	sand, silty
SM-K	medium-setting cationic aphalt emulsion
SP	sand, poorly graded
SP4	specialist fourth class
SPC	specialist
SS	slow setting
SS-K	slow-setting cationic asphalt emulsion
SSD	saturated, surface-dry
SSG	staff sergeant
ST	special text
stat	statute
SW	sand, well graded
TACOM	United States Army Tank-automotive and Armaments Command
temp	temperature
TM	technical manual
TO	theater of operations
TRADOC	United States Army Training and Doctrine Command
US	United States
USAES	United States Army Engineer School
USCS	Unified Soil Classification System
VA	Virginia

vol	volume
w/	with
w	moisture content
W	well graded
W/C ratio	ratio of water to cement
wt	weight

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Index

FM 5-472
Materials Testing

FORMS

DD FORM 1205	SOIL MOISTURE-CONTEENT CETERMINATION
DD FORM 1206	GRAIN-SIZE ANALYSIS (SIEVE METHOD)
DD FORM 1207	GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADATION CHART
DD FORM 1208	SPECIFIC-GRAVITY TESTS
DD FORM 1209	LIQUID- AND PLASTIC-LIMITS DETERMINATION
DD FORM 1210	LABORATORY COMPACTION CHARACTERISTICS OF SOIL (COMPACTION TEST)
DD FORM 1211	LABORATORY SOIL-COMPACTION TEST GRAPH
DD FORM 1212	LABORATORY CALIFORNIA BEARING RATIO (CBR) TEST DATA
DD FORM 1215	IN-PLACE DENSITY DETERMINATION - SAND-CONE METHOD
DD FORM 1216	SPECIFIC GRAVITY OF BITUMINOUS MIX COMPONENTS
DD FORM 1217	BITUMINOUS MIX DESIGN - AGGREGATE BLENDING
DD FORM 1218	MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES
DD FORM 1219	BITUMINOUS MIX CURVES
DD FORM 1793	DETERMINATION OF ASPHALT CONTENT
DD FORM 1794	GRAIN-SIZE ANALYSIS (HYDROMETER METHOD)
DD FORM 2463	CALIFORNIA BEARING RATIO (CBR) ANALYSIS
DD FORM 2464	REPORT OF FOUNDATION AND BORROW INVESTIGATION

SOIL MOISTURE-CONTENT DETERMINATION

1. PROJECT		**2. DATE**
3. JOB NUMBER	**4. TEST SITE**	**5. SAMPLE NUMBER**

TEST AVERAGE	**%**						
RUN NUMBER							
TARE NUMBER							
a. WEIGHT OF TARE + WET SOIL							
b. WEIGHT OF TARE + DRY SOIL							
c. WEIGHT OF WATER, W_w	*(a - b)*						
d. WEIGHT OF TARE							
e. WEIGHT OF DRY SOIL, W_s	*(b - d)*						
WATER CONTENT, w	*(c/e x 100)*	%	%	%	%	%	%

TEST AVERAGE	**%**						
RUN NUMBER							
TARE NUMBER							
a. WEIGHT OF TARE + WET SOIL							
b. WEIGHT OF TARE + DRY SOIL							
c. WEIGHT OF WATER, W_w	*(a - b)*						
d. WEIGHT OF TARE							
e. WEIGHT OF DRY SOIL, W_s	*(b - d)*						
WATER CONTENT, w	*(c/e x 100)*	%	%	%	%	%	%

TEST AVERAGE	**%**						
RUN NUMBER							
TARE NUMBER							
a. WEIGHT OF TARE + WET SOIL							
b. WEIGHT OF TARE + DRY SOIL							
c. WEIGHT OF WATER, W_w	*(a - b)*						
d. WEIGHT OF TARE							
e. WEIGHT OF DRY SOIL, W_s	*(b - d)*						
WATER CONTENT, w	*(c/e x 100)*	%	%	%	%	%	%

TEST AVERAGE	**%**						
RUN NUMBER							
TARE NUMBER							
a. WEIGHT OF TARE + WET SOIL							
b. WEIGHT OF TARE + DRY SOIL							
d. WEIGHT OF WATER, W_w	*(a - b)*						
d. WEIGHT OF TARE							
e. WEIGHT OF DRY SOIL, W_s	*(b - d)*						
WATER CONTENT, w	*(c/e x 100)*	%	%	%	%	%	%

6. REMARKS

WATER CONTENT $w = \frac{Ww}{Ws} \times 100$

7. TECHNICIAN *(Signature)*	**8. COMPUTED BY** *(Signature)*	**9. CHECKED BY** *(Signature)*

DD FORM 1205, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

GRAIN-SIZE ANALYSIS (SIEVE METHOD)

1. PROJECT		2. DATE
3. JOB NUMBER	4. EXCAVATION	5. DATE COMPLETED
6. NOTES ABOUT SAMPLE/DESCRIPTION		7. SAMPLE NUMBER
		8. ORIGINAL SAMPLE WEIGHT
9. PREWASHED ☐ YES ☐ NO	10. + #200 SAMPLE WEIGHT, WASHED	11. - #200 SAMPLE WEIGHT, WASHED

12. SIEVE SIZE	13. SIEVE WEIGHT	14. SIEVE-SAMPLE WEIGHT	15. WEIGHT RETAINED	16. CUMULATIVE WEIGHT RETAINED	17. PERCENT RETAINED	18. PERCENT PASSING

19. TOTAL WEIGHT RETAINED IN SIEVES *(Sum of column 15)*		24. ERROR *(8-23)*	25. ERROR IN PERCENTAGE $\left(\frac{24}{8}\right) \times 100 =$
20. WEIGHT SIEVED THROUGH #200 *(Weight in pan)*			
21. WASHING LOSS *(8 - [10+11])*			
22. TOTAL WEIGHT PASSING #200 SIEVE *(20+11)*			
23. TOTAL WEIGHT OF FRACTIONS *(19+22)*			

26. PERCENT GRAVEL *(% G)*	27. PERCENT SAND *(% S)*	28. PERCENT FINES *(% F)*	29. DECIMAL FINES *(% F ÷ 100)*

30. REMARKS

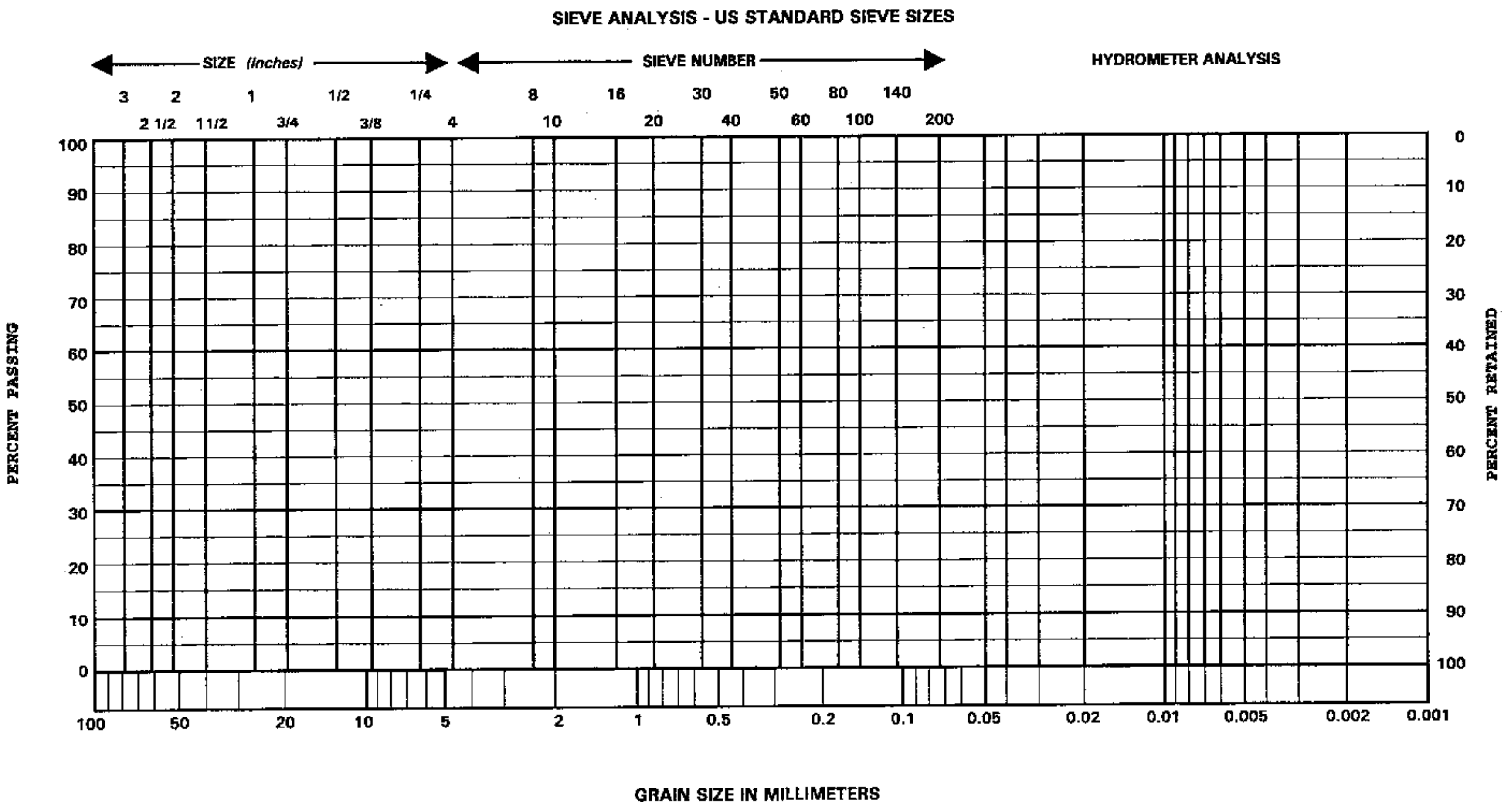
31. TECHNICIAN *(Signature)*	32. COMPUTED BY *(Signature)*	33. CHECKED BY *(Signature)*

DD FORM 1206, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

GRAIN SIZE DISTRIBUTION GRAPH - AGGREGATE GRADATION CHART

1. PROJECT

2. DATE



EXCAVATION NUMBER	SAMPLE NUMBER	LL	PL	PI	Cu (D_{60}/D_{10})	Cc $(D_{30})^2/(D_{60} \times D_{10})$	SOIL DESCRIPTION/REMARKS	CLASSIFICATION (USCS)

3. TECHNICIAN *(Signature)*

4. PLOTTED BY *(Signature)*

5. CHECKED BY *(Signature)*

DD FORM 1207, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

SPECIFIC-GRAVITY TESTS

1. PROJECT		2. DATE
3. BORING NUMBER	4. JOB NUMBER	5. EXCAVATION NUMBER

6. SPECIFIC GRAVITY OF SOLIDS (G_s)

FLASK CALIBRATION DATA	a. FLASK NUMBER	b. CLEAN, DRY WEIGHT, W_b Grams	c. FLASK + WATER WEIGHT, W_{bw} Grams	d. OBSERVED TEMPERATURE, T_i °C

e. SAMPLE OR DETERMINATION NUMBER					
DETERMINATION DATA	f. DISH NUMBER				
	g. WEIGHT OF DISH + DRY SOIL	Grams			
	h. WEIGHT OF DISH	Grams			
	i. WEIGHT OF DRY SOIL, W_s	Grams			
	j. WEIGHT OF FLASK + WATER + IMMERSED SOIL, W_{bws}	Grams			
	k. TEMPERATURE OF WATER, T_x	°C			
	l. CALCULATED WEIGHT OF FLASK + WATER AT T_x, W_{bw}	Grams			
	m. CORRECTION FACTOR FOR T_x, K				
n. SPECIFIC GRAVITY OF SOLIDS		$G_s = \frac{W_s K}{W_s + W_{bw} - W_{bws}}$			

7. APPARENT (G_a) AND BULK (G_m) SPECIFIC GRAVITY

a. SAMPLE OR SPECIMEN NUMBER					
b. TEMPERATURE OF WATER AND SOIL (°C) (must be within 23 ±1.7°C)					
c. TARE + SATURATED SURFACE - DRY SOIL					
DETERMINATION DATA	d. TARE				
	e. SATURATED SURFACE - DRY SOIL, *(B)*				
	f. (WIRE BASKET + SOIL) IN WATER				
	g. WIRE BASKET IN WATER				
	h. SATURATED SOIL IN WATER, *(C)*				
	i. TARE AND DRY SOIL				
	j. TARE				
	k. DRY SOIL, *(A)*				
l. APPARENT SPECIFIC GRAVITY		$G_a = (A) / (A - C)$			
m. BULK SPECIFIC GRAVITY		$G_m = (A) / (B - C)$			
n. BULK SPECIFIC GRAVITY, SATURATED SURFACE DRY (SSD)		$G_m = (B) / (B - C)$			

8. REMARKS

9. TECHNICIAN *(Signature)*	10. COMPUTED BY *(Signature)*	11. CHECKED BY *(Signature)*

DD FORM 1208, DEC 1999

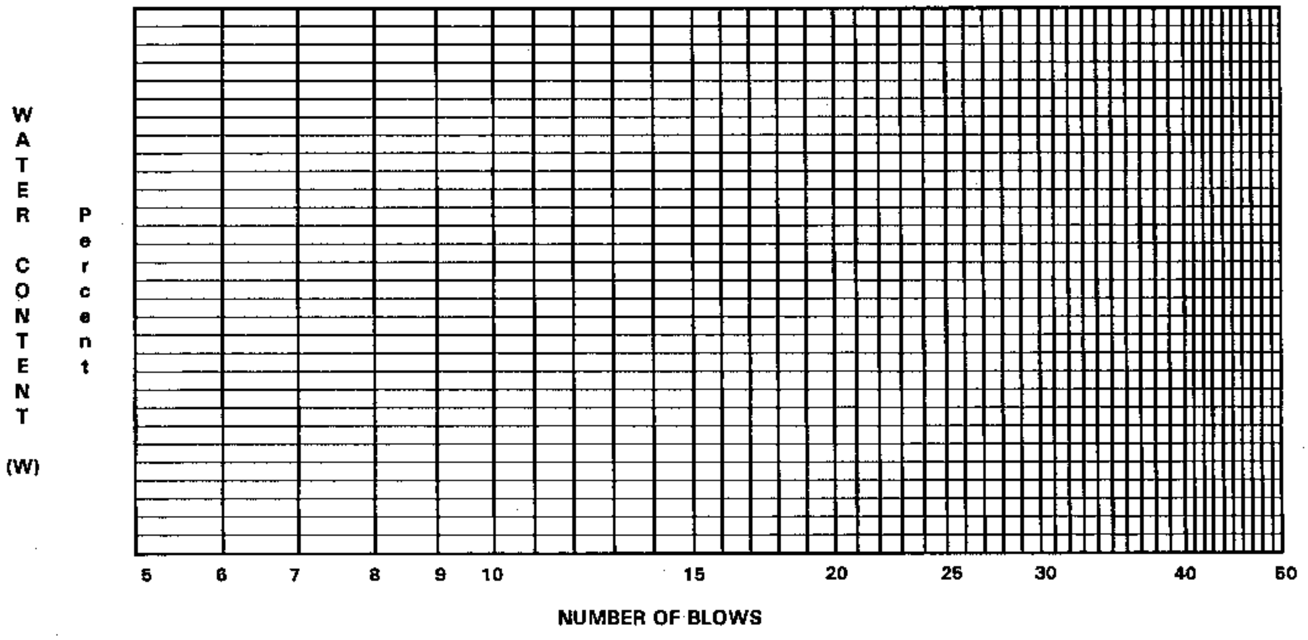
PREVIOUS EDITION IS OBSOLETE.

LIQUID- AND PLASTIC-LIMITS DETERMINATION

1. PROJECT		2. DATE
3. EXCAVATION NUMBER	4. JOB NUMBER	5. SAMPLE NUMBER

6. LIQUID LIMIT, LL

RUN NUMBER						
TARE NUMBER						
a. WEIGHT OF WET SOIL + TARE						
b. WEIGHT OF DRY SOIL + TARE						
c. WEIGHT OF WATER $W_w = a - b$						
d. WEIGHT OF TARE						
e. WEIGHT OF DRY SOIL $W_s = b - d$						
WATER CONTENT $w = \frac{W_w}{W_s} \times 100$						
NUMBER OF BLOWS						



7. PLASTIC LIMIT, PL

RUN NUMBER						
TARE NUMBER						
a. WEIGHT OF WET SOIL + TARE						
b. WEIGHT OF DRY SOIL + TARE						
c. WEIGHT OF WATER $W_w = a - b$						
d. WEIGHT OF TARE						
e. WEIGHT OF DRY SOIL $W_s = b - d$						
WATER CONTENT $w = \frac{W_w}{W_s} \times 100$						
PLASTIC LIMIT, PL *(Average w)*		▨	▨	▨	▨	▨

8. REMARKS

LL =	
PL =	
(LL - PL) PI =	

9. TECHNICIAN *(Signature)*	10. COMPUTED BY *(Signature)*	11. CHECKED BY *(Signature)*

DD FORM 1209, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

LABORATORY COMPACTION CHARACTERISTICS OF SOIL (COMPACTION TEST)

1. PROJECT	2. EXCAVATION NUMBER	3. SAMPLE NUMBER	4. DATE
	5. LAYERS/BLOWS PER LAYER /	6. WEIGHT OF TAMPER *(lb)*	7. HEIGHT OF DROP *(in)*
	8. SPECIFIC GRAVITY OF SOLIDS, G_s	9. DIAMETER OF MOLD *(in)*	10. VOLUME OF SOIL SAMPLE *(cu ft)* ☐ 0.0333 cu ft ☐ 0.0750 cu ft

11. RUN NUMBER		UNITS													
12. WEIGHT OF MOLD + WET SOIL		Grams													
13. WEIGHT OF MOLD		Grams													
14. WEIGHT OF WET SOIL	*(12 - 13)*	Grams													
15. WET UNIT WEIGHT, γwet	*((14/453.6)/10)**	Pcf													
16. TARE NUMBER															
a. WEIGHT OF TARE + WET SOIL		Grams													
b. WEIGHT OF TARE + DRY SOIL		Grams													
c. WEIGHT OF WATER, W_w	*(a - b)*	Grams													
d. WEIGHT OF TARE		Grams													
e. WEIGHT OF DRY SOIL, W_s	*(b - d)*	Grams													
f. WATER CONTENT, $w = \frac{W_w}{W_s} \times 100$	*(c / e x 100)*	Percent													
17. AVERAGE WATER CONTENT		Percent													
18. DRY UNIT WEIGHT, $\gamma d = \frac{\gamma wet}{1+(w/100)}$		Pcf													

19. REMARKS

* This formula contains the conversion from grams to pounds. Omit the conversion factor if the unit weight used is not grams.

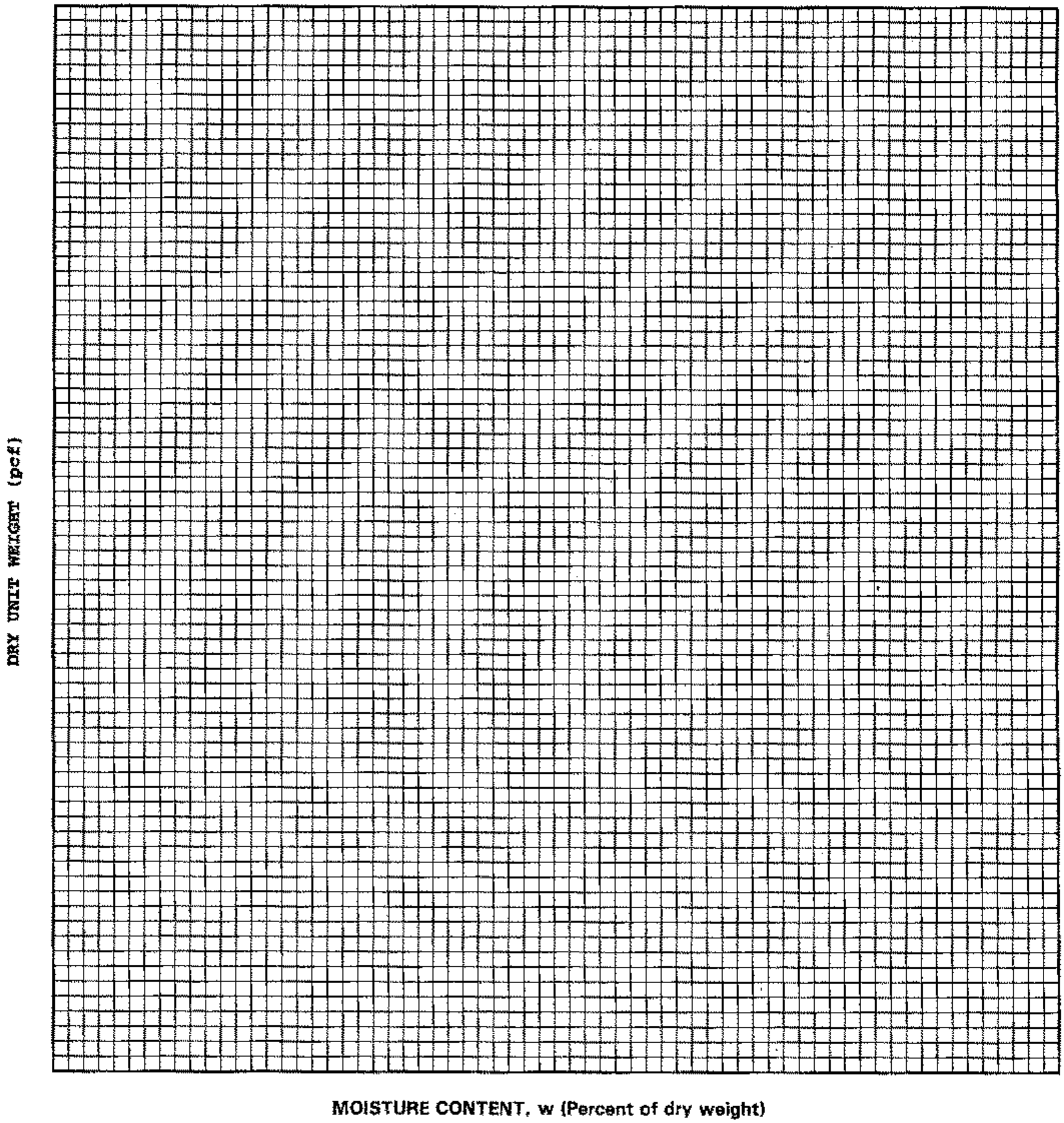
20. TECHNICIAN *(Signature)*	21. COMPUTED BY *(Signature)*	22. CHECKED BY *(Signature)*

DD FORM 1210, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

LABORATORY SOIL-COMPACTION TEST GRAPH

1. PROJECT	2. DATE	RESULTS
3. EXCAVATION NUMBER	4. SAMPLE NUMBER	7. MAXIMUM DRY UNIT WEIGHT
5. REMARKS *(Use back if more space is needed)*	6. SPECIFIC GRAVITY, G_s	8. OPTIMUM MOISTURE (WATER) CONTENT



9. TECHNICIAN *(Signature)*	10. PLOTTED BY *(Signature)*	11. CHECKED BY *(Signature)*

DD FORM 1211, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

LABORATORY CALIFORNIA BEARING RATIO (CBR) TEST DATA

1. PROJECT		2. DATE
3. EXCAVATION NUMBER	4. SAMPLE NUMBER	5. CONDITION ☐ DISTURBED ☐ UNDISTURBED

COMPACTION DATA	6. MOLD NUMBER		7. NUMBER OF LAYERS		8. BLOWS PER LAYER	
	9. PERCENT OF 3/4 in MATERIAL REPLACED		10. WEIGHT OF HAMMER *(lb)*		11. HEIGHT OF DROP *(in)*	
PROVING-RING DATA	12. NUMBER	13. CONSTANT	14. CAPACITY	15. SURCHARGE WEIGHT	16. SOAKING *(lb)*	17. PENETRATING *(lb)*
18. SWELL DATA *(Initial / Final)*	a. DATE	b. TIME	c. ELAPSED TIME	d. DIAL READING	e. INITIAL HEIGHT	f. SWELL PERCENT
			0.00			*(d / e x 100)*

19. PENETRATION DATA

a. PENETRATION *(in)*	b. STANDARD UNIT LOAD *(psi)*	c. PROVING RING DIAL READING *(in)*	d. CORRECTED RING DIAL READING *(in)*	e. TOTAL LOAD *(lb)*	f. UNIT LOAD *(psi)* *(e / 3.00)*	g. CORRECTED UNIT LOAD *(psi)*	h. CBR *(%)* *(g / b x 100)*
0.025	250						
0.050	500						
0.075	750						
0.100	1000						
0.125	1125						
0.150	1250						
0.175	1375						
0.200	1500						
0.300	1900						
0.400	2300						
0.500	2600						

WATER CONTENT AND UNIT WEIGHT DATA

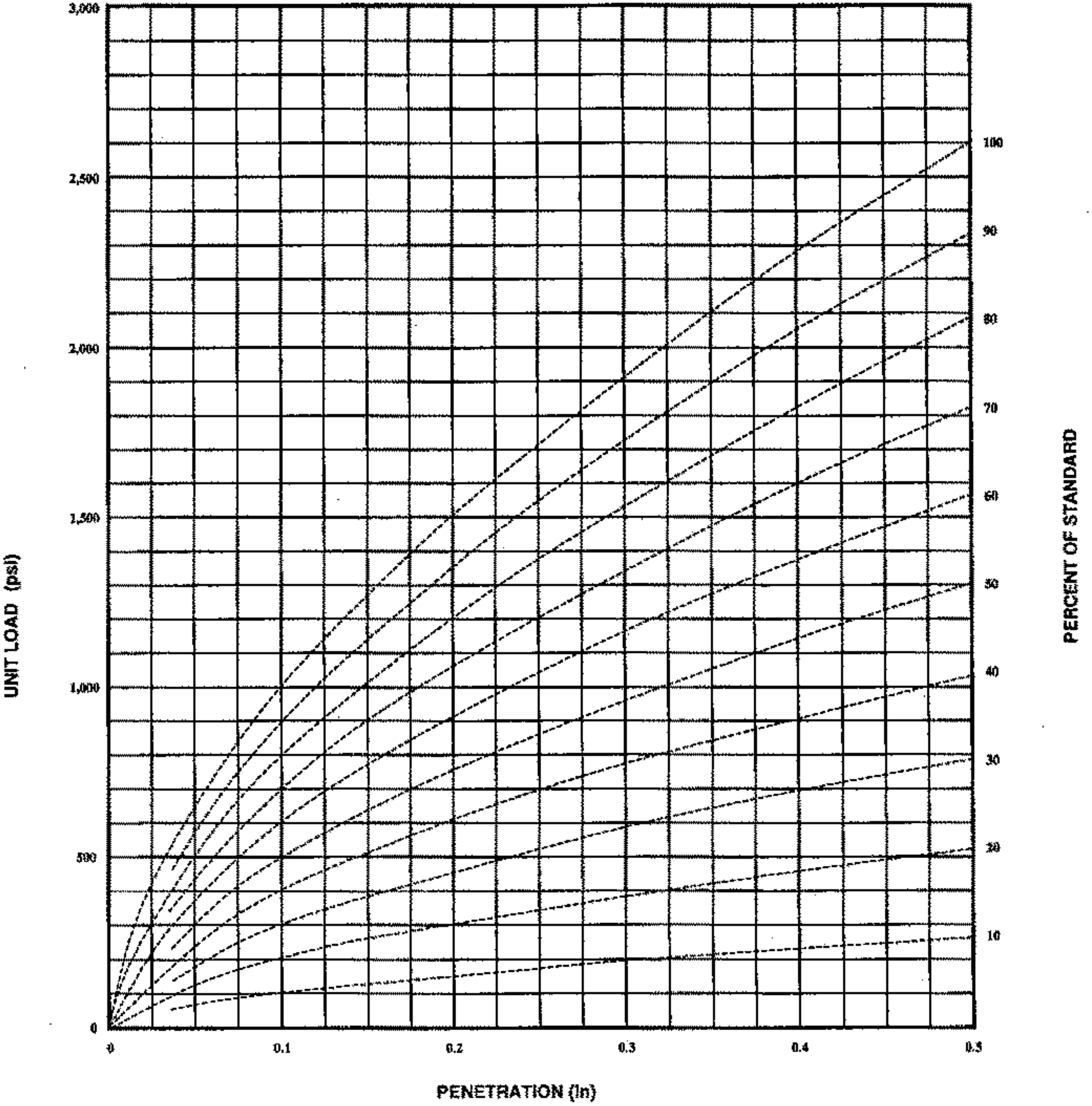
SAMPLES TAKEN	UNITS	BEFORE SOAKING		AFTER SOAKING	
20. WEIGHT OF MOLD + WET SOIL	Grams				
21. WEIGHT OF MOLD	Grams				
22. WEIGHT OF WET SOIL *(20 - 21)*	Grams				
23. WET UNIT WEIGHT, γ wet *([22 / 453.6] / 0.075)*	Pcf				
24. TARE NUMBER / SAMPLE TAKEN		BEFORE COMPACTION	AFTER COMPACTION	TOP 1 INCH	FROM MOLD
a. WEIGHT OF TARE + WET SOIL	Grams				
b. WEIGHT OF TARE + DRY SOIL	Grams				
c. WEIGHT OF WATER, W_w *(a - b)*	Grams				
d. WEIGHT OF TARE	Grams				
e. WEIGHT OF DRY SOIL, W_s *(b - d)*	Grams				
f. WATER CONTENT, $w = \frac{W_w}{W_s} \times 100$ *(c / e x 100)*	Percent				
25. AVERAGE WATER CONTENT	Percent				
26. DRY UNIT WEIGHT, $\gamma d = \frac{\gamma \text{ wet}}{1 + (w / 100)}$	Pcf				

DD Form 1212, DEC 1999

EDITION OF AUG 57 IS OBSOLETE.

CBR TEST GRAPH

(Plot test curve below to obtain corrected unit load)



27. REMARKS

28. TECHNICIAN *(Signature)*	29. COMPUTED BY *(Signature)*	30. CHECKED BY *(Signature)*

IN-PLACE DENSITY DETERMINATION - SAND-CONE METHOD

1. PROJECT

2. DATE

3. JOB NUMBER

4. TEST SITE

5. SAMPLE NUMBER

6. ADDITIONAL SPECIFICATIONS

CONVERSION FACTORS: 1 in = 2.54 cm 1 cu ft = 1728 cu in
1 lb = 453.6 gm
Unit weight of water: 1 cc = 1 gram
1 cu ft = 62.4 lb

CALIBRATION OF SAND (STANDARD MATERIAL)

APPARATUS OR TARE NUMBER	**UNITS**						
7. WEIGHT OF APPARATUS OR TARE FILLED	**Grams**						
8. WEIGHT OF APPARATUS OR TARE EMPTY	**Grams**						
9. WEIGHT OF MATERIAL *(7 - 8)*	**Grams**						
10. VOLUME OF APPARATUS OR TARE	**Cu ft**						
11. UNIT WEIGHT OF MATERIAL *([9/453.6]/10)**	**Pcf**						
12. AVERAGE UNIT WEIGHT OF MATERIAL	**Pcf**						

CALIBRATION OF APPARATUS		**TEMPLATE NUMBER**	**CONE NUMBER**
13. INITIAL WEIGHT OF APPARATUS + SAND	**Grams**		
14. FINAL WEIGHT OF APPARATUS + SAND	**Grams**		
15. WEIGHT OF SAND IN TEMPLATE AND/OR CONE *(13 - 14)*	**Grams**		

VOLUME OF THE HOLE

16. INITIAL WEIGHT OF APPARATUS + SAND	**Grams**		
17. FINAL WEIGHT OF APPARATUS + SAND	**Grams**		
18. WEIGHT OF SAND RELEASED *(16 - 17)*	**Grams**		
19. WEIGHT OF SAND IN THE HOLE *(18 - 15)*	**Grams**		
20. VOLUME OF THE HOLE *([19/453.6]/12)**	**Cu ft**		

WATER-CONTENT DETERMINATION

TARE NUMBER					
21. WEIGHT OF WET SOIL AND TARE	**Grams**				
22. WEIGHT OF DRY SOIL AND TARE	**Grams**				
23. WEIGHT OF WATER *(21 - 22)*	**Grams**				
24. WEIGHT OF TARE	**Grams**				
25. WEIGHT OF DRY SOIL *(22 - 24)*	**Grams**				
26. WATER CONTENT *([23/25] x 100)*	**Percent**				
27. AVERAGE WATER CONTENT	**Percent**				

UNIT-WEIGHT DETERMINATION

TARE NUMBER			
28. WEIGHT OF WET SOIL AND TARE	**Grams**		
29. WEIGHT OF TARE	**Grams**		
30. WEIGHT OF WET SOIL *(28 - 29)*	**Grams**		
31. WET UNIT WEIGHT *([30/453.6]/20)**	**Pcf**		
32. DRY UNIT WEIGHT *(31 x [100/(100+27)])*	**Pcf**		

33. REMARKS

* This formula contains the conversion from grams to pounds. Omit the conversion factor if the unit weight used is not grams.

34. TECHNICIAN *(Signature)*

35. COMPUTED BY *(Signature)*

36. CHECKED BY *(Signature)*

SPECIFIC GRAVITY OF BITUMINOUS MIX COMPONENTS	DATE

PROJECT	JOB

COARSE AGGREGATE	**UNITS** *(Grams)*			
MATERIAL PASSING_________SIEVE AND RETAINED ON_______SIEVE				
SAMPLE NUMBER				
1. WEIGHT OF OVEN - DRY AGGREGATE				
2. WEIGHT OF SATURATED AGGREGATE IN WATER				
3. DIFFERENCE *(Line 1 minus 2)*				
APPARENT SPECIFIC GRAVITY, $G = \frac{(\text{Line } 1)}{(\text{Line } 3)}$				
FINE AGGREGATE	**UNITS** *(Grams)*			
MATERIAL PASSING NUMBER________SIEVE				
SAMPLE NUMBER				
4. WEIGHT OF OVEN - DRY MATERIAL				
5. WEIGHT OF FLASK FILLED WITH WATER AT 20° C				
6. SUM (Line 4 + 5)				
7. WEIGHT OF FLASK + AGGREGATE + WATER AT 20° C,				
8. DIFFERENCE *(Line 6 minus 7)*				
APPARENT SPECIFIC GRAVITY, $G = \frac{(\text{Line } 4)}{(\text{Line } 8)}$				
FILLER	**UNITS** *(Grams)*			
SAMPLE NUMBER				
9. WEIGHT OF OVEN - DRY MATERIAL				
10. WEIGHT OF FLASK FILLED WITH WATER AT 20° C,				
11. SUM *(Line 9 + 10)*				
12. WEIGHT OF FLASK + AGGREGATE + WATER AT 20° C,				
13. DIFFERENCE *(Line 11 minus 12)*				
APPARENT SPECIFIC GRAVITY, $G = \frac{(\text{Line } 9)}{(\text{Line } 13)}$				
BINDER	**UNITS** *(Grams)*			
SAMPLE NUMBER				
14. WEIGHT OF PYCNOMETER FILLED WITH WATER				
15. WEIGHT OF EMPTY PYCNOMETER				
16. WEIGHT OF WATER *(Line 14 minus 15)*				
17. WEIGHT OF PYCNOMETER + BINDER				
18. WEIGHT OF BINDER *(Line 17 minus 15)*				
19. WEIGHT OF PYCNOMETER + BINDER + WATER TO FILL PYCNOMETER				
20. WEIGHT OF WATER TO FILL PYCNOMETER *(Line 19 minus 17)*				
21. WEIGHT OF WATER DISPLACED BY BINDER *(Line 16 minus 20)*				
APPARENT SPECIFIC GRAVITY, $G = \frac{(\text{Line } 18)}{(\text{Line } 21)}$				

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*

DD Form 1216, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

BITUMINOUS MIX DESIGN - AGGREGATE BLENDING

DATE

PROJECT

JOB

AGGREGATE GRADATION NUMBER

GRADATION OF MATERIAL

SIEVE SIZE *(To be entered by Technician):* →												
MATERIAL USED	PERCENT PASSING											
DESIRED:												

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE *(To be entered by Technician):* →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE *(To be entered by Technician):* →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

DD Form 1217, DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE *(To be entered by Technician):* →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

COMBINED GRADATION FOR BLEND - TRIAL NUMBER ____________

SIEVE SIZE *(To be entered by Technician):* →													
MATERIAL USED	% USED	PERCENT PASSING											
BLEND:													
DESIRED:													

REMARKS

TECHNICIAN *(Signature)*	COMPUTED BY *(Signature)*	CHECKED BY *(Signature)*

DD Form 1217 Reverse, DEC 65

MARSHALL METHOD - COMPUTATION OF PROPERTIES OF ASPHALT MIXTURES

DATE OF COMPUTATION

JOB NUMBER

PROJECT

DESCRIPTION OF BLEND

SPECIMEN NUMBER	ASPHALT CEMENT *(Percent)*	THICK-NESS *(Inches)*	WEIGHT *(Grams)*		VOLUME CC	SPECIFIC GRAVITY		AC BY VOLUME *(Percent)*	VOIDS *(Percent)*		UNIT WEIGHT TOTAL MIX *(Lb./Cu.Ft.)*	STABILITY *(Pounds)*		FLOW UNITS OF 1/100 IN.
			IN AIR	IN WATER		ACTUAL	THEO-RIZED		TOTAL MIX	FILLED		MEASURED	CON-VERTED	
a	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*	*l*	*m*	*n*	*o*
					$(d - e)$	$\frac{(d)}{(f)}$		$\frac{(b \times g)}{(Sp. Gr. of AC)}$	$(100 - 100\frac{(g)}{(h)})$	$\frac{(i)}{(i + j)}$	$(g \times 62.4)$		*	

**From conversion table*

COMPUTED BY

CHECKED BY

DD Form 1218, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE.

BITUMINOUS MIX CURVES

DATE

PROJECT

JOB

RECAP OF MARSHALL TEST RESULTS

PERCENT BITUMEN CONTENT					
UNIT WEIGHT - AGGREGATE ONLY					
UNIT WEIGHT - TOTAL MIX					
STABILITY					
FLOW					
PERCENT VOIDS - AGGREGATE ONLY					
PERCENT VOIDS - TOTAL MIX					
PERCENT OF TOTAL VOIDS FILLED WITH BITUMEN					

FLOW *(1/100 Inch)*

PERCENT BITUMEN

UNIT WEIGHT - TOTAL MIX *(Lb. Per Cu. Ft.)*

PERCENT BITUMEN

PERCENT VOIDS - TOTAL MIX

PERCENT BITUMEN

STABILITY *(Lb.)*

PERCENT BITUMEN

PERCENT TOTAL VOIDS FILLED WITH BITUMEN

PERCENT BITUMEN

TECHNICIAN *(Signature)*

PLOTTED BY *(Signature)*

CHECKED BY *(Signature)*

DD Form 1219, 1 DEC 65 PREVIOUS EDITION OF THIS FORM IS OBSOLETE

DETERMINATION OF ASPHALT CONTENT
(Dulin - Rotarex Extractor)

PROJECT

DATE

LINE NO.	ITEM		UNIT	1	2
1.	Weight of original sample and tare		g		
2.	Weight of tare		g		
3.	Weight of original sample	(1-2)	g		
4.	Weight of clean aggregate and tare		g		
5.	Weight of tare		g		
6.	Weight of clean aggregate	(4 - 5)	g		
7.	Final weight of filter paper		g		
8.	Initial weight of filter paper		g		
9.	Weight of filler in filter paper		g		
10.	Total amount of solvent		cm^3		
11.	Amount of solvent evaporated and ignited		cm^3		
12.	Final weight of evaporation dish and residue		g		
13.	Initial weight of clean evaporation dish		g		
14.	Amount of filler in evaporation dish	(12 - 13)	g		
15.	Amount of filler in total solvent	$(\frac{10}{11} \times 14)$	g		
16.	Total amount of aggregate in sample	(6+9+15)	g		
17.	Percentage asphalt	$(\frac{3-16}{6} \times 100)$	%		

REMARKS

TECHNICIAN	COMPUTED BY	CHECKED BY

DD Form 1793, FEB 71

GRAIN-SIZE ANALYSIS (HYDROMETER METHOD)

1. PROJECT		**2. DATE**
3. BORING NUMBER	**4. SAMPLE OR SPECIMEN NUMBER**	**5. CLASSIFICATION**
6. DISH NUMBER	**7. GRADUATE NUMBER**	**8. HYDROMETER NUMBER/TYPE** *(151H/152H)*
9. DISPERSING AGENT USED		**10. QUANTITY**
11. COMPOSITE CORRECTION	**12. DECIMAL FINES** *(Block 29, DD Form 1206)*	**13. SPECIFIC GRAVITY OF SOLIDS** *(Block 6n, DD Form 1208)* G_s =

14. TIME	15. ELAPSED TIME, *(T)* *minutes*	16. ACTUAL HYDROMETER READING *(R¹)*	17. CORRECTED READING *(R)*	18. TEMP *(°C)*	19. TEMPERATURE AND SPECIFIC GRAVITY CONSTANT *(K)*	20. EFFECTIVE DEPTH *(L)*	21. PARTICLE DIAMETER *(D), mm*	22. PERCENT FINER	
								a. PARTIAL	b. TOTAL

WEIGHT *(Grams)*	23. DISH + DRY SOIL	
	24. DISH	
	25. DRY SOIL *(W_s)*	

The particle diameter (D) is calculated from Stokes' equation using the corrected hydrometer reading. Use the following formula to solve for particle diameter (D): $D = K\sqrt{L/T}$

Corrected hydrometer reading (R) = actual hydrometer reading (R¹) + composite correction

W_s = Oven-dry weight (in grams) of soil used for hydrometer analysis

Hydrometer graduated in specific gravity (151H)

$$\text{Partial Percent Finer} = \left[\frac{G_s}{G_s - 1} \times \frac{100{,}000}{W_s}\right](R - 1)$$

Total Percent Finer = Partial Percent Finer x Decimal fines (Block 12)

Hydrometer graduated in grams per liter (152H)

$$\frac{(R)(a)}{W_s} \times 100$$

(a = specific gravity of solids correction factor)

26. REMARKS

FROST GROUP:

27. TECHNICIAN *(Signature)*	**28. COMPUTED BY** *(Signature)*	**29. CHECKED BY** *(Signature)*

CALIFORNIA BEARING RATIO (CBR) ANALYSIS

1. PROJECT

2. DATE

3. SOIL CLASSIFICATION

4. LOCATION OF SOIL

5. GROUP NUMBER

DATA SUMMARY

MOLD NUMBER	BLOWS PER LAYER	MOISTURE CONTENT, *W (percent)*	DRY DENSITY	CORRECTED CBR *(percent)*	SWELL *(percent)*

CBR - MOISTURE

CORRECTED CBR IN PERCENT

MOLDING WATER CONTENT (w)
(IN PERCENT) OF DRY WEIGHT

DENSITY - MOISTURE

DRY DENSITY IN POUNDS PER CU FT

MOLDING WATER CONTENT (w)
(IN PERCENT) OF DRY WEIGHT

6. TECHNICIAN *(Signature)*

7. COMPUTED BY *(Signature)*

8. CHECKED BY *(Signature)*

DD FORM 2463, DEC 1999

PREVIOUS EDITION IS OBSOLETE.

CBR FAMILY OF CURVES

W *(Percent)*		BLOWS/LAYER	BLOWS/LAYER	BLOWS/LAYER
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
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	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			
	DRY DENSITY, in pcf			
	CORRECTED CBR, in percent			

CBR FAMILY OF CURVES

CORRECTED CBR PERCENT

MOLDED DRY DENSITY, in pcf

DESIGN CBR

W *(Percent)*	LOWEST CBR	W PERCENT RANGE ________ %	ASSURED CBR
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	
		% TO %	

COMPACTION DATA

MAXIMUM DRY DENSITY (MDD) AT CE 56 ____________

RANGE ________ % TO ________ %

DENSITY ________ PCF TO ________ PCF

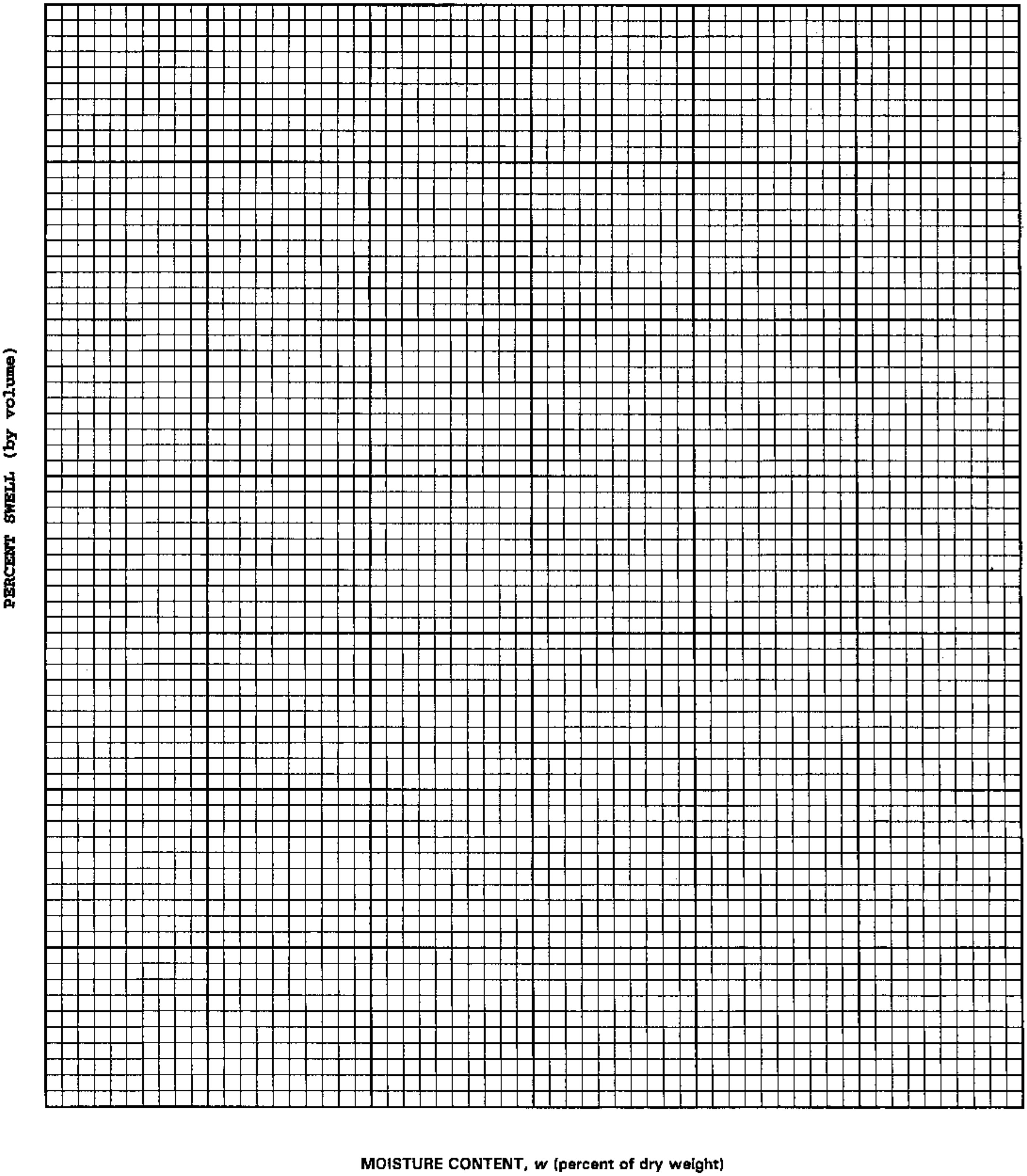
DESIGN CBR

(HIGHEST ASSURED) ____________

DESIGN MOISTURE

RANGE ________ % TO ________ %

CBR SWELL DATA



NOTE: Maximum acceptable swell is 3 percent.

REPORT OF FOUNDATION AND BORROW INVESTIGATION

1. PROJECT		2. DATE
3. JOB NUMBER	4. TEST SITE	5. LOCATION
6. TYPE OF EXPLORATION	7. BORING NUMBER	8. GROUND ELEVATION
9. PURPOSE OF EXPLORATION		

10. DEPTH BELOW SURFACE	11. ELEVATION	12. SAMPLE NUMBER	13. GRAPHIC LOG	14. GROUP SYMBOL	15. IN-PLACE CBR LIMITS	16. DESCRIPTION, TEST DATA, AND REMARKS

17. DEPTH TO WATER TABLE	18. PREPARED BY *(Signature)*	19. CHECKED BY *(Signature)*

DD FORM 2464, DEC 1999 PREVIOUS EDITION IS OBSOLETE.

FM 5-472
NAVFAC MO 330
AFJMAN 32-1221(I)
27 OCTOBER 1999

By Order of the Secretary of the Army:

ERIC K. SHINSEKI
General, United States Army
Chief of Staff

Official:

JOEL B. HUDSON
Administrative Assistant to the
Secretary of the Army
9917401

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